ASCE STANDARD

ASCE/SEI
41-23

Seismic Evaluation and Retrofit of Existing Buildings

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

STRUCTURAL
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ASCE STANDARD

ASCE/SEI

41-23

Seismic Evaluation and Retrofit of Existing Buildings

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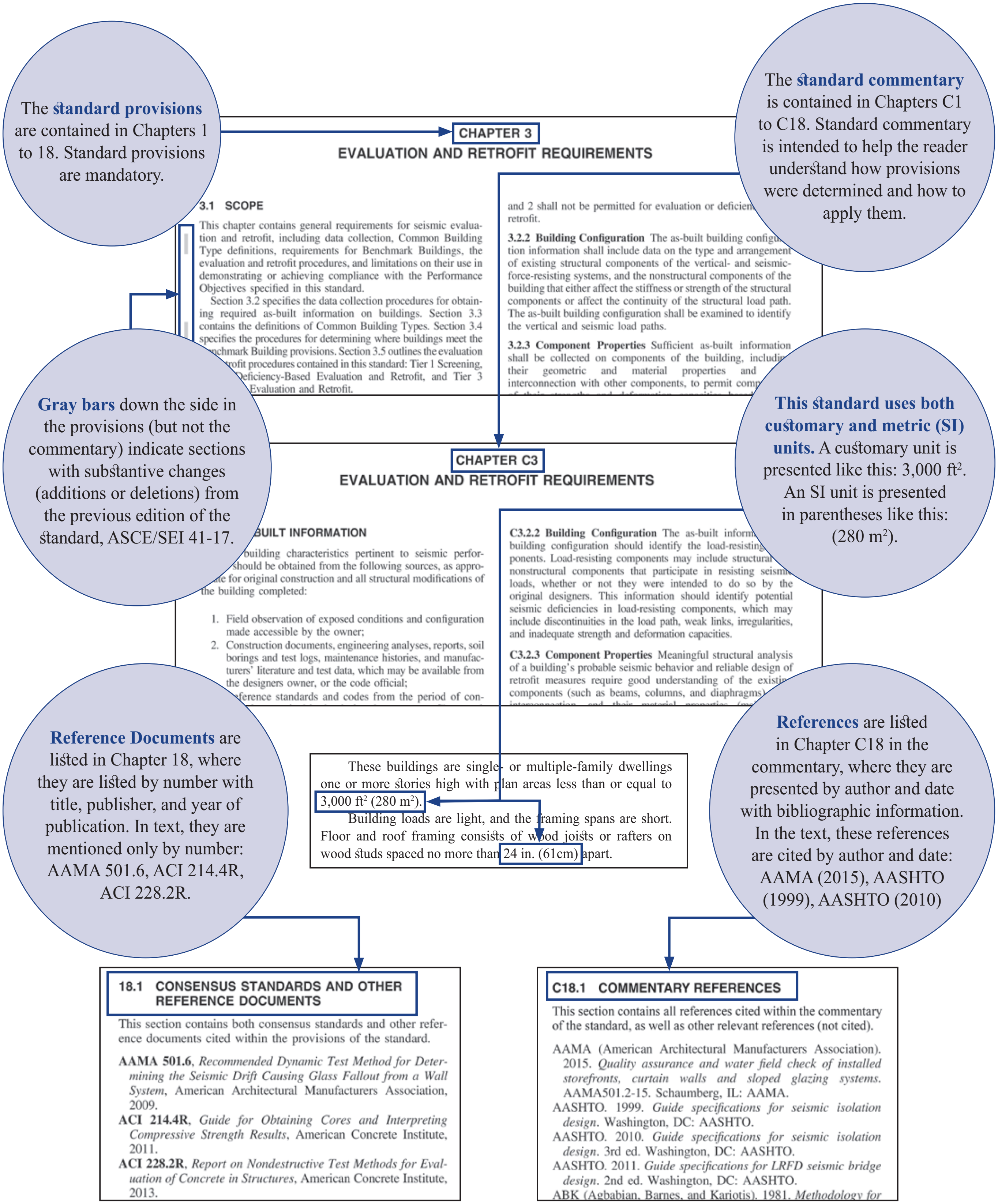


The **standard provisions** are contained in Chapters 1 to 18. Standard provisions are mandatory.

CHAPTER 3

EVALUATION AND RETROFIT REQUIREMENTS

3.1 SCOPE

This chapter contains general requirements for seismic evaluation and retrofit, including data collection, Common Building Type definitions, requirements for Benchmark Buildings, the evaluation and retrofit procedures, and limitations on their use in demonstrating or achieving compliance with the Performance Objectives specified in this standard.

Section 3.2 specifies the data collection procedures for obtaining required as-built information on buildings. Section 3.3 contains the definitions of Common Building Types. Section 3.4 specifies the procedures for determining where buildings meet the nchmark Building provisions. Section 3.5 outlines the evaluation trofit procedures contained in this standard: Tier 1 Screening, Deficiency-Based Evaluation and Retrofit, and Tier 3 Evaluation and Retrofit.

and 2 shall not be permitted for evaluation or deficien retrofit.

3.2.2 Building Configuration The as-built building configu tion information shall include data on the type and arrangement of existing structural components of the vertical- and seismic-force-resisting systems, and the nonstructural components of the building that either affect the stiffness or strength of the structural components or affect the continuity of the structural load path. The as-built building configuration shall be examined to identify the vertical and seismic load paths.

3.2.3 Component Properties Sufficient as-built information shall be collected on components of the building, includir their geometric and material properties and interconnection with other components, to permit comp

The **standard commentary** is contained in Chapters C1 to C18. Standard commentary is intended to help the reader understand how provisions were determined and how to apply them.

Gray bars down the side in the provisions (but not the commentary) indicate sections with substantive changes (additions or deletions) from the previous edition of the standard, ASCE/SEI 41-17.

CHAPTER C3

EVALUATION AND RETROFIT REQUIREMENTS

BUILT INFORMATION

building characteristics pertinent to seismic perfor- should be obtained from the following sources, as appro- ate for original construction and all structural modifications of the building completed:

1. Field observation of exposed conditions and configuration made accessible by the owner;
2. Construction documents, engineering analyses, reports, soil borings and test logs, maintenance histories, and manufacturers' literature and test data, which may be available from the designers owner, or the code official;

eference standards and codes from the period of con-

C3.2.2 Building Configuration The as-built inform building configuration should identify the load-resisting ponents. Load-resisting components may include structural nonstructural components that participate in resisting seismi loads, whether or not they were intended to do so by the original designers. This information should identify potential seismic deficiencies in load-resisting components, which may include discontinuities in the load path, weak links, irregularities, and inadequate strength and deformation capacities.

C3.2.3 Component Properties Meaningful structural analysis of a building's probable seismic behavior and reliable design of retrofit measures require good understanding of the existi components (such as beams, columns, and diaphragms)

This standard uses both customary and metric (SI) units. A customary unit is presented like this: 3,000 ft^2. An SI unit is presented in parentheses like this: (280 m^2).

These buildings are single- or multiple-family dwellings one or more stories high with plan areas less than or equal to 3,000 ft^2 (280 m^2).

Building loads are light, and the framing spans are short. Floor and roof framing consists of wood joists or rafters on wood studs spaced no more than 24 in. (61cm) apart.

Reference Documents are listed in Chapter 18, where they are listed by number with title, publisher, and year of publication. In text, they are mentioned only by number: AAMA 501.6, ACI 214.4R, ACI 228.2R.

References are listed in Chapter C18 in the commentary, where they are presented by author and date with bibliographic information. In the text, these references are cited by author and date: AAMA (2015), AASHTO (1999), AASHTO (2010)

18.1 CONSENSUS STANDARDS AND OTHER REFERENCE DOCUMENTS

This section contains both consensus standards and other reference documents cited within the provisions of the standard.

AAMA 501.6, *Recommended Dynamic Test Method for Determining the Seismic Drift Causing Glass Fallout from a Wall System*, American Architectural Manufacturers Association, 2009.

ACI 214.4R, *Guide for Obtaining Cores and Interpreting Compressive Strength Results*, American Concrete Institute, 2011.

ACI 228.2R, *Report on Nondestructive Test Methods for Evaluation of Concrete in Structures*, American Concrete Institute, 2013.

C18.1 COMMENTARY REFERENCES

This section contains all references cited within the commentary of the standard, as well as other relevant references (not cited).

AAMA (American Architectural Manufacturers Association). 2015. *Quality assurance and water field check of installed storefronts, curtain walls and sloped glazing systems*. AAMA501.2-15. Schaumberg, IL: AAMA.

AASHTO. 1999. *Guide specifications for seismic isolation design*. Washington, DC: AASHTO.

AASHTO. 2010. *Guide specifications for seismic isolation design*. 3rd ed. Washington, DC: AASHTO.

AASHTO. 2011. *Guide specifications for LRFD seismic bridge design*. 2nd ed. Washington, DC: AASHTO.

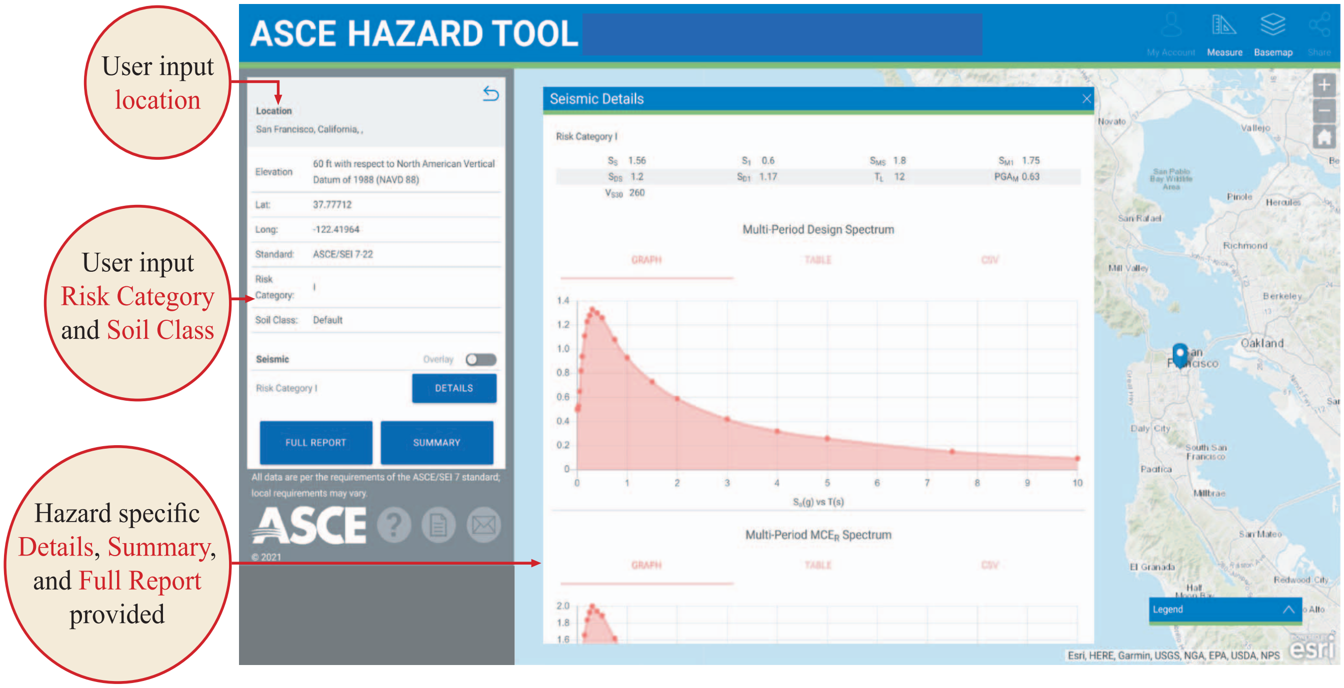
ABK (Agbabian, Barnes, and Kariotis). 1981. *Methodology for*

Supplements, errata, and interpretations may become available in the future.
Please check for important new materials at https://doi.org/10/16/9780784416112.

Tips for Using the ASCE Hazard Tool

asce7hazardtool.online

The ASCE Hazard Tool provides access to the digital data defined in the hazard Geodatabases required by ASCE standards. The digital data required for flood, ice, rain, seismic, snow, tornado, and wind are available at https://asce7hazardtool.online/. Digital data required for tsunami is available at https://asce7tsunami.online/.



Location
San Francisco, California, ,

Elevation: 60 ft with respect to North American Vertical Datum of 1988 (NAVD 88)
Lat: 37.77712
Long: -122.41964
Standard: ASCE/SEI 7-22
Risk Category: I
Soil Class: Default

Seismic
Risk Category I

All data are per the requirements of the ASCE/SEI 7 standard; local requirements may vary.

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Seismic Details

Risk Category I

S_S 1.56	S_1 0.6	S_{MS} 1.8	S_{M1} 1.75
S_{DS} 1.2	S_{D1} 1.17	T_L 12	PGA_M 0.63
V_{S30} 260			

Multi-Period Design Spectrum

$S_a(g)$ vs $T(s)$

Multi-Period MCE_R Spectrum

Esri, HERE, Garmin, USGS, NGA, EPA, USDA, NPS

User input location

User input Risk Category and Soil Class

Hazard specific Details, Summary, and Full Report provided

Digital Data: The ASCE Hazard Tool provides digital data required by ASCE Standards:

- **NEW!** Seismic hazard data from ASCE/SEI 41-23 and 41-17, including coefficients and response spectra grouped by different hazard level responses (BSE-2N, BSE-1N, etc…)
- Flood: Flood zone and static base flood elevation, plus direct links to additional information
- Tsunami: Whether the site is in a mapped tsunami design zone per the ASCE Tsunami Design Geodatabase, and link to ASCE Tsunami Design Geodatabase if required for design
- Snow: Ground snow load and winter wind parameter
- Rain: Median 15-minute and 60-minute duration rainfall intensities for 100-year mean recurrence interval
- Ice: Radial ice thickness with concurrent 3-second gust speeds and temperature concurrent with ice thickness due to freezing rain
- Seismic: Seismic coefficients S_S, S_1, S_{MS}, S_{M1}, S_{DS}, S_{D1}, T_L, PGA_M, and V_{S30}, plus the seismic design category, as well as the multi-period spectrum, the multi-period MCE_R spectrum, the two-period design spectrum, and the two-period MCE_R spectrum
- Wind: Three-second gust wind speeds at 33 feet (10 meters) above ground for Exposure Category C, including identification of hurricane-prone and wind-borne debris regions
- Tornado: Tornado wind speeds for 1,700-, 3,000-, 10,000-, 100,000-, 1,000,000-, and 10,000,000-year MRI, and for 1-, 2,000-, 10,000-, 40,000-, 100,000-, 250,000-, 1,000,000-, and 4,000,000-ft$_2$. target areas


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CONTENTS

Commentary to Standard ASCE/SEI 41-23

PREFACE

This 2023 edition of ASCE/SEI 41 *Seismic Evaluation and Retrofit of Existing Buildings* is a revision to the 2017 edition. A summary of the most significant changes that are in the ASCE/SEI 41-23 standard includes the following:

Chapter 1

- Revised the chapter to move significant material to commentary
- Changed the quality assurance, testing, and structural observation provisions to align with the *International Building Code*

Chapter 2

- Reorganized the chapter sections to place performance levels and Seismic Hazard Levels before Performance Objectives
- Adopted the 2018 USGS seismic hazard model and multi-period spectra
- Pointed to ASCE 7-22 for seismic hazard information, including new site class designations

Chapter 3

- Revised the Common Building Type definitions for wood-framed buildings
- Added criteria related to changes in Seismic Hazard Level for Benchmark Buildings
- Revised the Benchmark Building code editions
- Added Benchmark Building criteria for Risk Category III structures

Chapter 4

- Changed several of the Tier 1 Quick Check procedures

Chapter 5

- Aligned the Tier 2 Knowledge Factor with the Tier 3 requirements
- Updated the Tier 2 evaluation requirements for Steel Deck diaphragms
- Updated the Tier 2 Deficiency-Based Retrofit requirements to include retrofit-specific requirements on the resulting structure, design and detailing requirements, and definition of the scope of evaluation requirements for existing components

Chapter 6

- Revised the condition assessment and data collection requirements
- Eliminated the dependence of performance level for data collection and material testing
- Granted permission to use material property bounding in a nonlinear analysis in lieu of material testing

Chapter 7

- Aligned the dead and live load specifications with those of ASCE 7
- Aligned the snow load specifications with the new risk-targeted snow loads of ASCE 7
- Updated the viscous damping specifications
- Clarified that diaphragm ties, interconnection, wall out-of-plane anchorage, and wall out-of-plane demands are force-controlled actions
- Updated diaphragm specifications to better account for force transfer between offset vertical elements, to eliminate the linear static floor on linear dynamic forces, to allow diaphragm forces to be taken directly from a linear dynamic model or nonlinear static model, and to allow limited deformation-controlled acceptance for components modeled as linear elements in nonlinear static or dynamic analysis
- Revised the limitations for linear analysis to categorically allow linear analysis for certain simple model building types and to allow linear analysis for in-plane and out-of-plane discontinuities if the elements are treated as force-controlled
- Revised the linear lateral force specifications to be based on the new multi-period response spectra of ASCE 7
- Eliminated the J factor and added a minimum demand/capacity-based alternate for force-controlled actions
- Created specifications for modeling and acceptance of fiber elements
- Clarified the definitions of critical and noncritical elements
- Defined the valid range of modeling for unacceptable response
- Added a transient response limitation for unacceptable response
- Separated project-specific testing from general testing specifications
- Created specifications for the development of modeling parameters and acceptance criteria based on large data sets for general use
- Eliminated the use of monotonic testing except in the case of calibration of adaptive hinges
- Revised the specifications to explicitly set the Damage Control point on the generalized force-displacement curve
- Expanded the force-displacement curve beyond the Collapse Prevention point to the point of loss of vertical load-carrying capacity
- Revised the specifications to eliminate local acceptance criteria for Collapse Prevention of noncritical elements
- Added new requirements to check sliding at the soil-structure interface

Chapter 8

- Restructured the chapter to have a more logical flow when navigating the chapter based on the building foundation type, shallow or deep
- For buildings on shallow foundations, added a new section to select the appropriate analysis procedure for foundation evaluation based on foundation and superstructure characteristics prior to performing the analysis
- Added a simplified procedure for rapid evaluation of the foundation when certain conditions are met by idealizing the foundation into individual foundation segments
- Eliminated analysis procedures for shallow foundations using Methods 1-2 and 3, and foundation can be modeled as fixed base or a flexible base using linear or nonlinear analysis procedures
- Added a new section for selection of the analysis procedure
- Removed the requirement for building analysis using upper and lower bound soil properties
- Defined a new term to represent the soil short-term soil bearing capacity which is equivalent to the upper bound

soil bearing capacity value permitted to be used for foundations modeled as a fixed base or flexible base
- Determined foundation acceptance based on foundation action, either overturning axial load action, or overturning moment and axial load actions on the foundation
- Added different criteria when evaluating the foundation depending if the building is on isolated spread footings, combined footings, or mat foundations
- Added alternate provisions to determine the minimum foundation width to be used to calculate the soil stiffness for buildings on Mat foundations
- Expanded the foundation overturning moment capacity acceptance to include bidirectional moments on the footing
- For linear analysis where soil springs resist both tension and compression, spring stiffness values are half the expected stiffness of the soil which is the previous lower bound soil stiffness value
- Updated the requirements for seismic increment of earth pressure on retaining walls, which need to be considered only for performance objects higher than life safety

Chapter 9

- Chapter 9 now references AISC 342 for the modeling parameters and acceptance criteria for structural steel, composite steel-concrete, and cast and wrought iron components
- AISC 342 revises the default material strengths for various steels
- AISC 342 revises the material testing requirements for welded components
- AISC 342 revises the modeling parameters and acceptance criteria for steel columns
- AISC 342 revises the modeling parameters and acceptance criteria for beam–column connection panel zones
- AISC 342 revises the modeling parameters and acceptance criteria for pre-Northridge WUF-B beam–column connections
- AISC 342 revises the modeling parameters and acceptance criteria for welded bottom haunch with slab to include minimum requirements for the composite slab
- AISC 342 revises the modeling parameters and acceptance criteria for AISC 341 conforming beam–column connections
- AISC 342 revises the modeling parameters and acceptance criteria for steel braces in both tension and compression, with a particular impact on braces with thin walls
- AISC 342 adds explicit requirements to evaluate partial penetration welded column splices
- AISC 342 changes the designation of untopped steel deck diaphragms from force-controlled to deformation controlled and provides modeling parameters and acceptance criteria for them
- AISC 342 provides modeling parameters and acceptance criteria for concrete-filled steel deck diaphragms
- AISC 342 updates requirements for cast and wrought iron columns

Chapter 10

- 9 now references ACI 369.1 for the modeling parameters and acceptance criteria for structural steel, composite steel-concrete, and cast and wrought iron components
- ACI 369.1 revises the means to classify structural walls as shear or flexure controlled
- ACI 369.1 revises the modeling parameters and acceptance criteria for flexure controlled structural walls
- The standard modifies ACI 369.1 to revise the modeling parameters and acceptance criteria for structural walls governed by shear or shear friction at the base of the wall
- The standard modified ACI 369.1 to permit deformation-controlled actions in foundation components using modeling parameters and acceptance criteria for similar superstructure components

Chapter 11

- Revised the diagonal tension strength calculation for URM spandrels
- Clarified requirements for Comprehensive Testing of masonry
- Revised and expanded the provisions for anchorage to masonry walls
- Permitted the use of force redistribution in URM deformation-controlled lines of resistance
- Revised the linear *m*-factors for URM walls to permit evaluation of axial load ratios between 4% and 8%
- Revised the Collapse Prevention, Damage Control, and Limited Safety acceptance criteria for URM walls subject to out-of-plane actions to be consistent with the Life Safety procedure; a similar revision was also made to the Chapter 16 provisions for out-of-plane evaluation
- Completely rewritten provisions for Reinforced Masonry Walls and Wall Piers subject to in-plane actions
- Added provisions to allow the evaluation of nonconforming lap splices in Reinforced Masonry
- Added provisions for evaluation of masonry diaphragms

Chapter 12

- Revised Table 12.2-2 for single straight sheathed lumber diaphragms to clarify applicability of default properties whether the diaphragm is chorded or unchorded and accompanied by addition of a simplified diaphragm deflection equation
- Updated reference standards, including ASTM D245, ASTM D5457, US DOC PS 1, US DOC PS2, AWC National Design Specification (NDS) for Wood Construction, and AWC Special Design Provisions for Wind and Seismic (SDPWS)
- Updated criteria for determination of expected strength from SDPWS tabulated nominal strengths for shear walls and diaphragms to coordinate with reference to the 2021 Special Design Provisions for Wind and Seismic (SDPWS)
- Retitled Chapter 12 to “Wood” to reflect broad applicability of requirements beyond wood Light-frame construction; implemented consistent terminology for lumber sheathed systems throughout Chapter 12
- Revised Section 12.3.3.1 to clarify that demands on wood elements as well as bodies of metal connections are considered force-controlled actions

Chapter 13

- Reorganized the chapter to provide a more logical description of the process
- Revised Table 13-1 to eliminate the column for evaluation procedure and added section references
- Moved evaluation criteria from footnotes to Table 13-1 into the scope and acceptance criteria for the components
- Added tables of coefficients for calculation of seismic forces from ASCE 7-16
- Added a new section to clarify the requirements for determining capacity of new and existing nonstructural components
- Added a new procedure for evaluating overturning resistance for unanchored equipment

- Added criteria for evaluation of penthouses and clay tile roofs
- Clarified the requirements for evaluation of mechanical and electrical distribution systems
- Added a procedure for evaluation of multilevel steel storage racks

Chapter 14

- Revised the number of ground motions required and period range of interest for seismically isolated buildings that use the nonlinear dynamic procedure
- Editorially rewrote much of Chapter 14 for seismically isolated buildings for alignment with ASCE 7 Chapter 17
- Revised prototype test specimen adequacy/acceptance criteria for seismically isolated buildings

Chapter 15

- Revised the number of ground motions required and period range of interest for buildings with supplemental energy dissipation that use the nonlinear dynamic procedure
- Revised the criteria for deformation-controlled actions for buildings with supplemental energy dissipation which use the linear analysis procedures

Chapter 16

- Clarified and revised the requirements for New Vertical Elements in URM buildings using Chapter 16
- Added minimum requirements for the transfer of URM wall anchorage forces into diaphragms using Chapter 16

Chapter 17

- Revised and added to the Tier 1 structural checklist statements related to diaphragms
- Revised the Tier 1 structural checklist statements related to foundations and overturning
- Added Tier 1 nonstructural checklist statements for penthouses and tile roofs

ACKNOWLEDGMENTS

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DEDICATION

Michael Mahoney

ASCE 41-23 is dedicated to Mike Mahoney, who recently retired from the Federal Emergency Management Agency (FEMA) after nearly 40 years of service, mostly as a project officer for earthquake engineering programs. Mike's tireless work at FEMA led to many significant updates to this standard and its predecessor FEMA publications. He passionately advocated for FEMA to fund projects that addressed issues related to the seismic safety of new and existing buildings. Many of the FEMA-funded projects he championed and led resulted in material that greatly impacted this standard. In addition to his advocacy for the seismic safety of existing buildings, Mike was involved in or led many FEMA-funded projects that contributed to improvements in ASCE 7-22 *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. Following the Northridge earthquake, Mike served as FEMA's project officer for the SAC Steel project, resulting in the formation of much of the criteria embedded in ASCE 41 and its referenced standards for steel structures. Of all of Mike's contributions, the most significant to this standard may be his leadership in the formation of a FEMA-funded project, Update Seismic Retrofit Design Guidance, focused solely on technical development, advancement, and improvement of performance-based evaluation and retrofit provisions in ASCE 41. This project has already contributed significantly to ASCE 41-23, and ongoing work will help ensure that future editions remain a cutting-edge resource for performance-based treatment of existing buildings.

UNIT CONVERSIONS

SI Units	*Customary Units*
Measurement	
m = meter (SI base unit of length)	yd = yard
cm = centimeter	in. = inch
km = kilometer	mi = mile
ha = hectare	acre
L = liter (S.I. base unit of volume)	gal. = gallon
mL = milliliters	qt = quart
kg = kilogram (SI base unit of mass)	lb = pound
g = gram	oz = ounce
N = Newton (m·kg·s^2)	lbf = pound-force (lb/ft)
Pa = Pascals (N/m^2)	psi = pounds per square inch
kPa = kilopascals	atm = atmosphere
J = Joule ft	lbf = feet per pound-force
W = watt	Btu = British thermal unit
kW = kilowatt	hp = horsepower
s = second (S.I. base unit of time)	s = second
min = minute	min = minute
h = hour	h = hour
day	day
°C = degrees Celsius	°F = degrees Fahrenheit
ppm = parts per million	ppm = parts per million
Length	
1 m = 3.2808 ft = 1.0936 yd	1 ft = 3 yd = 0.3048 m
1 cm = 0.3937 in.	1 in. = 2.54 cm
1 km = 0.6214 mile	1 mile = 0.869 nautical mile = 1.6093 km
Area	
1 m^2 = 10.7643 ft^2	1 ft^2 = 0.0929 m^2
1 km^2 = 0.3861 mi^2	1 mi^2 = 2.59 km^2
1 ha = 2.4710 acre	1 acre = 43,560 ft^2 = 0.4047 ha
Volume 1 L = 0.2642 gal.	1 gal. = 4 qt = 3.7854 L
1 ml = 1 cm^3	1 ft^3 = 7.481 gal. = 28.32 L
Mass	
1 g = 0.0353 oz	1 oz = 28.3495 g
1 kg = 2.2046 lb	1 lb = 0.4536 kg
Force	
1 N = 0.2248 lb/ft	1 lbf = 4.4482 N
Density	
1 kg/m^2 = 0.2048 lb/ft^2	1 lb/ft^2 = 4.882 kg/m^2
1 kg/m^3 = 6.2427 lb/ft^3	1 lb/ft^3 = 16.018 kg/m^3
Pressure	
1 kPa = 0.145 psi	1 psi = 6.8948 kPa
1 atm = 14.7	1 psi = 101.35 kPa
Energy and Power	
1 J = 1.00 W·s = 0.7376 ft·lbf	1 ft·lbf = 1.3558 J
1 kJ = 0.2778 W·h = 0.948 Btu	1 Btu = 1.0551 kJ
1 W = 0.7376 ft·lbf/s = 3.4122 Btu/h	1 ft·lbf/s = 1.3558 W
1 kW = 1,3410 hp	1 hp = 550 ft·lb/s = 0.7457 kW
Flow Concentration Temperature	1 L/s = 15.85 gal./min = 2.119 ft^3/min 1 gal./min = 0.1337 ft^3/min = 0.0631 L/s mg/L = ppm$_m$ (in dilute solutions) °C = (°F − 32) × 5/9 °F = (°C × 9/5) + 32 Acceleration of gravity 32.2 ft/s^2 = 9.81 m/s^2
Fundamental Constants and Relationships	Density of water (at 4 °C) = 1,000 kg/m^3 = 1 g/cm^3 Specific weight of water (15 °C) = 62.4 lb/ft^3 = 9,810 N/m^3 Weight of water 1 gal. = 8.345 lb = 3.7854 kg

CHAPTER 1
GENERAL REQUIREMENTS

1.1 SCOPE

This standard, *Seismic Evaluation and Retrofit of Existing Buildings*, referred to herein as "this standard," specifies provisions for the seismic evaluation and retrofit of buildings. Seismic evaluation and retrofit of existing buildings shall comply with requirements of this standard to demonstrate compliance or noncompliance with, or achievement of Performance Objectives. Definitions and notation used throughout this standard are contained in Section 1.2. References used throughout this standard are cited separately in Chapter 18. Where standards are referenced and no edition or date is appended, then the edition or dated document listed in Chapter 18 is to be used. The processes for using this standard for seismic evaluation and retrofit and the associated procedures are defined in Sections 1.3 and 1.4, respectively.

1.2 DEFINITIONS AND NOTATION

1.2.1 Definitions

Acceleration-Sensitive Component: A component that is sensitive to, and subject to, damage from inertial loading.

Acceptance Criteria: Limiting values of properties, such as drift, strength demand, and inelastic deformation, used to determine the acceptability of a component at a given Performance Level.

Action: An internal moment, shear, torque, axial force, deformation, displacement, or rotation corresponding to a displacement caused by a structural degree of freedom; designated as force or deformation controlled.

Active Fault: A fault for which there is an average historic slip rate of 0.04 in. (1 mm) per year or more and evidence of seismic activity within Holocene times (the last 11,000 years).

Adaptive Model: An element model where component nonlinear action is represented by a force–deformation curve whose points change in the mathematical model based on the previous loading undergone in the mathematical model. Adaptive models should be capable of acceptable representing monotonic and cyclic response, and cyclic response to different loading protocols, as demonstrated by laboratory test data.

Aspect Ratio: Ratio of full height to length for concrete and masonry shear walls; ratio of story height to length for wood shear walls; ratio of span to depth for horizontal diaphragms.

Assembly: Two or more interconnected components.

Authority Having Jurisdiction: The organization, political subdivision, office, or individual legally charged with responsibility for administering and enforcing the provisions of this standard.

Balloon Framing: Continuous stud framing from sill to roof, with intervening floor joists nailed to studs and supported by a let-in ribbon.

Base: The level at which the horizontal seismic ground motions are considered to be imparted to the structure.

Basic Performance Objective for Existing Buildings (BPOE): A series of defined Performance Objectives based on a building's risk category meant for evaluation and retrofit of existing buildings. See Section 2.2.1.

Basic Performance Objective Equivalent to New Building Standards (BPON): A series of defined Performance Objectives based on a building's risk category meant for evaluation and retrofit of existing buildings to achieve a level of performance commensurate with the intended performance of buildings designed to a standard for new construction. *See* Chapter 2.

Beam: A structural member whose primary function is to carry loads transverse to its longitudinal axis.

Bearing Wall: A wall that supports gravity loads of at least 200 lb/ft (2,919 N/m) from floors or roofs.

Bed Joint: The horizontal layer of mortar on which a masonry unit is laid.

Benchmark Building: A building designed and constructed or evaluated to a specific performance level using an acceptable code or standard listed in Table 4-6.

Boundary Component: A structural component at the boundary of a shear wall or a diaphragm or at an edge of an opening in a shear wall or a diaphragm that possesses tensile or compressive strength to transfer lateral forces to the seismic-force-resisting system.

Braced Frame: A structural system consisting of vertical, horizontal, and diagonal structural components joined by concentric or eccentric connections. *See* **Concentrically Braced Frame** or **Eccentrically Braced Frame**.

BSE-1E: Basic Safety Earthquake-1 for use with the Basic Performance Objective for Existing Buildings, taken as a seismic hazard with a 20% probability of exceedance in 50 years, but not greater than the BSE-1N, at a site.

BSE-1N: Basic Safety Earthquake-1 for use with the Basic Performance Objective Equivalent to New Building Standards, taken as two-thirds of the BSE-2N at a site.

BSE-1X: Basic Safety Earthquake-1, either the BSE-1E or BSE-1N.

BSE-2E: Basic Safety Earthquake-2 for use with the Basic Performance Objective for Existing Buildings, taken as a seismic hazard with a 5% probability of exceedance in 50 years, but not greater than the BSE-2N, at a site.

BSE-2N: Basic Safety Earthquake-2 for use with the Basic Performance Objective Equivalent to New Building Standards, taken as the ground shaking based on the Risk-Targeted Maximum Considered Earthquake (MCE_R) per ASCE 7 at a site.

BSE-2X: Basic Safety Earthquake-2, either the BSE-2E or BSE-2N.

Building Performance Level: A limiting damage state for a building, considering structural and nonstructural components, used in the definition of Performance Objectives.

Capacity: The permissible strength or deformation for a component action.

Cast Iron: A hard, brittle, nonmalleable iron–carbon alloy containing 2.0% to 4.5% carbon. Shapes are obtained by reducing iron ore in a blast furnace, forming it into bars (or pigs), and remelting and casting it into its final form.

Cavity Wall: A masonry wall with an air space between wythes.

Checklist: Set of evaluation statements that shall be completed as part of the Tier 1 screening. Each statement represents a potential deficiency based on performance in past earthquakes.

Chord: See **Diaphragm Chord**.

Clay Tile Masonry: Masonry constructed with hollow units made of clay tile.

Clay-Unit Masonry: Masonry constructed with solid, cored, or hollow units made of clay; can be ungrouted or grouted.

Closed Stirrups or Ties: Transverse reinforcement defined in ACI 318 consisting of standard stirrups or ties with 90 degree hooks and lap splices in a pattern that encloses longitudinal reinforcement.

Code Official: The individual representing the Authority Having Jurisdiction who is legally charged with responsibility for administering and enforcing the provisions of a legally adopted regulation, building code, or policy.

Collar Joint: Vertical longitudinal joint between wythes of masonry or between masonry wythe and backup construction; can be filled with mortar or grout.

Collector: *See* **Diaphragm Collector**.

Column (or Beam) Jacketing: A retrofit method in which a concrete column or beam is encased in a steel or concrete “jacket” to strengthen or repair the member by confining the concrete.

Common Building Type: A building classification that groups buildings with common seismic-force-resisting systems and performance characteristics in past earthquakes.

Component: A part of an architectural, mechanical, electrical, or structural system of a building.

Composite Masonry Wall: Multiwythe masonry wall acting with composite action.

Composite Panel: A structural panel composed of thin wood strands or wafers bonded together with exterior adhesive.

Concentrated Plasticity Model: An element model where the nonlinear action, as represented by a force–deformation curve or a fiber model, occurs at a discrete location along the element’s length.

Concentrically Braced Frame (CBF): Braced frame in which component work lines of diagonal braces intersect at a single work point, or at multiple work points where the distance between intersecting work lines (or eccentricity) is less than or equal to the width of the smallest component connected at the joint.

Concrete Masonry: Masonry constructed with solid or hollow units made of concrete; can be ungrouted or grouted.

Connection: A link that transmits actions from one component or element to another component or element, categorized by type of action (moment, shear, or axial).

Connection Hardware: Proprietary or custom-fabricated body of a component that is used to link wood components.

Connectors: Nails, screws, lags, bolts, split rings, shear plates, headed studs, and welds used to link components to other components.

Contents: Items within the building introduced by the owner or occupants.

Continuity Plates: Column stiffeners at the top and bottom of a panel zone.

Control Node: A node located at the center of mass at the roof of a building used in the nonlinear static procedure (NSP) to measure the effects of earthquake shaking on a building.

Controlling Action: The component action that reaches its elastic limit at the lowest level of lateral deflection or loading of the structure or of a story in a structure.

Coupling Beam: A component that ties or couples adjacent shear walls acting in the same plane.

Cripple Studs: Short studs between a header and top plate at openings in wall framing, or studs between the base and sill of an opening.

Cripple Wall: Short wall between the foundation and the first-floor framing.

Critical Action: A component action whose failure results in a disproportionate collapse of a portion of a building or a significant change to the lateral-force-resisting system.

Cross Tie: A component that spans the width of the diaphragm and delivers out-of-plane wall forces over the full depth of the diaphragm.

Cross Wall: A wood-framed wall sheathed with lumber, structural panels, or gypsum wallboard.

Damping Device: An element of the damping system that dissipates energy due to relative motion of each end of the device. Damping devices include all pins, bolts, gusset plates, brace extensions, and other components required to connect damping devices to the other elements of the structure. Damping devices are classified as either displacement dependent or velocity dependent, or a combination thereof, and are permitted to be configured to act in either a linear or nonlinear manner.

Damping System: The collection of structural elements that includes all the individual damping devices, all structural elements or bracing required to transfer forces from damping devices to the base of the structure, and the structural elements required to transfer forces from damping devices to the seismic-force-resisting system.

Decay: Decomposition of wood caused by action of wood-destroying fungi. The term *dry rot* is used interchangeably with *decay*.

Decking: Solid sawn lumber or glue-laminated decking, nominally 2 to 4 in. (50.8 to 101.6 mm) thick and 4 in. (101.6 mm) or wider. Decking may be tongue-and-groove or connected at longitudinal joints with nails or metal clips.

Deep Foundation: Driven piles made of steel, concrete, or wood, cast-in-place concrete piers, or drilled shafts of concrete.

Deformability: The ratio of the ultimate deformation to the limit deformation.

Deformation-Controlled Action: An action that has an associated deformation that is allowed to exceed the yield value of the element being evaluated. The extent of permissible deformation beyond yield is based on component modification factors (*m*-factors).

Deformation-Sensitive Component: A component that is sensitive to deformation imposed by the drift or deformation of the structure, including deflection or deformation of diaphragms.

Demand: The amount of force or deformation imposed on an element or component.

Design Earthquake: A user-specified earthquake for the evaluation or retrofit of a building that has ground-shaking criteria described in Chapter 2.

Design Professional: The individual in responsible charge of the evaluation or retrofit design being performed using this standard.

Diagonal Brace: Inclined structural component designed to primarily carry axial force, enabling a braced frame to act as a truss to resist lateral forces.

Diaphragm: A horizontal (or nearly horizontal) structural element, such as a floor or roof system, used to transfer inertial lateral forces to vertical elements of the seismic-force-resisting system.

Diaphragm Chord: A boundary component perpendicular to the applied force that is provided to resist tension or compression caused by the diaphragm moment.

Diaphragm Collector: A component parallel to the applied force that transfers lateral forces from the diaphragm of the structure to vertical elements of the seismic-force-resisting system.

Diaphragm Ratio: *See* **Aspect Ratio**.

Diaphragm Strut: *See* **Diaphragm Tie**.

Diaphragm Tie: A component parallel to the applied load that is provided to transfer wall anchorage or diaphragm inertial forces within the diaphragm. Also called **Diaphragm Strut**. See **Cross Tie**, for the case where **Diaphragm Tie** spans the entire diaphragm width.

Differential Compaction: An earthquake-induced process in which soils become more compact and settle in a nonuniform manner across a site.

Dimensioned Lumber: Lumber from nominal 2 through 4 in. (50.8 through 101.6 mm) thick and nominal 2 in. (50.8) or wider.

Displacement-Dependent Damping Device: A damping device in which dissipated energy is primarily a function of the relative displacement between each end of the device. The response is substantially independent of the relative velocity between each of the devices and/or the excitation frequency.

Distributed Plasticity Model: An element model where the nonlinear action, as represented by a fiber model or a series of force–deformation curves, occurs over a distributed region of or the entire element's length.

Dowel-Type Fasteners: Bolts, lag screws, wood screws, nails, and spikes.

Drag Strut: *See* **Diaphragm Collector**.

Drift: Horizontal deflection at the top of the story relative to the bottom of the story.

Dry Rot: *See* **Decay**.

Dry Service: Structures wherein the maximum equilibrium moisture content does not exceed 19%.

Eccentrically Braced Frame (EBF): Braced frame where the distance between intersecting work lines of diagonal braces (or eccentricity) exceeds the width of the smallest component connected at the joint.

Edge Distance: The distance from the edge of the member to the center of the nearest fastener.

Effective Damping: The value of equivalent viscous damping corresponding to the energy dissipated by the building, or element thereof, during a cycle of response.

Effective Stiffness: The value of the lateral force in the building, or an element thereof, divided by the corresponding lateral displacement.

Effective Void Ratio: Ratio of collar joint area without mortar to the total area of the collar joint.

Element: An assembly of structural components that act together in resisting forces, including gravity frames, moment-resisting frames, braced frames, shear walls, and diaphragms.

Energy Dissipation Device: *See* **Damping Device**.

Energy Dissipation System: *See* **Damping System**.

Evaluation: An approved process or methodology of evaluating a building for a selected Performance Objective.

Expected Strength: The mean value of resistance of a component at the deformation level anticipated for a population of similar components, including consideration of the variability in material strength as well as strain-hardening and plastic section development.

Exterior Envelope: A nonstructural wall assembly that provides weather protection for the building. The exterior envelope includes the exterior finish material, glazing systems, and the nonstructural backup framing that attaches the exterior elements to the structural framing.

Fair Condition: Masonry found during condition assessment to have mortar and units intact but with minor cracking.

Fault: Plane or zone along which earth materials on opposite sides have moved differentially in response to tectonic forces.

Fiber Model: A representation of a component action's nonlinear behavior based on a series of unidirectional fibers located over the cross section of a component at a specific location with a specified gage length or over the entire length of the component.

Flexible Component: A component, including its attachments, having a fundamental period greater than 0.06 s.

Flexible Connection: A link between components that permits rotational or translational movement without degradation of performance, including universal joints, bellows expansion joints, and flexible metal hose.

Flexible Diaphragm: A diaphragm with horizontal deformation along its length twice or more than twice the average story drift.

Force-Controlled Action: An action that is not allowed to exceed the nominal strength of the element being evaluated.

Force–Deformation Curve: A representation of the nonlinear behavior of a component action based on a smooth or multilinear curve at a discrete location along the length of the component.

Foundation Component: The structural component in contact with the supporting soil at the base of a vertical element (column, or wall).

Foundation Overturning: Action from seismic lateral forces that causes or produces rotation or vertical displacement (settlement or uplift) of the foundation at the soil–footing interface.

Foundation System: An assembly of structural components, located at the soil–structure interface, that transfers loads from the superstructure into the supporting soil.

Full Flange Action: The flange and the web of a wall section deform as a single plane section during bending, Section 11.3.4.1.

Fundamental Period: The natural period of the building in the direction under consideration that has the greatest mass participation.

Gauge or Row Spacing: The center-to-center distance between fastener rows or gauge lines.

Global System: The primary components of a building that collectively resist seismic forces.

Glulam Beam: Shortened term for glue-laminated beam, which is a wood-based component made up of layers of wood bonded with adhesive.

Good Condition: Masonry found during condition assessment to have mortar and units intact and no visible cracking.

Grade: The classification of lumber with regard to strength and utility, in accordance with the grading rules of an approved agency.

Grading Rules: Systematic and standardized criteria for rating the quality of wood products.

Gypsum Wallboard: A sheet product consisting of a core primarily of gypsum with paper surfacing and used primarily as an interior surface sheathing material (also known as drywall).

Hazardous Materials: Toxic or Highly Toxic, as defined in 29 CFR 1910.1200, Appendix A, or explosive substances where the quantities exceed the threshold quantities established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released.

Head Joint: Vertical mortar joint placed between masonry units in the same wythe.

Header Course: A course where the masonry units are oriented perpendicular to those in the course above or below to tie the wythes of the wall together, typically with the masonry unit long dimension perpendicular to the wall.

High-Deformability Component: A component whose deformability is not less than 3.5 when subjected to four fully reversed cycles at the limit deformation.

Hollow Masonry Unit: A masonry unit with net cross-sectional area in every plane parallel to the bearing surface less than 75% of the gross cross-sectional area in the same plane.

Hoops: Transverse reinforcement defined in Chapter 21 of ACI 318 consisting of closed ties with 135 degree hooks embedded into the core and no lap splices.

In-Plane Wall: *See* **Shear Wall**.

Infill: A panel of masonry placed within a steel or concrete frame. Panels separated from the surrounding frame by a gap are termed *isolated infills*. Panels that are in full contact with a frame around its full perimeter are termed *shear infills*.

Isolation Interface: The boundary between the upper portion of the structure (superstructure), which is isolated, and the lower portion of the structure, which is assumed to move rigidly with the ground.

Isolation System: The collection of structural components that includes all individual isolation system devices, all structural components that transfer force between components of the isolation system, and all connections to other structural components. The isolation system also includes the wind-restraint system, if such a system is used.

Isolation System Device: An isolator or supplemental energy dissipation device used as part of the isolation system.

Isolator: A horizontally flexible and vertically stiff structural component of the isolation system that permits large lateral deformations under seismic load.

Joint: An area where ends, surfaces, or edges of two or more components are attached; categorized by type of fastener or weld used and method of force transfer.

Knee Joint: A joint that in the direction of framing has one column and one beam.

Landslide: A downslope mass movement of earth resulting from any cause.

Level of Seismicity: A degree of expected seismic hazard. For this standard, levels are categorized as very low, low, moderate, or high, based on mapped acceleration values and site amplification factors, as defined in Chapter 2.

Light Framing: Repetitive framing with small, uniformly spaced members.

Lightweight Concrete: Structural concrete that has an air-dry unit weight not exceeding 115 lb/ft^3 (1,840 kg/m^3).

Limit Deformation: Two times the initial deformation that occurs at a load equal to 40% of the maximum strength.

Limited-Deformability Component: A component that is neither a low-deformability nor a high-deformability component.

Linear Dynamic Procedure (LDP): A Tier 2 or Tier 3 response-spectrum-based modal analysis procedure, the use of which is required where the distribution of lateral forces is expected to depart from that assumed for the linear static procedure.

Linear Static Procedure (LSP): A Tier 2 or Tier 3 lateral force analysis procedure using a pseudolateral force. This procedure is used for buildings for which the linear dynamic procedure is not required.

Link Beam: A component between points of eccentrically connected members in an eccentrically braced frame element.

Liquefaction: An earthquake-induced process in which saturated, loose, granular soils lose shear strength and liquefy as a result of increase in pore-water pressure during earthquake shaking.

Load and Resistance Factor Design: A method of proportioning structural components (members, connectors, connections, and assemblages) using load factors and strength reduction factors such that no applicable limit state is exceeded when the structure is subjected to all design load combinations.

Load Duration: The period of continuous application of a given load, or the cumulative period of intermittent applications of load.

Load Path: A path through which seismic forces are delivered from the point at which inertial forces are generated in the structure to the foundation and, ultimately, the supporting soil.

Load Sharing: The load redistribution mechanism among parallel components constrained to deflect together.

Load/Slip Constant: The ratio of the applied load to a connection and the resulting lateral deformation of the connection in the direction of the applied load.

Local Component: A specific element or connection in a building's global system.

Low-Deformability Component: A component whose deformability is 1.5 or less.

Lower-Bound Strength: The mean minus one standard deviation of the yield strengths, Q_y, for a population of similar components.

Lumber: The product of the sawmill and planing mill, usually not further manufactured other than by sawing, resawing, passing lengthwise through a standard planing machine, cross-cutting to length, and matching.

Masonry: The assemblage of masonry units, mortar, and possibly grout or reinforcement; classified with respect to the type of masonry unit, including clay-unit masonry, concrete masonry, or hollow-clay tile masonry.

Maximum Considered Earthquake, Risk-Targeted ($\mathbf{MCE_R}$): An extreme Seismic Hazard Level set forth in ASCE 7 and determined for the orientation that results in the largest maximum response to horizontal ground motions and with adjustments for a targeted risk.

Mean Return Period: The average period of time, in years, between the expected occurrences of an earthquake of specified severity.

Means of Egress: A path for exiting a building, including but not limited to doors, corridors, ramps, and stairways.

Moisture Content: The weight of the water in wood expressed as a percentage of the weight of the oven-dried wood.

Moment-Resisting Frame (MRF): A structural system capable of resisting horizontal forces caused by the members (beams and columns) and joints resisting forces primarily by flexure.

Narrow Wood Shear Wall: Wood shear walls with an aspect ratio (height to width) greater than 2:1.

Nominal Size: The approximate rough-sawn commercial size by which lumber products are known and sold in the market. Actual rough-sawn sizes vary from nominal. Reference to standards or grade rules is required to determine nominal to actual finished size relationships, which have changed over time.

Nominal Strength: The capacity of a structure or component to resist the effects of loads, as determined by (1) computations using specified material strengths and dimensions, and formulas derived from accepted principles of structural mechanics, or (2) field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

Nonbearing Wall: A wall that supports gravity loads less than 200 lb/ft (2,919 N/m).

Noncompact Member: A steel section that has width-to-thickness ratios exceeding the limiting values for compactness specified in AISC 360.

Noncomposite Masonry Wall: Multiwythe masonry wall acting without composite action.

Noncritical Action: Any component action that is not a critical action.

Nonstructural Component: An architectural, mechanical, or electrical component of a building that is permanently installed in, or is an integral part of, a building system.

Nonstructural Performance Level: A limiting damage state for nonstructural building components used to define Performance Objectives.

Normal Wall: A wall perpendicular to the direction of seismic forces.

Occupancy: The purpose for which a building, or part thereof, is used or intended to be used, designated in accordance with the governing regulation, building code, or policy.

Open Front: An exterior building wall plane on one side only, without vertical elements of the seismic-force-resisting system in one or more stories.

Ordinary Moment Frame: A moment-resisting frame that meets the minimum requirements for an "ordinary moment frame" defined in AISC 341.

Oriented Strand Board: A mat-formed wood structural panel comprised of thin, rectangular wood strands arranged in cross-aligned layers with surface layers normally arranged in the long panel direction and bonded with waterproof adhesive.

Other Damping Devices: Devices not classified as displacement or velocity dependent shall be classified as "other."

Out-of-Plane Wall: A wall that resists lateral forces applied normal to its plane.

Overturning: Action that results when moment or axial loads are produced at the base of lateral-force-resisting elements, which is resisted by, the building weight above the point where overturning is evaluated, and the capacity of the connection to the lower structure or foundation.

Owner: The individual(s) or entity having legal possession or rights to sanction evaluation or retrofit of a building.

P-Δ (P-Delta) Effect: The secondary effect of vertical loads and lateral deflection on the shears and moments in various components of a structure.

Panel: A sheet-type wood product.

Panel Rigidity or Stiffness: The in-plane shear rigidity of a panel; the product of panel thickness and modulus of rigidity.

Panel Shear: Shear stress acting through the panel thickness.

Panel Zone: A structural component bounded by beam and column flanges within a moment-resisting beam-to-column connection.

Parapet: Portions of a wall extending above the roof diaphragm.

Partially Grouted Masonry Wall: A masonry wall containing grout in some of the cells.

Particleboard: A panel manufactured from small pieces of wood, hemp, and flax, bonded with synthetic or organic binders and pressed into flat sheets.

Perforated Wall or Perforated Infill Panel: A wall or panel not meeting the requirements for a solid wall or infill panel.

Performance Objective: One or more pairings of a selected Seismic Hazard Level with both an acceptable or desired Structural Performance Level and an acceptable or desired Nonstructural Performance Level.

Pier: Vertical portion of a wall between two horizontally adjacent openings. Piers resist axial stresses from gravity forces and bending moments from combined gravity and lateral forces.

Pitch or Spacing: The longitudinal center-to-center distance between any two consecutive holes or fasteners in a row.

Platform Framing: Construction method in which stud walls are constructed one floor at a time, with a floor or roof joist bearing on top of the wall framing at each level.

Ply: A single sheet of veneer, or several strips laid with adjoining edges that form one veneer lamina in a glued plywood panel.

Plywood: A structural panel composed of plies of wood veneer arranged in cross-aligned layers bonded with adhesive cured upon application of heat and pressure.

Pointing: The partial reconstruction of the mortar joints of a masonry wall by removing unsound mortar and replacing it with new mortar.

Pole: A round timber of any size or length, usually used with the larger end in the ground.

Pole Structure: A structure framed with generally round, continuous poles that provide the primary vertical frame and lateral-load-resisting system.

Poor Condition: Masonry found during condition assessment to have degraded mortar, degraded masonry units, or significant cracking.

Pounding: The action of two adjacent buildings coming into contact with each other during earthquake excitation as a result of their close proximity and differences in dynamic response characteristics.

Preservative: A chemical that, when suitably applied to wood, makes the wood resistant to attack by fungi, insects, marine borers, or weather conditions.

Pressure-Preservative-Treated Wood: Wood products pressure-treated by an approved process and preservative.

Primary Component: An element that is required to resist the seismic forces and accommodate seismic deformations for the structure to achieve the selected Performance Level.

Primary (Strong) Panel Axis: The direction that coincides with the length of the panel.

Probability of Exceedance: The chance, expressed as a percentage (%), that a more severe event will occur within a specified period, expressed in number of years.

Pseudo Seismic Force (V): The calculated lateral force used for the Tier 1 Quick Checks and for the Tier 2 Linear Static Procedure. The pseudolateral force represents the force required, in a linear analysis, to impose the expected actual deformation of the structure in its yielded state where subjected to the design earthquake motions.

Punched Metal Plate: A light steel plate fastener with punched teeth of various shapes and configurations that are pressed into wood members to effect force transfer.

Quick Check: Analysis procedure used in Tier 1 screenings to determine if the seismic-force-resisting system has sufficient strength or stiffness.

Redundancy: The quality of having alternative load paths in a structure by which lateral forces can be transferred, allowing the structure to remain stable following the failure of any single element.

Reentrant Corner: Plan irregularity in a diaphragm, such as an extending wing, plan inset, or E-, T-, X-, or L-shaped configuration, where large tensile and compressive forces can develop.

Reinforced Masonry: Masonry with the following minimum amounts of vertical and horizontal reinforcement: vertical reinforcement of at least 0.20 in.^{2} (129 mm^2) in cross section at each corner or end, at each side of each opening, and at a maximum spacing of 4 ft (1.2 m) throughout. Horizontal reinforcement of at least 0.20 in.^{2} (129 mm^2) in cross section at the top of the wall, at the top and bottom of wall openings, at structurally connected roof and floor openings, and at a maximum spacing of 10 ft (3.0 m).

Repointing: A method of repairing cracked or deteriorating mortar joints in which the damaged or deteriorated mortar is removed, and the joints are refilled with new mortar.

Representative Earthquake Loading Protocol: Subassemblage laboratory test using a cyclic loading pattern that simulates demands on a component or component action imposed by an earthquake.

Required Member Resistance (or Required Strength): Action on a component or connection, determined by structural analysis, resulting from the factored loads and the critical load combinations.

Resistance: The capacity of a structure, component, or connection to resist the effects of loads.

Resistance Factor: A reduction factor applied to member resistance that accounts for unavoidable deviations of the actual strength from the nominal value and for the manner and consequences of failure.

Retrofit: Improving the seismic performance of structural or nonstructural components of a building.

Retrofit Measures: Modifications to existing components, or installation of new components, that correct deficiencies identified in a seismic evaluation as part of a scheme to rehabilitate a building to achieve a selected Performance Objective.

Rigid Component: A component, including attachments, having a fundamental period less than or equal to 0.06 s.

Rigid Diaphragm: A diaphragm with horizontal deformation along its length less than half the average story drift.

Risk Category: A categorization of a building for determination of earthquake performance based on the governing regulation, building code, or policy or in lieu of an applicable regulation, building code, policy, the IBC, or ASCE 7.

Row of Fasteners: Two or more fasteners aligned with the direction of load.

Running Bond: A pattern of masonry where the head joints are staggered between adjacent courses by at least one-quarter of the length of a masonry unit.

Scragging: The process of subjecting an elastomeric bearing to one or more cycles of large-amplitude displacement.

Secondary Component: An element that accommodates seismic deformations but is not required to resist the seismic forces it may attract for the structure to achieve the selected Performance Level.

Seismic Evaluation: The process or methodology of evaluating a building for conformance or nonconformance with a specific Building, Structural, or Nonstructural Performance Objective or to identify deficiencies that prevent the building from conforming to a specific Building, Structural, or Nonstructural Performance Objective.

Seismic-Force-Resisting System: Those elements of the structure that provide its basic strength and stiffness to resist seismic forces.

Seismic Hazard Level: Ground-shaking demands of specified severity, developed on either a probabilistic or deterministic basis.

Seismic Retrofit: Measures that alter a building's structure or nonstructural components and systems so that the altered structure and nonstructural components and systems improve seismic performance.

Shallow Foundation: Isolated or continuous spread footings or mats.

Sheathing: Lumber or panel products that are attached to parallel framing members, typically forming wall, floor, ceiling, or roof surfaces.

Short Captive Column: A column with a height-to-depth ratio less than 75% of the nominal height-to-depth ratios of the typical columns at that level.

Shrinkage: Reduction in the dimensions of wood caused by a decrease of moisture content.

Site Class: A classification assigned to a site based on the types of soils present and their engineering properties, as defined in ASCE 7, Chapter 20.

Slip-Critical Joint: A bolted joint in which slip resistance of the connection is required.

Solid Masonry Unit: A masonry unit with net cross-sectional area in every plane parallel to the bearing surface equal to 75% or more of the gross cross-sectional area in the same plane.

Solid Wall or Solid Infill Panel: A wall or infill panel with openings not exceeding 5% of the wall surface area. The maximum length or height of an opening in a solid wall must not exceed 10% of the wall width or story height. Openings in a solid wall or infill panel must be located within the middle 50% of a wall length and story height and must not be contiguous with adjacent openings.

Special Moment Frame (SMF): A moment-resisting frame that meets the minimum requirements for a "special moment frame" defined in AISC 341.

Stack Bond: A placement of masonry units such that the head joints in successive courses are aligned vertically.

Stiff Diaphragm: A diaphragm that is neither flexible nor rigid.

Standard Cyclic Loading Protocol: Subassemblage lab test using a loading pattern having fully reversed cyclic displacements with progressively increasing amplitudes.

Storage Racks: Industrial pallet racks, movable shelf racks, and stacker racks made of cold-formed or hot-rolled structural members; does not include other types of racks, such as drive-in and drive-through racks, cantilever wall-hung racks, portable racks, or racks made of materials other than steel.

Story: The portion of a structure between the tops of two successive finished floor surfaces and, for the topmost story, from the top of the floor finish to the top of the roof structural element.

Story Shear Force: Portion of the pseudolateral force carried by each story of the building.

Strength: The maximum axial force, shear force, or moment that can be resisted by a component.

Stress Resultant: The net axial force, shear, or bending moment imposed on a cross section of a structural component.

Strong-Back System: A secondary system, such as a frame, commonly used to provide out-of-plane support for an unreinforced or under-reinforced masonry wall.

Strong Column–Weak Beam: A connection where the capacity of the column in any moment frame joint is greater than that of the beams, ensuring inelastic action in the beams.

Structural Component: A component of a building that provides gravity- or lateral-load resistance as part of a continuous load path to the foundation, including beams, columns, slabs, braces, walls, wall piers, coupling beams, and connections; designated as primary or secondary.

Structural Performance Level: A limiting structural damage state; used in the definition of Performance Objectives.

Structural System: An assemblage of structural components that are joined together to provide regular interaction or interdependence.

Structural Wall: A wall that resists lateral forces applied parallel with its plane; also known as an **In-Plane Wall**.

Stud: Vertical framing member in interior or exterior walls of a building.

Subassemblage: A representative assembly of components or of a specific component that is used to perform laboratory testing of a component's response to lateral forces or of a specific action within the component's response to lateral forces.

Subdiaphragm: A portion of a larger diaphragm used to distribute loads between diaphragm ties, struts, or cross ties.

Superstructure: In a building with a seismic isolation system, the portion of the structure above the isolation interface.

Target Displacement: An estimate of the maximum expected displacement of the roof of a building calculated for the design earthquake.

Tie: *See* **Diaphragm Tie**.

Tie-Down: A device used to resist uplift of the chords of light-framed shear walls.

Tie-Down System: For seismically isolated structures, the collection of structural connections, components, and elements that provide restraint against uplift of the structure above the isolation system.

Tier 1 Screening: Completion of checklists of evaluation statements that identify potential deficiencies in a building based on performance of similar buildings in past earthquakes.

Tier 2 Evaluation: An approach applicable to certain types of buildings and Performance Objectives based on specific evaluation of potential deficiencies to determine if they represent actual deficiencies that may require mitigation. Analysis of the response of the entire building may not be required.

Tier 2 Retrofit: The mitigation of deficiencies identified in the Tier 1 screening.

Tier 3 Evaluation: An approach to evaluation in which complete analysis of the response of the building to seismic hazards is performed, implicitly or explicitly recognizing nonlinear response.

Tier 3 Retrofit: An approach to retrofitting in which complete analysis of the response of the building to seismic hazards is performed, implicitly or explicitly recognizing nonlinear response.

Timber: Lumber of nominal cross-section dimensions of 5 in. (127 mm) or more.

Transverse Wall: A wall that is oriented transverse to in-plane shear walls and resists lateral forces applied normal to its plane; also known as an **Out-Of-Plane Wall**.

Ultimate Deformation: The deformation at the point where gravity-load support cannot be maintained.

Unreinforced Masonry (URM) Bearing Wall: An unreinforced masonry wall that provides vertical support for a floor or roof for which the total superimposed vertical load exceeds 100 lb/ft (1.45 kN/m) of wall.

Unreinforced Masonry (URM) Wall: A masonry wall containing less than the minimum amounts of reinforcement as defined for reinforced masonry walls; assumed to resist gravity and lateral loads solely through resistance of the masonry materials.

USGS SEISMIC DESIGN GEODATABASE: The U.S. Geological Survey (USGS) database of geocoded values of seismic design parameters S_{XS} and S_{X1} and geocoded sets of multi-period 5%-damped response spectra for the Seismic Hazard Levels specified in this standard.

User Note: The USGS Seismic Design Geodatabase is intended to be accessed through a USGS Seismic Design Web Service that allows the user to specify the site location, by latitude and longitude, and the site class to obtain the seismic design data. The USGS web service spatially interpolates between the gridded data of the USGS geodatabase. Both the USGS geodatabase and the USGS web service can be accessed at https://doi.org/10.5066/F7NK3C76.

Valid Range of Modeling: The maximum positive and negative deformations of a deformation-controlled action specified in Chapters 7 through 12 or based on subassemblage test data from a similar component action.

Velocity-Dependent Damping Device: A damping device in which dissipated energy is primarily a function of the relative velocity between each end of the device.

Veneer: A masonry wythe that provides the exterior finish of a wall system and transfers out-of-plane load directly to a backing but is not considered to add load-resisting capacity to the wall system.

Vertical Irregularity: A discontinuity of strength, stiffness, geometry, or mass in one story with respect to adjacent stories.

Wall, Flanged: A wall or wall segment with gross moment of inertia of the wall cross section bounded by the effective flange width as defined in Section 3.1.3 is at least 1.5 times the gross moment of inertia of the rectangular portion of the section. Flanged walls include barbell, C-shaped, T-shaped, and other nonrectangular shapes.

Wall Pier: Vertical portion of a wall between two horizontally adjacent openings.

Wind-Restraint System: The collection of structural components that provides restraint of the seismic-isolated structure for wind loads; may be either an integral part of the isolators or a separate device.

Wood Structural Panel: A wood-based panel product bonded with waterproof adhesive, meeting the requirements of DOC PS 1 or PS 2, including plywood, oriented strand board, and composite panels.

Wrought Iron: An easily welded or forged iron containing little or no carbon. Initially malleable, it hardens quickly when rapidly cooled.

Wythe: A continuous vertical section of a wall, one masonry unit in thickness.

Yield Story Drift: The lateral displacement of one level relative to the level above or below at which yield stress is first developed in a frame member.

1.2.2 Notation

1.2.2.1 Uppercase Notation

A = Cross-sectional area of a pile, Equation (8-13)
= Cross-sectional area of shear wall boundary members or diaphragm chords, in.^{2}, Equations (12-2), (12-3), (12-4), and (12-5)
A_a The effective peak acceleration coefficient as determined by the codes and standards referenced in Tables 3-6 and 3-7
A_b Sum of net mortared area of bed joints above and below the test unit, Equation (11-1)
A_{base} Area of foundation footprint if the foundation components are interconnected laterally, Equation (8-30)
A_{br} Average cross-sectional area of the diagonal brace, Equation (4-19)
A_c = Summation of the cross-sectional area of all columns in the story under consideration, Equation (4-7)
= Critical contact area of a footing required to support vertical gravity and overturning loads, Equation (8-13)
A_{col} Area of the end column in a frame, Equation (4-11)
A_f Actual area of the footing or foundation, Equations (8-5) and (8-8)
A'_f Minimum required soil bearing area to support the applied axial load, Section 8.4.5.2.2.1
A_n Area of net mortared or grouted section of a wall or wall pier, Chapters 11 and 16
A_{op} Area of opening in a masonry infill wall, Equation (11-31)
A_p = Area of wall tributary to the connection, Equation (4-12)
= Gross area of prestressed concrete elements, Equation (4-13)

AR Infill height to infill length ratio h_{inf}/L_{inf}, Table 11-9, Section 11.4.2.3

A_{rect} Area of the smallest rectangle that covers the footing footprint, Figure 8-4, Table 8-7 and Table 8-8

$A_{s,flange}$ Area of flexural reinforcement located within the effective width of the flange in tension oustide the web area, Equation (11-33c)

$A_{s,web}$ Area of flexural reinforcement located within the web, Equation (11-33c), Table 11-6

A_v = Shear area of masonry wall pier, Equations (C11-1) and (C11-2), Section 11.3.4

= The effective peak velocity-related coefficient as determined by the codes and standards referenced in Table 3-6

A_w = Summation of the net horizontal cross-sectional area for concrete and masonry wall or length for wood of all shear walls in the direction of loading, Equation (4-8)

= Area of infill wall, Equation (11-30)

A_{Wtot} Total area of a frame bay infilled with masonry, including openings in the infill wall, Equation (11-44)

A_x Accidental torsion amplification factor, Equation (7-4)

B Half the smaller dimension of the base of the structure, Section 8.6.2.

B_1 Damping coefficient used to adjust spectral response for the effect of viscous damping, Equation (2-3)

B_{bsa} Bessel function used to compute base slab averaging effects, Equations (8-27) and (8-28)

B_f Width of footing, typically taken as the dimension perpendicular to the direction of seismic force unless noted otherwise, Section 8.2.1.4, Equations (8-10), (8-14), and (8-24)

B'_f Effective footing width, Section 8.4.5.2.2.1

B_{fw} Width of the flange for an I-shaped footing, Equation (8-16)

B_X Numerical coefficient equal to the value of B_1 per Section 2.3.2 for the effective damping of the isolation system, β_X, at the displacement D_X, Chapter 14

C = Modification factor to relate expected maximum inelastic displacements calculated for linear elastic response, Section 4.4.2.1

= Compliant, per Chapters 3 and 17

= Damping coefficient for an energy dissipation device, Chapter 15

= Pseudo seismic compression load on the footing, Figure C8-8

C_0 = Modification factor to relate spectral displacement of an equivalent single-degree-of-freedom (SDOF) system to the roof displacement of the building multi-degree-of-freedom (MDOF) system, Equations (7-28) and (C7-4)

= Damping coefficient for fluid viscous device, Chapter 15

C_1 Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response, Chapter 7

C_2 Modification factor to represent the effects of pinched hysteresis shape, cyclic stiffness degradation and strength deterioration on the maximum displacement response, Chapter 7

C_a Modification factor for axial loads acting on the wall, Section 11.3.3.3

C'_a Modification factor for stiff and flexible diaphragms, Section 11.3.3.3

C_{cw} Modification factor for cross walls, Section 11.3.3.3

CF_1/CF_2 Stage combination factors for use with velocity-dependent energy dissipation devices, Chapter 15

C_g Modification factor for ground-level walls, Section 11.3.3.3

C_k Modification factor to convert the elastic fundamental period of the building, T, to the effective fundamental period, Section 7.4.1.3.1

C_m Effective mass factor to account for higher modal mass participation effects, Chapter 7

C_p Horizontal force factor, Equation (16-10) and Chapter 16, Table 16-4

C_{pl} Modification factor for Performance Level, Table 11-5 and Section 11.3.3.3

C_t = Numerical value for adjustment of period T, Equations (4-4) and (7-18)

= Modification factor for thin walls, Section 11.3.3.3

C_v Coefficient of variation, defined as the standard deviation divided by the mean

C_{vx} Vertical distribution factor, based on story weights and heights for the pseudo seismic force, Equations (7-24) and (7-25)

D = Generalized deformation metric

= Calculated dead load including self-weight of the foundation, Equation (8-8)

= Relative displacement between two ends of an energy dissipation device, Chapter 15

= Depth of diaphragm, Sections 16.2.3.1, 16.2.3.2, 16.2.3.3.1, and 16.2.3.4

= In-plane width dimension of masonry, in inches (millimeters), Section 16.2.3.3.2

D^- Maximum negative displacement of an energy dissipation device, Chapter 15

D^+ Maximum positive displacement of an energy dissipation device, Chapter 15

$\dot{D}$ Relative velocity between each end of the device, Chapter 15

D_{clear} Clearance from the frame, Equations (13-10) and (13-11)

D_f Depth to the foundation–soil interface, Section 8.2.1.4 and Section 8.4

DCR Demand–capacity ratio, computed in accordance with Equation (7-16)

$\overline{\mathrm{DCR}}$ Average demand–capacity ratio for elements in a story, computed in accordance with Equation (7-17)

DCR_i Controlling action demand–capacity ratio for element i in accordance with Equation (7-17)

$\mathrm{DCR}_{\max}$ = Largest demand–capacity ratio for any primary component of a building in the direction under consideration, Section 7.4.1.3.1

= Maximum demand–capacity ratio of the elements of the lateral-force-resisting system in the direct load path of the footing being evaluated, Equation (8-15)

$\mathrm{DCR}_{\min}$ The minimum demand–capacity ratio of all the deformation-controlled component actions in the load path to or from the component with the force-controlled action under consideration, Equation (7-36)

D_p Relative seismic displacement that the component must be designed to accommodate, Equations (13-9), (13-10), (13-12), and (13-13)

D_r = Quick Check drift ratio for moment frames, Equation (4-6)

= Drift ratio for nonstructural components, Equation (13-8)

D_s Depth of soft soil layer overlaying a stiff layer, Equation (8-45)

D_{TX} Total displacement, in inches (millimeters), of the isolation system in the direction under consideration, Chapter 14

D'_{TX} Minimum total displacement, in inches (millimeters), for the linear dynamic procedure of the isolation system in the direction under consideration, Chapter 14

D'_X Target displacement, in inches (millimeters), for the nonlinear static procedure at the center of rigidity of the isolation system in the direction under consideration, Chapter 14

D_X Displacement, in inches (millimeters), at the center of rigidity of the isolation system in the direction under consideration, Chapter 14

D_y Effective bilinear yield displacement of an isolation system device, Chapter 14

E = Young's modulus of elasticity

= Modulus of elasticity of the pile, Equation (8-25)

E_{fe} Expected elastic modulus of frame material, Chapter 11

E_{loop} Energy dissipated, in an isolation system device or energy dissipation device during a full cycle of reversible load over a test displacement range from Δ^+ to Δ^-, as measured by the area enclosed by the loop of the force–deflection curve, Chapters 14 and 15

E_m Elastic modulus of masonry determined in accordance with TMS 402 using expected masonry compressive strength, Equation (11-29)

E_{me} Expected elastic modulus of masonry in compression, Chapter 11

F^- Negative force in an isolation system device or energy dissipation device during a single cycle of testing at a displacement amplitude of Δ^-, Chapter 14 and 15

F^+ Positive force in an isolation system device or energy dissipation device during a single cycle of testing at a displacement amplitude of Δ^+, Chapter 14 and 15

F_1 Pseudolateral seismic force, in kips (kN), applied at Level 1, the base level, Chapter 14

F_d Total inertial force on a flexible diaphragm, Equation (C7-1)

F_i Lateral pseudo seismic force at level i, Equations (4-2a), (7-26), (C7-2), and (15-16)

F_{mc} The bearing (compressive) strength of the infill, Equation (C11-10)

F_{mi} The mth mode horizontal inertia force at level i, Equation (15-23)

F_p = Axial tensile force for the evaluation or retrofit of ties between the diaphragm and chords or boundaries, Equation (7-7)

= Horizontal seismic force for design of a structural or nonstructural component and its connection to the structure, Equation (7-8)

= Horizontal seismic force for anchorage of a wall to a diaphragm, Section 4.4.3.7 and Equation (7-9)

= Out-of-plane force per unit area for evaluation or retrofit of a wall spanning between two out-of-plane supports, Equation (7-13)

= Component seismic design force applied horizontally at the center of gravity of the component or distributed according to the mass distribution of the component, Chapter 13

F_{pe} Effective prestressing force of a prestressing tendon, Chapter 4

$F_{p,\min}$ = Minimum horizontal seismic force for anchorage of a wall to a diaphragm, Equation (7-10)

= Minimum out-of-plane force per unit area for evaluation or retrofit of a wall spanning between two out-of-plane supports, Equation (7-14)

F_{pv} Component seismic design force applied vertically at the center of gravity of the component or distributed according to the mass distribution of the component, Chapter 13

F_{px} Diaphragm inertial force at floor level x, Equation (7-26)

FRP Fiber-reinforced polymer, Chapter C11

F_{wx} Force applied to a wall at level x, Chapter 7

F_x Pseudolateral seismic force applied at floor level x, Chapters 4, 7, 14, and 15

F_y = Specified minimum yield stress for the type of steel being used, Chapters 9 and 17

= Effective bilinear yield force of an isolation system device, Chapter 14

G Effective soil shear modulus, Section 8.2.1.4

G_0 Initial or maximum soil shear modulus, Equations (8-1), (8-2), (8-3), and (8-4)

$G_v t_v$ Shear stiffness of wood structural panels, in lb/in.^{2} (kN/m^2), Equations (12-2), (12-4), and (12-5)

G_d Shear stiffness of shear wall or diaphragm assembly, Equations (9-1), (12-1), and (12-3), and Tables 12-1 and 12-2

G_m Shear modulus of masonry, Equation (11-29) and Chapter 11

$G_{\max}$ Small-strain soil shear modulus of soil, Section 8.2.1.1

G_{me} Expected shear modulus of masonry, Chapter 11

H = Horizontal load on footing

= Least clear height of opening on either side of pier, Chapter 16

H_{rw} Height of the retaining wall, Section C8.7

I Moment of inertia
I_c Effective moment of inertia of a column, Equation (11-36)
I_{ce} Equivalent moment of inertia of transformed column section, Equation (11-29)
I_f Moment of inertia of most flexible frame member confining infill panel, Chapter 11
I_g Moment of inertia of gross concrete or masonry section about centroidal axis, neglecting reinforcement, Chapters 10 and 11
IO Immediate Occupancy Performance Level
I_p Component performance factor; 1.0 shall be used for the Life Safety and Position Retention Nonstructural Performance Levels, and 1.5 shall be used for the Operational Nonstructural Performance Level, Equations (13-1), (13-2), and (13-3)

K Dimension used to calculate reinforcement development, in inches (millimeters), defined in TMS 402, Chapter C11
K' Storage stiffness of a solid viscoelastic device, Equation (15-13)
K'' Loss stiffness of a solid viscoelastic device, Chapter 15
K_e Effective lateral stiffness of the building in the direction under consideration, for use with the NSP, Section 7.4.3.2.5
K_F Format conversion factor for calculating LRFD reference resistance based on allowable stress factor, Chapter 12
K_{fl} Flexural stiffness of the equivalent composite cantilever column, Equations (11-28) and (11-29)
K_i Elastic stiffness of the building in the direction under consideration, for use with the NSP, Equation (7-27)
K_{inf} In-plane stiffness of infilled frame with unreinforced masonry infill panel, Equation (11-32)
$K_{\mathrm{ini}}^{\mathrm{solid}}$ Initial in-plane stiffness of an uncracked infilled frame with solid unreinforced masonry infill panel, Equation (11-28)
K_p Approximate stiffness of the support system of the component, its bracing, and its attachment, determined in terms of load per unit deflection at the center of gravity of the component, Equation (13-4)
K_{sh} Horizontal spring stiffness, Chapter 8
K_{shl} Shear stiffness of the equivalent composite cantilever column, Equation (11-28)
K_{xx} Rotational foundation stiffness, Equation (8-39)
K_y Translational foundation stiffness, Equation (8-38)

L = Total length of a frame, Equation (4-11)
= Unreduced live load from the original construction period, Equation (8-8)
= Length of pile in vertical dimension, Equation (8-25)
= Half the larger dimension of the base of the structure, Section 8.6.2
= Length of beam, center-to-center of columns, Chapter 4
= Length of wall or wall pier, Chapter 11
= Diaphragm span, distance between shear walls or collectors, Equations (12-3), (12-4), and (12-5)
= Span of diaphragm between shear wall and open front, in ft (m), Equation (16-21)
= Diaphragm span, Section C16.2.3.2.3 and Figure C16-1
L_{br} Average length of the diagonal brace, Equation (4-9)
L_c = Length of cross wall
= Critical length of foundation or foundation segment required to support the applied axial load, Equations (8-12) and (8-16), and Section 8.4.4.1.1.3.2
L_d Distance between lateral supports for a diaphragm, Equation (C7-1)
L_f = Span, in feet, of a flexible diaphragm that provides lateral support for a wall; the span is between vertical primary seismic-force-resisting elements that provide lateral support to the flexible diaphragm in the direction considered, Equation (7-11)
= Length of footing in the direction perpendicular to the axis of overturning action; Section 8.2.1.4, Equation (8-14), and Section 8.4.5.1
L_f' Effective footing length, Section 8.4.5.2.2.1
L_{fs} Length of foundation segment in the direction perpendicular to the axis of overturning action, Equation (8-10)
L_i Effective span for an open-front building, Equation (16-21)
L_{inf} Length of infill panel, Tables 11-8 and 11-9 and Figure C11-6
L_p Reinforced masonry wall plastic hinge length, Section 11.3.4.3
$L_{1,2,3\ldots}$ Distance from centroid of axial load on each footing segment from the point about which the moment capacity of the footing is calculated, Section C8.4.4.1.1.1.1
LS Life Safety Performance Level

$[M]$ Diagonal mass matrix, Equation (C7-4)
M^* Effective mass for the first mode, Equations (8-36) and (8-37)
M_c $0.50M_{\max}$, Section 11.3.4.3
$\mathrm{MCE_R}$ Risk-Targeted Maximum Considered Earthquake per ASCE 7
M_{CE} Moment capacity of the foundation or foundation segment, Equations (8-10), (8-14), and (8-17)
M_{CE_Ftg} The moment capacity of the footing, Equation (8-18)
$M_{CE,x}$ Moment capacity of the foundation for rocking about the minor or x-axis, Equation (C8-7)
$M_{CE,x\_\mathrm{uniaxial}}$ Uniaxial moment capacity of the foundation about the x-axis, Equation (8-23)
$M_{CE,y}$ Moment capacity of the foundation for rocking about the y-axis, Equation (8-17) and Equation (C8-5)
$M_{CE,y\_\mathrm{uniaxial}}$ Uniaxial moment capacity of the foundation about the y-axis, Equation (8-23)
M_{crd} Elastic critical distortional buckling moment, Equation (9-11)
M_{crl} Elastic critical local buckling moment, Equation (9-6)
M_{Ftg} The local moment demand on the footing, Equation (8-18)

M_{gj}	Moment in girder at level j, Equation (4-10)
$M_{\max}$	Expected moment capacity of the wall section at which the plastic hinge develops determined in accordance with the strength design provisions in TMS 402, using expected material strengths, or calculated with the nondimensionalized moment $M'_{\max}$, Section 11.3.4.3
$M'_{\max}$	Nondimensionalized moment determined according to Table 11-6, used in Equation (11-34), Section 11.3.4.3
M_n	Nominal moment strength at section, Chapter 10
M_{OT}	Total overturning moment induced on the element by seismic forces applied at and above the level under consideration, Equations (7-5) and (7-6)
M_p	Plastic moment of the gross section, Equations (9-3) and (9-8), Column plastic moment capacity, Equation (11-36)
M_s	Tier 1 system modification factor, Chapter 4.
M_{ST}	Stabilizing moment produced by dead loads acting on the element, Equations (7-5) and (7-6)
M_{UD}	= Deformation-controlled moment demand
	= The overturning moment demand on a foundation or foundation segment, Equations (8-11) and (8-22)
$M_{UD,x}$	Component of overturning moment determined about the x-axis or minor axis of overturning, Equations (8-17), (8-22), (8-23) and Section C8.4.4.1.1.1.3
$M_{UD,y}$	Component of overturning moment determined about the y-axis or major axis of overturning, Equations (8-22) and (8-23)
M_{uFy}	Bending moment in the member about the y-axis, calculated in accordance with Equation (9-12)
M_y	Yield moment of the gross section, Equations (9-2) through (9-12), (9-13), and (9-17)
M_{yE}	M1 to M4, Mcri, Chapter 9
M_{75}	$0.75M_{\max}$, Section 11.3.4.3
N	= Number of piles in a pile group, Equation (8-25)
	= Number of isolation system devices Chapter 14
N_{60}	SPT blow count corrected to an equivalent hammer energy efficiency of 60%, Equation (8-2), Section 8.2.1.4
$(N_1)_{60}$	SPT blow count normalized for an effective stress of 1 ton/ft^2 (ton/m^2) and corrected to an equivalent hammer energy efficiency of 60%, Equation (8-3)
N/A	Not applicable
N_{br}	Number of diagonal braces in tension and compression if the braces are designed for compression, number of diagonal braces in tension if the braces are designed for tension only, Equation (4-9)
NC	Noncompliant, Chapters 3 and 17
NL	No limit, Table 3-5
NP	Not permitted, Table 3-5
P	= Axial compressive force in a wall, Section 8.7 and Equation (11-33b) and (11-34)
	= Mobilized passive pressure, Figure 8-6
P_c	Lower bound of vertical compressive strength for wall or wall pier
P_{CE}	Expected gravity compressive force applied to a wall or pier component stress
P_{CL}	Lower-bound axial strength of a column, wall, or wall pier
P_D	= Superimposed dead load at the top of the wall or wall pier under consideration, Chapters 11 and 16
	= Axial load action caused by dead load, Section 8.4.4.1.1.2
P_D^{ftg}	Expected axial gravity load at the soil–footing interface determined as $1.0D$, where D is the dead load that includes the weight of the foundation., Equation (8-15)
P_{D+L}	Gravity compressive stress at the test location considering actual dead plus live loads in place at time of testing, Equations (11-1) and (16-1)
P_E	Seismic component of axial load on the footing, Equation (8-15), and Figure C8-20
P_G	= Gravity load in column
	= Gravity load at the soil–footing interface including footing weight, Equation (8-20), and Chapter C8
P_L	Axial load action caused by live load, Section 8.4.4.1.1.2
P_S	Axial load action caused by snow load, Section 8.4.4.1.1.2
P_T	Ratio of the effective translational period of the isolation system to the effective torsional period of the isolation system, Chapter 14
P_{test}	Splitting test load of masonry sample, Equation (16-2)
P_U	The expected vertical axial load on the soil at the footing interface, Equation (8-15)
$P_{U1,2,3\ldots}$	The expected vertical axial load on the soil at the footing interface, for each individual footing segment, Section 8.4.1.1.1
P_{UD}	Expected vertical load on soil at the footing interface caused by gravity and seismic loads and includes footing weight, Equation (8-10)
P_{ult}	Ultimate passive pressure, Figure 8-6
P_W	Self-weight of wall, Equations (11-8) and (11-11)
Q	Generalized force in a component
Q_c	$0.50Q_{\max}$, Figure 11-5a, Section 11.3.4
Q_{CE}	Expected strength of a deformation-controlled action of an element at the deformation level under consideration
$Q_{CE,F}$	Expected final lateral strength of URM walls or pier components, Equation (11-10)
Q_{CL}	Lower-bound estimate of the strength of a force-controlled action of an element at the deformation level under consideration
Q_D	Action caused by dead loads, Equations (7-1), (7-2), and (7-3)
Q_E	Action caused by the response to the selected Seismic Hazard Level, Equations (7-36), (7-37) and (7-38), and Chapter 14
Q_G	Action caused by gravity loads, Equations (7-1), (7-2), and (7-3)
Q_{Gf}	Expected bearing load on footing because of gravity loads, including load caused by overburden soil above the footing, Equation (8-5)
Q_L	Action caused by live load, Equations (7-1) and (7-3)
$Q_{\max}$	= Expected strength of reinforced masonry wall components, Figure 11-5a and b, Section 11.3.4

	= Maximum soil pressure under the footing, Section C8.4.4.1.1.3.2
$Q_{\min}$	Minimum soil pressure under the footing, Section C8.4.4.1.1.3.2
Q_r	Residual shear strength, Figure 11-5b, Section 11.3.4.4
Q_S	Action caused by effective snow load, Equations (7-1) and (7-3)
Q_{UD}	Deformation-controlled action caused by gravity loads and earthquake forces
Q_{UF}	Force-controlled action caused by gravity loads and earthquake forces
Q_y	= Yield strength of a component, Section 7.5.1.2
	= $0.80Q_{\max}$, Figure 11-5a, Section 11.3.4
	= $0.5Q_{\max}$, Figure 11-5b, Section 11.3.4
Q_{yL}	Mean minus one standard deviation strength for a force-controlled action determined from a series of representative subassembly tests, Section 7.6.3
Q_{75}	$0.75Q_{\max}$, Figure 11-5a, Section 11.3.4
$R_{dist,\max}$	Maximum redistribution ratio, Section C11.3.2.3.1
R_p	Nonstructural component response modification factor from Equation (13-1)
R_t	Cold-formed steel factor to translate from nominal to expected tensile stress, Table 9-1
R_y	Cold-formed steel factor to translate from nominal yield stress to expected stress, Table 9-1
RM	Reinforced Masonry, Chapter 11
RRS_{bsa}	Ratio of response spectra factor for base slab averaging, Equation (8-27)
RRS_e	Ratio of response spectra factor for embedment, Equation (8-31)
S	The elastic section modulus of a member
S_1	Spectral response acceleration parameter at a 1 s period
S_a	Spectral response acceleration
$S_a(T_X)$	5% damped spectral acceleration parameter in units of g at the effective period, T_X, Chapter 14
$\mathrm{S}_{a\ \mathrm{DIAPH}}$ (1)	URM wall out-of-plane stability factor, as function of diaphragm flexibility, Equation (11-27)
S_{a1}	Spectral response acceleration at 1 s period, Section C11.3.3.3
S_{DS}	Design short-period spectral response acceleration parameter, adjusted for Site Class, for determining Level of Seismicity, Equation (2-4)
S_{D1}	Design spectral response acceleration parameter at a 1 s period, adjusted for Site Class, for determining Level of Seismicity, Equation (2-5)
S_n	Distance between nth pile and axis of rotation of a pile group, Equation (8-26)
SPAF	System property adjustment factor, Chapter 15
SRSS	Square root sum of squares
S_S	Spectral response acceleration parameter at short periods
S_{X1}	Spectral response acceleration parameter at a 1 s period for any Seismic Hazard Level and any damping, adjusted for Site Class
S_{XS}	Spectral response acceleration parameter at short periods for the selected Seismic Hazard Level and damping, adjusted for Site Class

T	= Fundamental period of the building in the direction under consideration, in seconds
	= Fundamental period of the building using a model with a fixed base, in seconds, Section 8.6.2
	= Pseudo seismic tension load on the footing, Figure C8-8
$\tilde{T}$	Fundamental period of the building using a model with a flexible base, in seconds, Section 8.6.2
$\tilde{T}_{\mathrm{eff}}/T_{\mathrm{eff}}$	Effective period lengthening ratio, Equations (8-32) and (8-34)
T_0	Period at which the constant acceleration region of the design response spectrum begins at a value $= 0.2T_S$, Chapter 2
$T_{0.9\max}$	Period at which the multi-period design spectrum is at 90% of the maximum spectral acceleration, Section 7.4.2.2.2
T1	Tier 1 Evaluation
T2	Tier 2 Evaluation
T3	Tier 3 Evaluation
T_{90}	Period of the highest mode in the same direction as T to achieve a 90% modal mass participation, Section C7.4.4.2.3
T_C	Connection force for concrete or masonry walls to a flexible diaphragm, Equation (4-12)
T_{DIAPH}	Diaphragm period, in seconds, Section 11.3.3.3
T_e	Effective fundamental period of the building in the direction under consideration, in seconds, for use with the NSP, Equations (7-27), (7-28), and (7-29)
T_{fb}	The fundamental period, in seconds, of the structure above the isolation interface, Chapter 14
T_i	Elastic fundamental period of the building in the direction under consideration, for use with the NSP, Equation (7-27)
T_L	The long-period transition parameter, to be obtained from published maps, site-specific response analysis, or any other method approved by the Authority Having Jurisdiction
T_m	The mth mode period of the building including the stiffness of the velocity-dependent devices, Chapter 14
$T_{\max}$	Period at which the multi-period design spectrum is at the maximum spectral acceleration, Section 7.4.2.2.2
T_p	Fundamental period of the nonstructural component, Equation (13-4)
T_S	Characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum per Section 2.4
T_{ss}	Secant fundamental period of a building calculated using but replacing the effective stiffness (K_e) with the secant stiffness (K_s) at the target displacement, Chapter 15
T_X	Effective period of the seismically isolated building, in seconds, at the displacement D_X in the direction under consideration, Chapter 14
T_{xx}	Fundamental rotational period of SSI system, Equation (8-37)
T_y	Fundamental translational period of SSI system, Equation (8-36)
U	Unknown, Chapters 3 and 17

V^* Modified equivalent base shear, Chapter 15
V Pseudo seismic force, Chapters 4 and 7
V_a Shear strength of an unreinforced masonry pier, Chapter 16
V_b Lateral seismic force, in kips (kN), on the isolation system and structural elements below the base level, Chapter 14
V_{bjs1} Expected initial shear strength of wall or pier based on bed-joint sliding shear strength, Chapter 11
V_{bjs2} Expected final shear strength of wall or pier based on bed-joint sliding shear strength, Chapter 11
V_c Column shear force, Equation (4-6)
V_{ca} Total shear capacity of cross walls in the direction of analysis immediately above the diaphragm level being investigated, Chapter 16
V_{cb} Total shear capacity of cross walls in the direction of analysis immediately below the diaphragm level being investigated, Chapter 16
V_d = Base shear at Δ_d, Figure 7-3
= Diaphragm shear
V_{dt} Lower-bound shear strength based on diagonal tension stress for wall or wall pier, Equation (11-12)
V_{fre} Expected story shear strength of the bare frame taken as the shear capacity of the column, Chapter 11
V_g Shear caused by gravity loads
V_{GB1} Shear force in the grade beam segment, left, Section C8.4.4.1.1.1.1
V_{GB2} Shear force in the grade beam segment, right, Section C8.4.4.1.1.1.1
V_i The total calculated lateral shear force in an element i caused by earthquake response, assuming the structure remains elastic, calculated in accordance with Equation (7-17)
$V_{i,\text{initial}}$ Calculated shear force prior to redistribution in wall pier i, Section C11.3.2.3.1
$V_{i,\text{redistributed}}$ Shear force after redistribution in wall pier i, Section C11.3.2.3.1
V_j Story shear force, Chapter 4
V_n Nominal shear strength
V_p = Shear force at the development of the flexural capacity of an element
= Shear force on an unreinforced masonry wall pier, Equations (16-19) and (16-20)
V_r Expected shear strength of wall or wall pier based on rocking, Equations (11-8), (16-16), (16-17), and (16-20)
V_s Peak shear strength, Equations (11-24), (11-25), and (11-26)
V_{st} The total lateral seismic design force or shear on elements above the isolation system Chapter 14
V_{tc} Lower-bound shear strength based on toe crushing for a wall or wall pier, Equation (11-11)
V_{test} Test load at first observed movement of a masonry unit for an in-place masonry shear test, Equations (11-1) and (16-1)
V_{wx} Total shear force resisted by a shear wall at the level under consideration, Chapter 16
V_y Effective yield strength of the building in the direction under consideration, for use with the NSP, Section 7.4.3.2.4

W = Weight of a component, calculated as specified in this standard, Chapter 7
= Effective seismic weight of a building, including total dead load and applicable portions of other gravity loads listed in Sections 4.4.2.1 and 7.4.1.3.1
= Weight tributary to that portion of the diaphragm extending half of the distance to each adjacent tie or diaphragm boundary, Equation (7-7)
= Weight of the smaller portion of the building, Equation (7-8)
= Effective seismic weight, in kips (kN), of the building above the isolation interface, Chapter 14
W_D Area enclosed by one complete cycle of the force–displacement response of the device Chapter 15
W_d Total dead load tributary to a diaphragm, Chapter 16
W_j = Total seismic weight of all stories above level j
= Work done by an energy dissipating device, j, in one complete cycle corresponding to floor displacement Chapter 15
W_k Maximum strain energy in a frame Chapter 15
W_{mj} Work done by device j in one complete cycle of loading in the mth mode Chapter 15
W_{mk} Maximum strain energy in the frame in the mth mode, Chapter 15
W_p = Weight of the wall tributary to the wall anchor, Equations (7-9) and (7-10)
= Weight of the wall per unit area, Equations (7-13) and (7-14)
= Component operating weight, Chapter 13
W_s Effective seismic weight, in kips (kN), of the building above the isolation interface, excluding the effective seismic weight, in kips (kN), of the base level, Chapter 14
W_w Total dead load of an unreinforced masonry wall above the level under consideration or above an open front of a building, Chapter 16
W_{wx} Dead load of an unreinforced masonry wall assigned to level x, taken from midstory below level x to midstory above level x, Chapter 16

X Height of upper support attachment at level x as measured from grade, Equation (13-8), Length of rectangular distribution of soil pressure under the footing, Section C8.4.4.1.1.3.2
$X_{c.g.}$ Distance from the centroid of the footing to the edge of the footing in the direction of loading along the x-axis, Section C8.4.4.1.1.1.3
X, Y Height of lower support attachment at level x or y as measured from grade, Chapter 13

Y = Height of lower support attachment at level y as measured from grade, Equation (13-8)
= Length of triangular distribution of soil pressure under the footing, Section C8.4.4.1.1.3.2
$Y_{c.g}$ Distance from the centroid of the footing to the edge of the footing in the direction of loading along the y-axis, Section C8.4.4.1.1.1.3
Z = For columns, the sum of the plastic section moduli of all the frame columns at the level under consideration. For beams, it is the sum of the plastic section moduli of all the frame beams

with moment-resisting connections. If a beam has moment-resisting connections at both ends, then the contribution of that beam to the sum is twice the plastic section modulus of that beam (in^3), Equation (4-14)

= Seismic zone factor as determined by the reference code in Table 3-7

Y Height of lower support attachment at level y as measured from grade, Equation (13-8)

1.2.2.2 Lowercase Notation

a = Parameter used to measure deformation capacity in component load–deformation curves, Figures 7-4, C7-3, 9-1, 9-2, 12-1

= Site class factor, Equations (7-22) and (7-29)

= Longitudinal dimension of full footprint of building foundation, Chapter 8

a_0 Dimensionless frequency, Section 8.6.2

a_{75} Nondimensionalized parameter associated with ϕ_{75}, Table 11-6

a_c Nondimensionalized parameter associated with ϕ_c, Table 11-6

a_M Nondimensionalized parameter associated with $M'_{\max}$, Table 11-6

a_{m} Nondimensionalized parameter associated with ϕ_m, Table 11-6

a_n Diameter of masonry core multiplied by its length or area of the side of a square prism, Equation (16-2)

a_p Component amplification factor from Equation (13-1)

b = Parameter used to measure deformation capacity in component load–deformation curves, Figures 7-4, C7-3, 9-1, 9-2, 10-1, and 12-1

= Width of rectangular footings and the flange width of I-shaped footings, Table 8-3

= Shear wall length or width, Equations (9-1), (12-1), and (12-2)

= Diaphragm width, Equations (12-3), (12-4), and (12-5)

= Shortest plan dimension of the structure, in ft (mm), measured perpendicular to d, Chapter 14

b_0 Parameter relating effective foundation area to building period, Equation (8-29)

b_{75} Nondimensionalized parameter associated with ϕ_{75}, Table 11-6

b_c Nondimensionalized parameter associated with ϕ_c, Table 11-6

b_e Effective foundation size, in feet (meters), Equation (8-30)

b_M Nondimensionalized parameter associated with $M'_{\max}$, Table 11-6

b_{m} Nondimensionalized parameter associated with ϕ_m, Table 11-6

b_p Width of rectangular glass, Equation (13-11)

c = Parameter used to measure residual strength

= Radiation damping coefficient, Section 8.4.5.3.2

= Clearance (gap) between horizontal glass edges and the frame, Equation (13-11)

c_e Radiation damping coefficient, Equation (8-21)

c_M Nondimensionalized parameter associated with $M'_{\max}$, Table 11-6

d = Parameter used to measure deformation capacity, Figures 7-4, C7-3, 8-4, 11-1, and 12-1

= Distance from extreme compression fiber to centroid of tension reinforcement, in inches (millimeters), Chapters 4 and 17

= Width of a parapet, Figure 13-1

= Longest plan dimension of the structure, in feet (millimeters), Chapter 14

d_a = Elongation of anchorage at end of wall determined by anchorage details and load magnitude, Equations (9-1) and (12-1)

= Deflection at yield of tie-down anchorage or deflection at load level to anchorage at end of wall determined by anchorage details and dead load, in inches (millimeters), Equation (12-2)

d_b Nominal diameter of reinforcing bar, Chapter 17

e = Parameter used to measure deformation capacity, Figures 7-4, C7-3, 11-1, and 12-1

= Foundation embedment depth, in ft (meters), Equation (8-31)

= Actual eccentricity, in feet millimeters measured in plan between the center of mass of the structure above the isolation interface and the center of rigidity of the isolation system, plus accidental eccentricity taken as 5% of the longest plan dimension of the structure perpendicular to the direction of force under consideration, Chapter 14

e_{AC} Footing eccentricity when applied moment equals the moment capacity of the footing, Equation (C8-10)

e_n Nail deformation at yield load per nail for wood structural panel sheathing, Equations (12-2), (12-4), and (12-5)

e_v Void ratio, Equation (8-4)

f Parameter used to measure deformation capacity

f_1 Fundamental frequency of the building, Chapter 15

f_a Axial compressive stress caused by gravity loads, Equations (11-11) and (11-12)

f_{ae} Expected vertical compressive stress on a masonry wall, Chapter 11

f'_c Compressive strength of concrete

f'_{ce} Expected compressive strength of concrete, Table 8-11

f_d Flexible diaphragm inertial force per foot (meter), Equation (C7-1)

f'_{dt} Lower-bound masonry diagonal tension strength, Equation (11-12)

f_j^{avg} = Average axial stress in diagonal bracing elements at level j, Equation (4-9)

= Average flexural stress in the columns and beams at level j, Equation (4-15)

f'_{m} Lower-bound masonry compressive strength

f_{me} Expected compressive strength of masonry, Chapter 11

f_p The average prestress in prestressed or post-tensioned elements, Equation (4-13)

f_{sp} Tensile splitting strength of masonry, Chapters 11 and 16

f_{spe} Average mortar tensile splitting strength of masonry, Equation (11-5)

f_{spL} Mean minus one standard deviation mortar tensile splitting strength of masonry, Equation (11-7)

f'_t Lower-bound masonry tensile strength, Chapter 11

f_{te} Expected masonry flexural tensile strength, Chapter 11

f_y Specified yield stress for nonprestressed reinforcement, Chapters 4 and 17

f_{ye} Adjusted expected yield strength of reinforcing steel, Equation (11-8), Expected yield strength of reinforcing steel, Equation (11-32a)

g = Acceleration of gravity 386.1 in./s^2 (9,807 mm/s^2)

= Parameter used to measure deformation capacity

h = Average story height above and below a beam–column joint, Equations (4-6), (4-10), and (4-14)
= Effective structure height
= Clear height of wall between beams, Equation (9-33)
= Distance from inside of compression flange to inside of tension flange, Chapter 9
= Height of member along which deformations are measured
= Height of a column, pilaster, or wall, Equation (4-11) and Chapter 11
= Exponent used in accordance with Equation (7-32)
= Shear wall height, Chapters 9, 10, and 12
= Average roof elevation of structure, relative to grade elevation, Equation (13-1)
= Height of the parapet above the point of anchorage of the wall to the diaphragm, Figure 13-1
h^* Effective structural height, Equation (8-37)
h_{col} Height of column between beam centerlines, Figure C11-9
h_{eff} Effective height of wall or wall pier components under consideration, Chapters 11 and C11
h_i, h_x = Height from the base to floor level i or x, Equations (4-2a), (7-25), (8-22), and Chapter 14
= Height from the base of Building 2 to floor level i, Section 7.2.13.1
h_{inf} Height of infill panel, Chapters 11 and C11
h_n Height above base to roof level, in feet (meters), Equations (4-4), (4-11), (7-12), and (7-18)
h_p Height of rectangular glass, Equation (13-11)
h_x Height from base to floor level x, in feet (meters), Equation (7-25)
j Number of story level under consideration
k Exponent related to the building period, used to define the vertical distribution of lateral forces, Equations (4-2a) and (7-25), Wall stiffness, Equation (11-29)
k_1 = Distance from the center of the split-tee stem to the edge of the split-tee flange fillet, Equation (9-30)
= Lateral in-plane stiffness of a solid cantilevered shear wall, Equation (C11-1)
= Lateral in-plane stiffness of a wall pier between openings with full restraint against rotation at its top and bottom, Equation (C11-2)
= Elastic lateral stiffness of a reinforced masonry wall component, Figure 11-5 and Equation (11-28)
k_a Factor to account for diaphragm flexibility, Equations (7-9), (7-10), and (7-11)
k_b Stiffness of a representative beam, Equation (4-6)
k_c Stiffness of a representative column, Equation (4-6)
k_d = Postyield stiffness of an isolation system device, Chapter 14
= Stiffness of an isolation system device during unloading, Chapter 14
k_e Elastic rotational stiffness of the gross section, Equations (9-14) and (9-20)
k_{eff} Effective stiffness of an energy dissipation device, Chapter 15
k_{el} Total elastic stiffness, Equation (C11-3)
k_{fl} Flexural stiffness, Equation (C11-5)
k_h = Horizontal seismic coefficient in soil acting on retaining wall, Section C8.7
= Factor to account for variation in force over the height of the building when all diaphragms are rigid, Equations (7-9) and (7-12)
k_{ie} Initial elastic stiffness of the foundation spring, Section 8.4.5.3.2
k_s Shear stiffness, Equation (C11-4)
k_{sr} Winkler spring stiffness in overturning (rotation) for pile group, expressed as moment/unit rotation, Equation (8-26)
k_{sv} = Winkler spring stiffness in the vertical direction, expressed as force/unit displacement/unit area, Equation (8-24)
= Pile group axial spring stiffness expressed as force/unit displacement, Equation (8-25)
k_X Effective stiffness of the isolation system at displacement D_X in the direction under consideration, Chapter 14
k_{vn} Axial stiffness of nth pile in a pile group, Equation (8-26)
k_{z_sur} Winkler spring stiffness in the vertical direction, expressed as force/unit displacement/unit area, Section 8.4.5.1
l_b = Clear length of beam, Chapter 17
= Available length of straight development or lap splice, Chapter 11
l_{ceff} Assumed distance to infill strut reaction point for columns, Chapter 11
l_d Required splice length of a deformed bar, in in. (mm) defined in TMS 402, Equation (11-8)
l_w Length of entire wall or a segment of wall considered in the direction of shear force, in inches (millimeters), Table 8-11 and Chapter 11
m = Component demand modification factor to account for expected ductility associated with this action at the selected Structural Performance Level. m-factors are specified in Chapters 8 through 12
= Mass of soil associated with the foundation spring stiffness, Section 8.4.5.3.2
m_i Mass at level i, Equation (C7-5)
$m_{\max}$ Largest m-factor for all primary elements of the building in the direction under consideration, Chapter 7
n = Total number of stories in the vertical seismic framing above the base, Chapter 4 and Equation (C7-2), Total number of elements in a story, Equation (7-17)
= Shear wave velocity reduction factor, Section 8.6.1.2
= Total number of wall piers in the line of resistance, Section C11.3.2.3.1
n_c Total number of columns, Equations (4-7) and (4-10)
n_f Total number of frames, Equations (4-7) and (4-10)
n_p Number of prestressed strands, Equation (4-13)
p_a Atmospheric pressure, Equation (8-2)
q Vertical bearing pressure, Equations (8-10) and (8-14)
q_{allow} Allowable bearing pressure specified in the available design documents for the design of shallow foundations for gravity loads (dead plus live loads), Equation (8-7)
q_c Expected bearing capacity of shallow foundation expressed in load per unit area, Equations (8-7) and (8-8)
q_{cDA} Amplified expected soil bearing capacity for short-duration seismic loading, Equation (8-9)
r_I Radius of gyration of the isolation system, in ft (millimeters), Chapter 14
r_x Equivalent foundation radius for translation, Chapter 8, Section, 8.6.2
s = Average length of the braces, in feet (meters), Equation (4-9)
= Average span length of braced spans, Equation (4-9)
= Period of vibration, in seconds, Section 11.3.3.3
s_i Minimum separation distance between adjacent buildings at level i, Equation (7-15)
s_u Undrained shear strength of soil, Section 8.2.1.1
t = Thickness of footing, Equation (C8-3)
= Thickness of wall, Chapter 11
t_{inf} Thickness of infill panel, Chapter 11
t_w Thickness of wall, Table 8-11, Actual thickness of wall web, Section 11.3.4

t' Effective width of the wall section as defined in Sections 11.3.4.3.1 through 11.3.4.3.3 for reinforced masonry walls with rectangular and flanged sections, Equations (11-33b) and (11-34)

u Pore-water pressure, Equation (8-6)

v Maximum shear in the direction under consideration

v_a Shear stress for unreinforced masonry, Chapter 16

v_j^{avg} Average shear stress at level *j*, Equations (4-7) and (4-8)

v_c Unit shear strength for a cross wall, Chapter 16

v_{me} Expected masonry shear strength, Equations (11-2) and (11-9)

v_{mL} Lower-bound masonry shear strength, Equations (11-6), (11-7), (16-3), and (16-4)

v_s Effective shear wave velocity for site soil conditions, Sections 8.2 and 8.6

v_{s0} Shear wave velocity in soil at low strains, Equations (8-1) and (8-31), Section 8.6

v_s Average value of the soil shear wave velocity, Chapter 8

v_{te} Average of the bed-joint shear strength test values, Chapters 11 and 16

v_{tL} Mean minus one standard deviation of the mortar shear strength test values, v_{to}, Equations (11-6) and (16-4), Chapters 11 and 16

v_{to} Bed-joint shear strength from single test, Equations (11-1) and (16-1)

v_u Unit shear capacity for a diaphragm, Chapter 16

v_y Shear at yield in the direction under consideration

w_i = Portion of the effective seismic weight located on or assigned to floor level *i*, Equations (4-2a), (4-3a), (7-25), (7-26), (C7-2), and Chapter 15

= Portion of *Ws*, in kip (kN), that is located at Level *i*, Chapter 14

w_p Unit weight of the wall, Equation (4-12)

w_{px} Portion of the effective seismic weight tributary to the diaphragm located on or assigned to floor level *x*, Equation (7-26)

w_x = Portion of the effective seismic weight located on or assigned to floor level *x*, Equations (4-2a), (4-3a), and (7-25)

= Portion of *Ws*, in kip (kN), that is located at Level *x*, Equation (14-16)

x = Elevation in structure of component relative to grade elevation, Equation (13-1)

= Distance from the centerline of the flexible diaphragm, Equation (C7-1)

x_i = Horizontal distance, in feet (millimeters), from the center of mass to the *i*th isolation system device in the *x*-axis of the isolation system, Chapter 14

= Distance from centroid of cross section i of the footing to the y-axis, Section C8.4.4.1.1.1.3

y Distance, in inches (millimeters), between the centers of rigidity of the isolation system and the element of interest measured perpendicular to the direction of seismic loading under consideration, Chapter 14

y_i = Horizontal distance, in feet (millimeters), from the center of mass to the *i*th isolation system device in the *y*-axis of the isolation system, Chapter 14

= Distance from centroid of cross section i of the footing to the x-axis, Section C8.4.4.1.1.1.3

z_a Height, in feet (meters), of the wall anchor above the base of the structure, not to exceed h_n, Equation (7-12)

1.2.2.3 Greek Notation

Δ = Calculated deflection of diaphragm, wall, or bracing element

= Generalized deformation, Figure 12-1

= Total elastic and plastic displacement

= Calculated deflection of diaphragm, wall, or bracing element; or generalized deformation

Δ^- Negative displacement amplitude of an isolation system or energy dissipation device during a cycle of testing, Chapter 14

Δ^+ Positive displacement amplitude of an isolation system or energy dissipation device during a cycle of testing, Chapter 14

Δ_d = In-plane diaphragm displacement, in inches (millimeters), Equations (7-19) and (7-20)

= Lesser of the target displacement or displacement corresponding to the maximum base shear defined in Figure 7-3, Equation (7-32)

Δ_{eff} Differential displacement between the top and bottom of the wall or wall pier components under consideration over a height, h_{eff}, Figures C11-1 and C11-9. Chapter 11, Figure 11-5

Δ_{fallout} Relative seismic displacement (drift) causing glass fallout from the curtain wall, storefront, or partition, as determined in accordance with an approved engineering analysis method, Equations (13-12) and (13-13)

Δ_i Story displacement (drift) of story *i* divided by the story heigh

Δ_{i1} Lateral deflection of building 1 at level *i* relative to the ground for the selected Seismic Hazard Level, Equation (7-15)

Δ_{i2} Estimated lateral deflection of building 2 at level *i* relative to the ground using the provisions of this standard for the selected Seismic Hazard Level or other approved approximate procedures, Equation (7-15)

Δ_p Additional earth pressure on retaining wall caused by earthquake shaking, Section C8.7

Δ_{peak} Drift ratio at which the peak strength of an infilled frame is reached, Table 11-10

Δ_{res} Drift ratio at which the residual strength of an infilled frame is reached, Table 11-11

Δ_T Axial deformation at expected tensile yield load,

$\Delta_{tc,r}$ Lateral displacement associated with the onset of toe crushing $V_{tc,r}$, Table 11-4

Δ_w In-plane wall displacement, in inches (millimeters), Equation (7-19)

Δ_y = Calculated deflection of diaphragm, shear wall, or bracing element at yield, Equations (9-1), (12-1), (12-2), (12-3), (12-4), and (12-5)

= Displacement at effective yield strength, Figure 7-3, Equation (7-32)

= Generalized yield deformation, unitless, Figure 12-1

Γ_1 First modal mass participation factor, Equation (C7-4)

$\Sigma(\Delta_c X)$ Sum of individual chord-splice slip values on both sides of the diaphragm, each multiplied by its distance to the nearest support, Equations (12-3), (12-4), and (12-5)

ΣE_X Total energy dissipated, in kips-in. (kN-mm), in the isolation system during a full cycle of response at the displacement D_X, Chapter 14

$|\sum F_X^+|$ Absolute value of the sum, over all isolation system devices, of the force, in kips (kN), at a positive displacement equal to D_X, Chapter 14

$|\sum F_X^-|$ Absolute value of the sum, over all isolation system devices, of the force, in kips (kN), at a negative displacement equal to D_X, Chapter 14

α = Factor equal to 0.5 for fixed-free cantilevered shear wall, or 1.0 for fixed-fixed wall pier, Chapter 11
= Velocity exponent for a fluid viscous device, Chapter 15

$\bar{\alpha}$ Nondimensionalized parameter used to establish ϕ_m, ϕ_{75}, and ϕ_c, Table 11-6

$\bar{\bar{\alpha}}$ Nondimensionalized parameter used to establish $M'_{\max}$, Table 11-6

α_1 Positive postyield slope ratio equal to the positive postyield stiffness divided by the effective stiffness, Figure 7-3

α_2 Negative postyield slope ratio equal to the negative postyield stiffness divided by the effective stiffness, Figure 7-3, Equation (7-33)

α_e Effective negative postyield slope ratio equal to the effective postyield negative stiffness divided by the effective stiffness, Equations (7-32) and (7-33)

$\alpha_{\text{P-}\Delta}$ Negative slope ratio caused by P-Δ effects, Figure 7-3, Equation (7-33)

α_{xx} Dimensionless factor, function of dimensionless frequency a_0, Equation (8-44)

β = Effective viscous damping ratio of the structural system expressed as a decimal (as opposed to percent)
= Factor to adjust empirical fundamental period of the building, Equations (4-4) and (7-18)
= Ratio of expected frame strength to expected infill strength, Equation (11-12)

β_{eff} Effective damping of an energy dissipation device or system, Chapter 15

β_f Soil–structure interaction damping ratio, Equations (8-32) and (8-33)

β_{rd} Radiation damping ratio, Section 8.6.2

β_s Effective soil hysteretic damping ratio, Section 8.6.2

β'_s Soil hysteretic damping ratio, Equation (8-45)

β_{sp} Factor to adjust for spandrel length to height aspect ratio, Equation (11-18)

β_{SSI} Effective damping ratio of the structure–foundation system, Section 8.6.2, Equation (8-32)

β_X Effective damping of the isolation system at displacement D_X in the direction under consideration, Chapter 14

β_{xx} Rotational foundation damping ratio, Equation (8-42)

γ = Unit weight, weight/unit volume [lb/ft^3 (kg/m^3)], Equation (8-1)
= Load factor (Table 7-8)
= Reinforcement size factor defined in TMS 402, Chapter C11

γ_c Drift ratio at which Q_c is reached, Equation (11-30c)

γ_f Fraction of unbalanced moment transferred by flexure at slab–column connections, Chapter 10

γ_{fc} Drift ratio contributed by flexure at γ_c, Equation (11-31c)

γ_{fm} Drift ratio contributed by flexure at γ_m, Equation (11-31a)

γ_{f75} Drift ratio contributed by flexure at γ_{75}, Equation (11-31b)

γ_{m} Drift ratio at which $Q_{\max}$ is reached, Equation (11-30a)

γ_t Average total unit weight of overburden soil, Equation (8-6), Section C8.7

γ_{vc} Drift ratio contributed by shear at γ_c, Equation (11-32c)

γ_{vm} Drift ratio contributed by shear at γ_m, Equation (11-32a)

γ_{v75} Drift ratio contributed by shear at γ_{75}, Equation (11-32b)

γ_{75} Drift ratio at which Q_{75} is reached, Equation (11-30b)

δ Lateral displacement of shallow foundation, Figure 8-6

δ_{avg} The average of displacements at the extreme points of the diaphragm at level x, Chapter 7

δ_{cm} Displacement at the center of mass of the roof, Section 7.4.3.3.1

δ_i Displacement at level i caused by seismic force F_i, Equation (C7-2)

$\delta_{\max}$ The maximum displacement at any point of the diaphragm at level x, Chapter 7

δ_t Target displacement, Section 7.4.3.3.2, and Section 8.6.2

δ_{xA} Deflection at level x of building A, determined by analysis as defined in Equations (13-8) and (13-9)

δ_{xB} Deflection at building level x of building B, determined by analysis as defined in Equation (13-9)

δ_y Yield displacement of the building, Section 8.6.2

δ_{yA} Deflection at level y of building A, determined by analysis as defined in Equation (13-8)

ζ_f Reduction factor for the flexural stiffness term to account for the effect of masonry cracking, Section 11.3.4

ζ_v Reduction factor for the shear stiffness term to account for the effect of masonry cracking, Section 11.3.4

η Displacement multiplier, greater than 1.0, to account for the effects of torsion, Section 7.2.4.2.2

η_f Nondimensionalized parameter for flange to web reinforcing ratio, Equation (11-33c) and Table 11-6

θ = Generalized deformation, radians
= Theta 1: Rotation at Point B in Figure 9-2
= Theta 2: Rotation at Point C in Figure 9-2
= Theta 3: Rotation at Point D Figure 9-2
= Theta 4: Rotation at Point E in Figure 9-2

θ_i Story drift ratio, radian

θ_j Angle of inclination of energy dissipation device to the horizontal, Chapter 15

θ_{strut} Angle of the infill strut with respect to the horizontal, Chapter C11

θ_y = Generalized yield rotation, radians
= Rotation at which the gross section would reach the yield moment, Equations (9-13) through (9-16), (9-18), (9-19) through (9-22), and (9-24)

κ A knowledge factor used to reduce component strength based on the level of knowledge obtained for individual components during data collection, Sections 5.2.6, 6.2.4, and 8.4.2

λ = Near-field effect factor, Equation (7-33)
= Property modification factor for isolation system and energy dissipation devices, Chapters 14 and 15

$\lambda_{ae\ \max}$ Property modification factor for calculation of the maximum value of the isolation system device or energy dissipation device property of interest, used

	to account for aging effects and environmental conditions, Chapters 14 and 15
$\lambda_{ae\ \mathrm{min}}$	Property modification factor for calculation of the minimum value of the isolation system device or energy dissipation device property of interest, used to account for aging effects and environmental conditions, Chapters 14 and 15
λ_{max}	Property modification factor for calculation of the maximum value of the isolation system device or energy dissipation device property of interest, used to account for all sources of property variability, Chapters 14 and 15
λ_{min}	Property modification factor for calculation of the minimum value of the isolation system device or energy dissipation device property of interest, used to account for all sources of property variability, Chapters 14 and 15
$\lambda_{\mathrm{spec\ max}}$	Property modification factor for calculation of the maximum value of the isolation system device or energy dissipation device property of interest, used to account for permissible manufacturing variation, Chapters 14 and 15
$\lambda_{\mathrm{spec\ min}}$	Property modification factor for calculation of the minimum value of the isolation system device or energy dissipation device property of interest, used to account for permissible manufacturing variation, Chapters 14 and 15
$\lambda_{\mathrm{test\ max}}$	Property modification factor for calculation of the maximum value of the isolation system device or energy dissipation device property of interest, used to account for variation in vertical load, rate of loading or velocity effects, effects of heating during cyclic motion, history of loading, scragging (temporary degradation of properties with repeated cycling), and other potential sources of variation as measured by prototype testing, Chapters 14 and 15
$\lambda_{\mathrm{test\ min}}$	Property modification factor for calculation of the minimum value of the isolation system device or energy dissipation device property of interest, used to account for variation in vertical load, rate of loading or velocity effects, effects of heating during cyclic motion, history of loading, scragging (temporary degradation of properties with repeated cycling), and other potential sources of variation as measured by prototype testing, Chapters 14 and 15
μ	Expected ductility demand, Section 8.6.2
μ_{max}	Maximum strength ratio, Equation (7-32)
μ_{OT}	Response modification factor for overturning moment M_{OT}, Equation (7-6)
μ_{strength}	Ratio of the elastic strength demand to yield strength, Equations (7-23), (7-31), and (C7-3)
ν	Poisson's ratio, Section 8.2.1.1 and Equation (8-24)
ρ	Ratio of nonprestressed tension reinforcement, Chapters 8 and 11
$\rho_{f,\mathrm{web}}$	Ratio of cross-sectional area of flexural reinforcement located within the wall length l_w and web width t_w, to the cross-sectional area of the equivalent rectangular section, $l_w \times t'$, Equation (11-33a) and Table 11-6
ρ_g	Total of vertical reinforcement ratio plus horizontal reinforcement ratio in a wall or wall pier, Chapter 11
ρ_h	Horizontal reinforcement ratio in a wall or wall pier, Chapter 11
σ	Standard deviation of the variation of the material strengths, Section 7.5.1.4
$\sigma'_1, \sigma'_2, \sigma'_3$	Triaxial components of the state of stress the soil is under below the footing, Section 8.2.1.4
σ_a	Nondimensionalized parameter for axial stress, Equation (11-33b)
σ'_{mp}	Mean effective stress $(\sigma'_1 + \sigma'_2 + \sigma'_3)$ averaged over the relevant region below the footing, Equations (8-5) and (8-6)
σ'_{vo}	Effective vertical stress, Equation (8-6)
ϕ =	Strength reduction factor
=	Angle of shearing resistance for soil, Chapter 8
ϕ'	Effective stress friction angle, Section 8.2.1.1
ϕ_c	Wall curvature at which the moment reaches M_c, Section 11.3.4.3
ϕ_{fyE}	Curvature at section at first yield, defined as the curvature at which the yield strain of the reinforcing steel is first reached in tension, or a concrete strain of 0.002 is reached in compression; evaluated using expected material properties, Chapter 10
ϕ_{m}	Wall curvature at which the moment reaches M_{max}, Section 11.3.4.3
ϕ_{yE}	Curvature in the effective bilinear moment-curvature relationship associated with M_{yE}; evaluated using expected material properties, Chapter 10
ϕ_1	First mode shape vector, Equation (C7-4)
$\phi_{1,r}$	Ordinate of mode shape 1 at the roof control node, Equations (C7-4) and (C7-5)
ϕ_i	Modal displacement of floor i, Chapter 15
$\phi_{i,n}$	Ordinate of mode shape i at level n, Equation (C7-5)
ϕ_{rj}	First mode relative displacement in horizontal direction of energy dissipation device j, Chapter 15
ϕ_{75}	Wall curvature at which the moment reaches M_{75}, Section 11.3.4.3
χ =	A factor for calculation of out-of-plane wall anchorage forces, Equations (7-9), (7-10), (7-13), and (7-14)
=	A factor for adjusting action caused by response for the selected performance level, Equations (7-35) and (7-38)
Ψ =	A factor related to performance level for wall anchorage forces, Equation (4-12)
=	Dimensionless factor, function of Poisson's ratio, Equation (8-43)
Ω_0	Overstrength factor for the component, Equations (13-7a) and (13-7b)
$\bar{\omega}$	Nondimensionalized parameter for web reinforcing, Equation (11-33a) and Table 11-6
ω_1	Fundamental angular frequency equal to $2\pi f_1$, Chapter 15

1.3 SEISMIC EVALUATION PROCESS

Seismic evaluation shall be conducted in accordance with the process outlined in Sections 1.3.1 through 1.3.4.

1.3.1 Assignment of Performance Objective A seismic Performance Objective per Section 2.4, shall be assigned for the building.

1.3.2 Level of Seismicity The Level of Seismicity at the building site shall be determined in accordance with Section 2.5.

1.3.3 As-Built Information Available as-built information for the building shall be obtained per Section 3.2, and the applicable requirements of Chapters 4 through 6.

1.3.4 Evaluation Procedures Based on the Performance Objective, Level of Seismicity, and building type, an applicable evaluation procedure shall be selected in accordance with Section 3.4.

1.4 SEISMIC RETROFIT PROCESS

Seismic retrofit design of an existing building shall be conducted in accordance with the process outlined in Sections 1.4.1 through 1.4.5.

1.4.1 Assignment of Performance Objective A seismic Performance Objective per Section 2.4, shall be assigned for the building.

1.4.2 Level of Seismicity The Level of Seismicity of the building shall be determined in accordance with Section 2.5.

1.4.3 As-Built Information As-built information for the building shall be obtained as specified in Section 3.2, and Chapters 5 or 6.

1.4.4 Verification of Retrofit Design The design of retrofit measures shall be verified to meet the requirements of this standard for the assigned Performance Objective(s) through an analysis of the building, including the retrofit measures, consistent with the applicable retrofit procedures specified in Section 3.4.

1.4.5 Quality Assurance and Structural Observation Seismic retrofit work shall be checked for quality of construction and general compliance with the intent of construction documents for the seismic retrofit design. Quality assurance, including special inspection and tests, shall conform to the requirements Section 1.4.5.1 and structural observation shall be in accordance with the requirements of Section 1.4.5.2.

1.4.5.1 Special Inspections and Testing Special inspection and tests of the seismic retrofit work shall be in accordance with the provisions and reference standards in Chapters 8 through 15 and with the governing regulations, applicable building code, or policy. Where no governing regulation, applicable building code, or policy exists, Sections 1704 and 1705 of the International Building Code shall be used.

1.4.5.2 Structural Observation Structural observation shall be in accordance with the governing regulation, applicable building code, or policy or Section 1704.6 of the *International Building Code*, if no governing regulation, applicable building code or policy exists.

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CHAPTER 2
PERFORMANCE OBJECTIVES AND SEISMIC HAZARDS

2.1 SCOPE

The selection of a Performance Objective shall be in accordance with Section 2.4. A Performance Objective shall consist of one or more pairings of a selected Seismic Hazard Level, as defined in Section 2.3, with a target Structural Performance Level and a target Nonstructural Performance Level, as defined in Sections 2.2.1 and 2.2.2, respectively.

2.2 PERFORMANCE LEVELS

2.2.1 Structural Performance Levels and Ranges The Structural Performance Level of a building shall be selected from the following discrete Structural Performance Levels defined in Table 2-1.

Design procedures and acceptance criteria corresponding to these Structural Performance Levels shall be as specified in Chapters 4 through 16.

2.2.2 Nonstructural Performance Levels The target Nonstructural Performance Level for a building shall be selected from the following discrete Nonstructural Performance Levels: Operational (N-A), Position Retention (N-B), Life Safety (N-C), Hazards Reduced (N-D), and Not Considered (N-E) in Table 2-2. Design procedures and acceptance criteria corresponding to these Nonstructural Performance Levels shall be as specified in ASCE 7, Chapter 13.

2.3 SEISMIC HAZARD

The seismic hazard caused by ground shaking for the specified Seismic Hazard Level shall be based on the location of the building with respect to causative faults and the regional and site-specific geologic and geotechnical characteristics. Assessment of the site-failure hazards caused by earthquake-induced geologic and geotechnical conditions shall be performed in accordance with Chapter 8. The site class shall be classified in accordance with Chapter 20 of ASCE 7.

Seismic hazard caused by ground shaking shall be defined as acceleration response spectra or ground motion acceleration histories determined on either a probabilistic or deterministic basis. Acceleration response spectra shall be developed in accordance with either the general procedure of Section 2.3.2 or the site-specific procedure of Section 2.3.3. Ground motion acceleration histories shall be developed in accordance with Section 2.4.3. The level of seismicity of the site of the building shall be determined as specified in Section 2.5.

The site-specific procedure shall be used where required by Section 11.4.7 of ASCE 7.

2.3.1 Seismic Hazard Levels The performance levels in Section 2.2 shall be based on Seismic Hazard Levels defined in this section. The Seismic Hazard Level shall be represented by the general response spectra of Section 2.3.2 or the site-specific procedures of Section 2.3.3.

Table 2-1. Structural Performance Levels.

Structural Performance Level	Designation	Post-Earthquake Damage State Description
Immediate Occupancy	S-1	The structure remains safe to occupy and essentially retains its pre-earthquake strength and stiffness.
Damage Control	S-2	A damage state between Performance Levels S-3 and S-1. Acceptance criteria for evaluation or retrofit based on the Damage Control Structural Performance Level shall be taken as halfway between those for Immediate Occupancy and Life Safety.
Life Safety	S-3	The structure has damaged components but retains a margin of safety against the onset of partial or total collapse.
Limited Safety	S-4	A damage state between Performance Levels S-3 and S-5. Acceptance criteria for evaluation or retrofit based on the Limited Safety Structural Performance Level shall be taken halfway between those for Life Safety and Collapse Prevention.
Collapse Prevention	S-5	The structure has damaged components and continues to support gravity loads but retains no margin against collapse.
Structural Performance Not Considered	S-6	Used where an evaluation or retrofit does not address the structure.

Table 2-2. Nonstructural Performance Levels.

Nonstructural Performance Level	Designation	Post-Earthquake Damage State Description
Operational	N-A	Nonstructural components are able to provide the functions they provided in the building before the earthquake. Nonstructural components in compliance with the acceptance criteria of this standard for Operational Nonstructural Performance (N-A) and the requirements of ASCE 7, Chapter 13, where $I_p = 1.5$, are expected to achieve this post-earthquake state.
Position Retention	N-B	Nonstructural components might be damaged to the extent that they cannot immediately function but are secured in place so that damage caused by falling, toppling, or breaking of utility connections is avoided. Building access and Life Safety systems, including doors, stairways, elevators, emergency lighting, fire alarms, and fire suppression systems, generally remain available and operable, provided that power and utility services are available. Nonstructural components in compliance with the acceptance criteria of this standard for Position Retention Nonstructural Performance (N-B) and the requirements of ASCE 7, Chapter 13, are expected to achieve this post-earthquake state.
Life Safety	N-C	Nonstructural components may be damaged, but the consequential damage does not pose a life-safety threat. Nonstructural components in compliance with the acceptance criteria of this standard for Life Safety Nonstructural Performance (N-C) and the requirements of ASCE 7, Chapter 13, are expected to achieve this post-earthquake state.
Hazards Reduced	N-D	Nonstructural components are damaged and could potentially create falling hazards, but high-hazard nonstructural components identified in ASCE 7, Table 13.1-1, are secured to prevent falling into areas of public assembly or those falling hazards from those components could pose a risk to life safety for many people. Preservation of egress, protection of fire suppression systems, and similar life-safety issues are not addressed in this Nonstructural Performance Level.
Nonstructural Performance Not Considered	N-E	Used where an evaluation or retrofit does not address all nonstructural components to one of the levels in the previous sections.

2.3.1.1 BSE-2N Seismic Hazard Level The BSE-2N Seismic Hazard Level shall be the risk-targeted maximum considered earthquake (MCE_R) determined per Section 11.4, of ASCE 7.

2.3.1.2 BSE-1N Seismic Hazard Level The BSE-1N Seismic Hazard Level shall be taken as two-thirds of the values of the parameters for the BSE-2N Seismic Hazard Level, determined in accordance with Section 2.3.1.1.

2.3.1.3 BSE-2E Seismic Hazard Level The BSE-2E Seismic Hazard Level shall be based on a probabilistic hazard with a 5% in 50-year probability of exceedance. The response spectral ordinates for the BSE-2E need not be greater than those for BSE-2N at the corresponding periods.

2.3.1.4 BSE-1E Seismic Hazard Level The BSE-1E Seismic Hazard Level shall be based on a probabilistic hazard with a 20% in 50-year probability of exceedance The response spectral ordinates for the BSE-1E need not be greater than those for BSE-1N at the corresponding periods.

2.3.1.5 Seismic Hazard Levels for Other Probabilities of Exceedance, Risk Targets, or Deterministic Hazards Seismic Hazard Levels corresponding to other probabilities of exceedance, risk targets, or deterministic hazards shall be obtained from approved seismic hazard curves or a site-specific seismic hazard evaluation. When a Seismic Hazard Level other than the ones specified in Sections 2.3.1.1 through 2.3.1.4 is used as part of an evaluation or retrofit, the following information shall be documented:

1. The probabilistic or deterministic basis for the Seismic Hazard Level or ground motion accelerations;
2. Whether the Seismic Hazard Level was derived based on a USGS seismic hazard model or through site-specific procedures;
3. Whether the Seismic Hazard Level is represented by a maximum direction response spectrum, a geometric mean response spectrum or another response spectrum;
4. The basis for the site-specific procedure, including parameters such as ground motion models and how site effects were incorporated; and
5. The damping ratio used to develop the response spectrum.

2.3.2 General Response Spectrum A general horizontal response spectrum shall be developed using multiple periods as specified in Section 2.3.2.1. Where information is not available to develop a multi-period general response spectrum, a two-period general response spectrum shall be developed as indicated in Section 2.3.2.2. A general vertical response spectrum shall be developed as specified in Section 2.3.2.3.

2.3.2.1 Multi-Period General Horizontal Response Spectrum The multi-period design response spectrum shall be developed as follows:

1. At discrete values of period, T, equal to 0.0 s, 0.01 s, 0.02 s, 0.03 s, 0.05 s, 0.075 s, 0.1 s, 0.15 s, 0.2 s, 0.25 s, 0.3 s, 0.4 s, 0.5 s, 0.75 s, 1.0 s, 1.5 s, 2.0 s, 3.0 s, 4.0 s, 5.0 s, 7.5 s and 10 s, the 5%-damped design response spectral acceleration parameter, S_a, shall be taken as the multi-period 5%-damped

response spectrum at the specified Seismic Hazard Level from the USGS Seismic Design Geodatabase for the applicable site class.

2. At each response period, T, less than 10 s and not equal to one of the discrete values of period, T, listed in Item 1 above, S_a, shall be determined by linear interpolation between values of S_a, of Item 1 above.
3. At each response period, T, greater than 10 s, S_a, shall be taken as the value of S_a at the period of 10 s of Item 1 above, factored by $10/T$, where the value of T is less than or equal to that of the long-period transition period, T_L, and shall be taken as the value of S_a at the period of 10 s factored by $10T_L/T^2$, where the value of T is greater than that of the long-period transition period, T_L.

Where required in this standard S_{XS} and S_{X1} shall be defined from the multi-period response spectrum based on the provisions in Section 21.4 of ASCE 7, with S_{XS} replacing S_{DS} and S_{X1} replacing S_{D1}.

If a multi-period response spectrum with a damping ratio different than 5% is required, the multi-period response spectrum ordinates shall be divided by B_1 at all periods greater than $T_0 = 0.2\, S_{X1}/S_{XS}$ and by a factor linearly interpolated between 1.0 and B_1 between periods 0 and T_0 respectively, as follows:

$$B_1 = 4/[5.6 - \ln(100\beta)]$$

where β is the effective viscous damping ratio of the response spectra expressed as a decimal.

2.3.2.2 Two-Period General Horizontal Response Spectrum A two-period general horizontal response spectrum, as shown in Figure 2-1, shall be developed using Section 11.4.5.2, of ASCE 7 for spectral response acceleration, S_a, versus structural period, T, in the horizontal direction replacing S_{DS} and S_{D1} with S_{XS}/B_1 and S_{X1}/B_1, respectively, except where $T < T_0$ where S_a shall be interpolated between $0.4S_{XS}$ and S_{XS}/B_1 between 0 and T_0. S_{XS} and S_{X1} shall be adjusted for site class effects.

2.3.2.3 General Vertical Response Spectrum Where a vertical response spectrum is required for analysis per Chapter 7, it shall be developed in accordance with Section 11.9, of ASCE 7. Alternatively, it shall be permitted to develop a site-specific vertical response spectrum in accordance with Section 2.3.3.

2.3.3 Site-Specific Procedure for Hazards Caused by Ground Shaking Where site-specific ground-shaking characterization is used as the basis of evaluation or retrofit design, the characterization shall be developed in accordance with Chapter 21 of ASCE 7, replacing MCE_R for the specific Seismic Hazard Level being considered. For Seismic Hazard Levels based solely on probabilistic ground motions, deterministic ground motion shall not apply. Where deterministic ground motions are used, except in the determination of the BSE-1N and BSE-2N, a percentile other than specified in Section 21.2.2, of ASCE 7 is permitted and the deterministic lower limit need not apply.

Site-specific hazard response spectra for the BSE-1E and BSE-2E shall not be less than the percentage of the general response spectrum defined in Section 21.2.3, of ASCE 7 with the BSE-1E or BSE-2E replacing the MCE_R.

2.3.4 Ground Motion Acceleration Histories Development of ground motion acceleration histories shall be performed according to Section 16.2, of ASCE 7 with the following modification:

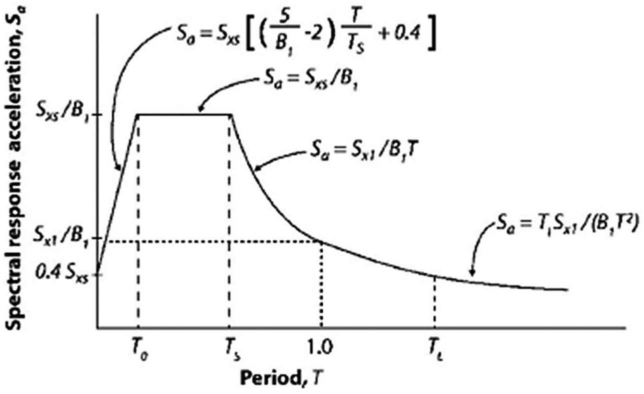

Figure 2-1. Two-Period General Horizontal Response Spectrum

1. Target Spectrum: Replace all references to MCE_R with the applicable target spectrum (BSE-1X or BSE-2X).
2. Kinematic Interaction: Kinematic interaction effects consisting of base slab averaging and foundation embedment shall be computed according to Section 8.6.1.
3. Period Range for Scaling or Matching: The period range shall be determined, corresponding to the vibration periods that significantly contribute to the building's lateral dynamic response. This period range shall have an upper-bound period greater than or equal to $1.5T_{\max}$ and a lower-bound period that does not exceed $0.2T_{\min}$, where $T_{\min}$ and $T_{\max}$ are the smallest and largest first-mode period for the two principal horizontal directions of response, respectively. The upper-bound period shall not be taken as less than 1 s. Where vertical response is considered in the analysis, the lower-bound period used for modification of vertical components of ground motion need not be taken as less than the larger of 0.1 s, or the lowest period at which significant vertical mass participation occurs.
4. Spectral Matching Limitation: Ground motion modification procedures, including spectral matching, shall not be used with Method 2 defined in ASCE 7, Section 16.2.1.2, unless the resulting suite retains a dispersion consistent with the unmodified suite of ground motions.

For buildings using seismic isolation systems, the requirements of Section 14.2.2.1, shall also apply. For buildings using supplemental energy dissipation systems, the requirements of Section 15.2.2.1, shall also apply.

2.4 PERFORMANCE OBJECTIVES

2.4.1 Basic Performance Objective for Existing Buildings (BPOE) When selected, the Basic Performance Objective for Existing Buildings (BPOE), which is a specified performance objective that varies with risk category, shall be in accordance with Table 2-3. Tier 1, Tier 2, or Tier 3 procedures are permitted to be used to demonstrate compliance with the BPOE based on the requirements in Table 2-4 and subject to the limitations on their use in Chapter 3.

2.4.2 Enhanced Performance Objectives A performance objective higher than the BPOE, including any performance objective described by one or more of the following, shall be designated as an Enhanced Performance Objective:

Table 2-3. Basic Performance Objective for Existing Buildings (BPOE).

Risk Category	BSE-1E	BSE-2E
I and II	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Collapse Prevention Structural Performance Hazards Reduced Nonstructural Performance* (5-D)
III	Damage Control Structural Performance Position Retention Nonstructural Performance (2-B)	Limited Safety Structural Performance Hazards Reduced Nonstructural Performance* (4-D)
IV	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Life Safety Structural Performance Hazards Reduced Nonstructural Performance* (3-D)

*Compliance with ASCE 7 provisions for new construction is deemed to comply.

Table 2-4. Scope of Assessment Required for Tier 1 and Tier 2 with the Basic Performance Objective for Existing Buildings (BPOE).

	Tiers 1 and 2[a]	
Risk Category	BSE-1E	BSE-2E
I and II	Not evaluated Life Safety Nonstructural Performance (3-C)	Collapse Prevention Structural Performance Hazards Reduced Nonstructural Performance[b] (5-D)
III	Not evaluated Position Retention Nonstructural Performance (2-B)	Limited Safety Structural Performance[c] Hazards Reduced Nonstructural Performance[b] (4-D)
IV	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Life Safety Structural Performance[d] Hazards Reduced Nonstructural Performance[b] (3-D)

[a] For Tier 1 and Tier 2 assessments of Risk Categories I through III, Structural Performance for the BSE-1E is not explicitly evaluated.
[b] Compliance with ASCE 7 provisions for new construction is deemed to comply.
[c] For Risk Category III, the Tier 1 screening checklists shall be based on the Collapse Prevention Performance Level (S-5), except that checklist statements using the Quick Check procedures of Section 4.4.3, shall be based on M_s factors taken as the average of the values for Life Safety and Collapse Prevention.
[d] For Risk Category IV, the Tier 1 screening checklists shall be based on the Collapse Prevention Performance Level (S-5), except that checklist statements using the Quick Check procedures of Section 4.4.3, shall be based on M_s factors for Life Safety.

1. Target Structural Performance Levels or Nonstructural Performance Levels that exceed those of the BPOE at the BSE-1E hazard level, the BSE-2E hazard level, or both, given the building's risk category.
2. Target Structural Performance Levels or Nonstructural Performance Levels of the BPOE using a Seismic Hazard Level greater than either the BSE-1E or BSE-2E hazard level, or both, given the building's risk category.
3. Target Building Performance Levels of the BPOE using a risk category higher than the building would be assigned.

2.4.3 Limited Performance Objectives A performance objective lower than the BPOE, including any performance objective described by one or more of the following, shall be designated as a Limited Performance Objective:

1. Target Structural Performance Levels or Nonstructural Performance Levels that are less than those of the BPOE at the BSE-1E hazard level, the BSE-2E hazard level, or both, given the building's risk category.
2. Target Structural Performance Levels or Nonstructural Performance Levels of the BPOE using a Seismic Hazard Level less than either the BSE-1E or BSE-2E hazard levels, or both, given the building's risk category.
3. A performance objective that satisfies the BSE-1E or BSE-2E portion of the BPOE, but not both, except where specifically allowed by Section 2.4.1.
4. Building Performance Levels using the BPOE for a lower risk category than the building would be assigned.

2.4.4 Basic Performance Objective Equivalent to New Building Standards (BPON) When selected, the Basic Performance Objective Equivalent to New Building Standards (BPON), which is a specific performance objective to be used only with Tier 3 systematic evaluation or retrofit that varies with Risk Category, shall be in accordance with Table 2-5.

2.4.5 Partial Retrofit A partial retrofit, which addresses a portion or portions of the building without evaluating or rehabilitating the complete lateral-force-resisting system, shall meet all the following requirements:

1. Does not result in a reduction in the Structural Performance Level or Nonstructural Performance Levels of the existing building for the same Seismic Hazard Level;

Table 2-5. Basic Performance Objective Equivalent to New Building Standards (BPON).

	Seismic Hazard Level	
Risk Category	**BSE-1N**	**BSE-2N**
I and II	Life Safety Structural Performance Position Retention Nonstructural Performance (3-B)	Collapse Prevention Structural Performance Hazards Reduced Nonstructural Performance* (5-D)
III	Damage Control Structural Performance Position Retention Nonstructural Performance (2-B)	Limited Safety Structural Performance Hazards Reduced Nonstructural Performance* (4-D)
IV	Immediate Occupancy Structural Performance Operational Nonstructural Performance (1-A)	Life Safety Structural Performance Hazards Reduced Nonstructural Performance* (3-D)

*Compliance with ASCE 7 provisions for new construction is deemed to comply.

2. Does not create a new structural irregularity or make an existing structural irregularity more severe;
3. Does not result in an increase in the seismic forces to any component that is deficient in capacity to resist such forces; and
4. Incorporates structural elements that are connected to the existing structure in compliance with the requirements of this standard.

2.4.6 System-Specific Performance Procedures The system-specific performance procedures in Chapter 16 are permitted to be used to meet the Performance Objective as defined for that procedure in Chapter 16.

2.5 LEVEL OF SEISMICITY

The Level of Seismicity shall be defined as High, Moderate, Low, or Very Low as defined in Table 2-6, where S_{DS} and S_{D1} are defined as S_{XS} and S_{X1} of the BSE-1N Seismic Hazard Level.

Table 2-6. Level of Seismicity Definitions.

Level of Seismicity*	S_{DS}	S_{D1}
Very low	$<0.167\,g$	$<0.067\,g$
Low	$\geq 0.167\,g$ $<0.33\,g$	$\geq 0.067\,g$ $<0.133\,g$
Moderate	$\geq 0.33\,g$ $<0.50\,g$	$\geq 0.133\,g$ $<0.20\,g$
High	$\geq 0.50\,g$	$\geq 0.20\,g$

*The higher level of seismicity defined by S_{DS} or S_{D1} shall govern.

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CHAPTER 3
EVALUATION AND RETROFIT REQUIREMENTS

3.1 SCOPE

This chapter contains general requirements for seismic evaluation and retrofit, including data collection, Common Building Type definitions, requirements for Benchmark Buildings, the evaluation and retrofit procedures, and limitations on their use in demonstrating or achieving compliance with the Performance Objectives specified in this standard.

Section 3.2 specifies the data collection procedures for obtaining required as-built information on buildings. Section 3.3 contains the definitions of Common Building Types. Section 3.4 specifies the procedures for determining where buildings meet the Benchmark Building provisions. Section 3.5 outlines the evaluation and retrofit procedures contained in this standard: Tier 1 Screening, Tier 2 Deficiency-Based Evaluation and Retrofit, and Tier 3 Systematic Evaluation and Retrofit.

3.2 AS-BUILT INFORMATION

Before beginning an evaluation or retrofit in accordance with this standard, sufficient general information about the building shall be obtained to determine the permitted evaluation or retrofit procedures, in accordance with Section 3.5. This step includes determining the building type classification, in accordance with Section 3.2.1 as required for determining the allowable evaluation procedures in accordance with Section 3.5.

Once a procedure has been selected, the required building data to be collected shall be in accordance with the requirements of this section, in addition to any data required for the specific procedures as identified in Chapters 4, 5, and 6.

The as-built information on building configuration, building components, site and foundation, and adjacent structures shall be obtained in accordance with Sections 3.2.2, 3.2.3, 3.2.4, and 3.2.5, respectively. These data shall be obtained from available drawings, specifications, and other documents for the existing construction. Data collected from available documents shall be supplemented and verified by on-site investigations, including nondestructive examination and testing of building materials and components as required for the procedures in Chapters 4, 5, or 6.

At least one site visit shall be made to observe exposed conditions of building configuration, building components, site and foundation, and adjacent structures, made accessible by the owner, to verify that as-built information obtained from other sources is representative of the existing conditions.

3.2.1 Building Type Where required by this standard, the building shall be classified as one or more of the Common Building Types in accordance with Section 3.3 based on the seismic-force-resisting system and the diaphragm type. If the structural system does not comply with one or more of those defined in Section 3.3 in both principal directions, then Tiers 1 and 2 shall not be permitted for evaluation or deficiency-based retrofit.

3.2.2 Building Configuration The as-built building configuration information shall include data on the type and arrangement of existing structural components of the vertical- and seismic-force-resisting systems, and the nonstructural components of the building that either affect the stiffness or strength of the structural components or affect the continuity of the structural load path. The as-built building configuration shall be examined to identify the vertical and seismic load paths.

3.2.3 Component Properties Sufficient as-built information shall be collected on components of the building, including their geometric and material properties and their interconnection with other components, to permit computation of their strengths and deformation capacities based on the requirements of the selected procedure.

3.2.4 Site and Foundation Information Data on foundation configuration and soil surface and subsurface conditions at the site shall be obtained from existing documentation, visual site reconnaissance, or a program of site-specific subsurface investigation in accordance with Chapter 8. A site-specific subsurface investigation shall be performed where Enhanced Performance Objectives are selected, or where insufficient data are available to quantify foundation capacities or determine the presence of geologic site hazards identified in Section 8.2.2. Where historic information indicates that geologic site hazards have occurred in the vicinity of the site, a site-specific subsurface investigation shall be performed to investigate the potential for geologic site hazards at the site. Use of applicable existing foundation capacity or geologic site hazard information available for the site shall be permitted.

A site reconnaissance shall be performed to observe variations from existing building drawings, foundation modifications not shown on existing documentation, the presence of adjacent development or grading activities, and evidence of poor foundation performance.

3.2.5 Adjacent Buildings Sufficient data shall be collected on the configuration and separation of adjacent structures to permit investigation of the interaction issues identified in Sections 3.2.5.1 through 3.2.5.3 where required by the selected procedure. If the necessary information on adjacent structures is not available, the potential consequences of the interactions that are not being evaluated shall be documented.

3.2.5.1 Building Pounding Data shall be collected to permit evaluation of the effects of building pounding, wherever a portion of an adjacent structure is located within 4% of the height above grade at the location of potential impact.

3.2.5.2 Shared Element Condition Data shall be collected on adjacent structures that share common vertical- or seismic-force-resisting elements with the building to permit investigation of the implications of the adjacent structure's influence on the performance of the investigated building in accordance with the selected evaluation procedure.

3.2.5.3 Hazards from Adjacent Buildings Data on hazards posed to the subject building by adjacent buildings and their elements shall be collected to permit consideration of their potential to damage the subject building as a result of an earthquake. If there is a potential for such hazards from an adjacent building, the Authority Having Jurisdiction over the subject building shall be informed of the effect of such hazards on achieving the selected Performance Objective.

3.3 COMMON BUILDING TYPES

The Common Building Types defined in Table 3-1 shall be used to determine eligibility for the Benchmark Building provisions in Section 3.4 and the Tier 1 and Tier 2 procedures in Section 3.5. A building is permitted to be classified as separate Common

Table 3-1. Common Building Types.

Wood Light Frames, Small Residential W1	These buildings are detached one- or two-family dwellings one to three stories high with plan areas on each level less than or equal to 3,000 ft^2 (280 m^2) and a total plan area less than or equal to 6,000 ft^2 (560 m^2). Floor and roof framing consists of wood joists or rafters on wood studs spaced no more than 24 in. (61 cm) apart or wood post-and-beam construction. The first-floor framing is supported directly on an at-grade foundation or slab-on-grade or directly on concrete or masonry basement walls or is raised up on cripple studs and post-and-beam supports. Seismic forces are resisted by wood framed and sheathed diaphragms and shear walls. Floor and roof diaphragms consist of straight or diagonal lumber sheathing, tongue-and-groove planks, oriented strand board, plywood, or other materials. Shear walls consist of straight or diagonal wood sheathing, plank siding, oriented strand board, plywood, stucco, gypsum board, particleboard, fiberboard, or similarly performing materials.
Wood Frames, Large Residential, Commercial, Industrial, and Institutional W2	These buildings are one- and two-family dwellings that exceed the criteria for W1 buildings; multiunit residential buildings or commercial, industrial, or institutional buildings. Elevated floor and roof framing consists of wood or steel trusses, glulam or steel beams, and wood posts or steel columns. Ground or basement floors generally consist of concrete slab-on-grade. Seismic forces are resisted by flexible diaphragms and exterior walls sheathed with plywood, oriented strand board, stucco, plaster, or straight or diagonal wood sheathing; or walls are braced with various forms of wood bracing, such as knee-braced or cantilevered columns. Bracing with materials other than wood is considered a mixed system and is subject to the requirements in Section 3.5.1.2.2. Wall openings for storefronts and garages, where present, are framed by post-and-beam framing. In some cases, these building may be located over a podium level structure with concrete or masonry shear walls and can be evaluated as a mixed system subject to the requirements in Section 3.5.1.2.2.2.
Steel Moment Frames S1 (with Stiff Diaphragms)	These buildings consist of a frame assembly of steel beams, joists, open web joists, and/or trusses, and steel columns. Floor and roof diaphragms consist of cast-in-place concrete slabs or steel deck with reinforced structural concrete fill supported on the steel framing and are stiff relative to the moment frames. Seismic forces are resisted by steel moment frames that develop their stiffness through fully restrained or partially restrained beam–column connections.
S1a (with Flexible Diaphragms)	These buildings are similar to S1 buildings, except that diaphragms are bare steel deck or steel deck with fill other than reinforced structural concrete and are flexible relative to the frames.
Steel Braced Frames S2 (with Stiff Diaphragms)	These buildings consist of a frame assembly of steel beams, joists, open-web joists, and/or trusses, and steel columns. Floor and roof diaphragms consist of cast-in-place concrete slabs or steel deck with reinforced structural concrete fill supported on the steel framing and are stiff relative to the braced frames. Seismic forces are resisted by steel braced frames that develop their stiffness through bracing action of the diagonal members resisting axial loads. Three variations in the configuration and design of braced frames exist. These variations are as follows: • Concentrically braced frames: Component work lines intersect at a single point or at multiple points such that the distance between intersecting work lines (or eccentricity) is less than or equal to the width of the smallest component connected at the joint. • Eccentrically braced frames: Component work lines do not intersect at a single point, and the distance between the intersecting work lines (or eccentricity) exceeds the width of the smallest component connecting at the joint. Some of the members are subjected to shear and flexural stresses because of that eccentricity. • Buckling-restrained braced frames: Special types of concentrically braced frames where the steel bracing members are encased within a rigid casing that is intended to prevent buckling of the steel brace.

continues

Table 3-1 ***(Continued)*****. Common Building Types.**

S2a (with Flexible Diaphragms)	These buildings are similar to S2 buildings, except that diaphragms consist of wood or cold-formed steel framing, bare steel deck, or steel deck with fill other than reinforced structural concrete, and they are flexible relative to the braced frames.
Metal Building Frames S3	These buildings use transverse steel moment frames and sometimes contain wall panel shear elements or braced frames at the ends of the building. Lateral forces in the longitudinal direction typically rely on wall panel shear elements or rod bracing. The buildings are one story high, but they sometimes have mezzanines. The transverse moment frames typically consist of beams and columns that are either web-tapered or prismatic built-up sections with thin plates. The frames are built in segments and assembled in the field with bolted or welded joints. The roof and walls consist of lightweight metal, fiberglass, or cementitious panels. Diaphragm forces are resisted by bare steel deck, roof panel shear elements, or a system of tension-only rod bracing located in the plane of the roof framing.
Dual Frame Systems with Backup Steel Moment Frames and Stiff Diaphragms S4	These buildings consist of a gravity frame assembly of steel beams, joists, open-web joists, and/or trusses, and steel columns. The floor and roof diaphragms consist of cast-in-place concrete slabs or steel deck with reinforced structural concrete fill and are stiff relative to the vertical elements of the lateral system. Seismic forces are resisted primarily by either steel braced frames or cast-in-place concrete shear walls in combination with backup steel moment frames. The steel moment frames interact with the steel braced frames or concrete shear walls and resist seismic forces in proportion to their relative rigidity.
Steel Frames with Infill Masonry Shear Walls S5 (with Stiff Diaphragms)	These buildings consist of a gravity frame assembly of steel beams, joists, open-web joists, and/or trusses, and steel columns. The floor and roof diaphragms consist of cast-in-place concrete slabs or steel deck with reinforced structural concrete fill and are stiff relative to the walls. Walls consist of solid or perforated infill panels constructed of solid clay brick, concrete block, or hollow clay tile masonry which are in-plane with and infill within the structural frames.
S5a (with Flexible Diaphragms)	These buildings are similar to S5 buildings, except that diaphragms consist of wood sheathing or bare steel deck, or steel deck with fill other than reinforced structural concrete and are flexible relative to the walls.
Steel Plate Shear Walls S6	These buildings consist of a gravity frame assembly of steel beams, joists, open-web joists, and/or trusses, and steel columns. Floor and roof diaphragms consist of cast-in-place concrete slabs or steel deck with reinforced structural concrete fill supported on the steel framing and are stiff relative to the shear walls. Shear walls are constructed with steel plates with horizontal and vertical boundary elements adjacent to the webs.
Cold-Formed Steel Light-Frame Construction CFS1 (Shear Wall System)	These buildings have cold-formed steel light-frame walls supporting the majority of the lateral loads. Floor and roof framing consists of cold-formed steel joists or rafters on cold-formed steel studs spaced no more than 24 in. (61 cm) apart, wood or cold-formed steel trusses, structural steel or cold-formed steel beams, and structural steel or cold-formed steel columns. Seismic forces are resisted by wood structural panel or bare steel deck diaphragms, and wood structural panel sheathed shear walls or steel sheet sheathed shear walls. Cold-formed steel light-frame buildings that have precast concrete plank diaphragms shall not be permitted to be classified as this common building type.
Cold-Formed Steel Light-Frame Construction CFS2 (Strap-Braced Wall System)	These buildings have cold-formed steel light-frame strap walls supporting the majority of the lateral loads. Floor and roof framing consists of cold-formed steel joists or rafters on cold-formed steel studs spaced no more than 24 in. (61 cm) apart, wood or cold-formed steel trusses, structural steel or cold-formed steel beams, and structural steel or cold-formed steel columns. Seismic forces are resisted by diaphragms with wood structural panels or bare steel deck, and steel light-frame stud walls with diagonal flat strap bracing. Cold-formed steel light-frame buildings that have precast concrete plank diaphragms shall not be permitted to be classified as this common building type.
Concrete Moment Frames C1	These buildings consist of a frame assembly of cast-in-place reinforced concrete beams and columns. Floor and roof framing consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Seismic forces are resisted by concrete moment frames that develop their stiffness through monolithic beam–column connections. In some conditions the moment frames consist of slab-column frames in two-way flat slab systems.
Concrete Shear Walls C2 (with Stiff Diaphragms)	These buildings have floor and roof framing that consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Buildings may also have floor and roof framing consisting of steel beams, joists, open-web joists, trusses, and/or cold-formed steel light-frame construction that support diaphragms consisting of steel deck with reinforced structural concrete fill. Floor and roof framing is supported on concrete or steel columns and/or concrete bearing walls. Seismic forces are resisted by cast-in-place concrete shear walls.
C2a (with Flexible Diaphragms)	These buildings are similar to C2 buildings, except that diaphragms consist of wood sheathing or bare steel decking and are flexible relative to the walls.

continues

Table 3-1 ***(Continued)*****. Common Building Types.**

Concrete Frames with Infill Masonry Shear Walls C3 (with Stiff Diaphragms)	These buildings consist of a gravity frame assembly of cast-in-place concrete beams and columns. The floor and roof diaphragms consist of cast-in-place concrete slabs with concret joists and beams and are stiff relative to the walls. Walls consist of solid or perforated infill panels constructed of solid clay brick, concrete block, or hollow clay tile masonry which are in-plane with and infill within the structural frames.
C3a (with Flexible Diaphragms)	These buildings are similar to C3 buildings, except that diaphragms consist of wood sheathing or bare steel deck or steel deck with fill other than reinforced structural concrete and are flexible relative to the walls.
Precast or Tilt-Up Concrete Shear Walls PC1 (with Flexible Diaphragms)	These buildings have precast concrete perimeter wall panels and, in some conditions, interior walls, that are typically cast on site and tilted into place. The panels are interconnected by weldments, cast-in-place concrete pilasters, or collector elements. Floor and roof framing consists of wood purlins, joists, and girders; open-web wood or steel joists; or steel beams, girders, and/or trusses. Framing is supported on interior steel or wood columns and perimeter concrete bearing walls. Seismic forces are resisted by precast concrete shear walls. Diaphragms consist of wood sheathing, bare steel deck, or steel deck with fill other than reinforced structural concrete and are flexible relative to the walls.
PC1a (with Stiff Diaphragms)	These buildings are similar to PC1 buildings, except that diaphragms consist of precast elements, cast-in-place concrete, or steel deck with reinforced structural concrete fill and are stiff relative to the walls.
Precast Concrete Frames PC2 (with Shear Walls)	These buildings consist of a frame assembly of precast concrete beams, girders, and columns with the presence of concrete shear walls. Floor and roof framing consists of cast-in-place concrete slabs, precast concrete planks, tees, or double-tees supported on precast concrete girders, some or all of which could be pre- or post-tensioned. Seismic forces are resisted by precast or cast-in-place concrete shear walls, which also support gravity loads. Diaphragms consist of precast elements interconnected with welded inserts, cast-in-place closure strips, or reinforced concrete slabs or topping slabs.
PC2a (without Shear Walls)	These buildings are similar to PC2 buildings, except that concrete shear walls are not present. Seismic forces are resisted by precast concrete moment frames that develop their stiffness through beam–column joints rigidly connected by welded inserts or cast-in-place concrete closures. Diaphragms consist of precast elements interconnected with welded inserts, cast-in-place closure strips, or reinforced concrete slabs or topping slabs.
Reinforced Masonry Bearing Walls RM1 (with Flexible Diaphragms)	These buildings have bearing walls that consist of reinforced brick or concrete block masonry. Floor and roof framing consists of wood purlins, joists, and girders; open-web wood or steel joists; or steel beams, girders, and/or trusses. Framing is supported by reinforced masonry bearing walls, wood stud walls, cold-formed steel light-frame construction, or by steel, wood or masonry columns. Seismic forces are resisted by reinforced masonry shear walls. Diaphragms consist of wood sheathing, bare steel deck, or steel deck with fill other than reinforced structural concrete and are flexible relative to the walls.
Reinforced Masonry Bearing Walls RM2 (with Stiff Diaphragms)	These building are similar to RM1 buildings, except that the diaphragms consist of steel deck with reinforced structural concrete fill, precast concrete planks, tees, or double-tees, with or without a cast-in-place concrete topping slab, and are stiff relative to the walls. The floor and roof framing is supported on interior steel or concrete frames or interior reinforced masonry walls.
Unreinforced Masonry Bearing Walls URM (with Flexible Diaphragms)	These buildings have perimeter bearing walls that consist of unreinforced clay brick, stone, or concrete masonry. Interior bearing walls, where present, also consist of unreinforced clay brick, stone, or concrete masonry. Floor and roof framing consists of wood joists and beams, which are supported by wood, steel, or cast iron columns. Seismic forces are resisted by unreinforced masonry shear walls. The diaphragms consist of wood sheathing and are flexible relative to the masonry shear walls. Where they exist, ties between the walls and diaphragms consist of anchors or bent steel plates embedded in the mortar joints and attached to framing. Previously retrofitted buildings have wall anchors that consist of post-installed adhesive anchors or post-installed thru-bolts. Buildings with bearing and/or shear walls comprised of adobe shall not be permitted to be classified as this common building type.
URMa (with Stiff Diaphragms)	These buildings are similar to URM buildings, except that the diaphragms are stiff relative to the unreinforced masonry walls. Floor and roof framing consists of cast-in-place concrete slabs supported by concrete or concrete encased steel beams and columns; arched or flat brick or tile floors, with or without concrete topping slabs; or steel deck with reinforced structural concrete fill on steel framing and are stiff relative to the masonry shear walls. Buildings with bearing and/or shear walls comprised of adobe shall not be permitted to be classified as this common building type.

Building Types for each principal direction if the building has different seismic-force-resisting systems in different directions. If either principal direction does not conform with one of the Common Building Type definitions in Table 3-1, the building shall not be considered a Common Building Type when determining applicability of provisions in Sections 3.4 and 3.5.

3.4 BENCHMARK BUILDINGS

Buildings designed and constructed or evaluated in accordance with the benchmark provisions of this section shall be deemed to comply with the provisions of this standard for the Basic Performance Objective for Existing Buildings (BPOE). An

Table 3-2. BPOE Benchmark Building Codes and Standards for Risk Categories I and II.

	Building Seismic Design Provisions				Seismic Evaluation or Retrofit Provisions	
Building Type[a,b]	**NBC/SBC**	**UBC**	**IBC**	**NEHRP**	**FEMA 310[d]/ ASCE/SEI 31[d]**	**FEMA 356[e]/ ASCE/SEI 41[e]**
Wood Light Frames, Small Residential (Type W1)[g]	1993	1976	2000	1985	1998	2000
Wood Light Frames, Large Residential, Commercial, Industrial, and Institutional (Type W2)	*f*	1997	2000	1997	1998	2000
Steel Moment Frames (Types S1 and S1a)	*f*	1997	2000	1997	1998	2000
Steel Concentrically Braced Frames (Types S2 and S2a)	*f*	1997	2000	*f*	1998	2000
Steel Eccentrically Braced Frames (Types S2 and S2a)	*f*	1997	2000	1997	*f*	2000
Steel Buckling-Restrained Braced Frames (Types S2 and S2a)	*f*	*f*	2006	*f*	*f*	2000
Metal Building Frames (Type S3)	*f*	*f*	2000	*f*	1998	2000
Dual Frame Systems with Concrete Shear Walls and Backup Steel Moment Frames (Type S4)	1999	1997	2000	1997	1998	2000
Dual Frame Systems with Steel Braced Frames and Backup Steel Moment Frames (Type S4)	1999	1997	2000	1997	1998	2000
Steel Frames with Infill Masonry Shear Walls (Types S5 and S5a)	*f*	*f*	2000	*f*	1998	2000
Steel Plate Shear Walls (Type S6)	*f*	*f*	2006	*f*	*f*	2000
Cold-Formed Steel Light-Frame Construction: Shear Wall System (Type CFS1)	*f*	1997[h]	2000	1997[h]	*f*	2000[h]
Cold-Formed Steel Light-Frame Construction: Strap-Braced Wall System (Type CFS2)	*f*	*f*	2003	2003	*f*	*f*
Concrete Moment Frames (Type C1)	1999	1997	2000	1997	1998	2000
Concrete Shear Walls (Types C2 and C2a)	1999	1997	2000	1997	1998	2000
Concrete Frames with Infill Masonry Shear Walls (Types C3 and C3a)	*f*	*f*	2000	*f*	1998	2000
Precast or Tilt-Up Concrete Shear Walls (Types PC1 and PC1a)	*f*	1997	2000	*f*	1998	2000
Precast Concrete Frames (Types PC2 and PC2a)	*f*	*f*	2000	*f*	1998	2000
Reinforced Masonry Bearing Walls with Flexible Diaphragms (Type RM1)	*f*	1997	2000	*f*	1998	2000
Reinforced Masonry Baering Walls with Stiff Diaphragms (Type RM2)	1997	1997	2000	1997	1998	2000
Unreinforced Masonry Bearing Walls with Flexible Diaphragms (Type URM)	*f*	*f*	2000	*f*	*f*	2000
Unreinforced Masonry Bearing Walls with Stiff Diaphragms (Type URMa)	*f*	*f*	2000	*f*	1998	2000
Seismic Isolation or Supplemental Energy Dissipation[c]	*f*	1991	2000	*f*	*f*	2000

[a] Building type refers to one of the Common Building Types defined in Table 3-1.

[b] For buildings in areas defined as Very Low Seismicity, the benchmark provisions are limited to the IBC, FEMA 310/ASCE/SEI 31, and FEMA 356/ASCE/SEI 41.

[c] Applies to buildings with seismic isolation or supplemental energy dissipation systems that comply with the cited reference codes and standards.

[d] Life Safety Structural Performance Level for the seismic hazard as defined by those provisions.

[e] Life Safety Structural Performance Level for the BSE-1 Seismic Hazard Level as defined by those provisions.

[f] No benchmark year; buildings must be evaluated using this standard.

[g] W1 buildings located on hillside sites as defined by Table 17-4 cannot be considered Benchmark Buildings.

[h] Only cold-formed steel light-frame buildings with wood structural panel shear walls are permitted to be considered Benchmark Buildings.

Source: NBC = *National Building Code* (BOCA 1993, 1996, 1999); SBC = *Standard Building Code* (SBCC 1993, 1994, 1996, 1997, 1998, 1999); UBC = *Uniform Building Code* (ICBO 1976, 1979, 1982, 1985, 1988, 1991, 1994, 1997); IBC = *International Building Code* (ICC 2000, 2003, 2006, 2009, 2012, 2015, 2018, 2021); NEHRP = *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings* FEMA 95 (BSSC 1985), FEMA 95 (BSSC 1988), FEMA 222 (BSSC 1992), FEMA 222A (BSSC 1995), FEMA 302 (BSSC 1997), FEMA 368 (BSSC 2001). FEMA 310 (1998), ASCE/SEI 31-03 (2003), FEMA 356 (2000), ASCE/SEI 41-06 (2007), ASCE/SEI 41-13 (2014), and ASCE/SEI 41-17 (2017).

Table 3-3. BPOE Benchmark Building Codes and Standards for Risk Category III.

Building Type[a]	Building Seismic Design Provisions	Seismic Evaluation or Retrofit Provisions
	IBC[b]	FEMA 356[c]/ASCE/SEI 41[c]
Wood Light Frames, Small Residential (Type W1)	2000	2000
Wood Frames, Large Residential, Commercial, Industrial, and Institutional (Type W2)	2000	2000
Steel Moment Frames (Types S1 and S1a)	2000	2000
Steel Concentrically Braced Frames (Types S2 and S2a)	2000	2000
Steel Eccentrically Braced Frames (Types S2 and S2a)	2000	2000
Steel Buckling-Restrained Braced Frames (Types S2 and S2a)	2006	2000
Metal Building Frames (Type S3)	2000	2000
Dual Frame Systems with Concrete Shear Walls and Backup Steel Moment Frames (Type S4)	2000	2000
Dual Frame Systems with Steel Braced Frames and Backup Steel Moment Frames (Type S4)	2000	2000
Steel Frames with Infill Masonry Shear Walls (Types S5 and S5a)	*d*	2000
Steel Plate Shear Walls (Type S6)	2006	2000
Cold-Formed Steel Light-Frame Construction: Shear Wall System (Type CFS1)	2000	2000[e]
Cold-Formed Steel Light-Frame Construction: Strap-Braced Wall System (Type CFS2)	2003	*d*
Concrete Moment Frames (Type C1)	2000	2000
Concrete Shear Walls (Types C2 and C2a)	2000	2000
Concrete Frames with Infill Masonry Shear Walls (Types C3 and C3a)	2000	2000
Precast or Tilt-Up Concrete Shear Walls (Types PC1 and PC1a)	2000	2000
Precast Concrete Frames (Types PC2 and PC2a)	2000	2000
Reinforced Masonry Bearing Walls with Flexible Diaphragms (Type RM1)	2000	2000
Reinforced Masonry Baering Walls with Stiff Diaphragms (Type RM2)	2000	2000
Unreinforced Masonry Bearing Walls with Flexible Diaphragms (Type URM)	2000	2000
Unreinforced Masonry Bearing Walls with Stiff Diaphragms (Type URMa)	2000	2000
Seismic Isolation or Supplemental Energy Dissipation[f]	2000	2000

[a] Building type refers to one of the common building type Common Building Types defined in Table 3-1.

[b] Complying with the requirements for Occupancy Category III or Risk Category III as defined by that code.

[c] Damage Control Structural Performance Level for the BSE-1 Seismic Hazard Level as defined by those provisions.

[d] No benchmark year; buildings must be evaluation using this standard.

[e] Only cold-formed steel light-frame buildings with wood structural panel shear walls are permitted to be considered Benchmark Buildings.

[f] Applies to buildings with seismic isolation or supplemental energy dissipation systems that comply with the cited reference codes and standards.

Source: IBC = International Building Code (ICC 2000, 2003, 2006, 2009, 2012, 2015, 2018, 2021); FEMA 356 (2000), ASCE/SEI 41-06 (2007), ASCE/SEI 41-13 (2014), and ASCE/SEI 41-17 (2017).

evaluation of nonstructural elements in accordance with Section 17.19, shall be performed where required by this standard.

Compliance with this section shall consider the provisions under which the structure was originally designed, retrofitted, or previously evaluated. Buildings that have been retrofitted to meet an approved standard shall be evaluated using the standards used for the retrofit, not the original design provisions. The edition of a design code or provisions or the retrofit standard that sets the benchmark year shall be as indicated in Table 3-2 for buildings assigned to Risk Categories I and II, Table 3-3 for buildings assigned to Risk Category III, and Table 3-4 for buildings assigned to Risk Category IV.

3.4.1 Benchmark Procedure Checklist Where Table 3-2, Table 3-3, or Table 3-4 is used to establish compliance, the design professional shall complete the checklist in Table 3-5. A building shall be deemed to comply in accordance with Table 3-2, Table 3-3, or Table 3-4 only if each item in Table 3-5 is marked Compliant or Not Applicable.

3.4.2 Parameters for Benchmark Procedure

3.4.2.1 Level of Seismicity The current Level of Seismicity for the building site shall be determined in accordance with Section 2.5. The Level of Seismicity represented by the original design code or standard is the comparable Level of Seismicity corresponding to the seismic detailing requirements for which the building was designed or evaluated for purposes of the benchmark procedure. The original Level of Seismicity shall be determined from Table 3-6 based on the building code or standard being used to deem compliance with the benchmark procedure.

3.4.2.2 Seismic Force Provisions Where required by Table 3-5, the building site's current seismic response parameter shall be taken as S_{XS} for the BSE-1N. The original seismic response

Table 3-4. BPOE Benchmark Building Codes and Standards for Risk Category IV.

	Seismic Evaluation or Retrofit Provisions	
Building Type[a]	**FEMA 310[c]/ASCE/SEI 31[c]**	**FEMA 356[d]/ASCE/SEI 41[d]**
Wood Light Frames, Small Residential (Type W1)	1998	2000
Wood Frames, Large Residential, Commercial, Industrial, and Institutional (Type W2)	1998	2000
Steel Moment Frames (Types S1 and S1a)	1998	2000
Steel Concentrically Braced Frames (Types S2 and S2a)	1998	2000
Steel Eccentrically Braced Frames (Types S2 and S2a)	[e]	2000
Steel Buckling-Restrained Braced Frames (Types S2 and S2a)	[e]	2000
Metal Building Frames (Type S3)	1998	2000
Dual Frame Systems with Concrete Shear Walls and Backup Steel Moment Frames (Type S4)	1998	2000
Dual Frame Systems with Steel Braced Frames and Backup Steel Moment Frames (Type S4)	1998	2000
Steel Frames with Infill Masonry Shear Walls (Types S5 and S5a)	1998	2000
Steel Plate Shear Walls (Type S6)	[e]	2000
Cold-Formed Steel Light-Frame Construction: Shear Wall System (Type CFS1)	[e]	[e]
Cold-Formed Steel Light-Frame Construction: Strap-Braced Wall System (Type CFS2)	[e]	[e]
Concrete Moment Frames (Type C1)	1998	2000
Concrete Shear Walls (Types C2 and C2a)	1998	2000
Concrete Frames with Infill Masonry Shear Walls (Types C3 and C3a)	1998	2000
Precast or Tilt-Up Concrete Shear Walls (Types PC1 and PC1a)	1998	2000
Precast Concrete Frames (Types PC2 and PC2a)	1998	2000
Reinforced Masonry Bearing Walls with Flexible Diaphragms (Type RM1)	1998	2000
Reinforced Masonry Bearing Walls with Stiff Diaphragms (Type RM2)	1998	2000
Unreinforced Masonry Bearing Walls with Flexible Diaphragms (Type URM)	[e]	2000
Unreinforced Masonry Bearing Walls with Stiff Diaphragms (Type URMa)	1998	2000
Seismic Isolation or Supplemental Energy Dissipation[b]	[e]	2000

[a] Building type refers to one of the Common Building Types defined in Table 3-1.

[b] Applies to buildings with seismic isolation or supplemental energy dissipation systems that comply with the cited reference standards.

[c] Immediate Occupancy Structural Performance Level for the Seismic Hazard Level as defined by those provisions.

[d] Immediate Occupancy Structural Performance Level for the BSE-1 seismic hazard as defined by those provisions.

[e] No benchmark year; buildings must be evaluated using this standard.

Source: FEMA 310 (1998), ASCE/SEI 31-03 (2003), FEMA 356 (2000), ASCE/SEI 41-06 (2007), ASCE/SEI 41-13 (2014), and ASCE/SEI 41-17 (2017).

parameter for the building's site shall be determined from Table 3-7 based on the building code or standard being used to deem compliance with the benchmark provisions.

3.5 EVALUATION AND RETROFIT PROCEDURES

Seismic evaluation or retrofit of the building shall be performed to demonstrate compliance with the selected Performance Objective in accordance with the requirements of the following sections. Section 3.5.1 covers the limitations on the use of the Tier 1 and Tier 2 procedures. Section 3.5.2 addresses the Tier 1 Screening procedure for evaluation. Section 3.5.3 addresses the Tier 2 Deficiency-Based procedures for evaluation and retrofit. Section 3.5.4 addresses the Tier 3 systematic procedures for evaluation and retrofit.

3.5.1 Limitations on the Use of Tier 1 and Tier 2 Evaluation and Retrofit Procedures. The Tier 1 Screening and Tier 2 deficiency-based procedures shall only be used with a Performance Objective that satisfies at least one of the following conditions:

1. The Performance Objective involves a Seismic Hazard Level less than or equal to BSE-1E with a Structural Performance Level up to and including Immediate Occupancy (S-1) and/or a Nonstructural Performance Level up to and including Position Retention (N-B), and
2. The Performance Objective involves a Seismic Hazard Level greater than BSE-1E but less than or equal to BSE-2E with a Structural Performance Level up to and including Life Safety (S-3) and/or a Nonstructural Performance Level up to and including Life Safety (N-C).

The selected Seismic Hazard Level shall be compared to BSE-1E or BSE-2E by comparing the respective values of S_S and S_1.

In addition, the Tier 1 and Tier 2 procedures shall only be used for buildings that conform to the limitations of Table 3-8 and of Section 3.5.1.1 or 3.5.1.2.

Table 3-5. Benchmark Procedure Checklist.

Status	Benchmarking Statement
C NC U	EXISTING DOCUMENTS: Record drawings of the structure confirm that the primary elements of the seismic-force-resisting system and their detailing were intended to be designed in accordance with the applicable provisions listed in Table 3-2, Table 3-3, or Table 3-4.
C NC U	FIELD VERIFICATION: Field verification confirms the building was constructed in general conformance with record drawings and that no modifications have been made that significantly affect the expected performance of the seismic-force-resisting system.
C NC U	CONDITION ASSESSMENT: Field verification confirms that significant deterioration of structural materials or building settlement is not present.
C NC U	GEOLOGIC SITE HAZARDS: No liquefaction, slope failure, or surface fault rupture hazard exists at the site. Alternately, if such hazard is present, the hazard has been mitigated by the design of the seismic-force-resisting system, including foundations.
C NC U N/A	LEVEL OF SEISMICITY: The building site Level of Seismicity in accordance with this standard is Low, Moderate, or High and is not higher than the comparable Level of Seismicity represented by the original design code or standard, as determined in accordance with Section 3.4.2.1.
C NC U N/A	SEISMIC FORCE PROVISIONS: For buildings located in areas where the Level of Seismicity is defined as High in accordance with Section 2.5, the current seismic response parameter determined in accordance with Section 3.4.2.2 is not more than 1.5 times the original seismic response parameter, determined in accordance with Section 3.4.2.2.

Note: C = Compliant, NC = Noncompliant, U = Unknown, and N/A = Not Applicable.

Table 3-6. Original Level of Seismicity for Benchmark Procedure.

Benchmark Building Code or Standard and Seismicity *Parameters*[a]	Seismic Parameter[b]	Original Level of Seismicity for Use with Benchmark Procedure
ASCE 41, ASCE 31, FEMA 356, FEMA 310	Level of Seismicity = High	High
ASCE 7, IBC, NEHRP	Seismic Design Category = D, E, or F	
UBC	Seismic Zone = 2b, 3 or 4	
FEMA 178, 1985 NEHRP, NBC, or SBC	A_a, $A_v \geq 0.2$	
ASCE 41, ASCE 31, FEMA 356, FEMA 310	Level of Seismicity = Moderate	Moderate
ASCE 7, IBC, NEHRP	Seismic Design Category = C	
UBC	Seismic Zone = 2a	
FEMA 178, 1985 NEHRP, NBC, or SBC	A_a, $A_v = 0.15$	
ASCE 41, ASCE 31, FEMA 356, FEMA 310	Level of Seismicity = Low	Low
ASCE 7, IBC, NEHRP	Seismic Design Category = B	
UBC	Seismic Zone = 1	
FEMA 178, 1985 NEHRP, NBC, or SBC	A_a, $A_v = 0.05$ or 0.1	
ASCE 41, ASCE 31, FEMA 356, FEMA 310	Level of Seismicity = Very Low	Very Low
ASCE 7, IBC, NEHRP	Seismic Design Category = A	
UBC	Seismic Zone = 0	
FEMA 178, 1985 NEHRP, NBC, or SBC	A_a, $A_v = 0$	

[a] Refer to Tables 3-2, 3-3, and 3-4 for valid years of Benchmark Building or standard.
[b] The higher of A_a and A_v shall be used to determine the original Level of Seismicity.

3.5.1.1 Buildings Conforming to One of the Common Building Types Where a building conforms to one of the Common Building Types contained in Table 3-1, the limitations in Table 3-8 with regard to building size, Structural Performance Level, and Level of Seismicity determine whether the Tier 1 Screening and Tier 2 Deficiency-Based Procedures are allowed to demonstrate compliance with the Performance Objectives of this standard.

3.5.1.2 Buildings Composed of More than One of the Common Building Types The limitations in this section apply to mixed seismic-force-resisting systems defined as combinations of the Common Building Types in either the same or different directions. In all cases, each individual seismic-force-resisting system, as defined in the following sections, must conform to one of the Common Building Types. The Tier 1 and Tier 2 procedures are not permitted to demonstrate compliance with the Performance

Table 3-7. Original Seismic Response Parameter for Benchmark Procedure.

Benchmark Building Code or Standard	Original Seismic Response Parameter
ASCE 41-13 or ASCE 41-17	S_{XS} for BSE-1N
FEMA 356 or ASCE 41-06	S_{XS} for BSE-1
ASCE 31, FEMA 310, ASCE 7, IBC, or 1997 NEHRP	S_{DS}
1997 UBC	$2.5C_a$
1988–1994 UBC	$2.75Z$
1976–1985 UBC	$1.1Z$
FEMA 178, 1985 NEHRP, NBC, or SBC	$2.5A_a$

Table 3-8. Limitations on the Use of the Tier 1 and Tier 2 Procedures.

	Number of Stories[b] beyond which the Tier 3 Systematic Procedures Are Required							
	Level of Seismicity							
	Very Low		Low		Moderate		High	
Common Building Type[a]	**S-5**	**S-1**	**S-5**	**S-1**	**S-5**	**S-1**	**S-5**	**S-1**
Wood Frames								
Light Frames, Small Residential (W1)	NL	NL	NL	4	4	4	4	4
Large residential, commercial, industrial, and institutional (W2)	NL	NL	NL	6	6	6	6	4
Steel Moment Frames								
Stiff diaphragm (S1)	NL	NL	NL	12	12	8	8	6
Flexible diaphragm (S1a)	NL	NL	NL	12	12	8	8	6
Steel Braced Frames								
Stiff diaphragm (S2)	NL	NL	NL	8	8	8	8	6
Flexible diaphragm (S2a)	NL	NL	NL	8	8	8	8	6
Metal Building Frames (S3)	NL	1	1	1	1	1	1	1
Dual Systems with Backup Steel Moment Frames (S4)	NL	NL	NL	12	12	8	8	6
Steel Frames with Infill Masonry Shear Walls								
Stiff diaphragm (S5)	NL	NL	NL	12	12	8	8	4
Flexible diaphragm (S5a)	NL	NL	NL	12	12	8	8	4
Steel Plate Shear Wall (S6)	NP[c]	NP[c]	NP[c]	NP[c]	NP[c]	NP[c]	NP[c]	NP[c]
Cold-Formed Steel Light-Frame Construction								
Shear wall system (CFS1)	NL	NL	NL	6	6	6	6	4
Strap-braced wall system (CFS2)	NL	NL	NL	6	6	6	6	4
Concrete Moment Frames (C1)	NL	NL	NL	12	12	8	8	6
Concrete Shear Walls								
Stiff diaphragm (C2)	NL	NL	NL	12	12	8	8	6
Flexible diaphragm (C2a)	NL	NL	NL	12	12	8	8	6
Concrete Frame with Infill Masonry Shear Walls								
Stiff diaphragm (C3)	NL	NL	NL	12	12	8	8	4
Flexible diaphragm (C3a)	NL	NL	NL	12	12	8	8	4
Precast or Tilt-Up Concrete Shear Walls								
Flexible diaphragm (PC1)	NL	NL	3	2	2	2	2	2
Stiff diaphragm (PC1a)	NL	NL	3	2	2	2	2	2
Precast Concrete Frames								
With shear walls (PC2)	NL	NL	NL	6	6	NP	4	NP
Without shear walls (PC2a)	NL	NL	NL	6	6	NP	4	NP
Reinforced Masonry Bearing Walls								
Flexible diaphragm (RM1)	NL	NL	NL	8	8	8	8	6
Stiff diaphragm (RM2)	NL	NL	NL	8	8	8	8	6

continues

Table 3-8 *(Continued)*. Limitations on the Use of the Tier 1 and Tier 2 Procedures.

Common Building Type[a]	Very Low S-5	Very Low S-1	Low S-5	Low S-1	Moderate S-5	Moderate S-1	High S-5	High S-1
	Number of Stories[b] beyond which the Tier 3 Systematic Procedures Are Required — Level of Seismicity							
Unreinforced Masonry Bearing Walls								
Flexible diaphragm (URM)	NL	NL	6	4	6	NP	4	NP
Stiff diaphragm (URMa)	NL	NL	6	4	6	NP	4	NP
Seismic Isolation or Passive Dissipation	NP[c]	NP[c]	NP[c]	NP[c]	NP[c]	NP[c]	NP[c]	NP[c]

[a] Common Building Types are defined in Section 3.3.

[b] Number of stories shall be considered as the number of stories above lowest adjacent grade.

[c] No deficiency-based procedures exist for these building types. If they do not meet the Benchmark Building requirements, Tier 3 systematic procedures are required.

NL = No Limit (No limit on the number of stories).

NP = Not Permitted (Tier 3 systematic procedures are required).

Objectives of this standard for mixed systems except as indicated in the following sections.

3.5.1.2.1 Combinations of Systems in Different Directions It is acceptable to use the Tier 1 and Tier 2 procedures to demonstrate compliance with a Performance Objective for a building with a different seismic-force-resisting system in each principal direction provided the seismic-force-resisting systems in both directions conform to a common building type in Table 3-1 and the building satisfies the height limits in Table 3-8 for the system with the lesser of the allowed height limits in both directions.

3.5.1.2.2 Combinations of Systems in the Same Direction It is acceptable to use Tier 1 and Tier 2 procedures to demonstrate compliance with a Performance Objective for a building with a combination of different seismic-force-resisting systems in a single principal direction subject to the requirements of Sections 3.5.1.2.2.1 for horizontal combinations, 3.5.1.2.2.2 for vertical combinations, and 3.5.1.2.2.3 for combinations of stiff and flexible diaphragms. Otherwise, the Tier 3 procedures shall be used for such evaluations and retrofit.

Alternatively, the Tier 1 and Tier 2 procedures shall be permitted to demonstrate compliance for a building with more than one type of seismic-force-resisting system along a single axis of the building, including changes over the height of the building, if the building is being evaluated for performance that does not exceed the Life Safety Performance Level and all statements in the Basic Configuration Checklist of Section 17.1.2 are found to be "Compliant."

3.5.1.2.2.1 Horizontal Combinations The Tier 1 and Tier 2 procedures shall be permitted for a building with a horizontal combination of two seismic-force-resisting systems in the same direction, provided that the following criteria are satisfied:

- The Performance Level does not exceed Life Safety (S-3) Performance Level;
- The building possesses seismic-force-resisting systems conforming to one or two of the common building types in Table 3-1 in each principal direction;
- Each line of resistance in each direction conforms to one of the common building types in Table 3-1;
- The building has flexible diaphragms at all levels above the base of the structure;
- The building height complies with the lowest height limit in Table 3-8 for any system in the direction under consideration; and
- Where the Tier 1 checklists require the use of the Quick Check procedures in Section 4.4, seismic forces are distributed to the vertical elements of the seismic-force-resisting system based on tributary areas.

3.5.1.2.2.2 Vertical Combinations The Tier 1 and Tier 2 procedures shall be permitted for a building with a vertical combination of two seismic-force-resisting systems in the same direction, provided that the following criteria are satisfied:

- The Performance Level does not exceed the Life Safety (S-3) Performance Level,
- Each story consists of a seismic-force-resisting system conforming to one of the common building types in Table 3-1, and
- The total building height complies with the lowest height limit in Table 3-8 for any system in the direction under consideration.

3.5.1.2.2.3 Combinations of Stiff and Flexible Diaphragms The Tier 1 and Tier 2 procedures shall be permitted for a building with a seismic-force-resisting system with a stiff diaphragm on the lower floors and the same seismic-force-resisting system with a flexible diaphragm on the upper floors as long as the total building height meets the more restrictive limitation for the common building type in Table 3-8.

3.5.2 Tier 1 Screening Procedure Seismic evaluation using the Tier 1 and Tier 2 procedures shall begin with the Tier 1 Screening procedure, conducted in accordance with the requirements of Chapter 4.

3.5.3 Tier 2 Deficiency-Based Evaluation and Retrofit Procedures Where potential deficiencies were identified by the Tier 1 Screening, a Tier 2 deficiency-based evaluation or retrofit may be performed in accordance with this section and Chapter 5.

3.5.3.1 Evaluation Requirements For a Tier 2 deficiency-based evaluation, only the potential deficiencies identified by the noncompliant checklist statements need to be assessed.

If the Tier 2 evaluation procedure in Chapter 5 demonstrates compliance for all of the Tier 1 checklist statements that were identified as noncompliant, then the building is deemed to comply with the selected Performance Objective.

3.5.3.2 Retrofit Requirements The Tier 2 deficiency-based retrofit procedure may be used for the Basic Performance Objective for Existing Buildings (BPOE), as defined in Section 2.4.1.

Where the Tier 2 deficiency-based retrofit procedure is used to achieve a Partial Retrofit Objective as defined in Section 2.4.5, retrofit measures shall be developed in accordance with Section 5.8 such that selected deficiencies identified by the Tier 2 evaluation are eliminated. The deficiencies selected for mitigation shall be retrofitted to comply with the requirements of the Tier 2 retrofit procedures for the selected Performance Level.

Where the Partial Retrofit Objective addresses architectural, mechanical, and electrical components, retrofit measures shall be developed in accordance with Chapter 13 for the selected Nonstructural Performance Level.

3.5.4 Tier 3 Systematic Evaluation and Retrofit Procedures

3.5.4.1 Evaluation Requirements A Tier 3 systematic evaluation shall be performed in accordance with the requirements of Chapter 6 where required by Section 3.5.1.

3.5.4.2 Retrofit Requirements The Tier 3 systematic retrofit procedure in Chapter 6 shall be permitted for all retrofit designs and shall be required where Tier 2 deficiency-based retrofit is not permitted in accordance with Section 3.5.1.

The Tier 3 systematic retrofit procedure includes the following steps:

1. An evaluation shall be performed to identify potential seismic deficiencies;
2. A preliminary retrofit scheme shall be developed;
3. An analysis of the building, including retrofit measures, shall be performed, to verify that the retrofit design meets the selected Performance Objective; and
4. Construction documents, including drawings, specifications, and a quality assurance plan, shall be developed.

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CHAPTER 4
TIER 1 SCREENING

4.1 SCOPE

This chapter contains the requirements for performing a Tier 1 screening where it is permitted in accordance with Section 3.5. The Tier 1 process is shown schematically in Figure 4-1.

The Performance Level, Seismic Hazard Level, and Level of Seismicity shall be determined in accordance with Sections 4.1.1, 4.1.2, and 4.1.3, respectively.

Section 4.2 specifies the requirements for the level of investigation of as-built conditions, performing site visits, and determining the building type.

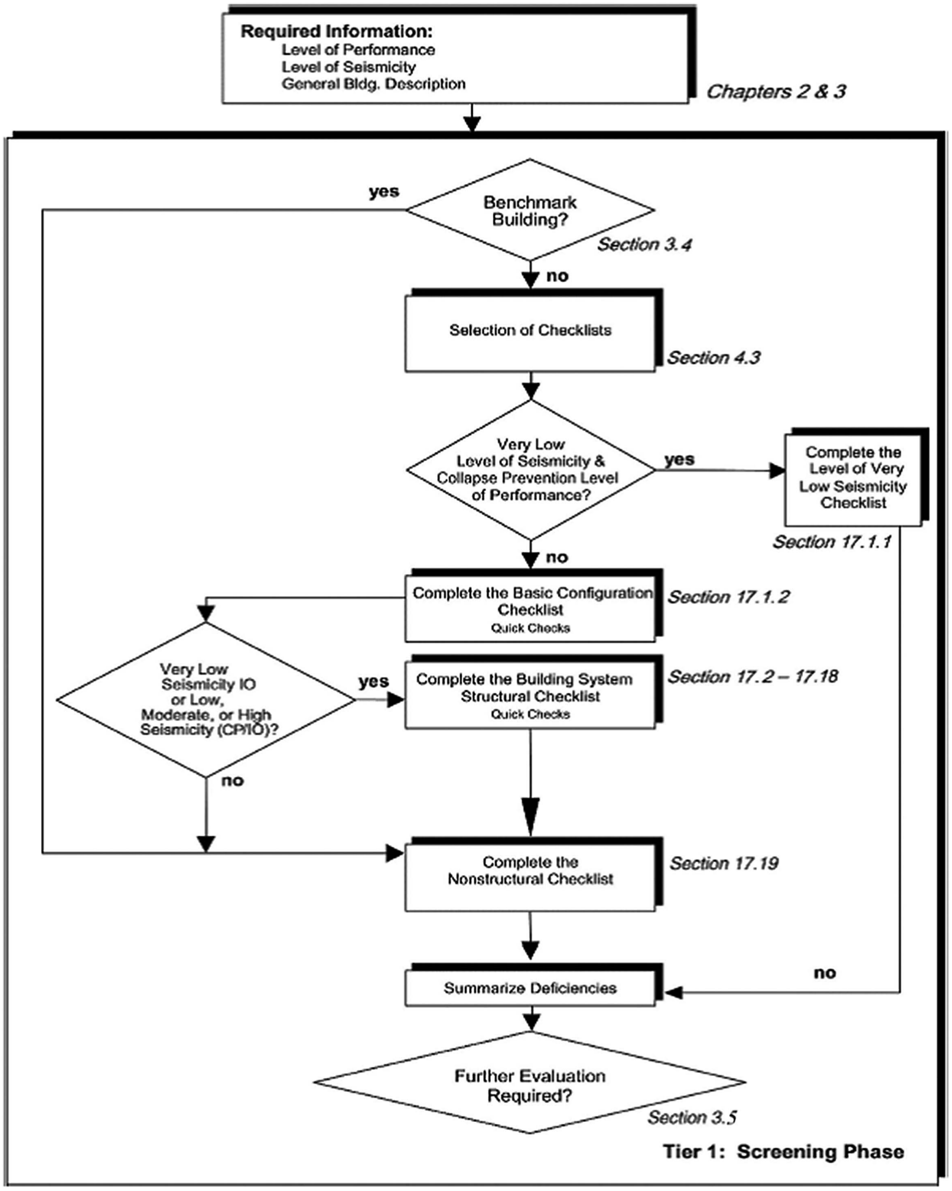


Figure 4-1. Tier 1 evaluation process.

The design professional shall select and complete the appropriate checklists in accordance with Section 4.3. The checklists themselves are contained in Chapter 17. Section 4.4 contains the Tier 1 analysis provisions for use with the Tier 1 checklists.

A list of potential deficiencies identified by evaluation statements for which the building was found to be noncompliant shall be compiled on completion of the Tier 1 checklists.

4.1.1 Performance Level A target Performance Level shall be defined in accordance with Section 2.2 before conducting a seismic evaluation using the Tier 1 screening procedure.

4.1.2 Seismic Hazard Level The Seismic Hazard Level for the Tier 1 screening shall be determined in accordance with Section 2.3

4.1.3 Level of Seismicity The Level of Seismicity of the building shall be defined as Very Low, Low, Moderate, or High in accordance with Section 2.5.

4.2 SCOPE OF INVESTIGATION REQUIRED

4.2.1 On-Site Investigation and Condition Assessment As-built information shall be obtained in accordance with Section 3.2 and the requirements of this section. Tier 1 screening shall be permitted to be based on available construction documents and other records, subject to the findings of an on-site investigation. An on-site investigation shall be conducted to verify general conformance of existing conditions to those described in available documents, to identify significant alterations or deviations from available documents, to supplement incomplete documents, to confirm the general quality of construction and maintenance, and otherwise as needed to complete the applicable Tier 1 checklists.

Where required, limited nondestructive investigation of a representative sample of relevant conditions shall be performed for all Tier 1 Quick Checks.

The on-site investigation shall include investigation of common, likely, or suspected construction defects and deterioration that could have significant effects on seismic performance. The scope of this investigation shall be permitted to be based on the judgment of the evaluator. The findings and documentation of this investigation shall be subject to the approval of the Authority Having Jurisdiction where required.

In setting the scope of this investigation, the evaluator shall consider at least the defect and deterioration types given in Table 4-1.

Table 4-1. Patterns of Defects and Deterioration.

Component or Material	Pattern	Tier 2 Reference Sections
Foundation	Evidence of settlement or heave	5.2.3, 5.4.3.2
Foundation elements	Deterioration caused by corrosion, sulfate attack, or material breakdown	5.2.3, 5.4.3.2
Wood	Decay, shrinkage, splitting, fire damage, or sagging in wood members. Deteriorated, broken, or loose metal connection hardware	5.2.3
Wood structural panel shear wall fasteners	Overdriven fasteners, omitted blocking, excessive fastener spacing, or inadequate edge distance	5.2.3
Steel ≥1/8 in. (3.18 mm) thick	Visible rusting, corrosion, cracking, or other deterioration	5.2.3
Steel <1/8 in. (3.18 mm) thick	Visible deformations, corrosion particularly near welds or fasteners, loose fasteners	5.2.3
Concrete	Visible deterioration of concrete or reinforcing steel	5.2.3
Concrete walls	Cracks that are 1/16 in. (1.6 mm) or wider, concentrated in one location or forming an X pattern	5.2.3
Concrete columns encasing masonry infill	Diagonal cracks wider than 1/16 in. (1.6 mm)	5.2.3
Unreinforced masonry units	Visible deterioration	5.2.3
Unreinforced masonry joints	Eroded mortar or mortar that is easily scraped away from the joints by hand with a metal tool	5.2.3
Unreinforced masonry walls	Voids or missing grout in collar joints along with the lack of header courses of multi-wythe walls	5.2.3
Infill masonry walls	Diagonal or stepped cracks more than 1/16 in. (1.6 mm) wide that extend throughout a panel, or out-of-plane offsets wider than 1/16 in. (1.6 mm) in masonry joints	5.2.3
Post-tensioning anchors	Corrosion or spalling in the vicinity of post-tensioning or end fittings	5.2.3
Precast concrete walls	Visible deterioration of concrete or reinforcing steel, or evidence of distress, especially at the connections	5.2.3
Reinforced masonry walls	Cracks that are 1/16 in.(1.6 mm) or wider, concentrated in one location or forming an X pattern	5.2.3
Masonry veneer	Deterioration, damage, or corrosion in connections	13.6.1
Masonry veneer	Eroded mortar or mortar that is easily scraped away from the joints by hand with a metal tool	13.6.1
Masonry veneer	Visible cracks or distortion in the masonry	13.6.1
Hazardous material equipment	Damaged supply lines	13.7
Mechanical or electrical equipment	Deterioration, damage, or corrosion in anchorage or supports	13.7
Cladding	Deterioration, damage, or corrosion in connections	13.6.1

4.2.2 Building Type The building type shall be classified as one or more of the Common Building Types listed in Table 3-1, based on the lateral-force-resisting system(s) and the diaphragm type. Separate building types shall be used for buildings with different lateral-force-resisting systems in different directions.

4.2.3 Default Material Values The use of default values is permitted for material properties for Tier 1 Quick Checks. The following default values are to be assumed unless otherwise indicated by the available construction documents, or by testing. Because these values and properties were taken from Chapters 9 and 10, refer to these chapters for values of material properties for uses other than Tier 1 Quick Checks.

f'_c See Table 4-2
f_y See Table 4-3
F_y See Tables 4-4 and 4-5
E Structural and cold-formed steel $= 29{,}000$ kip/in.^{2} (200 GPa)
$F_{pe} = 25$ kip (111.2 kN)

4.3 SELECTION AND USE OF CHECKLISTS

The Tier 1 checklists are provided in Chapter 17. Required checklists, as a function of Level of Seismicity and Performance Level, are listed in Table 4-6. Each of the required checklists designated in Table 4-6 shall be completed for a Tier 1 screening. Each of the evaluation statements on the checklists shall be marked “Compliant” (C), “Noncompliant” (NC), “Not Applicable” (N/A), or “Unknown” (U). Compliant statements identify issues that are acceptable according to the criteria of this standard, whereas noncompliant or unknown statements identify issues that require further investigation to demonstrate compliance with the applicable Performance Objective. Certain evaluation statements may not apply to the specific building being evaluated.

Table 4-2. Default Compressive Strengths (f'_c) of Structural Concrete (kip/in.^{2}).

Time Frame	Beams	Slabs and Columns	Walls
1900–1919	2	1.5	1
1920–1949	2	2	2
1950–1969	3	3	2.5
1970–Present	3	3	3

Table 4-4. Default Yield Strengths (F_y) of Archaic Materials.

Year	Material	Yield Strength (kip/in.^{2})
Pre-1900	Steel	24

Quick Checks for Tier 1 shall be performed in accordance with Section 4.4 where necessary to complete an evaluation statement.

The checklist for Very Low Seismicity, located in Section 17.1.1, shall be completed for buildings in Very Low Seismicity being evaluated to the Collapse Prevention Performance Level. For buildings in Very Low Seismicity being evaluated to the Immediate Occupancy Performance Level and buildings in levels of Low, Moderate, or High Seismicity, the appropriate structural and nonstructural checklists shall be completed in accordance with Table 4-6.

The appropriate structural checklists shall be selected based on the Common Building Types defined in Table 3-1. Buildings being evaluated to the Collapse Prevention Performance Level shall use the applicable checklists in Chapter 17 for the Collapse Prevention Performance Level. Buildings being evaluated to the Immediate Occupancy Performance Level shall use the applicable checklists in Chapter 17 for the Immediate Occupancy Performance Level. Refer to Table 2-4 for the use of the Collapse Prevention checklists for evaluating buildings to the Life Safety and Limited Safety Performance Levels as applicable.

A building with a different lateral-force-resisting system in each principal direction shall use two sets of structural checklists, one for each direction. A building with more than one type of lateral-force-resisting system along a single axis of the building being evaluated to the Collapse Prevention Performance Level, including changes in seismic-force-resisting system over the height, may be evaluated using the applicable checklists in Chapter 17 subject to the requirements in Section 3.5.1.

One nonstructural checklist is provided in Section 17.19, with a heading before each statement identifying if it applies to the Hazards Reduced, Life Safety, and Position Retention Performance Levels. Refer to Table 4-6 for the applicability of the nonstructural checklists.

Table 4-3. Default Yield Strengths (f_y) of Reinforcing Steel (kip/in.^{2}).[a]

		Structural[b]	Intermediate[b]	Hard[b]				
	Grade	33	40	50	60	65	70	75
Year	Minimum Yield (kip/in.^{2})	33	40	50	60	65	70	75
1911–1959		X	X	X		X		
1959–1966		X	X	X	X	X	X	X
1966–1987			X	X	X	X	X	
1987–present			X	X	X	X	X	X

[a] An entry of X indicates that the grade was available in those years.
[b] The terms structural, intermediate, and hard became obsolete in 1968.

Table 4-5. Default Yield Strengths (F_y) of Structural and Cold-Formed Steel.

Date	Specification	Remarks	Yield Strength[a] (kip/in.^{2})
1900	ASTM A9	Rivet steel	30
	Buildings	Medium steel	35
1901–1908	ASTM A9	Rivet steel	30
	Buildings	Medium steel	30
1909–1923	ASTM A9	Structural steel	28
	Buildings	Rivet steel	30
1924–1931	ASTM A7	Structural steel	30
		Rivet steel	30
	ASTM A9	Structural steel	30
		Rivet steel	25
1932	ASTM A140-32 T issued as a tentative revision to ASTM	Plates, shapes, bars	33
	A9 (Buildings)	Eyebar flats (unannealed)	36
1933	ASTM A140-32 T discontinued and ASTM A9 (Buildings) revised Oct. 30, 1933	Structural steel	30
	ASTM A141-32 T adopted as a standard	Rivet steel	30
1934-Present	ASTM A9	Structural steel	33
	ASTM A141	Rivet steel	30
1946–1967	ASTM A245 Grade C	Steel Sheet	33[b]
1961–1990	ASTM A36/A36M-04 (2004a)	Structural steel	37
1961-Present	ASTM A572/A572M-04 (2004b), Grade 50	Structural steel	50
1968–1995	ASTM A446 Grade A	Steel Sheet	42[c]
1968–1995	ASTM A446 Grade D	Steel Sheet	52[c]
1990-Present	ASTM A36/A36M-04 (2004a) and Dual Grade	Structural steel	49
1996-Present	ASTM A653 SS Grade 33	Steel Sheet	42[c]
1996-Present	ASTM A653 SS Grade 50	Steel Sheet	52[c]
1998-Present	ASTM A992/A992M-04 (2004c)	Structural steel	50
2000-Present	ASTM A1003 SS Grade 33	Steel Sheet	42[c]
2000-Present	ASTM A1003 SS Grade 50	Steel Sheet	52[c]

[a] Values are representative of material extracted from the flanges of wide flange shapes (i.e., for non-rivet steel).

[b] Values are based on minimum specified values.

[c] Values are based on mean minus one standard deviation values from statistical data.

Notes: Except as indicated in footnotes *b* and *c*, values for material before 1960 are based on minimum specified values. Values for material after 1960 are mean minus one standard deviation values from statistical data. Values are based on ASTM and AISC structural steel specification stresses.

4.4 TIER 1 ANALYSIS

4.4.1 Overview Analyses performed as part of the Tier 1 screening process are limited to Quick Checks. Quick Checks shall be used to calculate the stiffness and strength of certain building components to determine whether the building complies with certain evaluation criteria. Quick Checks shall be performed in accordance with Section 4.4.3 where they are triggered by evaluation statements from the checklists of Chapter 17. Seismic forces for use in the Quick Checks shall be computed in accordance with Section 4.4.2.

4.4.2 Seismic Forces

4.4.2.1 Pseudo Seismic Force The pseudo seismic force, in a given horizontal direction of a building, shall be calculated in accordance with Equation (4-1).

$$V = CS_aW \tag{4-1}$$

where

V = Pseudo seismic force;

C = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response; C shall be taken from Table 4-7;

S_a = Response spectral acceleration at the fundamental period of the building in the direction under consideration. The value of S_a shall be calculated in accordance with the procedures in Section 4.4.2.3; and

W = Effective seismic weight of the building, including the total dead load and applicable portions of other gravity loads listed below:

1. In areas used for storage, a minimum of 25% of the floor live load shall be applicable. The live load shall be permitted to be reduced for tributary area as approved by the code official. Floor live load in public garages and open parking structures need not be considered.
2. Where an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 10 lb/ft^2 (0.48 kN/m^2) of floor area, whichever is greater, shall be applied.

Table 4-6. Checklists Required for a Tier 1 Screening.

		Required Checklists[a]					
Level of Seismicity[b]	Level of Building Performance[c]	Very Low Seismicity Checklist (Section 17.1.1)	Basic Configuration Checklist (Section 17.1.2)	Collapse Prevention Checklist (Sections 17.2 through 17.18)	Immediate Occupancy Checklist (Sections 17.2 through 17.18)	Hazards Reduced or Life Safety Nonstructural Checklist (Section 17.19)	Position Retention Nonstructural Checklist (Section 17.19)
Very low	CP	X					
Very low	IO		X		X		X
Low	CP		X	X		X	
Low	IO		X		X		X
Moderate	CP		X	X		X	
Moderate	IO		X		X		X
High	CP		X	X		X	
High	IO		X		X		X

[a] An X designates the checklist that must be completed for a Tier 1 screening as a function of the Level of Seismicity and Level of Performance.
[b] Defined in Section 2.5.
[c] CP = Collapse Prevention Performance Level, and IO = Immediate Occupancy Performance Level (defined in Section 2.2.1).

Table 4-7. Modification Factor, *C*.

	Number of Stories			
Common Building Types*	1	2	3	≥4
Wood and cold-formed steel shear wall (W1, W2, CFS1)	1.3	1.1	1.0	1.0
Moment frame (S1, S3, C1, PC2a)				
Shear wall (S4, S5, C2, C3, PC1a, PC2, RM2, URMa)	1.4	1.2	1.1	1.0
Braced frame (S2)				
Cold-formed steel strap-brace wall (CFS2)				
Unreinforced masonry (URM) Flexible diaphragms (S1a, S2a, S5a, C2a, C3a, PC1, RM1)	1.0	1.0	1.0	1.0

*Defined in Table 3-1.

3. Total operating weight of permanent equipment.
4. Weight of landscaping and other materials at roof gardens and similar areas.
5. Weight of fluids and bulk material expected to be present during normal use.
6. Snow load per Section 7.2.3.3.

4.4.2.2 Story Shear Forces The pseudo seismic force calculated in accordance with Section 4.4.2.1 shall be distributed vertically in accordance with Equation (4-2a and b). For buildings six stories or fewer high, the value of k shall be permitted to be taken as 1.0.

$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V \tag{4-2a}$$

$$V_j = \sum_{x=j}^{n} F_x \tag{4-2b}$$

where

V_j = Story shear at story level j;
n = Total number of stories above ground level;
j = Number of story levels under consideration;
W = Total seismic weight, per Section 4.4.2.1;
V = Pseudo seismic force from Equation (4-1);
w_i = Portion of total building weight W located on or assigned to floor level i;
w_x = Portion of total building weight W located on or assigned to floor level x;
h_i = Height (ft) from the base to floor level i;
h_x = Height (ft) from the base to floor level x; and
k = 1.0 for $T \le 0.5$ s and 2.0 for $T > 2.5$ s; linear interpolation shall be used for intermediate values of k.

The story shear forces shall be distributed to the lateral-force-resisting elements in accordance with the Quick Checks for strength and stiffness in Section 4.4.3. For buildings with flexible diaphragms, story shear shall be distributed to each line of lateral resistance based on tributary area.

4.4.2.3 Spectral Acceleration Spectral acceleration, S_a, for use in computing the pseudo seismic force shall be computed in accordance with Equation (4-3).

$$S_a = \frac{S_{X1}}{T} \tag{4-3}$$

but S_a shall not exceed S_{XS}, where T is the fundamental period of vibration of the building, calculated in accordance with Section 4.4.2.4, and S_{X1} and S_{XS} are as defined in Section 2.3.2, for the Seismic Hazard Level specified in Section 4.1.2. Alternatively, a site-specific response spectrum shall be permitted to be developed according to Section 2.3.3 for the Seismic Hazard Level specified in Section 4.1.2.

4.4.2.4 Period The fundamental period of a building, in the direction under consideration, shall be calculated in accordance with Equation (4-4).

$$T = C_t h_n^{\beta} \tag{4-4}$$

where

T = Fundamental period (s) in the direction under consideration;
C_t = 0.035 for moment-resisting frame systems of steel (Building Types S1 and S1a);
= 0.018 for moment-resisting frames of reinforced concrete (Building Type C1);
= 0.030 for eccentrically braced steel frames (Building Types S2 and S2a);
= 0.020 for all other framing systems;
h_n = Height (ft) above the base to the roof level;
β = 0.80 for moment-resisting frame systems of steel (Building Types S1 and S1a);
= 0.90 for moment-resisting frame systems of reinforced concrete (Building Type C1); and
= 0.75 for all other framing systems.

Alternatively, for steel or reinforced-concrete moment frames of 12 stories or fewer, the fundamental period of the building may be calculated as follows:

$$T = 0.10n \tag{4-5}$$

where n is the number of stories above the base.

4.4.3 Quick Checks for Strength and Stiffness Quick Checks shall be used to compute the stiffness and strength of building components. Quick Checks are triggered by evaluation statements in the checklists of Chapter 17 and are required to determine the compliance of certain building components. The seismic forces used in the Quick Checks shall be calculated in accordance with Section 4.4.2.

4.4.3.1 Story Drift for Moment Frames Equation (4-6) shall be used to calculate the drift ratios of regular, multistory, multibay moment frames with columns continuous above and below the story under consideration. For other configurations of frames and frame elements, this quick check procedure shall not be used, and the checklist statement shall be marked as “noncompliant” unless the drift ratio is determined by a rational approach based on the principals of structural mechanics.

The drift ratio is based on the deflection caused by flexural displacement of a representative column, including the effect of end rotation caused by bending of the representative beam.

$$D_r = \left(\frac{k_b + k_c}{k_b k_c}\right)\left(\frac{h}{12E}\right)V_c \tag{4-6}$$

where

D_r = Drift ratio: interstory displacement divided by story height;
k_b = I/L for the representative beam;
k_c = I/h for the representative column;
h = Story height (inch);
I = Moment of inertia (in.^{4});
L = Beam length from center-to-center of adjacent columns (inch);
E = Modulus of elasticity (kip/in.^{2}); and
V_c = Shear in the column (kip).

The column shear forces are calculated using the story shear forces in accordance with Section 4.4.2.2.

Equation (4-6) shall be permitted to be used for the first floor of the frame if columns are fixed against rotation at the bottom. However, if columns are pinned at the bottom, the drift ratio determined using Equation (4-6) shall be multiplied by 2.

4.4.3.2 Shear Stress in Concrete Frame Columns The average shear stress, v_j^{avg}, in the columns of concrete frames shall be computed in accordance with Equation (4-7).

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{n_c}{n_c - n_f}\right)\left(\frac{V_j}{A_c}\right) \tag{4-7}$$

where

n_c = Total number of columns;
n_f = Total number of frames in the direction of loading;
A_c = Summation of the cross-sectional area of all columns in the story under consideration;
V_j = Story shear computed in accordance with Section 4.4.2.2; and
M_s = System modification factor; M_s shall be taken as equal to 2.0 for buildings being evaluated to the Collapse Prevention Performance Level, equal to 1.5 for buildings being evaluated to the Life Safety Performance Level, and equal to 1.0 for buildings being evaluated to the Immediate Occupancy Performance Level.

4.4.3.3 Shear Stress in Shear Walls The average shear stress in shear walls, v_j^{avg}, shall be calculated in accordance with Equation (4-8).

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right) \tag{4-8}$$

where

V_j = Story shear at level j computed in accordance with Section 4.4.2.2;
A_w = Summation of the horizontal cross-sectional area of all shear walls in the direction of loading. Openings shall be taken into consideration where computing A_w. For masonry walls, the net area shall be used. For wood-framed walls, the length shall be used rather than the area; and
M_s = System modification factor; M_s shall be taken from Table 4-8.

4.4.3.4 Diagonal Bracing The average axial stress in diagonal bracing elements, f_j^{avg}, shall be calculated in accordance with Equation (4-9).

Table 4-8. M_s Factors for Shear Walls.

	Level of Performance		
Wall Type	**CP***	**LS***	**IO***
Reinforced concrete, precast concrete, wood, reinforced masonry, and cold-formed steel	4.5	3.0	1.5
Unreinforced masonry	1.75	1.25	1.0

*CP = Collapse Prevention, LS = Life Safety, IO = Immediate Occupancy.

Table 4-9. M_s Factors for Diagonal Braces.

		Level of Performance		
Brace Type	**Width-to-Thickness Ratio**^{b}	**CP**^{a}	**LS**^{a}	**IO**^{a}
Tube^b	$<\lambda_{hd}90/(F_{ye})^{1/2}$	7.0	4.5	2.0
	$>\lambda_{md}190/(F_{ye})^{1/2}$	3.5	2.5	1.25
Pipe^c	$<\lambda_{hd}1{,}500/F_{ye}$	7.0	4.5	2.0
	$>\lambda_{md}6{,}000/F_{ye}$	3.5	2.5	1.25
Tension-only		3.5	2.5	1.25
Cold-formed steel strap-braced wall		3.5	2.5	1.25
All others		7.0	4.5	2.0

[a] CP = Collapse Prevention, LS = Life Safety, IO = Immediate Occupancy.

[b] Width-to-thickness ratios shall be determined in accordance with AISC 341 Table D1.1a.

[c] Interpolation to be used for tubes and pipes.

$$f_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{sN_{br}}\right)\left(\frac{L_{br}}{A_{br}}\right) \tag{4-9}$$

where

L_{br} = Average length of the braces (feet);
N_{br} = Number of braces in tension and compression if the braces are designed for compression, number of diagonal braces in tension if the braces are designed for tension only;
s = Average span length of braced spans (feet);
A_{br} = Average area of a diagonal brace (in.^{2});
V_j = Maximum story shear at each level (kip); and
M_s = System modification factor; M_s shall be taken from Table 4-9.

4.4.3.5 Precast Connections The strength of the connection in precast concrete moment frames shall be greater than the moment in the girder, M_{gj}, calculated in accordance with Equation (4-10).

$$M_{gj} = \frac{V_j}{M_s}\left(\frac{1}{n_c - n_f}\right)\left(\frac{h}{2}\right) \tag{4-10}$$

where

n_c = Total number of columns;
n_f = Total number of frames in the direction of loading;
V_j = Story shear at the level directly below the connection under consideration;
h = Typical column story height; and
M_s = System modification factor taken as equal to 2.5 for buildings being evaluated to the Collapse Prevention Performance Level, equal to 1.5 for buildings being evaluated to the Life Safety Performance Level, and equal to 1.0 for buildings being evaluated to the Immediate Occupancy Performance Level.

4.4.3.6 Column Axial Stress Caused by Overturning. The axial stress of columns in moment frames at the base subjected to overturning forces, p_{ot}, shall be calculated in accordance with Equation (4-11).

$$p_{ot} = \frac{1}{M_s}\left(\frac{2}{3}\right)\left(\frac{Vh_n}{Ln_f}\right)\left(\frac{1}{A_{col}}\right) \tag{4-11}$$

where

n_f = Total number of frames in the direction of loading;
V = Pseudo seismic force;
h_n = Height (feet) above the base to the roof level;
L = Total length of the frame (feet);
M_s = System modification factor taken as equal to 2.5 for buildings being evaluated to the Collapse Prevention Performance Level, equal to 1.5 for buildings being evaluated to the Life Safety Performance Level, and equal to 1.0 for buildings being evaluated to the Immediate Occupancy Performance Level; and
A_{col} = Area of the end column of the frame.

4.4.3.7 Flexible Diaphragm Connection Forces The horizontal seismic forces associated with the connection of a flexible diaphragm to either concrete or masonry walls, T_c, shall be calculated in accordance with Equation (4-12).

$$T_c = \Psi S_{XS} w_p A_p \tag{4-12}$$

where

w_p = Unit weight of the wall;
A_p = Area of wall tributary to the connection;
Ψ = 1.0 for Collapse Prevention Performance Level, 1.3 for Life Safety Performance Level, and 1.8 for Immediate Occupancy Performance Level; and
S_{XS} = Value specified in Section 4.4.2.3.

Exception: The force T_c, determined in accordance with Equation (4-12) need not be greater than F_p as determined in accordance with Section 7.2.13.1.

4.4.3.8 Prestressed Elements The average prestress in prestressed or post-tensioned elements, f_p, shall be calculated in accordance with Equation (4-13).

$$f_p = \frac{F_{pe} n_p}{A_p} \tag{4-13}$$

where

F_{pe} =
f_{pe} = Effective force of a prestressed strand,
n_p = Number of prestressed strands, and
A_p = Gross area of prestressed concrete elements.

4.4.3.9 Flexural Stress in Columns and Beams of Steel Moment Frames. The average flexural stress in the columns and beams of steel frames at each level shall be computed in accordance with Equation (4-14).

$$f_j^{avg} = V_j \frac{1}{M_s}\left(\frac{n_c}{n_c - n_f}\right)\left(\frac{h}{2}\right)\frac{1}{Z} \tag{4-14}$$

where

n_c = Total number of frame columns at the level, j, under consideration.
n_f = Total number of frames in the direction of loading at the level, j, under consideration.
V_j = Story shear computed in accordance with Section 4.4.2.2.
h = Story height (inch).
Z = For columns, the sum of the plastic section moduli of all the frame columns at the level under consideration. For beams, it is the sum of the plastic section moduli of all the frame beams with moment-resisting connections. If a beam has moment-resisting connections at both ends, then

the contribution of that beam to the sum is twice the plastic section modulus of that beam (in.^{3}).

M_s = System modification factor; M_s shall be taken as equal to 9.0 for buildings being evaluated to the Collapse Prevention Performance Level, equal to 6.0 for buildings being evaluated to the Life Safety Performance Level, and equal to 2.5 for buildings being evaluated to the Immediate Occupancy Performance Level for columns and beams satisfying the checklist items for compactness and column axial stress. If the columns or beams do not satisfy the checklist statements for compactness and column axial stress for the Immediate Occupancy Performance Level, then this item must be marked "Noncompliant."

CHAPTER 5

TIER 2 DEFICIENCY-BASED EVALUATION AND RETROFIT

5.1 SCOPE

This chapter contains the requirements for performing seismic evaluation and retrofit using the Tier 2 deficiency-based procedures. General requirements are specified in Section 5.2. Evaluation requirements and retrofit requirements are specified in Sections 5.3 and 5.8, respectively. The Tier 2 process is shown in Figure 5-1.

The Tier 2 deficiency-based evaluation requires additional analysis and evaluation of all the potential deficiencies identified in the Tier 1 screening (denoted by either "Noncompliant" or "Unknown" responses in the Tier 1 checklists). The additional analysis and evaluation of each potential deficiency shall be sufficient to either confirm the deficiency or demonstrate the adequacy of the structure as it relates to the potential deficiency. The evaluation shall, at a minimum, use the procedures specified in Sections 5.4 to 5.7.

The scope of the Tier 2 deficiency-based evaluation need not expand beyond the evaluation of the potential deficiencies identified in the Tier 1 screening.

The Tier 2 deficiency-based retrofit requires retrofit of the building such that the deficiencies identified in a Tier 1 screening or a Tier 2 evaluation are mitigated to achieve compliance with the selected Performance Objective(s). The scope of the Tier 2 deficiency-based retrofit need not expand beyond that necessary to modify the building to comply with a Tier 1 screening or a Tier 2 evaluation.

Tier 2 evaluation and retrofit of nonstructural components shall be performed in accordance with the provisions of Chapter 13.

5.2 GENERAL REQUIREMENTS

A Tier 1 screening (Chapter 4) shall be completed before performing a Tier 2 deficiency-based evaluation or retrofit. Use of deficiency-based procedures is subject to the limitations of Section 3.5.

5.2.1 Performance Level and Seismic Hazard Level The Performance Level and Seismic Hazard Level for evaluation or retrofit shall be the same as for the Tier 1 screening as specified in Sections 4.1.1 and 4.1.2.

If the Tier 2 deficiency-based evaluation demonstrates the adequacy of the structure with respect to all of the "Noncompliant" or "Unknown" statements in the Tier 1 screening, then the building complies with this standard for the corresponding Performance Objective. If the building is retrofitted in accordance with the deficiency-based retrofit procedure, then the retrofitted building complies with this standard for the corresponding Performance Objectives.

5.2.2 As-Built Information In addition to the information required for a Tier 1 screening in Chapter 4, sufficient information shall be collected for a Tier 2 evaluation or retrofit to complete the required procedures in this chapter. Destructive examination shall be conducted as required to complete the procedures for buildings being evaluated to the Immediate Occupancy Performance Level. Nondestructive examination of connections and conditions associated with all potential deficiencies shall be performed for all Tier 2 evaluations and retrofits.

For the purpose of this chapter, it is permitted to use the default material properties in Chapters 8 through 12 or to use material properties provided in available design drawings.

5.2.3 Condition Assessment Where the Tier 2 procedures are used to evaluate deterioration or damage identified in the Tier 1 screening phase or during a subsequent on-site investigation, the extent and the consequence of this deterioration or damage to the seismic-force-resisting system shall be determined. The adequacy of the damaged seismic-force-resisting system shall be evaluated considering the extent of the damage and the effect on the capacity of each damaged element. The effects of the condition of the materials on the seismic performance shall be permitted to be based on the judgment of the evaluator. The findings and documentation of this investigation shall be subject to the approval of the Authority Having Jurisdiction.

5.2.4 Tier 2 Analysis Methods Where the use of the Tier 2 procedures requires analysis of the structure or a component of the structure, the analysis shall conform to the following requirements of Chapter 7:

1. General analysis requirements shall be in accordance with Section 7.2, except that the scope of evaluation need not expand beyond the evaluation of the potential deficiencies identified in the Tier 1 screening.
2. Analysis procedures shall be in accordance with Section 7.2.1, utilizing either the linear static procedure (LSP) of Section 7.4.1 or the linear dynamic procedure (LDP) of Section 7.4.2. The limitations on the use of linear procedures in Section 7.3.1.1 need not apply to Tier 2 procedures. LDP shall be used when the LSP is limited in accordance with Section 7.3.1.2 or when the LDP is required by Tier 2 evaluation procedures.
3. Component gravity loads and load combinations shall be in accordance with Section 7.2.3.
4. Mathematical modeling shall be in accordance with Section 7.2.4.
5. The building's configuration and irregularities shall be included in accordance with Section 7.2.5.
6. Multidirectional seismic effects shall be included where required by Section 7.2.6.
7. P-Δ Effects shall be included in accordance with Section 7.2.7.

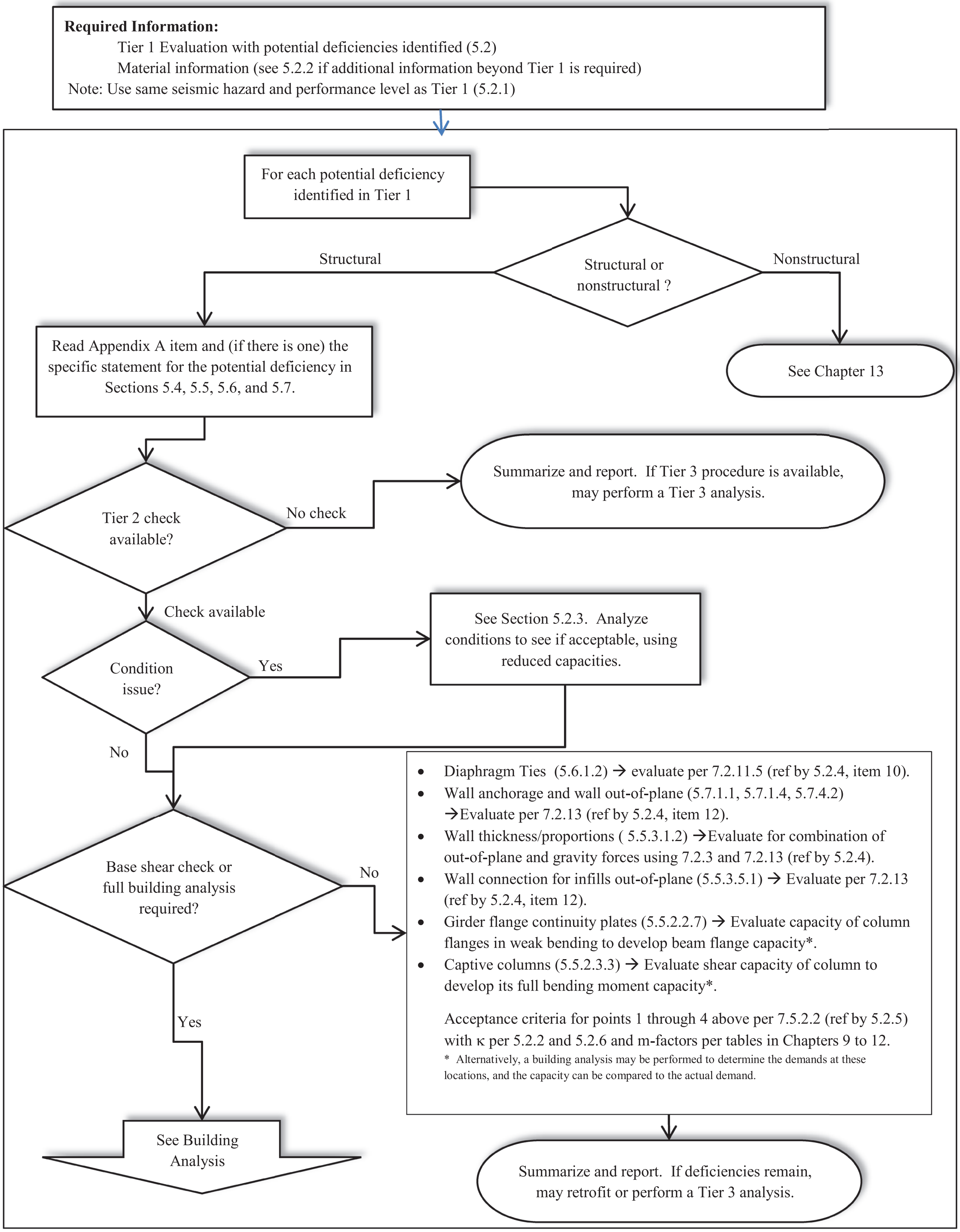
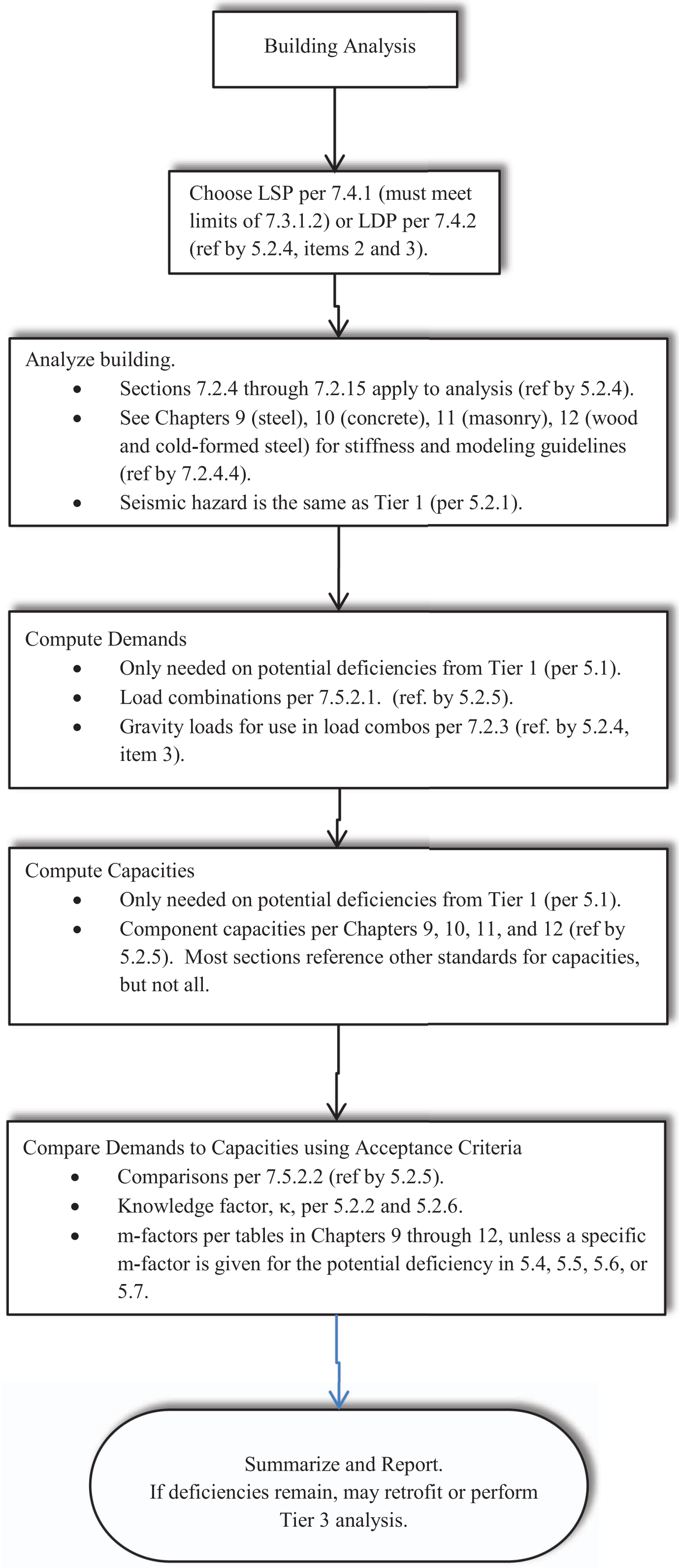


Figure 5-1. Tier 2 Evaluation Process.



Building Analysis

Choose LSP per 7.4.1 (must meet limits of 7.3.1.2) or LDP per 7.4.2 (ref by 5.2.4, items 2 and 3).

Analyze building.
- Sections 7.2.4 through 7.2.15 apply to analysis (ref by 5.2.4).
- See Chapters 9 (steel), 10 (concrete), 11 (masonry), 12 (wood and cold-formed steel) for stiffness and modeling guidelines (ref by 7.2.4.4).
- Seismic hazard is the same as Tier 1 (per 5.2.1).

Compute Demands
- Only needed on potential deficiencies from Tier 1 (per 5.1).
- Load combinations per 7.5.2.1. (ref. by 5.2.5).
- Gravity loads for use in load combos per 7.2.3 (ref. by 5.2.4, item 3).

Compute Capacities
- Only needed on potential deficiencies from Tier 1 (per 5.1).
- Component capacities per Chapters 9, 10, 11, and 12 (ref by 5.2.5). Most sections reference other standards for capacities, but not all.

Compare Demands to Capacities using Acceptance Criteria
- Comparisons per 7.5.2.2 (ref by 5.2.5).
- Knowledge factor, κ, per 5.2.2 and 5.2.6.
- m-factors per tables in Chapters 9 through 12, unless a specific m-factor is given for the potential deficiency in 5.4, 5.5, 5.6, or 5.7.

Summarize and Report.
If deficiencies remain, may retrofit or perform Tier 3 analysis.

Figure 5-1 (*Continued*). Tier 2 Evaluation Process

8. Soil–structure interaction need not be included, but if included it shall be in accordance with Section 7.2.8.
9. When Tier 2 evaluation procedures require evaluation of overturning effects, overturning shall be evaluated in accordance with Section 7.2.9.
10. Diaphragms shall be included in the model in accordance with Section 7.2.11. Diaphragms, chords, collectors, and ties shall be evaluated in accordance with Section 7.2.11 when required by Tier 2 evaluation procedures.
11. When Tier 2 evaluation procedures require evaluation of the continuity of structural elements to be tied together to form a complete load path, continuity shall be evaluated in accordance with Section 7.2.12.
12. When Tier 2 evaluation procedures require evaluation of walls and wall anchorage for out-of-plane forces, the evaluation shall be in accordance with Section 7.2.13.
13. When Tier 2 evaluation procedures require evaluation of vertical- or seismic-force-resisting elements common to two structures, the evaluation shall be in accordance with Section 7.2.14.
14. When Tier 2 evaluation procedures require evaluation of building separations, the evaluation shall be in accordance with Section 7.2.15.

The extent of modeling and analysis of the structure shall be as required to determine the forces or actions on the structural system or on each specific structural component addressed by the Tier 2 analysis.

5.2.5 Tier 2 Acceptance Criteria The acceptance criteria for Tier 2 procedures shall be in accordance with Section 7.5.2.2. Design actions shall be calculated in accordance with Section 7.5.2.1. Component capacities shall be in accordance with Section 7.5.2.2 and Chapters 8 through 12.

5.2.6 Knowledge Factor The knowledge factor, κ, shall be 0.75 unless data collection complies with the requirements for a knowledge factor of 0.9 or1.0 in accordance with Section 6.2.3.1.

5.3 TIER 2 DEFICIENCY-BASED EVALUATION REQUIREMENTS

The Tier 2 evaluation procedure shall consist of an evaluation in accordance with Sections 5.4 through 5.7 for the structural systems or components identified as "Noncompliant" or "Unknown" based on the Tier 1 screening checklists. The analysis shall be as required to determine the demands and capacities of all structural systems, components, and connections associated with the potential deficiency.

5.4 PROCEDURES FOR BASIC CONFIGURATION OF BUILDING SYSTEMS

This section provides Tier 2 deficiency-based evaluation procedures that apply to the Basic Configuration Checklists in Section 17.1.2.

5.4.1 General

5.4.1.1 Load Path No Tier 2 deficiency-based evaluation procedure is available for buildings without a compliant load path.

5.4.1.2 Adjacent Buildings An analysis should be performed in accordance with Section 5.2.4 to determine the drifts in the structure being evaluated. The drifts in the adjacent structures should be estimated using available information about the adjacent structure and the analysis procedures of this standard. Alternatively, it shall be permitted to assume that the adjacent building drift is 3% of the height of the diaphragm level under consideration. The square root of the sum of the squares combination of the drifts shall be less than the total separation at each diaphragm level. Buildings that have similar structural systems, have matching diaphragms, and do not differ in height by more than 50% of the height of the shorter building need not comply with this statement for the Life Safety Performance Level provided that impact between the two structures does not damage the facade or cladding of the building in such a manner as to create a Life Safety falling hazard.

5.4.1.3 Mezzanines The load path of the mezzanine to the main seismic-force-resisting system shall be identified. The adequacy of the load path shall be evaluated in accordance with Section 5.2.4. The adequacy of the elements of the main structure connected to the mezzanine shall be evaluated considering the magnitude and location of the mezzanine forces imparted on the main structure.

5.4.2 Building Configuration

5.4.2.1 Weak Story Irregularity An analysis shall be performed in accordance with Section 5.2.4, and the ability of the elements in the seismic-force-resisting system shall be evaluated to resist calculated demands, using Section 5.2.5 with m-factors from the appropriate material chapter, except that m-factors shall be divided by $(n + 1)$ where n is the number of stories above the story being considered. The m-factor need not be less than 1.

5.4.2.2 Soft Story Irregularity An analysis shall be performed in accordance with Section 5.2.4 using the linear dynamic procedure. The adequacy of all elements of the seismic-force-resisting system shall be evaluated in the noncompliant stories in accordance with Section 5.2.5. In addition, all gravity-load-carrying elements shall be evaluated considering the story drift. The building is deemed compliant with this statement if all elements in the noncompliant stories meet the acceptance criteria.

5.4.2.3 Vertical Irregularities An analysis shall be performed in accordance with Section 5.2.4, and the demand-capacity ratio (DCR) shall be determined in accordance with Section 7.3.1.1, for all elements of the seismic-force-resisting system in the noncompliant stories. The adequacy of the elements and connections below the vertical discontinuities shall be evaluated in accordance with Section 5.2.5 as force-controlled elements. The adequacy of struts and diaphragms to transfer loads to adjacent seismic-force-resisting elements as force-controlled elements shall be evaluated.

5.4.2.4 Geometric Irregularity An analysis shall be performed in accordance with Section 5.2.4 using the linear dynamic procedure. The adequacy of the seismic-force-resisting elements shall be evaluated in accordance with Section 5.2.5.

5.4.2.5 Mass Irregularity An analysis shall be performed in accordance with Section 5.2.4 using the linear dynamic procedure. The adequacy of the seismic-force-resisting elements shall be evaluated in accordance with Section 5.2.5.

5.4.2.6 Torsion Irregularity An analysis of the entire structure shall be performed in accordance with Section 5.2.4, including the effects of horizontal torsion. The adequacy of the seismic-force-resisting system, including the effects of horizontal torsion, shall be evaluated in accordance with Section 5.2.5. In addition, all vertical-load-carrying elements shall be adequate for their gravity loads combined with forces associated with story displacements that include torsion and P-delta effects.

5.4.3 Geologic Site Hazards and Foundation Components

5.4.3.1 Geologic Site Hazards No Tier 2 evaluation procedure is available for buildings subjected to liquefaction, slope failure, or surface fault rupture. The structure shall be evaluated for the effects of these hazards using the Tier 3 procedures in Chapters 6 and 8.

5.4.3.2 Foundation Performance The magnitude of differential movement in the foundation shall be evaluated, and an analysis of the building in accordance with Section 5.2.4 shall be performed. The adequacy of the structure shall be evaluated in accordance with Section 5.2.5 for all gravity loads and seismic forces in combination with the forces induced by the potential differential movement of the foundation.

5.4.3.3 Overturning An analysis shall be performed in accordance with Section 5.2.4. The adequacy of the foundation, including all gravity and seismic overturning forces, shall be evaluated in accordance with Section 5.2.5.

5.4.3.4 Ties between Foundation Elements The magnitude of differential movement in the foundation shall be evaluated, and an analysis of the building in accordance with Section 5.2.4 shall be performed. The adequacy of the structure shall be evaluated in accordance with Section 5.2.5 for all gravity and seismic forces in combination with the forces induced by the potential differential movement of the foundation.

5.5 PROCEDURES FOR SEISMIC-FORCE-RESISTING SYSTEMS

This section provides Tier 2 deficiency-based evaluation procedures that apply to all noncompliant seismic-force-resisting systems checklist evaluation statements.

5.5.1 General

5.5.1.1 Redundancy An analysis of the structure shall be performed in accordance with Section 5.2.4, and the adequacy of all elements and connections of the seismic-force-resisting system shall be evaluated for all noncompliant stories, in accordance with Section 5.2.5.

5.5.2 Procedures for Moment Frames

5.5.2.1 General Procedures for Moment Frames

5.5.2.1.1 Interfering Walls Where concrete and masonry walls are not isolated from moment-frame elements, an analysis shall be performed in accordance with Section 5.2.4 to compute the demands imparted by the structure to the interfering walls and the demands induced on the frame elements. The adequacy of the interfering walls and the frame to resist the induced forces shall be evaluated in accordance with Section 5.2.5.

5.5.2.1.2 Drift Check An analysis shall be performed in accordance with Section 5.2.4, and the adequacy of the moment-frame and slab–column frame elements, including P-delta effects and their associated connections, shall be evaluated in accordance with Section 5.2.5.

5.5.2.1.3 Axial Stress Check An analysis in accordance with Section 5.2.4 shall be performed. The gravity and overturning demands for noncompliant columns shall be calculated, and the adequacy of the columns to resist overturning forces shall be evaluated in accordance with Section 5.2.5.

5.5.2.1.4 Shear Stress Check An analysis in accordance with Section 5.2.4 shall be performed. The adequacy of the frame elements shall be evaluated in accordance with Section 5.2.5.

5.5.2.1.5 Strong Column–Weak Beam An analysis shall be performed in accordance with Section 5.2.4. If the percentage of strong column–weak beam joints in any story line of moment-resisting frames is greater than 50% of the joints, the ability of the columns at strong column–weak beam joints in that frame shall be evaluated to resist calculated demands, using Section 5.2.5 with m-values from the appropriate material chapter. If the percentage of strong column–weak beam joints in any story of any line of moment-resisting frames is less than 50% of the joints, the ability of all the columns in each noncompliant frame shall be evaluated to resist calculated demands in all noncompliant stories, using m-factors from the appropriate material chapter, except that m-values shall be divided by $(n + 1)$ where n is the number of stories above the story being considered. The m-factor need not be less than 1.

5.5.2.2 Procedures for Steel Moment Frames

5.5.2.2.1 Moment-Resisting Connections The demands on the noncompliant connections shall be computed in accordance with Section 5.2.4, and the connections shall be evaluated in accordance with Section 5.2.5.

5.5.2.2.2 Flexural Stress Check An analysis in accordance with Section 5.2.4 shall be performed. The adequacy of the frame elements shall be evaluated in accordance with Section 5.2.5.

5.5.2.2.3 Panel Zones The demands in noncompliant joints shall be calculated in accordance with Section 5.2.4, and the adequacy of the panel zones for web shear shall be evaluated in accordance with Section 5.2.5.

5.5.2.2.4 Column Splices The gravity and seismic demands shall be calculated in accordance with Section 5.2.4, and the adequacy of the splice connections shall be evaluated in accordance with Section 5.2.5.

5.5.2.2.5 Compact Members An analysis shall be performed in accordance with Section 5.2.4, and the adequacy of all noncompliant beams and columns that are part of a moment frame shall be evaluated in accordance with Section 5.2.5.

5.5.2.2.6 Beam Penetrations The shear and flexural demands on noncompliant beams shall be calculated in accordance with Section 5.2.4. The adequacy of the beams considering the strength around the penetrations shall be evaluated in accordance with Section 5.2.5.

5.5.2.2.7 Girder Flange Continuity Plates Forces in column flanges caused by flexure in the beam shall be calculated in accordance with Section 5.2.4. The adequacy of the column flange as a force-controlled component to transfer girder flange forces to the panel zone without continuity plates shall be evaluated in accordance with Section 5.2.5. Additionally, the adequacy of the moment frame beam-to-column connection shall be evaluated in accordance with Section 5.2.5 with the modification of the connection acceptance criteria per Chapter 9, Section 9.4.2.4.2.

5.5.2.2.8 Out-of-Plane Bracing at Beam–Column Joints An analysis shall be performed in accordance with Section 5.2.4, and the adequacy of the columns at the noncompliant joint to resist buckling between points of support for all gravity and seismic actions concurrent with a horizontal out-of-plane force equal to 6% of the critical column flange compression force applied at the noncompliant joint shall be evaluated in accordance with Section 5.2.5.

5.5.2.2.9 Bottom Flange Bracing An analysis shall be performed in accordance with Section 5.2.4. The adequacy of the beams shall be evaluated considering the potential for lateral-torsional buckling of the bottom flange between points of lateral support in accordance with Section 5.2.5.

5.5.2.3 Procedures for Concrete Moment Frames

5.5.2.3.1 Flat Slab Frames An analysis shall be performed in accordance with Section 5.2.4. The adequacy of the slab–column system for resisting seismic forces and punching shear shall be evaluated in accordance with Section 5.2.5.

5.5.2.3.2 Prestressed Frame Elements An analysis shall be performed in accordance with Section 5.2.4. The adequacy of the concrete frame, including prestressed elements, shall be evaluated in accordance with Section 5.2.5.

5.5.2.3.3 Captive Column Conditions The adequacy of the columns shall be evaluated for the shear force required to develop the moment capacity at the top and the bottom of the clear height of the column. Alternatively, an analysis shall be performed in accordance with Section 5.2.4, and the columns shall be evaluated as force-controlled elements in accordance with Section 5.2.5.

5.5.2.3.4 No Shear Failures The shear demands shall be calculated for noncompliant members in accordance with Section 5.2.4, and the adequacy of the members for shear shall be evaluated in accordance with Section 5.2.5.

5.5.2.3.5 Continuous Beam Bars The flexural demands shall be calculated at the ends and the middle of noncompliant beams in accordance with Section 5.2.4, and the adequacy of the beams using an *m*-factor equal to 1.0 shall be evaluated in accordance with Section 5.2.5.

5.5.2.3.6 Column and Beam Bar Splices The flexural demands at noncompliant beam and column splices shall be calculated in accordance with Section 5.2.4, and the adequacy of the beams and columns shall be evaluated in accordance with Section 5.2.5.

5.5.2.3.7 Column-Tie Spacing and Beam Stirrup Spacing The force demands in noncompliant beams and columns shall be calculated in accordance with Section 5.2.4, and the adequacy of the elements shall be evaluated in accordance with Section 5.2.5.

5.5.2.3.8 Joint Reinforcing The joint shear demands shall be calculated in accordance with Section 5.2.4, and the adequacy of the joint to develop the adjoining members' forces shall be evaluated in accordance with Section 5.2.5.

5.5.2.3.9 Joint Eccentricity The joint shear demands, including additional shear stresses from joint torsion, shall be calculated in accordance with Section 5.2.4, and the adequacy of the beam–column joints shall be evaluated in accordance with Section 5.2.5.

5.5.2.3.10 Stirrup and Tie Hooks The shear and axial demands in noncompliant members shall be calculated in accordance with Section 5.2.4, and the adequacy of the beams and columns shall be evaluated in accordance with Section 5.2.5.

5.5.2.4 Procedures for Precast Concrete Moment Frames For noncompliant Tier 1 statements related to precast concrete frame elements and connections, an analysis shall be performed in accordance with Section 5.2.4, and the adequacy of the precast frame elements or connections as force-controlled elements shall be evaluated in accordance with Section 5.2.5.

5.5.2.5 Procedures for Frames Not Part of the Seismic-Force-Resisting System

5.5.2.5.1 Complete Frames An analysis shall be performed in accordance with Section 5.2.4, and the shear walls for the combined gravity and seismic demands shall be evaluated in accordance with Section 5.2.5.

5.5.2.5.2 Deflection Compatibility An analysis shall be performed in accordance with Section 5.2.4, and the adequacy of all secondary components, including moment-frame elements and connections for the flexure and shear demands at the maximum interstory drifts for all noncompliant elements, shall be evaluated in accordance with Section 5.2.5.

5.5.2.5.3 Flat Slabs An analysis shall be performed in accordance with Section 5.2.4, and the column–slab joints for punching shear and shear transfer caused by moments at the maximum interstory drifts shall be evaluated in accordance with Section 5.2.5.

5.5.3 Procedures for Shear Walls

5.5.3.1 General Procedures for Shear Walls

5.5.3.1.1 Shear Stress Check An analysis shall be performed in accordance with Section 5.2.4, and the adequacy of the shear wall elements in the noncompliant stories and in any stories below a noncompliant story shall be evaluated in accordance with Section 5.2.5.

5.5.3.1.2 Wall Thickness and Proportions An analysis shall be performed in accordance with Section 5.2.4, and the adequacy of shear walls shall be evaluated in accordance with Section 5.2.5, including the adequacy to resist out-of-plane forces in combination with vertical loads.

5.5.3.1.3 Reinforcement Steel An analysis shall be performed in accordance with Section 5.2.4, and the adequacy of all noncompliant shear walls shall be evaluated in accordance with Section 5.2.5.

5.5.3.1.4 Overturning The overturning demands for noncompliant walls shall be calculated in accordance with Section 5.2.4, and the adequacy of all noncompliant shear walls shall be evaluated in accordance with Section 5.2.5.

5.5.3.1.5 Reinforcement at Openings The flexural and shear demands around all noncompliant shear walls shall be calculated in accordance with Section 5.2.4, and the adequacy of the piers and spandrels shall be evaluated in accordance with Section 5.2.5.

5.5.3.2 Procedures for Concrete Shear Walls

5.5.3.2.1 Coupling Beams An analysis shall be performed in accordance with Section 5.2.4, and the adequacy for flexure and shear of all noncompliant coupling beams shall be evaluated in accordance with Section 5.2.5. If the coupling beams are inadequate, the adequacy of the coupled walls shall be evaluated as if they are independent walls.

5.5.3.2.2 Confinement Reinforcement The shear and flexural demands on the noncompliant walls shall be calculated in accordance with Section 5.2.4, and the adequacy of the shear walls shall be evaluated in accordance with Section 5.2.5.

5.5.3.2.3 Wall Connections The shear and flexural demands on the shear walls shall be calculated in accordance with Section 5.2.4, and the adequacy of the connection to transfer shear between the walls and the steel frame shall be evaluated in accordance with Section 5.2.5.

5.5.3.2.4 Column Splices The tension demands caused by overturning forces on noncompliant columns shall be calculated in accordance with Section 5.2.4, and the adequacy of the splice connections shall be evaluated in accordance with Section 5.2.5.

5.5.3.3 Procedures for Precast Concrete Shear Walls

5.5.3.3.1 Wall Openings The adequacy of the remaining wall shall be evaluated for shear and overturning forces determined in accordance with Section 5.2.4, and the adequacy of the shear transfer connection between the diaphragm and the wall shall be evaluated in accordance with Section 5.2.5. The adequacy of the connection between any collector elements and the wall also shall be evaluated in accordance with Section 5.2.5.

5.5.3.3.2 Corner Openings An analysis shall be performed in accordance with Section 5.2.4. The adequacy of the diaphragm to transfer shear and spandrel panel forces to the remainder of the wall beyond the opening shall be evaluated in accordance with Section 5.2.5.

5.5.3.3.3 Panel-to-Panel Connections An analysis shall be performed in accordance with Section 5.2.4, and the adequacy of the welded inserts to transfer overturning forces as force-controlled elements shall be evaluated in accordance with Section 5.2.5. Alternatively, the panels shall be evaluated as independent elements without consideration of coupling between panels.

5.5.3.4 Procedures for Unreinforced Masonry Shear Walls

5.5.3.4.1 Masonry Layup When filled collar joints of multiwythe masonry walls have voids, an analysis shall be performed in accordance with Section 5.2.4, and the adequacy for in-plane shear demands shall be evaluated using only the inner wythe, or wythes when more than two wythes are present, of the wall for capacity. For out-of-plane demands, evaluate each wythe independently. Evaluate the anchorage of the outer wythe as a veneer in accordance with Chapter 13.

5.5.3.5 Procedures for Infill Walls in Frames

5.5.3.5.1 Wall Connections The out-of-plane demands on the wall shall be calculated in accordance with Section 5.2.4, and the adequacy of the connection to the frame shall be evaluated in accordance with Section 5.2.5.

5.5.3.5.2 Cavity Walls When infill walls are of cavity construction, an analysis shall be performed in accordance with Section 5.2.4, and the adequacy for in-plane shear demands using only the inner wythe of the wall for capacity shall be evaluated in accordance with Section 5.2.5. For out-of-plane demands, each wythe shall be evaluated independently. The anchorage of the outer wythe as a veneer shall be evaluated in accordance with Chapter 13.

5.5.3.5.3 Masonry Infill Walls When the infill wall does not extend to the soffit of the frame beam, the capacity of columns adjacent to nonconforming walls shall be evaluated for the shear force required to develop the flexural capacity of the column over the clear height above the infill. If the infill does not extend to columns, the beam shall be evaluated for the shear force required to develop the flexural capacity of the beam between the infill panel and the column.

5.5.3.6 Procedures for Walls in Wood Frame Buildings

5.5.3.6.1 Stucco, Gypsum Wallboard, Plaster, or Narrow Shear Walls The overturning and shear demands for noncompliant walls shall be calculated in accordance with Section 5.2.4, and the adequacy shall be evaluated in accordance with Section 5.2.5.

5.5.3.6.2 Shear Walls Connected through Floors The overturning and shear demands for noncompliant walls shall be calculated in accordance with Section 5.2.4, and the adequacy of the structure to transfer forces through the floors shall be evaluated in accordance with Section 5.2.5.

5.5.3.6.3 Hillside Site Conditions An analysis shall be performed in accordance with Section 5.2.4, and the shear and overturning demands on the shear walls, including torsion effects of the hillside, shall be calculated. The adequacy of the shear walls shall be evaluated in accordance with Section 5.2.5.

5.5.3.6.4 Cripple Walls The shear demand for noncompliant walls shall be calculated in accordance with Section 5.2.4, and the adequacy of the walls shall be evaluated in accordance with Section 5.2.5.

5.5.3.6.5 Openings The overturning and shear demands on noncompliant walls shall be calculated in accordance with Section 5.2.4, and the adequacy of the shear walls shall be evaluated in accordance with Section 5.2.5.

5.5.3.6.6 Hold-Down Anchors The overturning and shear demands for noncompliant walls shall be calculated in accordance with Section 5.2.4, and the adequacy of the shear walls shall be evaluated in accordance with Section 5.2.5.

5.5.3.7 Procedures for Cold-Formed Steel Light-Frame Construction, Shear Wall Systems

5.5.3.7.1 Stucco, Gypsum Wallboard, Plaster, or Narrow Shear Walls The overturning and shear demands for noncompliant shear walls shall be calculated in accordance with Section 5.2.4, and the adequacy shall be evaluated in accordance with Section 5.2.5.

5.5.3.7.2 Shear Walls Connected through Floors The overturning and shear demands for noncompliant shear walls shall be calculated in accordance with Section 5.2.4, and the adequacy of the structure to transfer forces through the floors shall be evaluated in accordance with Section 5.2.5.

5.5.3.7.3 Hillside Site Conditions An analysis shall be performed in accordance with Section 5.2.4, and the shear and overturning demands on the shear walls, including torsion effects of the hillside, shall be calculated. The adequacy of the shear walls shall be evaluated in accordance with Section 5.2.5.

5.5.3.7.4 Cripple Walls The shear demand for noncompliant shear walls shall be calculated in accordance with Section 5.2.4, and the adequacy of the shear walls shall be evaluated in accordance with Section 5.2.5.

5.5.3.7.5 Openings The overturning and shear demands on noncompliant shear walls shall be calculated in accordance with Section 5.2.4, and the adequacy of the shear walls shall be evaluated in accordance with Section 5.2.5.

5.5.3.7.6 Hold-Down Anchors The overturning and shear demands for noncompliant shear walls shall be calculated in accordance with Section 5.2.4, and the adequacy of the shear walls shall be evaluated in accordance with Section 5.2.5.

5.5.4 Procedures for Braced Frames

5.5.4.1 Axial Stress Check An analysis shall be performed in accordance with Section 5.2.4. The adequacy of the braced frame elements shall be evaluated in accordance with Section 5.2.5.

5.5.4.2 Column Splices The tension demands on noncompliant columns shall be calculated in accordance with Section 5.2.4, and the adequacy of the splice connections shall be evaluated in accordance with Section 5.2.5.

5.5.4.3 Slenderness of Diagonals The compression demands in noncompliant braces shall be calculated in accordance with Section 5.2.4, and the adequacy of the braces shall be evaluated for buckling in accordance with Section 5.2.5.

5.5.4.4 Connection Strength The demands on the noncompliant connections shall be calculated in accordance with Section 5.2.4, and the adequacy of the brace connections shall be evaluated in accordance with Section 5.2.5.

5.5.4.5 Out-of-Plane Restraint for Braced Frames An analysis shall be performed in accordance with Section 5.2.4, and the adequacy of the noncompliant beam for all gravity and seismic actions concurrent with a horizontal out-of-plane force equal to 2% of the brace compression force applied at the bottom flange of the beam shall be evaluated in accordance with Section 5.2.5.

5.5.4.6 K-Bracing and Chevron-Bracing Configurations An analysis shall be performed in accordance with Section 5.2.4, and the adequacy of all beams and columns, including the concurrent application of unbalanced forces resulting from the tensile strength of one brace, assuming that the other brace has buckled in compression, shall be evaluated.

5.5.4.7 Tension-Only Braces An analysis shall be performed in accordance with Section 5.2.4, and the adequacy of the tension-only braces shall be evaluated in accordance with Section 5.2.5.

5.5.4.8 Concentrically Braced Frame Joints An analysis shall be performed in accordance with Section 5.2.4. The axial, flexural, and shear demands, including the demands caused by eccentricity of the braces, shall be calculated. The adequacy of the joints shall be evaluated in accordance with Section 5.2.5.

5.5.4.9 Procedures for Cold-Formed Steel Light-Frame Construction, Strap-Braced Wall Systems

5.5.4.9.1 Narrow Cold-Formed Steel Strap-Braced Walls The overturning and shear demands for noncompliant cold-formed steel strap-braced walls shall be calculated in accordance with Section 5.2.4, and the adequacy shall be evaluated in accordance with Section 5.2.5.

5.5.4.9.2 Cold-Formed Steel Strap-Braced Walls Connected through Floors The overturning and shear demands for non compliant cold-formed steel strap-braced walls shall be calculated in accordance with Section 5.2.4, and the adequacy of the structure to transfer forces through the floors shall be evaluated in accordance with Section 5.2.5.

5.5.4.9.3 Hillside Site Conditions An analysis shall be performed in accordance with Section 5.2.4, and the shear and overturning demands on the cold-formed steel strap-braced walls, including torsion effects of the hillside, shall be calculated. The adequacy of the cold-formed steel strap-braced walls shall be evaluated in accordance with Section 5.2.5.

5.5.4.9.4 Hold-Down Anchors The overturning and shear demands for noncompliant cold-formed steel strap-braced walls shall be calculated in accordance with Section 5.2.4, and the adequacy of the cold-formed steel strap-braced walls shall be evaluated in accordance with Section 5.2.5.

5.5.4.9.5 Chord Stud Axial Stress An analysis shall be performed in accordance with Section 5.2.4, and the overturning demands on the end stud which the strap is attached to should be calculated. The adequacy of the end stud for the calculated loads shall be evaluated as a force-controlled action in accordance with Section 5.2.5.

5.5.4.9.6 Strap Brace Detailing An analysis of the building shall be performed in accordance with Section 5.2.4 excluding walls that do not conform to AISI S400 requirements for cold-formed steel strap tightness and strap-to-intermediate stud connection. The shear and overturning demands on the conforming cold-formed steel strap-braced walls shall be calculated.

5.6 PROCEDURES FOR DIAPHRAGMS

This section provides Tier 2 deficiency-based evaluation procedures that apply to all noncompliant diaphragm checklist evaluation statements.

5.6.1 General Procedures for Diaphragms

5.6.1.1 Diaphragm and Roof Chord Continuity The load path around the discontinuity shall be identified. The diaphragm shall be analyzed for the forces in accordance with Section 5.2.4, and the adequacy of the elements in the load path shall be evaluated in accordance with Section 5.2.5.

5.6.1.2 Diaphragm Cross Ties The out-of-plane forces shall be calculated in accordance with Section 7.2.13, and the adequacy of the existing connections, including development of the forces into the diaphragm, shall be evaluated in accordance with Section 5.2.5.

5.6.1.3 Openings in Diaphragms at Shear Walls, Braced Frames, and Moment Frames The diaphragm forces shall be calculated in accordance with Section 5.2.4, and the adequacy of the diaphragm to transfer the loads to the wall or frames, considering the available length and the presence of any drag struts, shall be evaluated in accordance with Section 5.2.5. For concrete and masonry walls, the adequacy of the wall and diaphragm connections to resist out-of-plane forces with the wall spanning out-of-plane between points of anchorage shall be evaluated in accordance with Section 5.2.5.

5.6.1.4 Plan Irregularities in Diaphragms The chord and collector demands at locations of plan irregularities shall be calculated by analyzing the diaphragm in accordance with Section 5.2.4. It shall be permitted to consider the relative movement of the projecting wings of the structure by applying the static base shear, assuming that each wing moves in the same direction or each wing moves in opposing directions, whichever is more severe. The adequacy of all elements that can contribute to the tensile capacity at the location of the irregularity shall be evaluated in accordance with Section 5.2.5.

5.6.1.5 Diaphragm Reinforcement at Openings The shear and flexural demands at major openings shall be calculated, and the resulting chord forces shall be determined in accordance with Section 5.2.4. The adequacy of the diaphragm elements to transfer forces around the opening shall be evaluated in accordance with Section 5.2.5.

5.6.2 Procedures for Wood Diaphragms For wood diaphragms with noncompliant spans or aspect ratios, an analysis of the entire diaphragm at each noncompliant level shall be performed in accordance with Section 5.2.4, and the adequacy of the diaphragm system shall be evaluated in accordance with Section 5.2.5. The diaphragm deflection shall be calculated, and the adequacy of the vertical-load-carrying elements at the maximum deflection, including P-delta effects, shall be evaluated.

5.6.3 Procedures for Steel Deck Diaphragms For diaphragms with noncompliant spans or aspect ratios consisting of bare steel deck or steel deck diaphragms with fill other than reinforced structural concrete, an analysis of the entire diaphragm shall be performed at each noncompliant level in accordance with Section 5.2.4, and the adequacy of the diaphragm system shall be evaluated in accordance with Section 5.2.5. The diaphragm deflection shall be calculated, and the adequacy of the vertical-load-carrying elements at the maximum deflection, including P-delta effects, shall be evaluated.

5.6.4 Procedures for Precast Concrete Diaphragms Noncompliant precast concrete diaphragms shall be evaluated for the forces determined in accordance with Section 5.2.4. The adequacy of the slab element interconnection and the shear capacity shall be evaluated in accordance with Section 5.2.5.

5.6.5 Diaphragms Other Than Wood, Steel Deck, Concrete, or Horizontal Bracing An analysis of the diaphragm system shall be performed in accordance with Section 5.2.4, and the adequacy of the system shall be evaluated in accordance with Section 5.2.5 or using available reference standards for the capacity of the diaphragm not covered by this standard.

5.7 PROCEDURES FOR CONNECTIONS

This section provides Tier 2 deficiency-based evaluation procedures that apply to all noncompliant connection checklist evaluation statements.

5.7.1 Anchorage for Normal Forces

5.7.1.1 Wall Anchorage Where the wall anchorage is noncompliant with the Tier 1 Quick Check procedure, a more detailed analysis of the wall anchorage system may be performed in accordance with Sections 5.2.4 and 5.2.5 to demonstrate compliance. Alternatively, the adequacy of non-load-bearing walls to span between points of anchorage may be evaluated.

5.7.1.2 Stiffness of Wall Anchors The amount of relative movement possible given the existing connection configuration shall be determined. The impact of this movement shall be evaluated by analyzing the elements of the connection for forces induced by the maximum potential movement.

5.7.1.3 Wood Ledgers with Cross-Grain Bending No Tier 2 procedure is available to demonstrate compliance of wood ledgers loaded in cross-grain bending.

5.7.1.4 Precast Concrete Panel Connections The stability of the wall panels for the out-of-plane forces in accordance with Section 5.2.4 shall be evaluated. The adequacy of the existing connections to deliver all forces into the diaphragm, including moments caused by eccentricities between the panel center of mass and points of anchorage, shall be evaluated.

5.7.2 Connections for Shear Transfer The diaphragm and wall demands shall be calculated in accordance with Section 5.2.4, and the adequacy of the connection to transfer the demands to shear walls, steel frames, or infill frames shall be evaluated in accordance with Section 5.2.5.

5.7.3 Connections for Vertical Elements

5.7.3.1 Steel and Concrete Columns The column demands, including any axial load caused by overturning, shall be calculated in accordance with Section 5.2.4, and the adequacy of the connection to transfer the demands to the foundation shall be evaluated in accordance with Section 5.2.5.

5.7.3.2 Shear Wall Boundary Columns Shear wall demands shall be determined in accordance with Section 5.2.4. The overturning resistance of the shear wall considering the dead load above the foundation and the portion of the foundation dead load that can be activated by the boundary column anchorage connection shall be evaluated in accordance with Section 5.2.5.

5.7.3.3 Wood or Cold-Formed Steel Posts and Wood Sills and Cold-Formed Steel Base Tracks No Tier 2 evaluation procedure is available for posts without positive connections to the foundation. For wood sills or cold-formed steel base tracks, it shall be permitted to evaluate the adequacy of alternate methods of shear attachment for seismic forces determined in accordance with Section 5.2.4.

5.7.3.4 Concrete Walls, Precast Wall Panels, and Other Wall Panels The wall demands shall be calculated in accordance with Section 5.2.4, and the adequacy of any load path to transfer the demands to the foundation shall be evaluated in accordance with Section 5.2.5.

5.7.3.5 Uplift at Pile Caps The axial forces caused by overturning and shear demands at the pile cap shall be calculated in accordance with Section 5.2.4, and the adequacy of the pile cap reinforcement and pile connections to transfer uplift forces to the piles shall be evaluated in accordance with Section 5.2.5.

5.7.4 Interconnection of Elements

5.7.4.1 Girder–Column Connection No Tier 2 procedure is available to demonstrate compliance of girder–column connections found noncompliant.

5.7.4.2 Girders Supported by Walls or Pilasters A determination shall be made as to whether the girder connection at the pilaster is required to resist wall out-of-plane forces. The adequacy of the connection to resist the anchorage forces in accordance with Section 5.2.4 shall be determined and shall be evaluated in accordance with Section 5.2.5.

5.7.4.3 Corbel Bearing and Connections The story drift shall be calculated in accordance with Section 5.2.4. For bearing length noncompliance, the bearing length shall be sufficient to provide support for the girders at maximum drift. The adequacy of the bearing support for all loads, including any additional eccentricity at maximum drift, shall be evaluated in accordance with Section 5.2.5. For welded connection noncompliance, the force in the welded connections induced by the story drift shall be calculated. The adequacy of the connections to resist these forces shall be evaluated. Calculated overstresses in these connections shall not jeopardize the vertical support of the girders or the seismic-force-resisting system.

5.7.4.4 Beam, Girder, and Truss Supported on Unreinforced Masonry (URM) Walls or URM Pilasters No Tier 2 procedure is available to demonstrate compliance of beams, girders, or trusses without a secondary load path.

5.7.5 Roof and Wall Panel Connections The panel demands shall be calculated in accordance with Section 5.2.4, and the adequacy of the panels to transfer the demands to the framing shall be evaluated in accordance with Section 5.2.5.

5.8 TIER 2 DEFICIENCY-BASED RETROFIT REQUIREMENTS

Where a Tier 2 deficiency-based retrofit is to be performed to achieve compliance with the selected performance objective(s), deficiencies identified by a Tier 1 screening or Tier 2 evaluation shall be mitigated by implementation of retrofit measures in accordance with this standard. The proposed retrofit measures shall satisfy the requirements of Sections 5.8.1 through 5.8.3. The scope of retrofit measures shall conform with Section 5.8.4.

5.8.1 Compliance with Deficiency-Based Evaluation The resulting building, including strengthening measures, shall conform to a Common Building Type and to the limitations for use of the Tier 1 and Tier 2 evaluation procedures of Section 3.5.1. A combination of common building types shall be permitted if the provisions of Section 3.5.1.2 are satisfied for the retrofitted building. It shall be permitted to waive the requirement of flexible diaphragms of Section 3.5.1.2.2.1. The retrofitted building shall comply with Tier 2 evaluation requirements for all statements identified in the original building as nonconforming based on a Tier 1 screening and Tier 2 evaluations.

If the modifications to the building for the retrofit change the original building from one Common Building Type to another or creates a combination of Common Building Types different from the original building, the resulting building shall satisfy the limitations of Section 3.5.1 for the use of Tier 2 procedures.

5.8.2 Additional Evaluation of the Resulting Building

5.8.2.1 Building Configuration If the retrofit creates a building configuration irregularity consisting of a Weak Story, Soft Story, Vertical Irregularity, Geometry, Mass, or Torsion condition as defined by noncompliance with the Building Configuration statements in Tables 17-2 and 17-3, the resulting building shall conform with the Tier 2 evaluation procedures in Section 5.4.2.

5.8.2.2 Increased Gravity Demands to Existing Elements The seismic retrofit measures shall not increase the gravity load demands to existing structural elements and foundations by more than 5% or reduce the capacity of existing structural elements or foundations to resist gravity loads unless it is demonstrated that the component complies with the applicable building code requirements for gravity loads.

5.8.2.3 Increased Seismic Demands to Existing Elements If the retrofit increases the seismic demands on any existing structural element, connection, or foundation in the seismic-force-resisting system by more than 10% as a result of added seismic mass or a change in seismic load path, such elements shall be demonstrated to be in compliance with Sections 5.2.4 and 5.2.5.

5.8.3 Evaluation of New and Modified Structural Elements and Connections A Tier 2 analysis and evaluation shall be performed as necessary to demonstrate the adequacy of all new structural elements, connections, and foundations added and all existing structural elements and connections modified as part of the retrofit. Tier 2 analysis methods of Section 5.2.4 and acceptance criteria of Section 5.2.5 shall be used in conjunction with the procedures in Sections 5.4 through 5.7.

5.8.4 Retrofit-Specific Requirements

5.8.4.1 General In addition to compliance with Tier 2 evaluation procedures, the retrofit measures shall conform with this section.

5.8.4.2 Design and Detailing Requirements New elements added to the seismic-force-resisting system of a retrofitted building shall conform with the following requirements:

1. New elements and systems shall conform with the Retrofit Measures requirements in Chapters 9 through 12.
2. New deformation-controlled components in the vertical elements of the seismic-force-resisting system shall be designed and detailed such that the corresponding element *m*-factor determined in accordance with Chapters 9 through 12 for a primary element and Collapse Prevention performance is no less than 2.0.
3. Regardless of the level of detailing and ductility, new deformation-controlled components in the vertical elements of the seismic-force-resisting system, and within the scope of the Tier 2 evaluation, shall have strength to achieve the required acceptance criteria using an *m*-factor that is not more than two times the lowest *m*-factor of the deformation-controlled components in the vertical primary elements of the seismic-force-resisting system with which they share load for the selected performance objective(s).
4. Connections between new elements and between new and existing elements in the seismic-force-resisting system shall be designed to meet force-controlled acceptance criteria for the selected performance objective(s) or otherwise meet acceptance criteria in accordance with the applicable Chapter 9 through 12.

5.8.4.3 Scope of Evaluation Requirements for Existing Components Existing elements of the seismic-force-resisting system of the resulting building as listed next shall be evaluated and demonstrated to be compliant with the selected performance objective(s) using the Tier 2 analysis methods of Section 5.2.4 and acceptance criteria of Section 5.2.5. The minimum scope of evaluation, in addition to the requirements elsewhere in this section, shall include the following:

1. Existing beams, columns, and connections that form part of new braced frame, new moment frame, or new shear wall systems.
2. Existing collectors, collector connections, and the collector connection to the diaphragms along the line of new and modified braced frames, moment frames, or shear walls. Connections are to be evaluated as force-controlled elements.
3. Existing diaphragm shear demands and connections to a line of new and modified braced frames, moment frames, or shear walls.
4. Existing columns below discontinuous braced frames, moment frames, and shear walls shall be evaluated as force-controlled elements.
5. Existing diaphragm or horizontal elements that transfer forces between vertical elements when new braced frames, moment frames, and shear walls create a horizontal offset.

CHAPTER 6
TIER 3 SYSTEMATIC EVALUATION AND RETROFIT

6.1 SCOPE

This chapter sets forth the requirements and procedures for performing Tier 3 systematic evaluations and retrofits. These procedures are to be used where systematic procedures are required in accordance with Chapter 3 and may be used as a further investigation of buildings where the deficiency-based evaluation procedures have been used.

Section 6.2 provides data collection requirements that are in addition to those in Section 3.2, and provisions to define the member capacities based on the available information about the building. Sections 6.3 and 6.4 provide requirements for Tier 3 evaluation and retrofit, respectively.

6.2 DATA COLLECTION REQUIREMENTS

Investigation of as-built conditions and data collection requirements shall be in accordance with Section 3.2 and the requirements of this section. Data shall be obtained from available drawings, specifications, and other documents for the existing construction. Where required by the provisions in this standard, data collected from available documents shall be supplemented and verified by on-site investigations, including nondestructive examination, and testing of building materials and components.

Data on the as-built condition of the structure, nonstructural components, site, and adjacent buildings shall be collected to perform the selected analysis procedure. The extent of data collected shall be in accordance with Sections 6.2.1, 6.2.2, and 6.2.3.

6.2.1 Construction Documentation Construction documents shall provide enough detailed information about the as-built conditions to carry out the selected analysis procedure in Chapter 7 or Chapter 13. At a minimum, the construction documents shall consist of design drawings or other sources of information that define the configuration of the gravity load supporting elements and seismic-force-resisting system, including foundation system, and identification of the member geometry, section properties, and material strengths and stiffnesses of existing structural components and their connections.

In addition to information on the building structure, the following information shall be collected:

1. Foundation and subsurface soil conditions,
2. Nonstructural components configuration and detailing, and
3. Configuration of adjacent structures when such structures have the potential to influence the seismic performance of the building being evaluated or retrofit.

Where construction documents include sufficient information to perform an evaluation of the primary and secondary structural components per Chapter 7, this information shall be verified by a visual condition assessment in accordance with Section 6.2.2.

Where construction documents are not available or do not provide the minimum information to perform an evaluation of the primary and secondary structural components per Chapter 7, the information shall be obtained or supplemented by a comprehensive condition assessment in accordance with Section 6.2.2 and material testing per Section 6.2.3.

6.2.2 Condition Assessment A condition assessment of the existing building and site conditions shall be performed as specified in this section and based on the requirements for either visual or comprehensive condition assessment in Chapters 9 through 13.

The condition assessment shall include direct visual observation of the following, as applicable:

1. Examination of the physical condition of a representative sample of primary and secondary structural components;
2. Verification of the presence and configuration of a representative sample of primary and secondary structural components and their connections, and the continuity of load paths in between;
3. Confirmation of component orientation, plumbness, and physical dimensions from a representative sample;
4. A review and documentation of other conditions, including adjacent buildings and presence of nonstructural components that could affect the behavior of the primary and secondary structural components;
5. Identification of prior repairs or alterations to the primary and secondary structural components based on a review of a representative sample;
6. Identification of damage, deterioration, or corrosion of the primary and secondary components based on a review of a representative sample of primary and secondary structural components; and
7. Identification of the seismic bracing or anchorage of the nonstructural components.

The required number of primary and secondary structural components that must be observed to constitute a representative sample shall be based on the requirements for visual or comprehensive condition assessment in accordance with the provisions of Chapters 9 through 12. The need to perform visual or comprehensive condition assessment shall be based on the completeness of the construction documents as specified in Section 6.2.1.

The requirements for observation of nonstructural components shall be in accordance with Chapter 13.

6.2.3 Material Properties Unless the condition assessment identifies damage or deterioration that could adversely affect the material properties, the material properties used in the evaluation or seismic retrofit of the building shall be based on the values specified on the construction documents or the default values specified in Chapters 9 through 12 used in conjunction with a knowledge factor in accordance with Section 6.2.3.1 for evaluation with linear procedures, or with bounding factors in accordance with Section 6.2.3.2 for evaluation with nonlinear procedures.

Testing to establish material property values for use as alternative values to those specified on the construction documents or the default values specified in Chapters 9 through 12 shall be permitted. If material testing is performed and the values from the testing are less than the values specified on the construction documents or the default values specified in Chapters 9 through 12, the values from the material testing shall be used.

Material testing programs shall conform to one of the following specified in this standard: usual or comprehensive. Usual and comprehensive testing requirements shall be in accordance with Chapters 9 through 12. In addition to the requirements in Chapters 9 through 12, comprehensive testing shall be performed where the following occurs:

1. Construction documents do not provide the information required by Section 6.2.1;
2. For specific material properties, where the lowest test value under Usual testing is less than 85% of the value specified on the design drawings or the default values;
3. Where values larger than the values specified on the drawings or the default values are proposed to be used in the evaluation or retrofit; or
4. Where the condition assessment identifies components with damage, corrosion, or deterioration.

If damage, corrosion, or deterioration is limited to one class of components or specific stories of a building, comprehensive testing shall only be required on the stories of the building where there is deterioration that affects the structural integrity of the components.

When comprehensive testing is used, statistical tests provided in accordance with ASTM E178 (2016a) shall be permitted to be used to determine whether a test value can be rejected as an outlier.

When comprehensive testing is performed, material strengths shall be based on the tested values. It shall be permitted to use the values specified on the construction documents for linear analysis procedures in lieu of the values obtained by material testing if the values specified on the construction documents are less than the values from material testing.

6.2.3.1 Knowledge Factor for Linear Procedures A knowledge factor, κ, shall be determined from Table 6-1 based on the information available on the construction documents. Knowledge factors shall be selected for each component as determined by the level of knowledge obtained for that component during data collection. It shall be permitted to use different knowledge factors for component types based on the level of testing performed for each component type. Usual and Comprehensive testing requirements shall be in accordance with Chapters 9 through 12.

6.2.3.2 Property Bounding for Nonlinear Procedures Where nonlinear procedures are used, data collection consistent with either the usual or comprehensive levels of knowledge shall be performed. Alternatively, it shall be permitted to perform bounding analyses to envelope the range of material properties in lieu of the required material testing for all building types except those containing unreinforced masonry as primary components (Table 6-2). The bounding analyses shall include at least two different mathematical models with representative lower- and upper-bound estimates of all the material properties. At a minimum, an analysis with all lower-bound property assumptions and an analysis with all upper-bound property assumptions shall be performed. Where the standard requires bounding of properties, the lower-bound properties of the specific component in the standard shall be incorporated with the lower-bound properties of the material properties and the same for upper-bound properties. Additional analyses shall be performed if there is a situation where a combination of lower-bound and upper-bound properties produces significantly different performance assessments than the lower-bound and upper-bound only models. Each analysis shall be assessed separately and the worst-case result for each component taken from the different bounding cases. Where Chapters 9 through 12 stipulate that material testing for a specific material property or component action is not required, that component action strength shall be permitted to be based on expected material properties without bounding factors.

When determining the appropriate bounds for material properties of the deformation-controlled components, the following shall be considered:

1. The variation of the material property as specified on the construction documents, and
2. The variation in the component action capacity based on the material property.

Table 6-1. Knowledge Factor for Linear Procedures.

Material Strength Specified on Construction Documents	Material Testing Performed	Material Strength Used in Evaluation	Knowledge Factor (κ)
Yes	None	Values specified in the construction documents	0.9
Yes	Usual	The values specified in the construction documents unless the minimum material testing result is less than 85% of the values specified on the construction documents	1.0
Yes	Comprehensive	Values based on the material testing results	1.0
No	None	Default values in Chapters 9 through 12	0.75
No	Usual	The default values in Chapters 9 through 12 unless the minimum material testing result is less than 85% of the default value	1.0
No	Comprehensive	Values based on the material testing results	1.0

Table 6-2. Property Bounding Requirements for Nonlinear Procedures.

Material Strength Specified on Construction Documents	Material Testing Performed	Material Strength Used in Evaluation	Bounding Analysis Required
Yes	None	Values specified in the construction documents	Yes
Yes	Usual	The values specified in the construction documents unless the minimum material testing result is less than 85% of the values specified on the construction documents	No
Yes	Comprehensive	Values based on the material testing results	No
No	None	Default values in Chapters 9 through 12	Yes
No	Usual	The default values in Chapters 9 through 12 unless the minimum material testing result is less than 85% of the default values	Yes
No	Comprehensive	The values based on material results	No

Table 6-3. Component Strength Modification Factor.

	Lower-Bound	Upper-Bound
Component Strength Modification Factor	0.75	1.25

For the bounding analysis, the strength and, if based on strength, the stiffness, of deformation-controlled components, Q_{ce}, shall be multiplied by factors to increase and decrease Q_{ce}. The factors shall be taken from Table 6-3 unless other factors can be justified by testing or analysis.

For the bounding analysis, the strength of force-controlled components, Q_{cl}, shall be multiplied by a lower-bound factor to decrease its value. The lower-bound factor shall be taken from Table 6-3, unless another factor can be justified by testing or analysis.

6.3 TIER 3 EVALUATION REQUIREMENTS

A Tier 3 evaluation shall consist of an analysis of an existing building performed in accordance with Chapter 7 for structural systems and Chapter 13 for nonstructural components. The scope of the structural analysis shall be in accordance with Section 7.1, based on the analysis requirements in Sections 7.2 and 7.3 and one or more of the analysis procedures specified in Section 7.4, using the acceptance criteria in Section 7.5. Foundation elements shall be evaluated in accordance with Chapter 8, and structural elements of the building shall be evaluated in accordance with the requirements of Chapters 9 through 12. Nonstructural elements shall be evaluated in accordance with Chapter 13. Chapters 14 and 15 shall be used where seismic isolation and supplemental energy dissipation devices are present in a building being evaluated.

A building meeting all provisions of these systematic evaluation procedures for a selected Performance Objective shall be deemed compliant with that Performance Objective.

6.4 TIER 3 RETROFIT REQUIREMENTS

The Tier 3 retrofit procedure shall consist of an analysis of a building, including retrofit measures, to demonstrate compliance with a selected Performance Objective. Where seismic deficiencies relative to a selected Performance Objective are identified by an evaluation performed in accordance with this standard or by other approved methods, a preliminary retrofit scheme shall be developed in accordance with Section 1.4.

An analysis of the building then shall be performed, including the retrofit measures, based on the procedure specified in this section. The scope of the analysis shall be in accordance with Section 7.1, based on the analysis requirements in Sections 7.2 and 7.3 and one or more of the analysis procedures specified in Section 7.4, using the acceptance criteria in Section 7.5. The analysis and acceptance criteria shall be used for both existing elements and new elements introduced as part of the retrofit. Foundation elements shall be evaluated in accordance with Chapter 8, and structural elements of the building shall be evaluated in accordance with the requirements of Chapters 9 through 12. Nonstructural elements shall be evaluated in accordance with Chapter 13. Chapters 14 and 15 shall be used where seismic isolation and supplemental energy dissipation devices are used as part of the retrofit measures. The results of this analysis shall be used to verify that the retrofit design meets the selected Performance Objective.

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CHAPTER 7

ANALYSIS PROCEDURES AND ACCEPTANCE CRITERIA

7.1 SCOPE

This chapter sets forth requirements for analysis of buildings using either the Tier 2 deficiency-based procedures or Tier 3 systematic procedures. Section 7.2 specifies general analysis requirements for the mathematical modeling of buildings, including basic assumptions, consideration of torsion, diaphragm flexibility, $P-\Delta$ effects, soil–structure interaction (SSI), multidirectional effects, and overturning as well as analysis of diaphragms, continuity, and structural walls. Section 7.3 describes how to select one of the four analysis procedures and sets limitations on their application. Section 7.4 specifies the requirements for the four analysis procedures. Section 7.5 defines component acceptance criteria, including behavior types and capacities. Section 7.6 specifies procedures for developing alternative modeling parameters and acceptance criteria.

For Tier 2 deficiency-based procedures in Chapter 5, the analysis need only be used to determine demands, capacities, and acceptance criteria for those elements that the Tier 2 procedures designate to be evaluated.

Analysis of buildings with seismic isolation or energy dissipation systems shall comply with the requirements of Chapters 14 and 15, respectively.

7.2 GENERAL ANALYSIS REQUIREMENTS

An analysis of the building shall be conducted in accordance with the requirements of this section.

7.2.1 Analysis Procedures An analysis of the building shall be performed using the linear static procedure (LSP), the linear dynamic procedure (LDP), the nonlinear static procedure (NSP), or the nonlinear dynamic procedure (NDP), selected based on the limitations specified in Section 7.3.

7.2.2 Effective Seismic Weight The effective seismic weight used to determine the pseudo seismic forces in the linear static procedure or to determine the mass applied to the mathematical model in the linear dynamic procedure, nonlinear static procedure, and nonlinear dynamic procedure shall include the following:

1. The total dead load in the building;
2. In areas used for storage, a minimum 25% of the floor live load, and the live load shall be permitted to be reduced for tributary area as approved by the Authority Having Jurisdiction;
3. Where an allowance for partition load is included in the floor live load, the actual partition weight or a minimum weight of 10 lb/ft^2 (0.48 kN/m^2) of floor area, whichever is greater;
4. Total operating weight of permanent equipment not included in the dead load;
5. Weight of landscaping and other materials at roof gardens and similar areas;
6. Weight of fluids and bulk material expected to be present during normal use; and
7. Snow load as defined in Section 7.2.3.3.

7.2.3 Component Gravity Loads and Load Combinations For linear procedures, the following actions caused by gravity loads, Q_G, shall be considered for combination with actions caused by seismic forces.

Where the effects or actions of gravity loads and seismic forces are additive, the action caused by gravity loads, Q_G, shall be obtained in accordance with Equation (7-1):

$$Q_G = 1.1(Q_D + Q_L + Q_S) \tag{7-1}$$

where

Q_D = Action caused by dead loads as defined in Section 7.2.3.1,

Q_L = Action caused by live load as defined in Section 7.2.3.2, and

Q_S = Action caused by effective snow load as defined in Section 7.2.3.3.

Where the effects or actions of gravity loads and seismic forces are counteracting, the action caused by gravity loads, Q_G, shall be obtained in accordance with Equation (7-2):

$$Q_G = 0.9Q_D \tag{7-2}$$

For nonlinear procedures, the following actions caused by gravity loads, Q_G, in accordance with Equation (7-3) shall be considered for combination with actions caused by seismic forces:

$$Q_G = Q_D + Q_L + Q_S \tag{7-3}$$

where Q_D, Q_L, and Q_S are as defined for Equation (7-1).

See Chapters 14 and 15 for gravity loads and load combinations for seismic isolation and energy dissipation systems, respectively.

7.2.3.1 Dead Load Dead load shall be determined based on the provisions in Chapter 3 of ASCE 7.

7.2.3.2 Live Load Live load shall be taken as 25% of the unreduced live load specified in Chapter 4 of ASCE 7, but not less than the actual live load present in the building. Roof live load specified in Chapter 4 of ASCE 7 shall not be included with live load except where the live load on a roof is due to occupancy-related live loads due to rooftop assembly, rooftop decks, or vegetative or landscaped roofs with occupiable areas.

7.2.3.3 Snow Load Where the flat roof snow load calculated in accordance with ASCE 7 exceeds 45 lb/ft^2 (2.16 kN/m^2), the effective snow load shall be taken as 15% of the calculated snow load. Where the flat roof snow load is less than or equal to

45 lb/ft^2 (2.16 kN/m^2), the effective snow load shall be permitted to be zero.

Ground snow load values obtained from ASCE 7 shall be based on the building's risk category. When the applicable building code does not specify a risk category, snow loads shall be permitted to be based on Risk Category II. Alternatively, snow loads shall be permitted to be determined in accordance with the performance-based procedures of ASCE 7, Section 1.3.1.3.

7.2.4 Mathematical Modeling

7.2.4.1 Basic Assumptions A building shall be modeled, analyzed, and evaluated as a three-dimensional assembly of components. Alternatively, use of a two-dimensional model shall be permitted if the building meets one of the following conditions:

1. The building has rigid diaphragms as defined in Section 7.2.11 and torsion effects do not exceed the limits specified in Section 7.2.4.2, or torsion effects are accounted for as specified in Section 7.2.4.2; or
2. The building has flexible diaphragms as defined in Section 7.2.11.

If two-dimensional models are used, the three-dimensional nature of components and elements shall be considered when calculating stiffness and strength properties.

If the building contains out-of-plane offsets in vertical seismic-force-resisting elements, the model shall explicitly account for such offsets in the determination of diaphragm demands.

Modeling stiffness of structural components shall be based on the stiffness requirements of Chapters 8 through 12.

For nonlinear procedures, a connection shall be explicitly modeled if the connection is weaker than or has less ductility than the connected components or if the flexibility of the connection results in an increase in the relative deformations between adjacent connections of more than 10%.

7.2.4.2 Torsion The effects of torsion shall be considered in accordance with this section. Torsion need not be considered in buildings with flexible diaphragms as defined in Section 7.2.11. The mathematical model shall account for the spatial distribution of gravity loads over the entire plan of the building at each floor and roof and $P-\Delta$ effects that produce additional plan rotation shall be considered.

7.2.4.2.1 Total Torsional Moment The total torsional moment at a story shall be equal to the sum of the actual torsional moment and the accidental torsional moment calculated as follows:

1. The actual torsional moment at a story shall be calculated by multiplying the seismic story shear force by the eccentricity between the center of mass and the center of rigidity measured perpendicular to the direction of the applied load. The center of mass shall be based on all floors above the story under consideration. The center of rigidity of a story shall include all vertical seismic-force-resisting elements in the story.
2. The accidental torsional moment at a story shall be calculated as the seismic story shear force multiplied by a distance equal to 5% of the horizontal dimension at the given floor level measured perpendicular to the direction of the applied load.

7.2.4.2.2 Consideration of Torsional Effects for Linear Procedures Effects of torsion shall be considered for the LSP and LDP in accordance with the following requirements:

1. Increased forces and displacements caused by actual torsion shall be calculated for all buildings.
2. The torsional amplification multiplier for displacements, η, for each level x shall be calculated as the ratio of the maximum displacement at any point on the level x diaphragm to the average displacement $\eta = \delta_{max}/\delta_{avg}$. Displacements shall be calculated for the applied forces.
3. Increased forces and displacements caused by accidental torsion need not be considered if either of the following conditions apply: (a) the accidental torsional moment is less than 25% of the actual torsional moment, or (b) the ratio of the displacement multiplier η caused by the actual plus accidental torsion and the displacement multiplier caused by actual torsion is less than 1.1 at every floor.
4. For linear analysis procedures, forces and displacements caused by accidental torsion shall be amplified by a factor, A_x, as defined by Equation (7-4), where the displacement multiplier η caused by actual plus accidental torsion exceeds 1.2 at any level.

$$A_x = \left(\frac{\eta}{1.2}\right)^2 \leq 3.0 \tag{7-4}$$

5. If the displacement multiplier η caused by actual plus accidental torsion at any level exceeds 1.5, two-dimensional models shall not be permitted, and three-dimensional models that account for the spatial distribution of mass and stiffness shall be used.
6. Where two-dimensional models are used, forces and displacements shall be amplified by the maximum value of η calculated for the building.
7. The effects of accidental torsion shall not be used to reduce force and deformation demands on components to be less than the forces and deformations determined without including accidental torsion.

7.2.4.2.3 Consideration of Torsional Effects for Nonlinear Procedures Effects of torsion shall be considered for the NSP and NDP in accordance with the following requirements:

1. The mathematical model shall consider the effect of actual torsion due to eccentricities between the center of mass of the diaphragm and the center of rigidity of the diaphragm at each floor and roof level.
2. The torsional amplification multiplier for displacements, η, for each level x shall be calculated as the ratio of the maximum displacement at any point on the level x diaphragm to the average displacement $\eta = \delta_{max}/\delta_{avg}$. Displacements shall be calculated for the applied forces. It shall be permitted to use either a linear or nonlinear analysis to compute η to be used in Items 3 through 5 and Item 7.
3. For NSP using three-dimensional models, accidental torsion shall be considered by using one of the following methods:
   (a) By using modal load patterns that are derived based on shifting the center of mass at each floor and roof by 5% of the horizontal dimension in the direction perpendicular to the direction the target displacement is determined in, or
   (b) Increasing the target displacement by multiplying the target displacement at the center of mass by the ratio of the η calculated using actual and accidental torsion of 5% of the horizontal dimension in the direction perpendicular to the direction of the target displacement producing the larger value of η to η calculated using only actual torsion.
4. For NSP using two-dimensional models, accidental torsion shall be considered by multiplying the target displacement

by factor η calculated using actual and accidental torsion in the direction perpendicular to the direction of the target displacement producing the larger torsional moment.

5. For NDP, accidental torsion shall be considered by using one of the following methods:
   (a) By creating four analysis permutations, in which the center of mass is shifted by plus and minus 5% of the horizontal dimensions of the floor and roof levels in each orthogonal direction corresponding to the directions of applied ground motions; or
   (b) If both orthogonal directions have a torsional strength irregularity, shifting the center of mass at each floor and roof by 5% of the horizontal dimension at the given floor level on the side of the center of rigidity that produces the maximum value of η in each of the orthogonal directions corresponding to the directions of applied ground motions. The mass shall be permitted to be shifted simultaneously in both orthogonal directions or as separate permutations in each orthogonal direction; or
   (c) If one or none of the orthogonal directions have a torsional strength irregularity, by modifying the mathematic model to shift the center of mass at each floor and roof by 5% of the horizontal dimension at the given floor level measured perpendicular to the applied ground motion acceleration record in the direction that produces the largest value of η or the direction that has a torsional strength irregularity per Section 7.3.1.1.4.
6. The effects of accidental torsion shall not be used to reduce force and deformation demands on components to be less than the forces and deformations determined without including accidental torsion.
7. It shall be permitted to exclude accidental torsion if the structure does not have a torsional strength irregularity defined in Section 7.3.1.1.4 in both orthogonal directions and one of the following conditions apply:
   (a) The accidental torsional moment is less than 25% of the actual torsional moment based on Section 7.2.4.2.1, or
   (b) The value of maximum value of η including accidental torsion is less than 1.2 at every level.
8. If Item 7 does not apply, when two or more Seismic Hazard Levels are evaluated and a three-dimensional model is used, it shall be permitted to exclude accidental torsion in the analyses for all but the highest hazard level if all of the following conditions are met:
   (a) The spectral acceleration at the highest hazard level is at least 150% of the spectral acceleration of the smaller hazard levels over the period range of interest used to scale or match the ground motion suite,
   (b) The performance level in any of the smaller hazard levels is not Damage Control or Immediate Occupancy, and
   (c) There are no unacceptable responses in the highest hazard level's analysis.
9. For nonlinear analysis of buildings that include seismic isolation or supplemental energy dissipation devices, accidental torsion requirements shall be in accordance with Chapters 14 or 15, as applicable.

7.2.4.3 Primary and Secondary Components Components that affect the lateral stiffness or distribution of forces in a structure and are required to resist seismic forces and accommodate deformations for the structure to achieve the selected Performance Level shall be classified as primary. Primary components shall be evaluated for earthquake-induced forces and deformations in combination with gravity load effects.

Structural components that accommodate seismic deformations but are not required to resist seismic forces for the structure to achieve the selected Performance Level shall be permitted to be classified as secondary. Secondary components shall be evaluated for earthquake-induced deformations in combination with gravity load effects. Secondary components shall be assessed using any analysis procedure in Section 7.4 to meet all of the following criteria:

1. The total initial lateral stiffness of secondary components in a building shall not exceed 20% of the total initial lateral stiffness of primary components at any story;
2. For diaphragms classified as other than flexible, inclusion of secondary components shall not cause the center of rigidity in a story to shift by more than 10% of the diaphragm dimension perpendicular to the direction of applied force or shall not cause a change in displacement of more than 10% at the points furthest from the center of rigidity in each direction at any story; and
3. Inclusion of secondary components shall not increase the force or deformation demands on a primary component by more than 10%.

If the secondary components do not meet the criteria in the preceding list, sufficient components shall be reassigned as primary components until all the remaining secondary components meet the preceding criteria.

Components shall not be selectively designated primary or secondary to change the configuration of a building from irregular to regular.

Nonstructural components shall be classified as secondary components unless they are required to be classified as primary components in accordance with this section. Nonstructural components that are attached at two or more floors shall be assessed to determine if they are acting as structural components with respect to resisting seismic forces and deformation. Where nonstructural components are classified as primary components, they shall be assessed based on the requirements of Section 7.5. Where nonstructural components can be classified as secondary components, they are permitted to be excluded from the mathematical model and instead shall be assessed based on the requirements of Chapter 13.

7.2.4.3.1 Linear Procedures Mathematical models for use with linear analysis procedures shall include the stiffness and resistance of all the primary components. It shall be permitted to include the stiffness and resistance of the secondary components in the mathematical model. Where mathematical models include only primary components, an evaluation of the secondary components shall be based on the linear static procedure in accordance with Section 7.5.2. In that evaluation, deformations from the mathematical model with the primary components shall be applied to a linear mathematic model of all the secondary components or a representative subset of the secondary components. The applied deformations shall be based on the maximum deformations from the mathematical model with the primary components at the locations where the unmodeled secondary components are.

7.2.4.3.2 Nonlinear Procedures Mathematical models for use with nonlinear procedures shall include the stiffness and resistance of primary and secondary components. The strength and stiffness degradation of primary and secondary components shall be modeled explicitly.

It is permitted to exclude the strength and stiffness of the secondary components from the nonlinear mathematical model if they are evaluated using a separate mathematical model of all or a representative subset of the secondary components in accordance with the following criteria:

1. When the NSP is used for the evaluation of the primary components, the secondary components shall be evaluated by either of the following options:
   (a) Evaluating a linear mathematical model of the secondary components in accordance with the LSP per Section 7.5.2 using deformation demands from the primary component nonlinear mathematical model at the locations of the secondary components when the primary component model is displaced to the target displacement in lieu of applying the pseudo seismic force, or
   (b) Evaluating a nonlinear mathematical model of the secondary components in accordance with the NSP per Section 7.5.3 using deformation demands from the primary component nonlinear mathematical model at the locations of the secondary components when the primary component model is displaced to the target displacement in lieu of displacing the secondary component mathematical model to the target displacement.
2. When the NDP is used for the evaluation of the primary components, the secondary components shall be evaluated by either of the following options:
   (a) Evaluating a linear mathematical model of the secondary components in accordance with the LSP per Section 7.5.2.1 using deformation demands from the nonlinear mathematical model of the primary components at the locations of the secondary components in lieu of applying the pseudo seismic force. The deformation demands from the primary component model shall be the mean of the maximum deformations from each ground motion record producing an acceptable response, or
   (b) By using the NSP based on both of the following procedures:
      (i) Evaluating the secondary component nonlinear mathematic model in accordance with Section 7.5.3 using deformation demands from the primary component nonlinear mathematical model at the locations of the secondary components. The deformation demands from the primary component model shall be the mean of the maximum deformations from each ground motion record producing an acceptable response.
      (ii) Displacing the nonlinear mathematical model of the secondary components to the maximum deformations of the primary component mathematical model at the locations of the secondary to confirm the secondary components' model does not exhibit an unacceptable response per Section 7.5.3.2.1. The maximum deformation of the primary component model shall be based on the maximum of the maximum deformations from each ground motion record producing an acceptable response.

7.2.4.4 Stiffness and Strength Assumptions Stiffness and strength properties of components shall be determined in accordance with the requirements of Chapters 8 through 12, 14, and 15.

7.2.4.5 Foundation Modeling The foundation system shall be modeled considering the degree of fixity provided at the base of the structure. Rigid or flexible base assumptions shall be permitted in accordance with the requirements for foundation acceptability in Section 8.4 and Section 8.5. Flexible base assumptions shall be required when the provisions of Section 8.6, are used. Foundation modeling shall consider movement caused by geologic site hazards specified in Section 8.2, and load-deformation characteristics specified in Sections 8.4 and 8.5.

7.2.4.6 Damping For linear static, linear dynamic, and nonlinear static procedures, 5% damped response spectra shall be used for the analysis of all buildings except those meeting the following criteria:

1. For structural steel buildings without exterior cladding and nonstructural interior partitions, an equivalent viscous damping ratio, β, equal to 2% of critical damping ($\beta = 0.02$) shall be assumed;
2. For buildings with wood diaphragms and cross walls that interconnect the diaphragm levels at a maximum spacing of 40 ft (12.2 m) on center transverse to the direction of motion, an effective viscous damping ratio, β, equal to 10% of critical damping ($\beta = 0.10$) shall be permitted;
3. For buildings using seismic isolation technology or enhanced energy dissipation technology, an equivalent viscous damping ratio, β, shall be calculated using the procedures specified in Chapters 14 and 15; or
4. There is sufficient analysis or test data based on the specific characteristics of the building to substantiate the use of an equivalent damping ratio other than 5% ($\beta = 0.05$).

Damping of the building system shall be implemented in the analysis procedure in accordance with the requirements of Sections 7.4.1.4 and 7.4.2.4 for linear procedures, Section 7.4.3.4 for the nonlinear static procedure, and as augmented by soil–structure interaction per Section 8.6.2.

Damping for the building system shall be implemented in the nonlinear dynamic analysis procedure in accordance with the requirements of Section 7.4.4.4.

For buildings using seismic isolation technology or enhanced energy dissipation technology, the effects of added viscous damping shall be incorporated directly in the nonlinear dynamic analysis in accordance with the procedures specified in Chapters 14 and 15.

7.2.5 Configuration Building irregularities defined in Section 7.3.1.1 shall be based on the plan and vertical configuration of the existing building for an evaluation or retrofit. Irregularities shall be determined, both with and without the contribution of secondary components.

7.2.6 Multidirectional Seismic Effects Buildings shall be evaluated or retrofitted to address seismic motion in any horizontal direction. Multidirectional seismic effects shall be considered to act concurrently, as specified in Section 7.2.6.1, for buildings meeting one of the following criteria:

1. The building has plan irregularities as defined in Section 7.3.1.1, or
2. The building has one or more primary columns that form a part of two or more intersecting frame or braced frame elements.

All other buildings shall be permitted to be evaluated or retrofitted for seismic motions acting nonconcurrently in the direction of each principal axis of the building.

7.2.6.1 Concurrent Seismic Effects Where concurrent multidirectional seismic effects must be considered, horizontally oriented, orthogonal *X*- and *Y*-axes shall be established. Components of the building shall be evaluated or retrofitted for combinations of forces and deformations from separate analyses performed for ground motions in *X*- and *Y*-directions as follows:

1. Where the LSP or LDP is used as the basis for analysis, elements and components shall be analyzed for (a) forces and deformations associated with 100% of the forces in the *X*-direction plus the forces and deformations associated with 30% of the forces in the *Y*-direction; and for (b) forces and deformations associated with 100% of the forces in the *Y*-direction plus the forces and deformations associated with 30% of the forces in the *X*-direction. Other combination rules shall be permitted where verified by experiment or analysis; and
2. Where the NSP is used as the basis for analysis, elements and components of the building shall be analyzed for (a) forces and deformations associated with 100% of the target displacement in the *X*-direction only, plus the forces (not deformations) associated with 30% of the displacements in the *Y*-direction only; and for (b) forces and deformations associated with 100% of the displacements in the *Y*-direction only, plus the forces (not deformations) associated with 30% of the displacements in the *X*-direction only. Forces and deformations shall be determined in accordance with Section 7.4.3 for the NSP.

   Alternatively, it shall be permitted to determine the forces and deformations associated with 100% of the displacements in any single direction that generate the maximum deformation and component action demands. Further concurrent seismic effects need not be considered in the critical direction(s). Other combination rules shall also be permitted where verified by experiment or analysis; and
3. Where the NDP is used as the basis for analysis with a two-dimensional model, elements and components of the building shall be evaluated for forces and deformations associated with the application of ground motions scaled by the maximum value of η calculated for the building. Forces and deformations shall be determined in accordance with Section 7.4.4 for the NDP; and
4. Where the NDP is used as the basis for analysis with a three-dimensional model, elements and components of the building shall be analyzed for forces and deformations associated with the application of the suite of ground motions as required by Section 2.3.3.

7.2.6.2 Vertical Seismic Effects The effects of the vertical response of a building to earthquake ground motion shall be considered for any of the following cases:

1. Horizontal cantilever components of buildings that provide gravity load support;
2. Horizontal prestressed components of buildings; and
3. Building components, excluding foundations, in which demands caused by gravity loads specified in Section 7.2.3 exceed 80% of the nominal capacity of the component.

For components requiring consideration of vertical seismic effects, the vertical response of a structure to earthquake ground motion need not be combined with the effects of the horizontal response.

7.2.7 P-delta Effects

$P-\Delta$ effects shall be included in linear and nonlinear analysis procedures. For nonlinear procedures, static $P-\Delta$ effects shall be incorporated in the analysis by including in the mathematical model the nonlinear force–deformation relationship of all components subjected to axial forces.

7.2.8 Soil–Structure Interaction

The effects of soil–structure interaction (SSI) shall be evaluated for those buildings in which an increase in fundamental period caused by SSI effects results in an increase in spectral accelerations. For other buildings, the effects of SSI need not be evaluated.

Calculation of SSI effects using the explicit modeling procedure shall be based on a mathematical model that includes the flexibility and damping of individual foundation components. Foundation stiffness parameters shall comply with the requirements of Sections 8.4 and 8.5. Damping ratios for individual foundation components shall be permitted to be used. In lieu of explicitly modeling damping for individual foundation elements, use of the effective damping ratio of the structure–foundation system, β_{SSI}, calculated in accordance with Section 8.6.2 shall be permitted for the LSP and LDP. For the NSP, the effective damping ratio of the foundation–structure system, β_{SSI}, calculated in accordance with Section 8.6.2 shall be used to modify spectral demands. For the NDP, foundation damping at individual foundation elements shall be explicitly included in the mathematical model.

The general or site-specific response spectrum shall be permitted to be reduced due to the effects of kinematic soil–structure interaction. Kinematic interaction effects shall be permitted to be calculated through explicit mathematical modeling of the soil–foundation–structure system, which accounts for spatial and depth variations in ground motion. Alternatively, kinematic interaction effects shall be permitted to be calculated per Section 8.5.1.

Combination of damping effects with kinematic interaction effects calculated in accordance with Section 8.6.1 shall be permitted, subject to the limitations of Section 8.6. Soil–structure interaction effects shall be limited based on the following requirements:

1. For LSP and LDP, the maximum pseudolateral force calculated including the effects of soil–structure interaction shall not be less than 70% of the pseudolateral force calculated, excluding soil–structure interaction effects; and
2. For NSP, the target displacement calculated including soil–structure interaction effects shall not be less than 70% of the target displacement calculated without the inclusion of soil–structure interaction effects.

7.2.9 Overturning

Buildings shall be evaluated or retrofitted to resist overturning effects caused by seismic forces. Each vertical-force-resisting element receiving earthquake forces caused by overturning shall be investigated for the cumulative effects of seismic forces applied at and above the level under consideration. The effects of overturning shall be evaluated at each level of the structure as specified in Section 7.2.9.1 for linear procedures and Section 7.2.9.2 for nonlinear procedures. The effects of overturning on foundations and geotechnical components shall be considered in the evaluation or retrofit of foundation regarding strengths and stiffnesses as specified in Chapter 8.

7.2.9.1 Overturning Effects for Linear Procedures Where linear procedures are used, overturning effects shall be resisted through the stabilizing effect of dead loads acting alone or in combination with positive connections of structural components to components below the level under consideration.

Where dead loads alone are used to resist the effects of overturning, Equation (7-5) shall be satisfied:

$$M_{ST} > \frac{M_{OT}}{\mathrm{DCR}_{\min}} \tag{7-5}$$

where

M_{OT} = Total overturning moment induced on the element by seismic forces applied at and above the level under consideration; overturning moment shall be determined based on seismic forces calculated in accordance with Section 7.4.1 for LSP and 7.4.2 for LDP;

M_{ST} = Stabilizing moment produced by dead loads acting on the element; and

$\mathrm{DCR}_{\mathrm{min}}$ = Coefficient defined in Section 7.5.2.1.2.

The quantity $M_{OT}/\mathrm{DCR}_{\mathrm{min}}$ need not exceed the overturning moment on the element, as limited by the expected strength of the structure. The element shall be evaluated for the effects of increased compression at the end about which it is being overturned. For this purpose, compression at the end of the element shall be considered a force-controlled action.

Alternatively, the load combination represented by Equation (7-6) shall be permitted for evaluating the adequacy of dead loads alone to resist the effects of overturning:

$$0.9M_{ST} > M_{OT}/(C_1 C_2 \mu_{OT}) \tag{7-6}$$

where

μ_{OT} = 10.0 for Collapse Prevention,
= 8.0 for Life Safety,
= 4.0 for Immediate Occupancy, and
C_1 and C_2 = Coefficients defined in Section 7.4.1.3.1

Where Equation (7-5) or (7-6) for dead load stability against the effects of overturning is not satisfied, positive attachment between elements of the structure at and immediately above and below the level under consideration shall be provided. Positive attachments shall be capable of resisting earthquake forces in combination with gravity loads as deformation- or force-controlled actions in accordance with Equation (7-36), (7-37), or (7-38) and applicable acceptance criteria of Equation (7-39) or (7-40).

7.2.9.2 Overturning Effects for Nonlinear Procedures Where nonlinear procedures are used, the effects of earthquake-induced uplift on the tension side of an element shall be included in the analytical model as a nonlinear degree of freedom. The adequacy of elements above and below the level at which uplift occurs shall be evaluated for any redistribution of forces or deformations that occurs as a result of this uplift.

7.2.10 Sliding at the Soil–Structure Interface For structures with shallow foundations and having Structural Performance Level of Life Safety or greater, the seismic lateral force demand along each line of frames or shear walls at the soil–structure interface shall be evaluated against the sliding resistance provided by soil friction and passive pressure along that line. If the demand is greater than the sliding resistance along any line of frames or shear walls, then the requirements of Section 7.2.10.1 shall be met. For linear procedures, the seismic lateral force demand for this check shall be taken as the sum of horizontal reactions along the line divided by the maximum DCR of the superstructure primary component actions along that line, calculated in accordance with Equation (7-16). The maximum DCR need not be taken as less than 1.0 for this check. For the nonlinear static procedure, the seismic lateral force demand shall be taken as the sum of reaction forces along the line calculated in accordance with Section 7.4.3.3. For the nonlinear dynamic procedure, the seismic lateral force demand along a line shall be treated as a single action and the mean value of this action computed in accordance with Section 7.4.4.3. It shall be permitted to omit this check if the structure complies with Section 7.2.10.1.

Evaluation of liquefaction and landsliding hazards shall satisfy the requirements in Chapter 8. Lateral resistance at the soil–structure interface for structures with deep foundations shall also be evaluated in accordance with Chapter 8.

Horizontal displacement of the structure relative to the soil (sliding displacement) need not be considered except as required for conformance with the selected Nonstructural Performance Level and the provisions of Chapter 13. External utility distribution lines, which connect into the building above or below grade and which are required to remain functional, shall be evaluated for capacity to accommodate expected movements of the building relative to the soil or shall be provided with flexible connections capable of accommodating the expected movement.

7.2.10.1 Foundation Interconnection Where required by Section 7.2.10, the building shall have a diaphragm or equivalent horizontal bracing (foundation ties) that has the capacity to redistribute horizontal forces between components of the lateral-load-resisting system and the points at the soil–structure interface that resist lateral force. The diaphragm or horizontal bracing shall be assessed in accordance with one of the following methods:

Method 1. The diaphragm or horizontal bracing elements shall be evaluated in each orthogonal direction as deformation-controlled elements subject to forces resulting from redistribution of the horizontal base shear between frame or shear wall lines and points of lateral resistance due to friction or passive pressure at the soil–structure interface. For this evaluation, the horizontal base shear shall not be taken as less than the smaller of the following:

- The total seismic horizontal reaction from the superstructure analysis,
- The total expected base shear capacity of the superstructure, or
- The upper bound total lateral resistance provided by soil friction and soil passive pressure.

Method 2. The behavior of the diaphragm or horizonal bracing and the compatibility of horizontal displacements at the soil–structure interface shall be accounted for in an analytical model of the building.

7.2.11 Diaphragms, Chords, Collectors, and Ties Diaphragms shall be defined as horizontal elements that transfer earthquake-induced inertial forces to vertical elements of the seismic-force-resisting systems through the collective action of diaphragm components including chords, collectors, and ties. Diaphragm chords, collectors, and ties shall be considered part of the diaphragm and are subject to the provisions of this standard for diaphragms in addition to any provisions specific to chords, collectors, and ties.

Diaphragms shall be provided at each level of the structure as necessary to connect building masses to the primary vertical elements of the seismic-force-resisting system. The analytical model of the building shall account for the behavior of the diaphragms as specified in this section.

Diaphragms and their connections to vertical elements providing lateral support shall comply with the requirements specified in Section 9.9 for metal diaphragms; Chapter 10 for concrete diaphragms and precast concrete diaphragms; and Section 12.5 for wood diaphragms.

7.2.11.1 Classification of Diaphragms Diaphragms shall be classified as flexible where the maximum horizontal deformation of the diaphragm along its length is more than twice the average story drift of the vertical seismic-force-resisting elements of the story immediately below the diaphragm.

Diaphragms shall be classified as rigid where the maximum lateral deformation of the diaphragm is less than half the average story drift of the vertical seismic-force-resisting elements of the story immediately below the diaphragm.

Diaphragms that are neither flexible nor rigid shall be classified as stiff.

For the purpose of classifying diaphragms, story drift and diaphragm deformations shall be calculated using the pseudo seismic force specified in Equation (7-21). The in-plane deflection of the diaphragm shall be calculated for an in-plane distribution of seismic force consistent with the distribution of mass and all in-plane seismic forces associated with offsets in the vertical seismic framing at that diaphragm level.

In lieu of classifying a diaphragm as flexible based on calculation, it shall be permitted to classify diaphragms constructed of bare steel decking or wood structural panels as flexible in accordance with Section 12.3.1.1 of ASCE 7.

7.2.11.2 Mathematical Modeling Mathematical modeling of buildings with rigid diaphragms shall account for the effects of torsion as specified in Section 7.2.4.2. Mathematical models of buildings with stiff or flexible diaphragms shall account for the effects of diaphragm flexibility by modeling the diaphragm as an element with in-plane stiffness consistent with the structural characteristics of the diaphragm system. Alternatively, for buildings with flexible diaphragms at each level, each seismic-force-resisting element in a vertical plane shall be permitted to be evaluated independently, with seismic masses assigned on the basis of tributary area.

7.2.11.3 Diaphragm Chords Except for diaphragms considered as unchorded, as specified in Chapter 12, a boundary component shall be provided at each diaphragm edge (either at the perimeter or at an opening) to resist tension or compression resulting from the diaphragm moment. This boundary component shall be a continuous diaphragm chord; a continuous component of a wall or frame element; or a continuous combination of wall, frame, and chord components. The boundary components shall be evaluated or retrofitted to transfer accumulated seismic forces at the diaphragm boundaries. At reentrant corners in diaphragms and at the corners of openings in diaphragms, diaphragm chords shall be extended distances sufficient to develop the accumulated diaphragm boundary forces into the diaphragm beyond the corners.

7.2.11.4 Diaphragm Collectors At each vertical element of the seismic-force-resisting system, a diaphragm collector shall be provided to transfer to the element accumulated diaphragm forces that are in excess of the forces transferred directly to the element in shear. The diaphragm collector shall be extended beyond the element and attached to the diaphragm to transfer the accumulated forces.

7.2.11.5 Diaphragm Ties Diaphragms shall be provided with continuous tension ties between chords or boundaries. Ties shall be evaluated or retrofitted for a minimum axial tension as a force-controlled action using Equation (7-37) with Q_E calculated using Equation (7-7) and χ, C_1, and C_2 in Equation (7-37) taken as 1.0:

$$F_p = 0.4 S_{XS} W \tag{7-7}$$

where

F_p = Axial tensile force for the evaluation or retrofit of ties between the diaphragm and chords or boundaries;

S_{XS} = Spectral response acceleration parameter at short periods, as determined in accordance with Section 2.3.2; and

W = Weight tributary to that portion of the diaphragm extending half the distance to each adjacent tie or diaphragm boundary.

Where diaphragms of timber, gypsum, or metal deck construction provide lateral support for walls of masonry or concrete construction, ties shall be evaluated or retrofitted for the wall anchorage forces specified in Section 7.2.13 for the area of wall tributary to the diaphragm tie.

7.2.12 Continuity All structural components shall be tied together to form a complete load path for the transfer of inertial forces generated by the dynamic response of portions of the structure to the rest of the structure.

1. Smaller portions of a building, such as outstanding wings, shall be connected to the structure as a whole. Component connections shall be capable of resisting, in any direction, the horizontal force as a force-controlled action per section 7.5.1.1 with Q_E calculated using Equation (7-8) and χ, C_1, C_2, and $\mathrm{DCR}_{\mathrm{min}}$ taken as 1.0. These connections are not required if the individual portions of the structure are self-supporting and are separated by a seismic joint permitting independent movement during dynamic response in accordance with Section 7.2.15:

   $$F_p = 0.133 S_{XS} W \tag{7-8}$$

   where

   F_p = Horizontal seismic force in any direction for the analysis of connections between two components of a building,

   S_{XS} = Spectral response acceleration parameter at short periods, as determined in accordance with Section 2.3.2, and

   W = Weight of the smaller portion of the building.

2. A positive connection for resisting horizontal force acting parallel to the member shall be provided for each beam, girder, or truss to its support. The connection shall be considered a force-controlled action per Section 7.5.1.1 with Q_E calculated as 5% of the dead load and live load reaction and χ, C_1, C_2, and $\mathrm{DCR}_{\mathrm{min}}$ taken as 1.0.
3. Where a sliding support is provided at the end of a component, gravity connections or supports for members spanning between structures or seismically separate portions of structures shall be designed for the maximum anticipated relative displacements.

7.2.13 Structural Walls and Their Anchorage Walls shall be evaluated or retrofitted for out-of-plane inertial forces as required by this section and as further required for specific structural systems in Chapters 9 through 12. Actions that result from application of the forces specified in this section shall be considered force controlled. Nonstructural walls shall be evaluated using the provisions of Chapter 13.

7.2.13.1 Out-of-Plane Wall Anchorage to Diaphragms Each wall shall be positively anchored to all diaphragms that provide lateral support for the wall or are vertically supported by the wall. Walls shall be anchored to diaphragms at horizontal distances not exceeding 8 ft (2.4 m), unless it can be demonstrated that the wall has adequate capacity to span horizontally between the supports for greater distances. Anchorage of walls to diaphragms shall be considered a force-controlled action using Equation (7-37), with Q_E calculated using Equation (7-9), and C_1 and C_2 in Equation (7-37) taken as 1.0, which shall be developed into the diaphragm. χ used in Equation (7-37) shall be 1.0, and χ used in Equations (7-9) and (7-10) shall be per Table 7-1. If subdiaphragms are used, each subdiaphragm shall be capable of transmitting the shear forces caused by wall anchorage to a continuous diaphragm tie. Subdiaphragms shall have length-to-depth ratios not exceeding 3:1. Where wall panels are stiffened for out-of-plane behavior by pilasters or similar components, anchors shall be provided at each such component, and the distribution of out-of-plane forces to wall anchors and diaphragm ties shall consider the stiffening effect and accumulation of forces at these components:

$$F_p = 0.4 S_{XS} k_a k_h \chi W_p \tag{7-9}$$

$$F_{p,\min} = 0.2 k_a \chi W_p \tag{7-10}$$

$$k_a = 1.0 + \frac{L_f}{100} \tag{7-11}$$

$$k_h = \frac{1}{3}\left(1 + \frac{2z_a}{h_n}\right) \tag{7-12}$$

where

F_p = Seismic force for anchorage of a wall to a diaphragm;
k_a = Factor to account for diaphragm flexibility, equal to 1.0 for rigid diaphragms and need not exceed 2.0 for flexible diaphragms;
L_f = The span, in feet, of a flexible diaphragm that provides lateral support for a wall; the span is between vertical primary seismic-force-resisting elements that provide lateral support to the flexible diaphragm in the direction considered;
k_h = Factor to account for variation in force over the height of the building when all diaphragms are rigid; for flexible diaphragms, use 1.0;
z_a = The height, in feet, of the wall anchor above the base of the structure, not to exceed h_n;
h_n = height, in feet, above the base to the roof level;
χ = Factor for calculation of out-of-plane wall forces, from Table 7-1, for the selected structural performance level;
S_{XS} = Spectral response acceleration parameter at short periods, as determined in accordance with Section 2.3.2 without any adjustment for soil–structure interaction; and
W_p = Weight of the wall tributary to the wall anchor.

7.2.13.2 Out-of-Plane Strength of Walls Wall components shall have adequate strength to span between locations of out-of-plane support when subjected to out-of-plane forces calculated using Equation (7-13) but not less than forces calculated using Equation (7-14). Demands on walls out of plane shall be considered a force-controlled action using Equation (7-37), with Q_E calculated using Equation (7-13), and C_1 and C_2 in Equation (7-37) taken as 1.0. χ used in Equation (7-37) shall be 1.0, and χ used in Equations (7-13) and (7-14) shall be per Table 7-2:

$$F_p = 0.4 S_{XS} \chi W_p \tag{7-13}$$

$$F_{p,\min} = 0.1 \chi W_p \tag{7-14}$$

where

F_p = Out-of-plane force per unit area for the evaluation or retrofit of a wall spanning between two out-of-plane supports;
χ = Factor for calculating out-of-plane wall forces, from Table 7-2, for the selected performance level;

Table 7-1. Factor χ for Calculation of Out-of-Plane Wall Anchorage Forces.

Structural Performance Level	χ
Collapse Prevention	0.9
Life Safety	1.3
Immediate Occupancy	2.0

Table 7-2. Factor χ for Calculation of Out-of-Plane Wall Strength.

Structural Performance Level	χ
Collapse Prevention	0.8
Life Safety	1.1
Immediate Occupancy	1.7

S_{XS} = Spectral response acceleration at short periods, as determined in accordance with Section 2.3.2 without any adjustment for soil–structure interaction; and
W = Weight of the wall per unit area.

7.2.14 Structures Sharing Common Elements Buildings sharing common vertical- or seismic-force-resisting elements shall be evaluated or retrofitted considering interconnection of the two structures, or they shall be separated as specified in this section.

7.2.14.1 Interconnection Buildings that share common elements, other than foundation elements, shall be thoroughly tied together so that they behave as an integral unit. Ties between the structures at each level shall be evaluated or retrofitted for the forces specified in Section 7.2.11. Analyses of the combined response of the buildings shall account for the interconnection of the structures and shall evaluate the structures as one integral unit.

If the shared common elements are foundation elements and the superstructures meet the separation requirements of Section 7.2.15, the structures need not be tied together. Shared foundation elements shall be evaluated or retrofitted considering an analysis of the combined response of the two buildings.

7.2.14.2 Separation Buildings that share common elements shall be completely separated by introducing seismic joints between the structures meeting the requirements of Section 7.2.15. Independent seismic-force-resisting systems shall be provided for each structure. Independent vertical support shall be provided on each side of the seismic joint, unless slide bearings are used and adequate bearing lengths are provided to accommodate the expected independent lateral movement of each structure. It shall be assumed for such purposes that the structures move out of phase with each other in opposite directions simultaneously. The shared elements shall be either completely removed or anchored to one of the structures in accordance with the applicable requirements of Section 7.2.12.

7.2.15 Building Separation

7.2.15.1 Minimum Separation Buildings shall be separated from adjacent structures to prevent pounding by a minimum distance s_i at any level i given by Equation (7-15), unless they are exempted as specified in Section 7.2.15.2:

$$s_i = \sqrt{\Delta_{i1}^2 + \Delta_{i2}^2} \tag{7-15}$$

where

Δ_{i1} = Lateral deflection of Building 1 under consideration, at level i, relative to the ground, calculated in accordance with the provisions of this standard for the selected Seismic Hazard Level; and
Δ_{i2} = Lateral deflection of an adjacent Building 2, at level i, relative to the ground, estimated using the provisions of this standard for the selected Seismic Hazard Level or other approved approximate procedure. Alternatively, it shall be permitted to assume that $\Delta_{i2} = 0.03h_i$ for any

structure in lieu of a more detailed analysis, where h_i is the height of level i above the base of building 2.

The value of s_i need not exceed 0.04 times the height of the level under consideration above the base of Building 1 at the location of potential impact.

Refer to Chapter 14 for building separation requirements for seismically isolated structures.

7.2.15.2 Separation Exceptions For Structural Performance Levels of Life Safety or lower, buildings adjacent to structures that have diaphragms located at the same elevation and differ in height by less than 50% of the height of the shorter building need not meet the minimum separation distance specified in Section 7.2.15.1.

Where an approved analysis procedure that accounts for the change in dynamic response of the structures caused by impact is used, the evaluated and retrofitted buildings need not meet the minimum separation distance specified in Section 7.2.15.1. Such an analysis shall demonstrate that

- The structures are capable of transferring forces resulting from impact for diaphragms located at the same elevation, or
- The structures are capable of resisting all required vertical and lateral forces considering the loss of any elements or components damaged by impact of the structures.

7.2.16 Verification of Analysis Assumptions Each component shall be evaluated to verify that locations of inelastic deformations assumed in the analysis are consistent with strength and equilibrium requirements along the component length. Each component shall also be evaluated for post-earthquake residual gravity load capacity by a rational analysis procedure approved by the Authority Having Jurisdiction that accounts for potential redistribution of gravity loads and reduction of strength or stiffness caused by earthquake damage to the structure.

7.3 ANALYSIS PROCEDURE SELECTION

An analysis of the building, including retrofit measures, shall be conducted to determine the forces and deformations induced in components of the building by ground motion corresponding to the selected Seismic Hazard Level or by other seismic geologic site hazards specified in Section 8.2.2.

The analysis procedure shall comply with one of the following:

1. Linear analysis subject to limitations specified in Section 7.3.1 and complying with the linear static procedure (LSP) in accordance with Section 7.4.1 or the linear dynamic procedure (LDP) in accordance with Section 7.4.2,
2. Nonlinear analysis subject to limitations specified in Section 7.3.2 and complying with the nonlinear static procedure (NSP) in accordance with Section 7.4.3 or the nonlinear dynamic procedure (NDP) in accordance with Section 7.4.4, or
3. Alternative rational analysis in accordance with Section 7.3.3.

The analysis results shall comply with the applicable acceptance criteria selected in accordance with Section 7.5.

7.3.1 Linear Procedures Linear procedures shall be permitted for any of the following:

1. Buildings classified as Common Building Types W1, W1a, W2, CFS1, CFS2, or PC1.
2. Buildings classified as URM Common Building Type, provided they have all of the following characteristics:
   (a) Flexible diaphragms at all levels above the base of the structure;
   (b) Vertical elements of the seismic-force-resisting system consisting of unreinforced masonry shear walls or a combination of predominantly unreinforced masonry and incidental concrete shear walls;
   (c) A minimum of two lines of walls in each principal direction, except for single-story buildings with an open front on one side; and
   (d) A maximum of six stories above the base of the structure.
3. For all other buildings, subject to the limitations in Section 7.3.1.1.

For buildings incorporating base isolation systems or supplemental energy dissipation systems, the additional limitations of Section 14.5.2, or Section 15.4, shall apply.

7.3.1.1 Method to Determine Limitations on Use of Linear Procedures The method presented in this section shall be used to determine the applicability or limitations of linear analysis procedures based on four configurations of irregularity defined in Section 7.3.1.1.1 through Section 7.3.1.1.4. The determination of irregularity shall be based on the configuration of the original or retrofit structure. A linear analysis to determine irregularity shall be performed by either an LSP in accordance with Section 7.4.1 or an LDP in accordance with Section 7.4.2. The results of this analysis shall be used to identify the magnitude and uniformity of distribution of inelastic demands on the primary elements and components of the seismic-force-resisting system.

The magnitude and distribution of inelastic demands for existing and added primary elements and components shall be defined by demand–capacity ratios (DCRs) and computed in accordance with Equation (7-16):

$$\mathrm{DCR} = \frac{Q_{UD}}{Q_{CE}} \tag{7-16}$$

where

Q_{UD} = Deformation-controlled action caused by gravity loads and earthquake forces calculated in accordance with Section 7.5.2; and

Q_{CE} = Expected strength of the deformation-controlled action of a component or element, calculated as specified in Chapters 8 through 13.

DCRs shall be calculated for each action (such as axial force, moment, or shear) of each primary component. The controlling action for the component shall be the one with the largest DCR. The DCR for this action shall be termed the controlling component DCR. The largest DCR for any element at a particular story is termed the controlling element DCR at that story. If an element at a particular story contains multiple components, then the component with the largest computed DCR shall define the controlling component for the element at that story.

A linear analysis shall be permitted to demonstrate nonconformance with a Performance Objective, even where nonlinear analysis is required to demonstrate compliance.

7.3.1.1.1 In-Plane Discontinuity Irregularity An in-plane discontinuity irregularity shall be considered to exist in any primary element of the seismic-force-resisting system wherever a seismic-force-resisting element is present in one story but does not continue, or is offset within the plane of the element, in the story immediately below. Figure 7-1 shows such a condition. For buildings that have an in-plane discontinuity irregularity, linear procedures shall not be used unless the structural elements

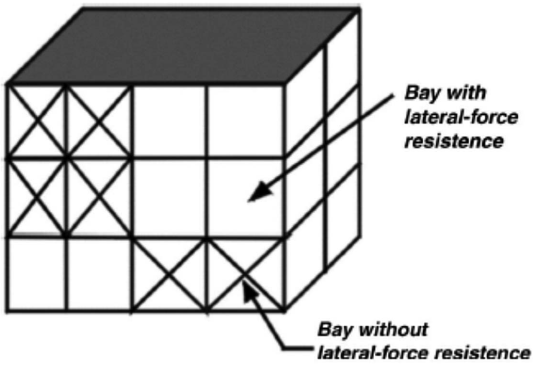


Figure 7-1. In-plane discontinuity in a seismic-force-resisting system.

supporting the discontinuous vertical lateral-force-resisting elements and the collector elements transferring forces between discontinuous vertical lateral-force-resisting elements are evaluated as force-controlled components.

7.3.1.1.2 Out-of-Plane Discontinuity Irregularity An out-of-plane discontinuity irregularity shall be considered to exist in any primary element of the seismic-force-resisting system where an element in one story is offset out of plane relative to that element in an adjacent story, as shown in Figure 7-2. For buildings that have an out-of-plane discontinuity, linear procedures shall not be used unless the structural elements supporting the discontinuous vertical lateral-force-resisting elements and the diaphragms and associated collector elements transferring forces between discontinuous vertical elements shall be evaluated as force-controlled components in accordance with Sections 7.4.1.3.4 and 7.4.2.3.2.

7.3.1.1.3 Weak Story Irregularity A weak story irregularity shall be considered to exist in any direction of the building if the ratio of the average shear DCR for elements of the vertical seismic-force-resisting system in any story to that of an adjacent story in the same direction exceeds 125%. The average DCR of a story shall be calculated by Equation (7-17):

$$\overline{\mathrm{DCR}} = \frac{\sum_{1}^{n} \mathrm{DCR}_i V_i}{\sum_{1}^{n} V_i} \qquad (7\text{-}17)$$

where

$\overline{\mathrm{DCR}}$ = Average DCR for elements in the story;

DCR_i = Controlling action DCR for element i of the story;

V_i = Total calculated lateral shear force in an element i caused by earthquake response, assuming that the structure remains elastic; and

n = Total number of elements in the story.

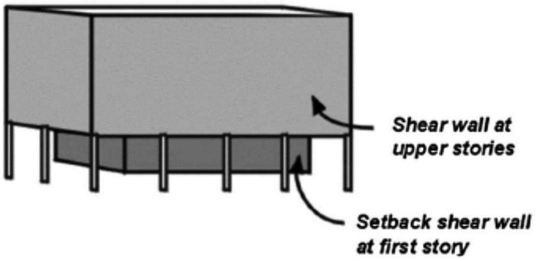


Figure 7-2. Typical building with out-of-plane offset irregularity.

For buildings with flexible diaphragms, each line of framing shall be independently evaluated.

If a weak story irregularity exists and any component DCR exceeds the lesser of 3.0 or the *m*-factor for the component action, then linear procedures shall not be used.

7.3.1.1.4 Torsional Strength Irregularity A torsional strength irregularity shall be considered to exist in any story if the diaphragm above the story under consideration is not flexible and, for a given direction, the ratio of the controlling element DCR for primary elements on one side of the center of resistance of a story to the controlling element DCR on the other side of the center of resistance of the story exceeds 1.5. If a torsional strength irregularity exists and any component DCR exceeds the lesser of 3.0 or the *m*-factor for the component action, then linear procedures shall not be used.

7.3.1.2 Limitations on Use of the Linear Static Procedure Where Section 7.3.1.1 permits the use of linear procedures, the linear static procedure shall not be used for a building with one or more of the following characteristics:

1. The fundamental period of the building, T, is greater than or equal to 3.5 times T_s;
2. The ratio of the horizontal dimension at any story to the corresponding dimension at an adjacent story exceeds 1.4 (excluding penthouses);
3. The building has a torsional stiffness irregularity in any story; a torsional stiffness irregularity exists in a story if the diaphragm above the story under consideration is not flexible and the results of the analysis indicate that the drift along any side of the structure is more than 150% of the average story drift;
4. The building has a vertical stiffness irregularity; a vertical stiffness irregularity exists where the average drift in any story (except penthouses) is more than 150% of that of the story above or below; and
5. The building has a nonorthogonal seismic-force-resisting system.

7.3.2 Nonlinear Procedures Nonlinear procedures shall be permitted for all buildings. Nonlinear procedures shall be used for analysis of buildings where linear procedures are not permitted. Data collection for use with nonlinear procedures shall be in accordance with Section 6.2.

7.3.2.1 Nonlinear Static Procedure The NSP shall be permitted for structures with all of the following characteristics:

1. The strength ratio μ_{strength}, calculated in accordance with Equation (7-32), is less than $\mu_{\max}$ calculated in accordance with Equation (7-33). If μ_{strength} exceeds $\mu_{\max}$, an NDP analysis shall be performed.
2. Higher mode effects are not significant, as defined in the following:
   - To determine if higher modes are significant, a modal response spectrum analysis shall be performed for the structure using sufficient modes to produce 90% mass participation. A second response spectrum analysis shall also be performed, considering only the first mode participation. Higher mode effects shall be considered significant if the shear in any story resulting from the modal analysis considering modes required to obtain 90% mass participation exceeds 130% of the corresponding story shear considering only the first mode response. It shall be permitted to use the soil–structure interaction modifications of Section 8.5, to demonstrate

compliance with this requirement, without requiring a site-specific hazard assessment.

- If higher mode effects are significant, the NSP shall be permitted if an LDP analysis is also performed to supplement the NSP. Buildings with significant higher mode effects must meet the acceptance criteria of this standard for both analysis procedures, except that an increase by a factor of 1.33 shall be permitted in the LDP acceptance criteria for deformation-controlled actions (*m*-factors) provided in Chapters 8 through 12. A building analyzed using the NSP, with or without a supplementary LDP evaluation, shall meet the acceptance criteria for nonlinear procedures specified in Section 7.5.3.

7.3.2.2 Nonlinear Dynamic Procedure The NDP shall be permitted for all structures. Where the NDP procedure is used, the Authority Having Jurisdiction shall consider the requirement of review and approval by an independent third-party engineer with experience in seismic design and nonlinear procedures.

7.3.3 Alternative Rational Analysis Use of an approved alternative analysis procedure that is rational and based on fundamental principles of engineering mechanics and dynamics shall be permitted. Such alternative analyses shall not adopt the acceptance criteria contained in this standard without first determining their applicability. All projects using alternative rational analysis procedures shall be reviewed and approved by an independent third-party engineer with experience in seismic design.

7.4 ANALYSIS PROCEDURES

Selection of an appropriate analysis procedure shall comply with Section 7.2.1.

7.4.1 Linear Static Procedure

7.4.1.1 Basis of the Procedure If the LSP is selected for seismic analysis of the building, the seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements shall be determined using a linearly elastic, static analysis in accordance with this section.

The pseudo seismic force defined in Section 7.4.1.3 shall be used to calculate internal forces and system displacements at the selected Seismic Hazard Level.

Results of the LSP shall be checked using the acceptance criteria of Section 7.5.2.

7.4.1.2 Period Determination for Linear Static Procedure The fundamental period of a building shall be calculated for the direction of response under consideration using one of the following analytical, empirical, or approximate methods specified in this section.

7.4.1.2.1 Method 1: Analytical Eigenvalue (dynamic) analysis of the mathematical model of the building shall be performed to determine the fundamental period of the building.

7.4.1.2.2 Method 2: Empirical The fundamental period of the building shall be determined in accordance with Equation (7-18):

$$T = C_t h_n^{\beta} \tag{7-18}$$

where

T = Fundamental period (in s) in the direction under consideration;
C_t = 0.035 for steel moment-resisting frame systems,
= 0.018 for concrete moment-resisting frame systems,
= 0.030 for steel eccentrically braced frame and steel buckling-restrained braced frame systems, and
= 0.020 for all other framing systems;
h_n = Height (in ft) above the base to the roof level; and
β = 0.80 for steel moment-resisting frame systems,
= 0.90 for concrete moment-resisting frame systems, and
= 0.75 for all other framing systems.

7.4.1.2.3 Method 3: Approximate The use of any of the following approximate methods shall be permitted:

1. For any building, use of Rayleigh's method or any other rational method to approximate the fundamental period shall be permitted.
2. For 1-story buildings with single-span flexible diaphragms, use of Equation (7-19) to approximate the fundamental period shall be permitted:

$$T = (0.1\Delta_w + 0.066\Delta_d)^{0.5} \tag{7-19}$$

   where Δ_w and Δ_d are in-plane wall and diaphragm displacements in inches because of a lateral force in the direction under consideration equal to the weight tributary to the diaphragm.
3. For 1-story buildings with multiple-span flexible diaphragms, use of Equation (7-19) shall be permitted as follows: a lateral force equal to the weight tributary to the diaphragm span under consideration shall be applied to calculate a separate period for each diaphragm span. The period that maximizes the pseudo seismic force shall be used for analysis of all walls and diaphragm spans in the building.
4. For unreinforced masonry buildings with single-span flexible diaphragms six stories or fewer high, use of Equation (7-20) to approximate the fundamental period shall be permitted:

$$T = (0.066\Delta_d)^{0.5} \tag{7-20}$$

   where Δ_d is the maximum in-plane diaphragm displacement in inches because of a lateral force in the direction under consideration equal to the weight tributary to the diaphragm.

7.4.1.3 Determination of Forces and Deformations for Linear Static Procedure Forces and deformations in elements and components shall be calculated for the pseudo seismic force of Section 7.4.1.3.1, using component stiffnesses calculated in accordance with Chapters 8 through 12. Pseudo seismic forces shall be distributed throughout the building in accordance with Sections 7.4.1.3.2 through 7.4.1.3.4. Alternatively, for unreinforced masonry buildings in which the fundamental period is calculated using Equation (7-20), pseudo seismic forces shall be permitted to be distributed in accordance with Section 7.4.1.3.5. Actions and deformations shall be modified to consider the effects of torsion in accordance with Section 7.2.4.2.

7.4.1.3.1 Pseudo Seismic Force for Linear Static Procedure The pseudo lateral force in a given horizontal direction of a building shall be determined using Equation (7-21). This force shall be used to evaluate or retrofit the vertical elements of the seismic-force-resisting system:

$$V = C_1 C_2 C_m S_a W \tag{7-21}$$

where

V = Pseudo lateral force, and
C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear-elastic response.

For fundamental periods less than 0.2 s, C_1 need not be taken as greater than the value at T = 0.2 s. For fundamental periods greater than 1.0 s, C_1 = 1.0:

$$C_1 = 1 + \frac{\mu_{\text{strength}} - 1}{a(C_k T)^2} \tag{7-22}$$

where

a = Site class factor;
= 130 site Class A or B;
= 110 site Class BC;
= 90 site Class C;
= 75 site Class CD;
= 60 site Class D, DE, E, or F; and
μ_{strength} = Ratio of elastic strength demand to yield strength coefficient calculated in accordance with Equation (7-32) with the elastic base shear capacity substituted for shear yield strength, V_y.

Alternatively, μ_{strength} is permitted to be calculated using

$$\mu_{\text{strength}} = \frac{\text{DCR}_{\text{max}}}{1.5} C_m \geq 1.0 \tag{7-23}$$

where
DCR_{max} is the largest DCR computed for any primary component of a building in the direction of response under consideration, taking $C_1 = C_2 = C_m = 1.0$;

T = Fundamental period of the building in the direction under consideration, calculated in accordance with Section 7.4.1.2, including modification for SSI effects of Section 7.2.8, if applicable;
C_k = Modification factor to convert the elastic fundamental period of the building, T, to the effective fundamental period;
= 1.1 if $\text{DCR}_{\text{max}} > 2.0$ for the Life Safety, Limited Safety, and Collapse Prevention Performance Levels;
= 1.0 in all other cases; and
C_2 = Modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on maximum displacement response. For fundamental periods less than 0.2 s, C_2 need not be taken as greater than the value at $T = 0.2$ s. For fundamental periods greater than 0.7 s, $C_2 = 1.0$:

$$C_2 = 1 + \frac{1}{800}\left(\frac{\mu_{\text{strength}} - 1}{C_k T}\right)^2 \tag{7-24}$$

Alternately, it shall be permitted to use C_1C_2 per Table 7-3, where m_{max} is the largest m-factor for all primary elements of the building in the direction under consideration.

C_m = Effective mass factor to account for higher modal mass participation effects obtained from Table 7-4. C_m shall be taken as 1.0 if the fundamental period, T, is greater than 1.0 s;
S_a = Response spectrum acceleration obtained from the procedure specified in Section 2.3 and determined at the fundamental period, T, and damping ratio of the building in the direction under consideration. Where S_a is taken from the multipoint spectrum or site-specific response spectrum and the period T is less than the period at which spectral acceleration is 90% of the maximum ($T_{0.9\text{max}}$), S_a shall be taken as 90% of the maximum value of spectral acceleration; and
W = Effective seismic weight of the building per Section 7.2.2.

Table 7-3. Alternate Values for Modification Factors C_1C_2.

Fundamental Period	$m_{\text{max}} < 2$	$2 \leq m_{\text{max}} < 6$	$m_{\text{max}} \geq 6$
$T \leq 0.3$	1.1	1.4	1.7
$0.3 < T \leq 1.0$	1.0	1.1	1.2
$T > 1.0$	1.0	1.0	1.0

7.4.1.3.2 Vertical Distribution of Seismic Forces for Linear Static Procedure The vertical distribution of the pseudo lateral force shall be as specified in this section for all buildings except unreinforced masonry buildings with flexible diaphragms and seismically isolated structures, for which the pseudo lateral force shall be permitted to be distributed in accordance with Section 7.4.1.3.5 and Section 14.5.2.5, respectively. The seismic force F_x applied at any floor level x shall be determined in accordance with Equations (7-25) and (7-26):

$$F_x = C_{vx} V \tag{7-25}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{7-26}$$

where

C_{vx} = Vertical distribution factor;
k = 2.0 for $T \geq 2.5$ s;
= 1.0 for $T \leq 0.5$ s (linear interpolation shall be used to calculate values of k for intermediate values of T);
V = Pseudo lateral force from Equation (7-21);
w_i = Portion of the effective seismic weight W located on or assigned to level i;
w_x = Portion of the effective seismic weight W located on or assigned to level x;
h_i = Height from the base to level i; and
h_x = Height from the base to level x.

7.4.1.3.3 Horizontal Distribution of Seismic Forces for Linear Static Procedure The seismic forces at each floor level of the building calculated using Equation (7-25) shall be distributed according to the distribution of mass at that floor level.

Table 7-4. Values for Effective Mass Factor C_m.

No. of Stories	Concrete Moment Frame	Concrete Shear Wall	Concrete Pier-Spandrel	Steel Moment Frame	Steel Concentrically Braced Frame	Steel Eccentrically Braced Frame	Other
1–2	1.0	1.0	1.0	1.0	1.0	1.0	1.0
3 or more	0.9	0.8	0.8	0.9	0.9	0.9	1.0

Note: C_m shall be taken as 1.0 if the fundamental period, T, in the direction of response under consideration is greater than 1.0 s.

7.4.1.3.4 Diaphragms for Linear Static Procedure Diaphragms and their connections to the vertical elements of the seismic-force-resisting system shall meet the requirements of Section 7.2.11 and this section. Diaphragms shall be evaluated or retrofitted to resist the combined effects of the lateral inertial force, F_{px}, calculated in accordance with Equation (7-27), and horizontal forces resulting from offsets in, or changes in the stiffness of, the vertical seismic framing elements above and below the diaphragm. Actions resulting from offsets in or changes in the stiffness of the vertical seismic framing elements shall be added directly to the diaphragm inertial forces.

$$F_{px} = \left(\frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} \right) w_{px} \tag{7-27}$$

where

F_{px} = Diaphragm inertial force at level x,
F_i = Lateral inertial force applied at level i given by Equation (7-25),
w_i = Portion of the effective seismic weight W located on or assigned to floor level i, and
w_{px} = Portion of the effective seismic weight W tributary to the diaphragm located on or assigned to floor level x.

The seismic force on each flexible diaphragm shall be distributed along the span of that diaphragm, proportional to its displaced shape.

Forces in diaphragm collectors and connections between the diaphragm and the vertical seismic-force-resisting elements shall be the larger of the forces calculated from Equation (7-27) applied to the diaphragm at each level individually plus any forces resulting from offsets in, or changes in the stiffness of, the vertical seismic framing elements above and below the diaphragm using the vertical distribution of seismic forces of Section 7.4.1.3.2; or

Diaphragms transferring horizontal forces from discontinuous vertical elements shall be taken as force controlled. Actions on other diaphragms shall be considered force or deformation controlled as specified for diaphragm components in Chapters 9 through 12.

7.4.1.3.5 Distribution of Seismic Forces for Unreinforced Masonry Buildings with Flexible Diaphragms for Linear Static Procedure For unreinforced masonry buildings with flexible diaphragms for which the fundamental period is calculated using Equation (7-20), it shall be permitted to calculate and distribute the pseudolateral force as follows:

1. The period shall be calculated from Equation (7-10) for each span of the building and at each level.
2. The pseudo seismic force for each span shall be calculated by Equation (7-21).
3. The pseudo seismic forces calculated for all spans shall be applied and forces in the vertical seismic-force-resisting elements shall be calculated using tributary forces.
4. The diaphragm forces for evaluation of diaphragms shall be determined from the results of Step 3 and distributed along the diaphragm span considering its deflected shape.
5. The diaphragm deflections shall not exceed 6 in. (152 mm) for this method of distribution of pseudo seismic force to be applicable.

7.4.1.4 Damping for Linear Static Procedure For buildings analyzed using the linear static procedure, the response spectra shall be based on the damping specified in Section 7.2.4.6.

7.4.2 Linear Dynamic Procedure

7.4.2.1 Basis of the Procedure If the LDP is selected for seismic analysis of the building, the seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements shall be determined using a linearly elastic, dynamic analysis in compliance with the requirements of this section.

Buildings shall be modeled with linearly elastic stiffness and equivalent viscous damping values consistent with components responding at or near yield level, as defined in Section 7.5.1. Modeling and analysis procedures to calculate forces and deformations shall be in accordance with Section 7.4.2.2.

Results of the LDP shall be checked using the acceptance criteria of Section 7.5.2.

7.4.2.2 Modeling and Analysis Considerations for Linear Dynamic Procedure

7.4.2.2.1 General The ground motion characterized for dynamic analysis shall comply with the requirements of Section 7.4.2.2.2. The dynamic analysis shall be performed using the response spectrum method in accordance with Section 7.4.2.2.3 or the response history method in accordance with Section 7.4.2.2.4.

7.4.2.2.2 Ground Motion Characterization for Linear Dynamic Procedure The horizontal ground motion shall be characterized by the requirements of Section 2.3 and shall be one of the following:

1. A response spectrum as specified in Section 2.3.2. Where S_a is taken from the multipoint spectrum and the fundamental period in a horizontal direction is less than the period where spectral acceleration is 90% of the maximum ($T_{0.9\max}$), S_a shall be taken as 90% of the maximum value of spectral acceleration for the fundamental mode in that direction at periods below $T_{0.9\max}$. Where a two-period response spectrum is required and the period of the fundamental mode in a horizontal direction is less than T_0, S_a shall be taken as S_{xs} for all periods less than T_s in that direction.
2. A site-specific response spectrum as specified in Section 2.3.3. When the fundamental period in a horizontal direction is less than the period where spectral acceleration is 90% of the maximum ($T_{0.9\max}$), S_a shall be taken as 90% of the maximum value of spectral acceleration for the fundamental mode in that direction at periods below $T_{0.9\max}$.
3. Ground motion acceleration histories as specified in Section 2.3.4.

7.4.2.2.3 Response Spectrum Method for Linear Dynamic Procedure Dynamic analysis using the response spectrum method shall calculate peak modal responses for sufficient modes to capture at least 90% of the participating mass of the building in each of two orthogonal principal horizontal directions of the building.

Peak member forces, displacements, story forces, story shears, and base reactions for each mode of response shall be combined by either the square root sum of squares (SRSS) rule or the complete quadratic combination (CQC) rule.

Multidirectional seismic effects shall be considered in accordance with the requirements of Section 7.2.6.

7.4.2.2.4 Linear Response History Method For the LDP, response history analysis shall be performed in accordance with the requirements for the nonlinear response history method specified in Section 7.4.4.2.3.

7.4.2.3 Determination of Forces and Deformations for Linear Dynamic Procedure

7.4.2.3.1 Modification of Demands for Linear Dynamic Procedure All forces and deformations calculated using either the response spectrum or the response history method shall be multiplied by the product of the modification factors C_1 and C_2 defined in Section 7.4.1.3 and further modified to consider the effects of torsion in accordance with Section 7.2.4.2.

7.4.2.3.2 Diaphragms for Linear Dynamic Procedure Diaphragms and their connections to the vertical elements of the seismic-force-resisting system shall meet the requirements of Section 7.2.11 and this section. Diaphragms shall be evaluated or retrofitted to resist the combined effects of the seismic forces calculated by the LDP and the horizontal forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements above and below the diaphragm. The seismic forces in the diaphragms shall be calculated by one of two methods:

1. Directly from the LDP analysis model when the diaphragms are explicitly modeled and mass is distributed over the entire diaphragm at each level.
2. The diaphragm seismic inertial forces calculated using Equation (7-27) with F_i taken as the difference in the LDP story shear forces between the story below the diaphragm and the story above the diaphragm in lieu of forces from Equation (7-25) plus any horizontal forces resulting from offsets in, or changes in the stiffness of, the vertical seismic framing elements above and below the diaphragm. It shall be permitted to evaluate the diaphragm using Section 7.4.1.3.4. Actions resulting from offsets in or changes in stiffness of the vertical seismic framing elements shall be added directly to the diaphragm inertial forces.

Forces in diaphragm collectors and connections between the diaphragm and the vertical seismic-force-resisting elements shall be the larger of the following:

1. The forces calculated from Equation (7-27) applied to the diaphragm at each level individually plus any forces resulting from offsets in, or changes in the stiffness of, the vertical seismic framing elements above and below the diaphragm using the vertical distribution of seismic forces of the LDP.
2. The difference in forces from the LDP analysis between the shear in the vertical elements of the seismic force-resisting system below and above the diaphragm to which the collector elements and connections are delivering load when the diaphragm is explicitly modeled in a three-dimensional mathematical model.

Diaphragms receiving horizontal forces from discontinuous vertical elements shall be taken as force controlled. Actions on other diaphragms shall be considered force or deformation controlled as specified for diaphragm components in Chapters 9 through 12.

7.4.2.4 Damping for Linear Dynamic Procedure For buildings analyzed using the response spectrum method, modal damping ratios shall be determined in accordance with Section 7.2.4.6.

For buildings analyzed using the linear response history method, damping shall be modeled in accordance with the nonlinear dynamic procedures in Section 7.4.4.4. Target damping ratios shall be determined in accordance with Section 7.2.4.6.

7.4.3 Nonlinear Static Procedure

7.4.3.1 Basis of the Procedure If the NSP is selected for seismic analysis of the building, a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components of the building shall be subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a target displacement is exceeded. Mathematical modeling and analysis procedures shall comply with the requirements of Section 7.4.3.2. The target displacement shall be calculated by the procedure in Section 7.4.3.3.

7.4.3.2 Modeling and Analysis Considerations for Nonlinear Static Procedure

7.4.3.2.1 General Requirements for Nonlinear Static Procedure Selection of a control node, selection of seismic force patterns, determination of the fundamental period, and application of the analysis procedure shall comply with the requirements of this section.

The relation between base shear force and lateral displacement of the control node shall be established for control node displacements ranging between 0 and 150% of the target displacement, δ_t.

The component gravity loads shall be included in the mathematical model for combination with seismic forces as specified in Section 7.2.3. The seismic forces shall be applied in both the positive and negative directions, and the maximum seismic effects shall be used for analysis.

The analysis model shall be discretized to represent the force–deformation response of each component along its length to identify locations of inelastic action.

Primary and secondary components of seismic-force-resisting elements shall be included in the model, as specified in Section 7.2.4.3.

The force-displacement behavior of all deformation-controlled component actions shall be explicitly included in the model using backbone curves per Section 7.5.1 that include strength degradation and residual strength, if any. Chapters 8 through 12 provide parameters to develop backbone curves for typical components found in buildings. It shall be permitted to use the provisions of Section 7.6 to develop backbone curves from test data as an alternate to the backbone curves in Chapters 8 through 12 or for component actions not specified in those chapters. The valid range of modeling shall be established for all backbone curves incorporated in the mathematical model. Where Chapters 8 through 12 do not specifically stipulate a valid range of modeling or suitable test data is not available, the maximum deformations specified in Chapters 8 through 12 shall be used. It shall be permitted to extend the valid range of modeling of an element beyond the maximum deformation specified in Chapters 8 through 12 if the component action's strength and stiffness are degraded to a value less than or equal to 5% of Q_{CE}, once the maximum deformation specified in Chapters 8 through 12 is reached and it can be verified that component's ability to support gravity loads is maintained or the mathematical model simultaneously adjusts to the component's loss of gravity load support.

All component actions not explicitly modeled with a force-displacement relationship per Section 7.5.1 shall be considered force-controlled.

The NSP shall be used in conjunction with the acceptance criteria of Sections 7.5.3.2.2 and 7.5.3.2.3.

7.4.3.2.2 Component Modeling for Nonlinear Static Procedure Nonlinear component modeling using lumped or distributed plasticity models shall represent component force–deformation relationships specified in Chapters 8 through 15 or otherwise be shown to represent experimentally obtained cyclic response characteristics in accordance with Section 7.6. The nonlinear model shall be discretized such that any identified locations of inelastic action represent all potential local mechanisms that affect component response.

Fiber models are permitted in lieu of the force–deformation curve described in Section 7.5.1.2. Fiber models shall be calibrated, within the expected range of behavior, to represent force–deformation response in accordance with component action prescribed in Chapters 8 through 12 or in accordance with Section 7.6. Fiber models shall exhibit similar force–deformation response in the complete building analysis to the force–deformation curve specified in Chapters 8 through 12 or derived per Section 7.6.

Components not explicitly represented in the nonlinear model shall be evaluated outside the nonlinear model for potential inelastic actions and acceptance criteria per 7.5.3.2, and representative stiffness in the nonlinear model shall capture load distribution effects and any potential inelastic mechanisms on modeled components. Elastic components in the nonlinear model shall represent effective stiffness prescribed in the respective Chapters 8 through 12 to appropriately capture load distribution, and acceptance criteria for those elements shall be in accordance with 7.5.3.2.3.

7.4.3.2.3 Control Node Displacement for Nonlinear Static Procedure The control node shall be located at the center of mass at the roof of a building. For buildings with a penthouse, the floor of the penthouse shall be regarded as the level of the control node. The displacement of the control node in the mathematical model shall be calculated for the specified seismic forces.

7.4.3.2.4 Lateral Load Distribution for Nonlinear Static Procedure Lateral loads shall be applied to the mathematical model in proportion to the distribution of mass in the plane of each floor diaphragm. The vertical distribution of these forces shall be proportional to the shape of the fundamental mode in the direction under consideration.

7.4.3.2.5 Idealized Force–Displacement Curve for Nonlinear Static Procedure The nonlinear force–displacement relationship between base shear and displacement of the control node shall be replaced with an idealized relationship to calculate the effective lateral stiffness, K_e, and effective yield strength, V_y, of the building, as shown in Figure 7-3.

The first line segment of the idealized force–displacement curve shall begin at the origin and have a slope equal to the effective lateral stiffness, K_e. The effective lateral stiffness, K_e, shall be taken as the secant stiffness calculated at a base shear force equal to 60% of the effective yield strength of the structure. The effective yield strength, V_y, shall not be taken as greater than the maximum base shear force at any point along the force–displacement curve.

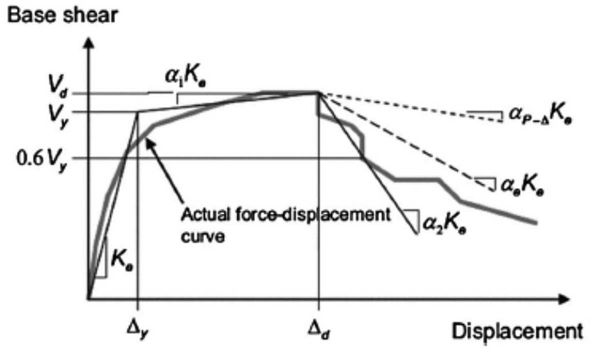


Figure 7-3. Idealized force–displacement curves.

The second line segment shall represent the positive post-yield slope ($\alpha_1 K_e$), determined by a point (V_d, Δ_d) and a point at the intersection with the first line segment such that the areas above and below the actual curve are approximately balanced. (V_d, Δ_d) shall be a point on the actual force–displacement curve at the calculated target displacement, or at the displacement corresponding to the maximum base shear, whichever is least.

The third line segment shall represent the negative post-yield slope ($\alpha_2 K_e$), determined by the point at the end of the positive post-yield slope (V_d, Δ_d) and the point at which the base shear degrades to 60% of the effective yield strength.

7.4.3.2.6 Period Determination for Nonlinear Static Procedure The effective fundamental period in the direction under consideration shall be based on the idealized force–displacement curve defined in Section 7.4.3.2.5. The effective fundamental period, T_e, shall be calculated in accordance with Equation (7-28):

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \tag{7-28}$$

where

T_i = Elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis,
K_i = Elastic lateral stiffness of the building in the direction under consideration calculated using the modeling requirements of Section 7.2.4.4, and
K_e = Effective lateral stiffness of the building in the direction under consideration.

7.4.3.2.7 Analysis of Mathematical Models for Nonlinear Static Procedure Separate mathematical models representing the framing along two orthogonal axes of the building shall be developed for two-dimensional analysis. A mathematical model representing the framing along two orthogonal axes of the building shall be developed for three-dimensional analysis.

The effects of torsion shall be evaluated in accordance with Section 7.2.4.2.

Independent analysis along each of the two orthogonal principal axes of the building shall be permitted unless concurrent evaluation of multidirectional effects is required by Section 7.2.6.

7.4.3.3 Determination of Forces, Displacements, and Deformations for Nonlinear Static Procedure

7.4.3.3.1 General Requirements for Nonlinear Static Procedure For buildings with rigid diaphragms at each floor level, the target displacement, δ_t, shall be calculated in accordance with Equation (7-29) or by an approved procedure that accounts for the nonlinear response of the building.

For buildings with nonrigid diaphragms at each floor level, diaphragm flexibility shall be explicitly included in the model. The target displacement shall be calculated as specified for rigid diaphragms, except that it shall be amplified by the ratio of the maximum displacement at any point on the roof to the displacement at the center of mass of the roof ($\delta_{\max}/\delta_{cm}$). $\delta_{\max}$ and δ_{cm} shall be based on a response spectrum analysis of a three-dimensional model of the building. The target displacement so calculated shall be no less than that displacement given by Equation (7-29). No line of vertical seismic framing shall be evaluated for displacements smaller than the target displacement.

Alternatively, for buildings with flexible diaphragms at each floor level, a target displacement shall be calculated for each line of vertical seismic framing. The target displacement for an individual line of vertical seismic framing shall be as specified for buildings with rigid diaphragms, except that the masses shall be assigned to each line on the basis of tributary area.

Element forces and deformations corresponding to the control node displacement equaling or exceeding the target displacement shall comply with acceptance criteria of Section 7.5.3.

7.4.3.3.2 Target Displacement for Nonlinear Static Procedure The target displacement, δ_t, at each floor level shall be calculated in accordance with Equation (7-29) and as specified in Section 7.4.3.3.1:

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \qquad (7\text{-}29)$$

where

S_a = Response spectrum acceleration at the effective fundamental period and damping ratio of the building in the direction under consideration, as calculated in Section 2.3.2 or 2.3.3;

g = Acceleration of gravity;

C_0 = Modification factor to relate spectral displacement of an equivalent single-degree-of-freedom (SDOF) system to the roof displacement of the building multiple-degree-of-freedom (MDOF) system calculated using one of the following procedures:

- The first mode mass participation factor multiplied by the ordinate of the first mode shape at the control node,
- The mass participation factor calculated using a shape vector corresponding to the deflected shape of the building at the target displacement multiplied by ordinate of the shape vector at the control node, or
- The appropriate value from Table 7-5, and

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear-elastic response calculated per Equation (7-30). For periods less than 0.2 s, C_1 need not be taken as greater than the value at T = 0.2 s. For periods greater than 1.0 s, C_1 = 1.0.

$$C_1 = 1 + \frac{\mu_{\text{strength}} - 1}{a T_e^2} \qquad (7\text{-}30)$$

where

a = Site class factor:
= 130 for Site Class A or B;
= 90 for Site Class C;
= 60 for Site Class D, E, or F;

T_e = Effective fundamental period of the building in the direction under consideration, in seconds;

μ_{strength} = Ratio of elastic strength demand to yield strength coefficient calculated in accordance with Equation (7-32); use of the NSP is not permitted where μ_{strength} exceeds $\mu_{\max}$, per Section 7.3.2.1; and

C_2 = Modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on the maximum displacement response calculated per Equation (7-31). For periods greater than 0.7 s, C_2 = 1.0.

$$C_2 = 1 + \frac{1}{800}\left(\frac{\mu_{\text{strength}} - 1}{T_e}\right)^2 \qquad (7\text{-}31)$$

The strength ratio μ_{strength} shall be calculated in accordance with Equation (7-32):

Table 7-5. Values for Modification Factor C_0.

	Shear Buildings*		Other Buildings
Number of Stories	**Triangular Load Pattern (1.1, 1.2, 1.3)**	**Uniform Load Pattern (2.1)**	**Any Load Pattern**
1	1.0	1.0	1.0
2	1.2	1.15	1.2
3	1.2	1.2	1.3
5	1.3	1.2	1.4
10+	1.3	1.2	1.5

*Buildings in which, for all stories, story drift decreases with increasing height.

Note: Linear interpolation shall be used to calculate intermediate values.

$$\mu_{\text{strength}} = \frac{S_a}{V_y / W} . C_m \qquad (7\text{-}32)$$

where S_a is defined previously, and

V_y = Yield strength of the building in the direction under consideration calculated using results of the NSP for the idealized nonlinear force–displacement curve developed for the building in accordance with Section 7.4.3.2.5;

W = Effective seismic weight, as calculated in Section 7.2.2; and

C_m = Effective mass factor from Table 7-4. Alternatively, C_m, taken as the effective modal mass participation factor calculated for the fundamental mode using an eigenvalue analysis, shall be permitted. C_m shall be taken as 1.0 if the fundamental period, T, is greater than 1.0 s.

For buildings with negative post-yield stiffness, the maximum strength ratio, $\mu_{\max}$, shall be calculated in accordance with Equation (7-33):

$$\mu_{\max} = \frac{\Delta_d}{\Delta_y} + \frac{|\alpha_e|^{-h}}{4} \qquad (7\text{-}33)$$

where

Δ_d = Lesser of the target displacement, δ_t, or displacement corresponding to the maximum base shear defined in Figure 7-3;

Δ_y = Displacement at effective yield strength defined in Figure 7-3;

h = $1 + 0.15 \ln T_e$; and

α_e = Effective negative post-yield slope ratio defined in Equation (7-34).

The effective negative post-yield slope ratio, α_e, shall be calculated in accordance with Equation (7-34):

$$\alpha_e = \alpha_{P-\Delta} + \lambda(\alpha_2 - \alpha_{P-\Delta}) \qquad (7\text{-}34)$$

where

α_2 = Negative post-yield slope ratio defined in Figure 7-3; this ratio includes $P-\Delta$ effects, in-cycle degradation, and cyclic degradation;

$\alpha_{P-\Delta}$ = Negative slope ratio caused by $P-\Delta$ effects; and

λ = Near-field effect factor:
= 0.8 if $S_{X1} \geq 0.6$ for BSE-2N; and
= 0.2 if $S_{X1} \leq 0.6$ for BSE-2N.

7.4.3.3.3 Modification of Demands for Nonlinear Static Procedure The target displacement shall be modified to consider the effects of torsion in accordance with Section 7.2.4.2.

7.4.3.3.4 Diaphragms for Nonlinear Static Procedure Diaphragms and their connections to the vertical elements of the seismic-force-resisting system shall meet the requirements of Section 7.2.11 and this section. Diaphragms shall be evaluated or retrofitted to resist the combined effects of the horizontal forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements above and below the diaphragm. If the diaphragms are not explicitly modeled in the NSP with nonlinear force–deformation properties, the seismic forces in the diaphragms shall be calculated by the LSP using Equation (7-27) with F_i computed from the NSP at the target displacement and, if different, at the displacement that produces the maximum base shear up to the target displacement in lieu of forces from Equation (7-25). Horizontal forces resulting from offsets in, or changes in the stiffness of, the vertical seismic framing elements above and below the diaphragm shall be added to forces in the diaphragm calculated using Equation (7-27). The larger diaphragm forces computed from Equation (7-27) at the target displacement and, if different, at the displacement that produces the maximum base shear up to the target displacement shall be used.

Forces in diaphragm collectors and connections between the diaphragm and the vertical seismic-force-resisting elements shall be the larger of the following:

1. The forces calculated from diaphragm inertial forces using Equation (7-27) applied to the diaphragm at each level individually plus any forces resulting from offsets in, or changes in the stiffness of, the vertical seismic framing elements above and below the diaphragm the vertical distribution of seismic forces of the NSP; or
2. The difference in forces from the NSP analysis between the shear in the vertical elements of the seismic-force-resisting system below and above the diaphragm to which the collector elements and connections are delivering load to.

Diaphragms receiving horizontal forces from discontinuous vertical elements, diaphragm collector elements, and connections between the diaphragm and collectors and vertical elements of the seismic-force-resisting system shall have their component actions considered force controlled unless a three-dimensional mathematical model is used and the diaphragm component actions are explicitly modeled with nonlinear force-displacement curves or fiber models. Actions on other diaphragms shall be considered force or deformation controlled as specified for diaphragm components in Chapters 9 through 12.

If the diaphragm is idealized as rigid or modeled as semirigid with linear-elastic elements within the mathematical model, and if the diaphragm does not receive forces from discontinuous vertical lateral force-resisting elements or have vertical lateral-force-resisting elements that do not continue above the diaphragm, except roof diaphragms, and is permitted to be considered deformation controlled, it shall be permitted to evaluate the diaphragm using forces from Equation (7-39) with m-factors taken as specified in Chapters 9 through 12 for the Performance Level being assessed but not greater than 2.

Alternatively, it shall be permitted to evaluate the diaphragms with the LSP or LDP using forces determined using either Section 7.4.1.3.4 or Section 7.4.2.3.2.

7.4.3.4 Damping for Nonlinear Static Procedure For buildings analyzed using the nonlinear static procedure, the damping shall be in accordance with Section 7.2.4.6.

7.4.4 Nonlinear Dynamic Procedure

7.4.4.1 Basis of the Procedure If the NDP is selected for seismic analysis of the building, a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components of the building shall be subjected to earthquake shaking represented by ground motion acceleration histories in accordance with Section 2.4.3 to obtain forces and displacements.

Calculated displacements and forces shall be compared directly with acceptance criteria specified in Section 7.5.3.

7.4.4.2 Modeling and Analysis Considerations for Nonlinear Dynamic Procedure

7.4.4.2.1 General Requirements for Nonlinear Dynamic Procedure The modeling and analysis requirements specified in Section 7.4.3.2 for the NSP shall apply to the NDP, excluding considerations of control node and target displacements. The mathematical model of the component action shall account for the hysteretic shape of the force–deformation relationship per Section 7.4.4.2.4.

7.4.4.2.2 Ground Motion Characterization for Nonlinear Dynamic Procedure For the NDP, earthquake shaking shall be characterized by discretized recorded or synthetic earthquake records as base motion meeting the requirements of Section 2.4.3.

7.4.4.2.3 Nonlinear Response History Method for Nonlinear Dynamic Procedure For the NDP, response history analysis shall be performed using horizontal ground motion acceleration histories prepared according to the requirements of Section 2.4.3.

If Ritz vector-based nonlinear response history analysis is adopted as the integration solution, the analysis shall include sufficient modes to capture at least 90% mass participation, the time step shall be sufficiently small to ensure convergence to a mathematically accurate solution, and sufficient vectors shall be included to capture accurately local dynamic response in the nonlinear elements.

Response parameters shall be calculated for each response history analysis. The number of analyses required, method of computing results, and treatment of concurrent effects shall be accounted for in accordance with Section 7.2.6.

7.4.4.2.4 Cyclic Response in Nonlinear Dynamic Procedure Cyclic force–deformation envelopes of nonlinear components shall incorporate changes to the unloading and reloading stiffness that capture effects of hysteretic pinching for the representative inelastic action in the component. The force–deformation behavior of the component action shall be shown to capture related effects on hysteretic shape throughout the range of component response in the analysis. Component hysteretic behavior actions shall be identified as one of the following by comparing the hysteretic area of the subassemblage test to the equivalent elastic perfectly plastic test per Figure 7-4:

1. Negligible Pinching: The hysteretic shape is 75% or more of the area encompassed by the envelope of the hysteretic shape assuming fully elastic-plastic behavior in all four quadrants of the force-displacement plot when displaced cyclically to the "a" parameter.
2. Low Pinching: The hysteretic shape envelops 50% to 75% of the area encompassed by the envelope of the hysteretic shape assuming fully elastic-plastic behavior in all four quadrants of the force-displacement plot when displaced cyclically to the "a" parameter.
3. Moderate Pinching: The hysteretic shape envelopes 30% to 50% of the area encompassed by the envelope of the

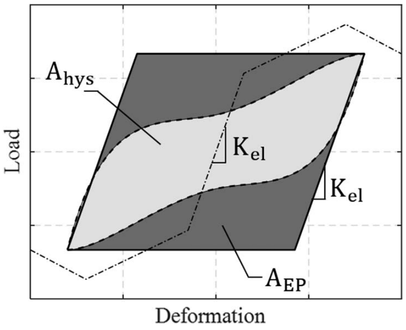


Figure 7-4. Comparison of hysteretic energy.

hysteretic shape assuming fully elastic-plastic behavior in all four quadrants of the force-displacement plot when displaced cyclically to the "a" parameter.

4. Significant Pinching: The hysteretic shape envelopes less than 30% of the area encompassed by the envelope of the hysteretic shape assuming fully elastic-plastic behavior in all four quadrants of the force-displacement plot. In a significantly pinched hysteretic shape, there is virtually no envelope of the force-displacement response in the second and fourth quadrants of the force-displacement plot.

As an alternative, the level of pinching shall be permitted to be based on a subassemblage test of a similar component. It shall be permitted to represent inelastic actions as having moderate pinching behavior and shall be used where experimental data is not available for the component type or Chapters 8 through 12 do not provide guidance for the inelastic action being modeled.

7.4.4.2.5 Adaptive Models in Nonlinear Dynamic Procedure It shall be permitted to use adaptive force–deformation models in lieu of force–deformation curve models or fiber models permitted in Section 7.4.3.2.1. Adaptive force–deformation models shall conform to the requirements in Section 7.6.5.

7.4.4.3 Determination of Forces and Deformations for Nonlinear Dynamic Procedure Dynamic analysis performed using the nonlinear response history method shall calculate building response at discrete time steps using discretized recorded or synthetic ground motion acceleration histories. Average component actions, included in forces and deformations, shall be determined as follows:

1. Where component response is independent of the direction of action, the average shall be calculated as the mathematical mean of the maximum absolute response from each response history analysis.
2. Where component response is dependent on the direction of action, the average response parameter shall be calculated independently for each direction and axis as the mathematical means of the maximum positive and minimum negative response from each response history analysis.

7.4.4.3.1 Modification of Demands for Nonlinear Dynamic Procedure The effects of torsion shall be considered in accordance with Section 7.2.4.2.

7.4.4.3.2 Diaphragm Forces for Nonlinear Dynamic Procedure Diaphragms, including their chords, collectors, ties, and connections to the vertical elements of the seismic-force-resisting system shall meet the requirements of Section 7.2.11 and this section. Diaphragms shall be evaluated or retrofitted to resist the effects of the seismic forces calculated based on the average of the maximum acceleration in each ground motion plus the average of the maximum horizontal forces in each ground motion resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements above and below the diaphragm.

Forces in diaphragm collectors and connections between the diaphragm and the vertical seismic-force-resisting elements shall be the difference in forces between the shear in the vertical elements of the seismic-force-resisting system below and above the diaphragm to which the collector element and connections are delivering load. The diaphragm collector and connection design forces shall be the average of the maximum difference in forces above and below the diaphragm considering the suite of ground motions.

Diaphragms receiving horizontal forces from discontinuous vertical elements, diaphragm collector elements, and connections between the diaphragm and collectors and vertical elements of the seismic-force-resisting system shall have their component actions considered force controlled unless a three-dimensional mathematical model is used and the diaphragm component actions are explicitly modeled with nonlinear force-displacement curves or fiber models. Actions on other diaphragms shall be considered force or deformation controlled as specified for diaphragm components in Chapters 9 through 12.

If the diaphragm is idealized as rigid, or modeled as semirigid with linear-elastic elements within the mathematical model (without nonlinear force–deformation properties), and if the diaphragm does not receive forces from discontinuous vertical lateral-force-resisting elements or have vertical lateral-force-resisting elements that do not continue above the diaphragm, except roof diaphragms, it shall be permitted to evaluate the diaphragm using Equation (7-36) with m taken as specified in Chapters 9 through 12 for the Performance Level being assessed but not greater than 2.

Alternatively, it shall be permitted to evaluate the diaphragms with the LSP or LDP using forces determined using either Section 7.4.1.3.4 or Section 7.4.2.3.2.

7.4.4.4 Damping for Nonlinear Dynamic Procedure For the nonlinear dynamic procedure, the target elastic equivalent viscous damping ratio shall be calculated using Equation (7-35):

$$\beta = \frac{0.36}{\sqrt{h}} \leq 0.05 \qquad (7\text{-}35)$$

EXCEPTIONS:

1. For the Life Safety, Limited Safety, and Collapse Prevention Performance Levels, the equivalent viscous damping ratio calculated per Equation (7-35) shall not be less than 2.5% ($\beta = 0.025$).
2. For structural steel buildings without exterior cladding, the target elastic equivalent viscous damping ratio shall not exceed 1% ($\beta = 0.01$).
3. For buildings using seismic isolation technology or enhanced energy dissipation technology, an equivalent viscous damping ratio, β, shall be calculated using the procedures specified in Chapters 14 and 15;
4. Higher target elastic equivalent viscous damping ratios shall be permitted if substantiated through analysis or test data.

Equivalent viscous damping shall be modeled using Rayleigh damping, modal damping, or other rational methodology. Where equivalent viscous damping is implemented using mass and stiffness proportional methods, the target equivalent viscous damping ratios shall be applied such that:

1. The average equivalent viscous damping ratio, weighted by mass participation over the modes required to achieve 90% mass participation, shall not exceed the target equivalent viscous damping ratio; and
2. No more than eight times the first translational mode damping is provided in the highest translational mode required to achieve 90% mass participation, unless substantiated through analysis or test data; and
3. The total elastic equivalent viscous damping ratio in the range of 0.2 times and 1.5 times the fundamental period in each direction is no more than the target elastic effective viscous damping ratio.

7.5 ACCEPTANCE CRITERIA

7.5.1 General Requirements The acceptability of force- and deformation-controlled actions shall be evaluated for each component in accordance with the requirements of this section. Each component shall be classified as primary or secondary in accordance with Section 7.2.4.3, and each component's action shall be classified as deformation controlled (ductile) or force controlled (nonductile) in accordance with Section 7.5.1.1 and critical or noncritical in accordance with Section 7.5.1.2. Component strengths, material properties, and component capacities shall be determined in accordance with Sections 7.5.1.3, 7.5.1.4, and 7.5.1.5, respectively. Component acceptance criteria shall be determined in accordance with provisions in Chapters 8 through 15. Component acceptance criteria not specified in this standard shall be determined by reference to suitable laboratory test data in accordance with Section 7.6.

To achieve a selected Performance Objective, the building shall be provided with at least one continuous load path to transfer seismic forces, induced by ground motion in any direction, from the point of application of the seismic force to the final point of resistance. All primary and secondary components shall be capable of resisting force- and deformation-controlled actions within the applicable acceptance criteria of the selected Performance Level.

Components analyzed using the linear procedures of Sections 7.4.1 and 7.4.2 shall satisfy the requirements of Section 7.5.2. Components analyzed using the nonlinear procedures of Sections 7.4.3 and 7.4.4 shall satisfy the requirements of Section 7.5.3. Foundations shall satisfy the criteria specified in Chapter 8.

7.5.1.1 Deformation-Controlled and Force-Controlled Actions All actions shall be classified as either deformation controlled or force controlled using the component force versus deformation curves shown in Figure 7-5.

Deformation-controlled actions are defined in Chapters 8 through 12 of this standard by the designation of linear and nonlinear acceptance criteria. Where linear or nonlinear acceptance criteria are not specified in the standard or developed in accordance with Section 7.6, actions shall be classified as force controlled.

The Type 1 curve depicted in Figure 7-5 shall have an elastic range (Points A to B on the curve) and a plastic range (Points B to E), followed by loss of seismic-force-resisting strength at Point E and effective loss of lateral force or gravity-load-resisting strength at Point F. The plastic range shall have either a positive or negative postelastic slope (Points B to C) and a strength-degraded region with residual strength greater than 5% the strength at Point B to resist seismic forces and gravity loads (Points C to D). Component actions exhibiting Type 1 curve behavior shall be classified as deformation controlled if the plastic range is such that $e \geq 2g$; otherwise, they shall be classified as force controlled.

The Type 2 curve depicted in Figure 7-5 shall have an elastic range (Points A to B on the curve) and a plastic range (Points B to C). The plastic range shall have either a positive or negative postelastic slope (Points B to C), followed by loss of seismic-force-resisting strength at Point D to a residual strength less than 5% of the strength at Point B. Loss of lateral force or gravity-load-resisting strength shall occur at the deformation associated with Point F. Component actions exhibiting Type 2 curve behavior shall be classified as deformation controlled if the plastic range is such that $e \geq 2g$; otherwise, they shall be classified as force controlled.

The Type 3 curve depicted in Figure 7-5 shall have an elastic range (Points A to B on the curve) followed by loss of seismic-force-resisting strength at Point D and loss of lateral force or gravity-load-resisting strength at the deformation associated with Point F. Component actions exhibiting this behavior shall be classified as deformation controlled if the loss of lateral-force-resisting strength is greater than 5% of the strength at Point B and $f \geq 2g$; otherwise, they shall be classified as force controlled.

For nonlinear procedures, force-controlled actions are permitted to be reclassified as deformation-controlled actions with a residual strength less than 5% of the strength at Point B if represented in the mathematical modeling using a Type 2 or 3 curve, provided all of the following criteria are met:

1. The component action being reclassified exhibits the Type 3 behavior defined in this section,
2. The gravity-load-resisting load path is not altered or an alternate load path is provided to ensure that local stability is maintained in accordance with the load combinations of Section 7.2.3 at the anticipated maximum displacements predicted by the analysis, and
3. The total gravity load supported by all components that are reclassified from force controlled to deformation controlled

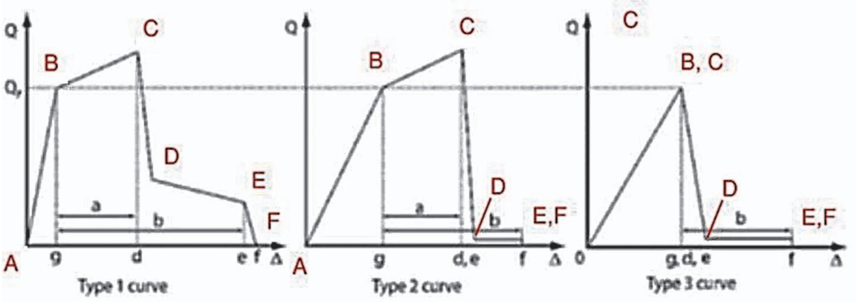


Figure 7-5. Component force versus deformation curves.

does not exceed 5% of the total gravity load being supported at that story.

When nonlinear procedures are used, any component action that is modeled as linear elastic, not explicitly represented in the mathematical model with a Type 1, Type 2, Type 3 or other nonlinear response curve conforming to Section 7.6, or not explicitly included in the mathematical model shall be treated as a force-controlled action.

Where overstrength of Type 3 components alters the expected mechanism in the building, the analysis shall be repeated with the affected Type 3 component strengths increased by the ratio Q_{CE}/Q_y, and all components shall be rechecked.

7.5.1.2 Critical and Noncritical Actions Component actions shall be classified as critical or noncritical. A component action shall be classified as critical if any of the following conditions apply:

1. The component's loss of vertical load carrying capacity would result in the collapse of more than one structural bay or one bay on more than one story.
2. The component's loss of lateral force resistance would result in any of the following:
   (a) A reduction in strength of the lateral-force-resisting system that results in a weak story per Section 7.3.1.1.3,
   (b) A torsional strength irregularity per Section 7.3.1.1.4 when evaluated using the LSP or LDP, or
   (c) A reduction in lateral-force-resisting strength in a story of 15% or more.

All other component actions shall be classified as noncritical.

7.5.1.3 Expected and Lower-Bound Strengths In Figure 7-5, Q_y represents the yield strength of the component. Where evaluating the behavior of deformation-controlled actions, the expected strength, Q_{CE}, shall be used. Q_{CE} is defined as the mean value of resistance of a component at the deformation level anticipated for a population of similar components, including consideration of the variability in material strength and strain hardening and plastic section development. Where evaluating the behavior of force-controlled actions, a lower-bound estimate of the component strength, Q_{CL}, shall be used. Q_{CL} is defined as the mean minus one standard deviation of the yield strengths, Q_y, for a population of similar components.

7.5.1.4 Material Properties Expected material properties shall be based on mean values of tested material properties. Lower-bound material properties shall be based on mean values of tested material properties minus one standard deviation, σ.

Nominal material properties, or properties specified in construction documents, shall be taken as lower-bound material properties unless otherwise specified in Chapters 8 through 12, 14, and 15. Corresponding expected material properties shall be calculated by multiplying lower-bound values by appropriate factors specified in Chapters 8 through 12, 14, and 15 to translate from lower-bound to expected values.

7.5.1.5 Component Capacities

7.5.1.5.1 General Detailed criteria for calculation of individual component force and deformation capacities shall comply with the requirements in individual materials chapters as follows:

1. Foundations: Chapter 8;
2. Components composed of steel or cast iron: Chapter 9;
3. Components composed of reinforced concrete: Chapter 10;
4. Components composed of reinforced or unreinforced masonry: Chapter 11;
5. Components composed of timber, cold-formed steel light-frame, gypsum, or plaster products: Chapter 12;
6. Nonstructural (architectural, mechanical, and electrical) components: Chapter 13; and
7. Seismic isolation systems and energy dissipation systems: Chapters 14 and 15.

Elements and components composed of combinations of materials are covered in the chapters associated with each material.

7.5.1.5.2 Linear Procedures If linear procedures are used, capacities for deformation-controlled actions shall be defined as the product of *m*-factors, κ-factors, and expected strengths, Q_{CE}. Capacities for force-controlled actions shall be defined as lower-bound strengths, Q_{CL}, as summarized in Table 7-6.

7.5.1.5.3 Nonlinear Procedures If nonlinear procedures are used, component capacities for deformation-controlled actions shall be taken as permissible inelastic deformation limits. Component capacities for force-controlled actions shall be taken as lower-bound strengths, Q_{CL}, as summarized in Table 7-7.

7.5.2 Linear Procedures

7.5.2.1 Forces and Deformations Component forces and deformations shall be calculated in accordance with linear analysis procedures of Sections 7.4.1 or 7.4.2.

Table 7-6. Calculation of Component Action Capacity: Linear Procedures.

Parameter	Deformation Controlled	Force Controlled
Existing material strength	Expected mean value with allowance for strain hardening	Lower-bound value (approximately mean value minus 1σ level)
Existing action capacity	κQ_{CE}	κQ_{CL}
New material strength	Expected material strength	Specified material strength
New action capacity	Q_{CE}	Q_{CL}

Table 7-7. Calculation of Component Action Capacity: Nonlinear Procedures.

Parameter	Deformation Controlled	Force Controlled
Deformation capacity (existing component)	κ × Deformation limit	N/A
Deformation capacity (new component)	Deformation limit	N/A
Strength capacity (existing component)	N/A	$\kappa \times Q_{CL}$
Strength capacity (new component)	N/A	Q_{CL}

7.5.2.1.1 Deformation-Controlled Actions for Linear Static Procedure or Linear Dynamic Procedure Deformation-controlled actions, Q_{UD}, shall be calculated in accordance with Equation (7-36):

$$Q_{UD} = Q_G + Q_E \tag{7-36}$$

where

Q_{UD} = Deformation-controlled action caused by gravity loads and earthquake forces,

Q_G = Action caused by gravity loads as defined in Section 7.2.3, and

Q_E = Action caused by the response to the selected Seismic Hazard Level calculated using either Section 7.4.1 or Section 7.4.2.

7.5.2.1.2 Force-Controlled Actions for Linear Static Procedure or Linear Dynamic Procedure Force-controlled actions, Q_{UF}, shall be calculated using one of the following methods:

1. Q_{UF} shall be taken as the maximum action that can be developed in a component based on a limit-state analysis considering the expected strength of the components in the load path of the component with the force-controlled action under consideration, or the maximum action developed in the component as limited by the nonlinear response of the building.
2. Q_{UF} shall be calculated in accordance with Equation (7-37) or Equation (7-38):

$$Q_{UF} = Q_G \pm \frac{\chi Q_E}{C_1 C_2} \tag{7-37}$$

$$Q_{UF} = Q_G \pm \frac{Q_E}{\mathrm{DCR_{min}}} \tag{7-38}$$

where

Q_{UF} = Force-controlled action caused by gravity loads in combination with earthquake forces;

χ = Factor for adjusting action caused by response for the selected Structural Performance Level;
= 1.0 for Collapse Prevention;
= 1.15 for Limited Safety; or
= 1.3 for Life Safety, Damage Control, or Immediate Occupancy;

C_1 = Equation (7-30) or per Table 7-3;

C_2 = Equation (7-31) or per Table 7-3; and

$\mathrm{DCR_{min}}$ = The minimum DCR, calculated in accordance with Equation (7-16), of all the deformation-controlled component actions in the load path to or from the component with the force-controlled action under consideration from pseudo-seismic forces applied in the direction of the vertical elements of the seismic force-resisting system containing that component. It is permitted to exclude the DCR of horizontal diaphragm and collector components in the load path if they are less than 1.0 in the determination of $\mathrm{DCR_{min}}$. $\mathrm{DCR_{min}}$ shall not be taken less than 1.0.

The smaller value of Q_{UF} from Equations (7-37) and (7-38) shall be used.

7.5.2.2 Acceptance Criteria for Linear Procedures

7.5.2.2.1 Acceptance Criteria for Deformation-Controlled Actions for LSP or LDP The Acceptance Ratio for deformation-controlled actions for LSP or LDP in primary and secondary components shall satisfy Equation (7-39).

$$\text{Acceptance Ratio} = Q_{UD}/m\kappa Q_{CE} \leq 1.0 \tag{7-39}$$

where

m = Component capacity modification factor to account for expected ductility associated with this action at the selected structural Performance Level

Q_{CE} = Expected strength of component deformation-controlled action of an element at the deformation level under consideration; Q_{CE}, the expected strength, shall be determined considering all coexisting actions on the component under the loading condition by procedures specified in Chapters 8 through 15; and

κ = Knowledge factor defined in Section 6.2.3.

m-factors for Immediate Occupancy, Life Safety, and Collapse Prevention are specified in Chapters 8 through 12, 14, and 15. The *m*-factor for Damage Control shall be taken as the average of the *m*-factor for Immediate Occupancy and Life Safety for Primary Components. The *m*-factors for Limited Safety shall be taken as the average of the *m*-factors for Life Safety and Collapse Prevention for primary and secondary components respectively.

7.5.2.2.2 Acceptance Criteria for Force-Controlled Actions for LSP or LDP The Acceptance Ratio for force-controlled actions for LSP or LDP in primary and secondary components shall satisfy Equation (7-40):

$$\text{Acceptance Ratio} = \frac{Q_{UF}}{\kappa Q_{CL}} \leq 1.0 \tag{7-40}$$

where Q_{CL} is the lower-bound strength of a force-controlled action of an element at the deformation level under consideration. Q_{CL}, the lower-bound strength, shall be determined considering all coexisting actions on the component under the loading condition by procedures specified in Chapters 8 through 12, 14, and 15.

7.5.2.2.3 Verification of Analysis Assumptions for Linear Static Procedure or Linear Dynamic Procedure In addition to the requirements in Section 7.2.16, the following verification of analysis assumptions shall be made:

1. Where moments caused by gravity loads in horizontally spanning primary components exceed 75% of the expected moment strength at any location, the possibility for inelastic flexural action at locations other than member ends shall be specifically investigated by comparing flexural actions with expected member strengths; and
2. Where linear procedures are used, formation of flexural plastic hinges away from member ends shall not be permitted.

7.5.3 Nonlinear Procedures

7.5.3.1 Forces and Deformations Component forces and deformations shall be calculated in accordance with nonlinear analysis procedures of Sections 7.4.3 or 7.4.4.

7.5.3.2 Acceptance Criteria for Nonlinear Procedures

7.5.3.2.1 Unacceptable Response for Nonlinear Dynamic Procedures Unacceptable response to ground motion shall not be permitted for NDP. Any one of the following shall be deemed to be an unacceptable response:

1. Computational solution fails to converge;
2. Predicted demands on deformation-controlled actions exceed the valid range of modeling, unless the component backbone curve is modeled such that its strength and stiffness are degraded to a residual strength of 5% or less of Q_{CE} once the maximum deformation specified in Chapters 8 through 12 is reached and gravity load support can be maintained at the maximum predicted deformation or the mathematical model simultaneously adjusts to the component's loss of gravity load support;
3. Predicted demands on force-controlled actions that are not explicitly included in the mathematical model with a nonlinear force-displacement curve and reclassified as deformation-controlled per Section 7.5.1.1 exceed their expected capacity;
4. Predicted deformation demands on element actions not explicitly modeled exceed the deformation limits at which the members are no longer able to carry their gravity loads; or
5. Predicted peak transient story drift exceeds 6% in any story, unless it can be demonstrated the structure can remain stable at the peak predicted transient drift.

EXCEPTION: For Limited Safety and Collapse Prevention Performance Levels, not more than one ground motion per 11 ground motions within a suite corresponding to a specific response spectrum shall be permitted to produce unacceptable response. When a ground motion produces unacceptable response, the average response shall be computed as 120% of the median value from ground motion records with acceptable and unacceptable responses, but not less than the mean value from only ground motion records producing acceptable responses. When computing the median value from ground motion records with acceptable and unacceptable responses, values from the ground motion records with unacceptable responses should be treated as larger than the median regardless of the actual value from the ground motion records.

7.5.3.2.2 Acceptance Criteria for Deformation-Controlled Actions for NSP or NDP Primary and secondary components shall have expected deformation capacities not less than maximum deformation demands calculated per the NSP or NDP. Primary and secondary component demands shall be within the acceptance criteria for nonlinear components at the selected structural performance level. Acceptance criteria for Immediate Occupancy, Life Safety, and Collapse Prevention are given in Chapters 8 through 12, 14, and 15. Acceptance criteria for Damage Control shall be taken as the a or e point specified in the tables that specify force–deformation curve modeling parameters in Chapters 8 through 12, 14, and 15. Acceptance criteria for Limited Safety shall be the average of the acceptance criteria for Life Safety and Collapse Prevention. It shall be permitted to allow noncritical deformation-controlled actions to exceed their Collapse Prevention limit in an NDP provided the analysis satisfies the acceptable response criteria in Section 7.5.3.2.1 is satisfied. Expected deformation capacities shall be determined considering all coexisting forces and deformations in accordance with Chapters 8 through 15.

Fiber model acceptance criteria shall be calibrated such that deformations over the component length in which inelastic action is expected does not exceed the force–deformation curves prescribed in Chapters 8 through 12 or based on test data in accordance with Section 7.6. The acceptance criteria resulting from the deformations of elements comprising fiber-sections shall not exceed acceptance criteria limits prescribed in Chapters 8 through 12 or based on test data in accordance with Section 7.6. It shall be permitted to exceed the force–deformation curves prescribed in Chapters 8 through 12 and develop acceptance criteria based on strain limits, if such limits are calibrated to test data per Section 7.6.

7.5.3.2.3 Acceptance Criteria for Force-Controlled Actions for NSP or NDP Force-controlled components that are not explicitly included in the mathematical model with nonlinear force–deformation properties per Section 7.5.1.1 shall satisfy Equation (7-41). Lower-bound strengths shall be determined considering all coexisting forces and deformations by procedures specified in Chapters 8 through 12, 14, and 15.

$$\gamma\chi(Q_{UF}-Q_G)+Q_G\le Q_{CL} \tag{7-41}$$

where

Q_{UF} = Force-controlled demand determined per section 7.4.3.3 or 7.4.4.3 for the NSP or NDP, respectively;
Q_G = Gravity load demand per Section 7.2.2;
Q_{CL} = Lower-bound component strength per Chapters 8 through 12, 14, and 15;
γ = Load factor obtained from Table 7-8 based on critical or noncritical designation per Section 7.5.1.2; and

χ is taken as 1.0 for Collapse Prevention or 1.3 for Life Safety and Immediate Occupancy.

EXCEPTIONS:

1. For actions other than shear in structural walls, the nominal element strength need not exceed the effects of gravity load plus the force demand determined by plastic mechanism analysis, where the analysis is based on expected material properties.
2. The product $\gamma\chi$ need not exceed a value of 1.5.

Where a lower value of $\gamma\chi$ results in a higher demand-capacity ratio, the check should also be performed using this lower value, except that the product $\gamma\chi$ need not be taken as less than 1.0.

When χ is greater than 1.0 and the NDP is used, it shall be permitted to perform an analysis with the ground motion records at the Seismic Hazard Level, where χ is greater than 1.0 amplified by the χ and the demands on force-controlled component actions computed with $\chi = 1.0$ in Equation (7-41), unless the analysis with the ground motions amplified produces more unacceptable responses than are permitted in Section 7.5.3.2.1 for the performance level being considered.

7.5.3.2.4 Verification of Analysis Assumption for NSP or NDP In addition to the requirements in Section 7.2.16, the following verification of analysis assumption shall be made:

Flexural plastic hinges shall not form away from component ends unless they are explicitly accounted for in modeling and analysis.

Table 7-8. Load Factor for Force-Controlled Behaviors.

Action Type	γ
Critical	1.3
Noncritical	1.0

7.6 EXPERIMENTALLY DERIVED MODELING PARAMETERS AND ACCEPTANCE CRITERIA

Modeling parameters and acceptance criteria for component actions not specifically addressed in this standard or as alternatives to those provided in this standard shall be derived using the experimentally obtained cyclic response characteristics of subassemblage tests, determined in accordance with this section. This section specifies rules for developing parameters for general use for various representative components and for project-specific testing to be used in conjunction with a component similar to the subassemblage tested. Where the modeling parameters are intended for general use, the procedures of Section 7.6.1 shall be followed. Where the modeling parameters are intended to be used on an individual project application, the procedures of Section 7.6.2 shall be followed. The provisions of this section shall not apply to seismic isolation or supplemental damping components. Modeling parameters and acceptance critera for seismic isolation and supplemental energy dissipation devices shall be in accordance with the requirements of Chapters 14 and 15, respectively.

7.6.1 Criteria for General Use Parameters This section provides requirements to develop modeling parameters and acceptance criteria for

1. Component actions not listed in this standard or other standards referenced in Chapter 18, or
2. Revising or providing alternate parameters for component actions in this standard or other standards referenced in Chapter 18.

7.6.1.1 Experimental Test Data The criteria for the component action shall be based on data from subassemblage. Load-deformation behavior shall be normalized to develop modeling parameters and acceptance criteria in accordance with Section 7.6.3. Parameters that affect the deformation capacity of the component action shall be identified and their effect on the modeling parameters quantified. When multiple component actions affect the deformation of a subassemblage, they shall be discretized, or provisions shall be provided to account for the influence of actions in the force–deformation space of other actions. Ranges of component configurations and the parameters affecting the component action shall be stipulated.

Standard cyclic or representative earthquake loading protocols shall be permitted. Subassemblage tests having at least two cycles at each increasing target drift level shall be included in the data set used to determine modeling parameters in accordance with Sections 7.6.3, 7.6.4, or 7.6.5. For each component action, a minimum of three tests for a given member size and specific component action is required.

Tests shall be conducted to the point at which the component action being tested resists negligible lateral force or the component loses the ability to support gravity loads. The deformation at which the component loses the ability to support gravity load shall be defined as the valid range of modeling. Tests that have not been conducted to the deformation at which the component loses the ability to support gravity load are permitted to be included, provided the maximum deformation tested is recorded as the valid range of modeling, or the valid range of modeling is established from other similar tests or a combination of similar tests and analytical simulations calibrated to tests at which the component resists negligible lateral force or cannot support gravity loads.

Tests covering the range of expected cyclic loading demands and histories shall be included where modeling parameters are developed in accordance with Section 7.6.4 or 7.6.5 for use with mathematical models that explicitly adapt behavior based on loading history.

7.6.1.2 Analytical Model Data It is permitted to supplement subassemblage test data in Section 7.6.1.2 with analytical results when determining parameters in accordance with Sections 7.6.3, 7.6.4, or 7.6.5. Analytical results shall not be used without being validated with physical subassemblage tests.

7.6.2 Criteria for Individual Project Testing Development of modeling parameters and acceptance criteria for specific component actions or for specific conditions on an individual project based on subassemblage testing is permitted in accordance with this section. The subassemblage test shall be based on the provisions of Section 7.6.2.1, and the development of modeling parameters and acceptance criteria from the test shall be based on Section 7.6.3. Peer review of this process shall be conducted in accordance with Section 7.6.2.3.

7.6.2.1 Experimental Setup Each tested subassemblage shall be an identifiable portion of the structural element or component, the stiffness and strength of which is required to be modeled as part of the structural analysis process. The objective of the experiment shall be to estimate the seismic-force-displacement relationships. These properties shall be used in developing an analytical model of the structure to calculate its response to selected earthquake shaking and other hazards and in developing acceptance criteria for strength and deformations. The limiting strength and deformation capacities shall be determined from an experimental program using multiple tests performed for the same configuration. Three or more tests shall be performed to determine the component behaviors throughout its expected range of performance.

The experimental setup shall replicate the construction details, support and boundary conditions, and loading conditions expected in the building. In cases where deformation components, such as flexure or shear, are modeled separately, test instrumentation shall be provided to enable backbone curves for each action to be derived from the overall test force–deformation relations. The tests shall include cyclic loading protocols with the number of cycles and displacement levels based on the expected response of the structure. At least two tests shall use the same cyclic loading protocol.

Loading protocols shall be representative of the seismic hazard, including but not limited to site effects, expected ground motions, component loading history that change material properties or preloading condition, and strong shaking duration.

Loading protocols shall test components to the point where the action under consideration ceases to resist lateral forces or, where applicable, the component can no longer support gravity loads. The deformation at which the component loses the ability to support gravity load shall be defined as the valid range of modeling. If the loading protocol does not test the component to failure, the maximum deformation of the component test shall be identified as the valid range of modeling.

Tests using monotonic loading are permitted to develop an adaptive model or are permitted as an augmentation to a minimum of three cyclic tests.

7.6.2.2 Data Reduction and Reporting A report shall be prepared for each series of subassemblage tests. The report shall include the following:

1. Description of the subassemblage being tested;
2. Description of the experimental setup, including the following:

   2.1. Details on fabrication of the subassemblage,
   2.2. Location and date of testing,
   2.3. Description of instrumentation used,
   2.4. Name of the person in responsible charge of the test, and
   2.5. Photographs of the specimen, taken before testing;
3. Description of the loading protocol used, including the following:
   3.1. Increment of loading or deformation applied,
   3.2. Rate of loading application, and
   3.3. Duration of loading at each stage;
4. Description, including photographic documentation, and limiting deformation value for all important behavior states observed during each subassemblage test, including the following, as applicable:
   4.1. Elastic range with effective stiffness reported,
   4.2. Plastic range,
   4.3. Onset of visible damage,
   4.4. Loss of seismic-force-resisting strength,
   4.5. Loss of gravity-load-carrying ability,
   4.6. Force–deformation plot for the subassemblage (noting the various behavior states), and
   4.7. Description of limiting behavior states defined as the onset of specific damage mode, change in stiffness or behavior (such as initiation of cracking or yielding), and failure modes.

Project documentation should include the force–deformation relationship for each test in accordance with Section 7.6.3 including the following:

1. Idealized force–deformation curve for each subassemblage test;
2. Mean force–deformation curve aggregated from all subassemblage tests;
3. The standard deviation or coefficient of variation of each point on the idealized force–deformation curve;
4. Hysteretic shape and amount of hysteretic pinching in accordance with Section 7.4.4.2.4;
5. Where applicable, recommended method to calibrate fiber model or adaptive model to based on the subassemblage tests; and
6. Recommended acceptance criteria for each performance level applicable to the project.

7.6.2.3 Peer Review Peer review of the development of modeling parameters and acceptance criteria for specific component actions or for specific conditions on an individual project based on subassemblage testing shall be conducted in accordance with this section.

The peer reviewer shall be an independent engineer or engineers approved by the Authority Having Jurisdiction. The peer reviewers shall be experienced with the use of test data in design and analysis of structures.

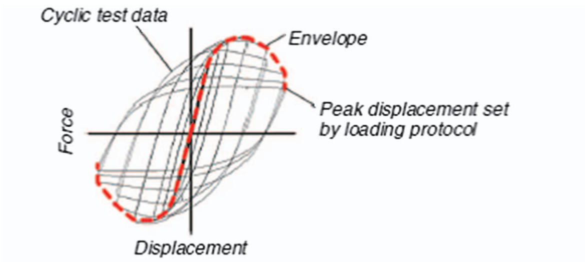
Upon completion of the review, the peer reviewer(s) shall provide the Authority Having Jurisdiction and the registered design professional a letter attesting to the scope of the review performed, concurrence with the alternative modeling parameters and acceptance criteria resulting from the test program, limits on the applicability of the proposed parameters and criteria, and inspection requirements, if required.

7.6.3 Modeling Parameters and Acceptance Criteria for Nonadaptive Force–Deformation Curves The following procedure shall be followed to develop modeling parameters and acceptance criteria for component actions represented by nonadaptive force–deformation curves based on experimental data:

1. An idealized force–deformation curve shall be developed from the experimental data. The backbone curve shall be plotted in a single quadrant. If the component action exhibits different response in opposite quadrants, separate backbone curves shall be plotted and parameters developed for the direction-specific actions. The backbone curves shall be constructed as follows:
   1.1. Envelope curves shall be drawn through each point of peak displacement during the first cycle of each increment of loading or deformation, as indicated in Figure 7-6. A smooth backbone curve shall be established by taking the mean of the value at each control point on the envelope curves as depicted in Figure 7-7.
   1.2. The backbone curve shall be approximated by a smooth curve or a series of linear segments, drawn to form a multisegmented curve conforming to one of the types indicated in Figure 7-5. When the backbone curve is idealized as a series of linear segments, it shall conform to Figure 7-8.

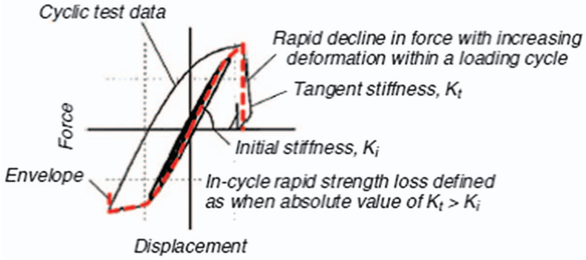
   The points located on Figure 7-8(a) shall be established as follows:

   - Point B: The effective yield point of the component action.
   - Point C: The deformation where the strength begins to deteriorate significantly due to approaching failure modes and in which the strength is never larger than the maximum strength in the tests. The displacement where the strength is 80% of the maximum strength on the backbone curve is permitted to be used with a corresponding strength equal to the maximum strength.
   - Point D: The deformation at which strength degradation levels out and a residual strength resisting lateral forces is reached. This point is permitted to be



Ductile behavior

(a)



Rapid Strength Loss Behavior

(b)

Figure 7-6. Envelopes from subassemblage test data.

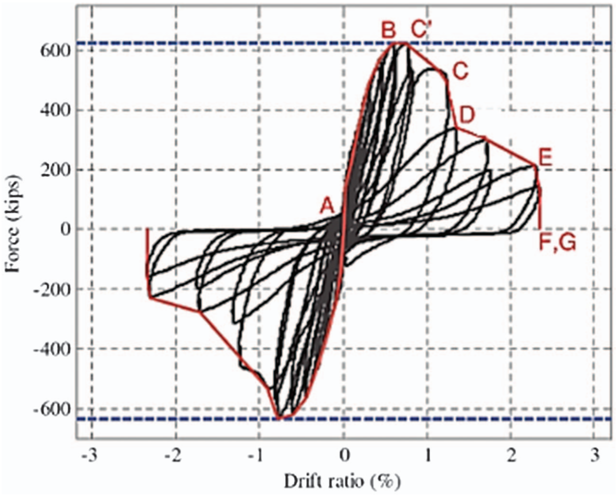


Figure 7-7. Backbone curves derived from envelopes of experimental test data.

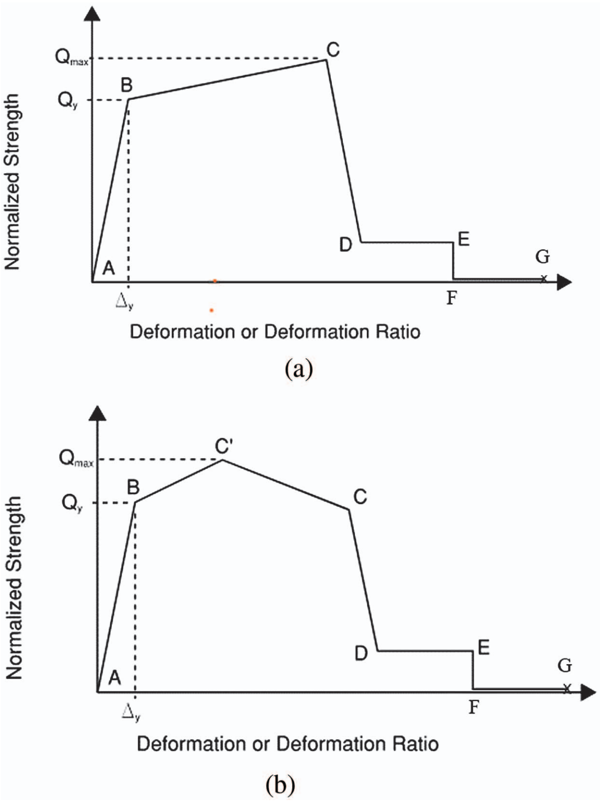


Figure 7-8. Multisegment linear idealized backbone curve: (a) common idealization, (b) idealization including maximum strength.

excluded and to connect Points C and E in the backbone curve if the test data indicate there is no region of leveled-off residual strength.
- Point E: The deformation at which the component begins to degrade rapidly from the residual strength to Point F. This point is permitted to be excluded and to extend the backbone curve from either Point C or D to Point F if the test data indicate no region of leveled-off residual strength.
- Point F: The deformation at which either the component action degrades to the point where resistance to lateral loads is less than 5% of the strength at Point B or the maximum displacement used in the tests.
- Point G: The deformation at which the component loses the ability to support gravity load, if the component supports gravity load. If the subassemblage tests were not conducted to a deformation sufficient to establish Point G or the component does not support gravity load, Point G shall be taken the same as Point F.

The points located on Figure 7-8(b) shall be established as follows:

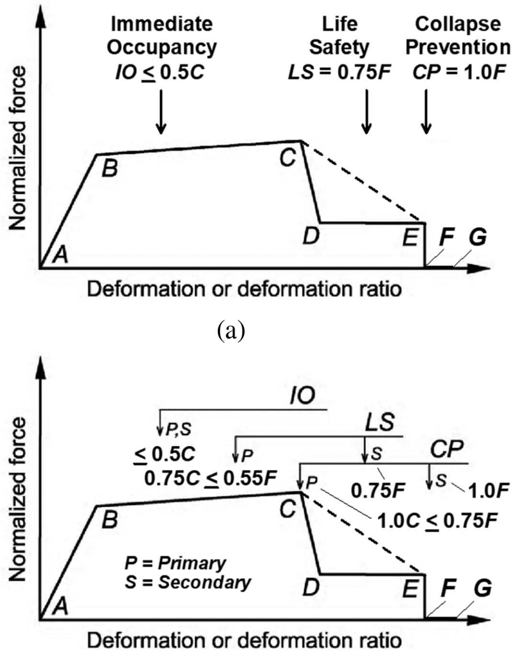
- Point B: The effective yield point of the component action.
- Point C′: The deformation at which the maximum strength is reached.
- Point C: The deformation where the strength begins to deteriorate significantly due to approaching failure modes and in which the strength is never larger than the resistance at C′. The point where the strength is 80% of the maximum strength on the backbone curve is permitted to be used with a corresponding strength equal to the maximum strength.
- Point D: The deformation at which strength degradation levels out and a residual strength is reached. This point is permitted to be excluded and to connect Points C and E in the backbone curve if the test data indicate there is no region of leveled-off residual strength.
- Point E: The deformation at which the component begins to degrade rapidly from the residual strength to Point F. This point is permitted to be excluded and to extend the backbone curve from either Point C or D to Point F if the test data indicate no region of leveled-off residual strength.
- Point F: The deformation at which either the component action degrades to the point where resistance to lateral loads is less than 5% of the strength at Point B or the maximum displacement used in the tests.
- Point G: The deformation at which the component loses the ability to support gravity load, if the component supports gravity load. If the subassemblage tests were not conducted to a deformation sufficient to establish Point G or the component does not support gravity load, Point G shall be taken the same as Point F.

1.3. When developing component action backbone curves for general use parameters in accordance with Section 7.6.1, the backbone curve shall be represented by the median of each point described in Item 1.2 in each test of similar configurations and parameters that control the component behavior. When developing component action backbone curves based on project-specific testing in accordance with Section 7.6.2, the backbone curve shall be represented by the mean of each point described in Item 1.2 in each test of similar configurations. Where the test data represent different subassemblages, the points shall be normalized based on the effective yield point, Point B in Figure 7-5. If equations are used to predict the values of Points B through G based on aggregating the results of different subassemblage tests, they shall provide an estimate of the median value and provide a range of component sizes that the parameters are applicable over and specify the coefficient of variation for each point. Backbone curves derived from monotonic tests shall only be included where permitted in Section 7.6.1 or 7.6.2. Even if the component action backbone curve is idealized as a smooth curve, the points on the curve in Figure 7-8 shall be derived to allow computation of the acceptance criteria in this section.

2. The stiffness of the subassemblage for use in linear procedures shall be taken as the slope of the first segment of the composite curve. The composite multilinear force–deformation curve shall be used for modeling in nonlinear procedures.
3. For the purpose of determining acceptance criteria, subassemblage actions shall be classified as being either force controlled or deformation controlled as specified in Section 7.5.1.1. Subassemblage actions shall be classified as deformation controlled unless the component exhibits Type 3 behavior in accordance with Figure 7-5 and either of the following conditions apply:
   - The residual strength is less than 20% of Q_{CE}; or
   - The deformation at Point F is less than two times the deformation at Point B in Figure 7-5.
4. The strength capacity, Q_{CL}, for force-controlled actions evaluated using either the linear or nonlinear procedures shall be taken as the strength of the force-controlled action calculated from the reference material standards in Chapters 8 through 12 using lower-bound material properties for the specific material. If the strength under consideration cannot be ascertained by the reference material standard, Q_{CL} shall be taken as the 16th percentile value or the mean minus one standard deviation determined from the subassemblage tests.
5. The acceptance criteria for deformation-controlled actions used in nonlinear procedures shall be the deformations corresponding with the following points on the curves of Figure 7-8:
   5.1. Immediate Occupancy:
   The deformation at which permanent, visible damage occurred in the experiments, but not greater than 0.50 times the deformation at Point C.
   5.2. Damage Control:
   The deformation at Point C.
   5.3. Life Safety:
   The 10th percentile of the plastic deformation at Point G, but not less than 0.75 times the deformation at Point F.
   5.4. Limited Safety:
   The 18th percentile of the plastic deformation at Point G, but not less than 0.88 times the deformation at Point F.
   5.5. Collapse Prevention:
   The 25th percentile of the plastic deformation at Point G, but not less than the deformation at Point F.

Acceptance criteria derived using percentile of deformation limits shall only be permitted for general use parameters if there are more than 30 subassemblage tests used to develop the points on the component action backbone curve.

Figure 7-9a illustrates criteria for nonlinear procedures.



(a)

(b)

Figure 7-9. Acceptance criteria. (a) Nonlinear procedures (NSP and NDP). See text for criteria expressed as test data percentiles. (b) Linear procedures (LSP and LDP).

6. The *m*-factors used as acceptance criteria for deformation-controlled actions in linear procedures shall be determined as follows: (a) obtain the deformation acceptance criteria given in items 6.1 through 6.4; (b) then determine the ratio of this deformation to the deformation at yield, represented by the deformation Point B in the curves shown in Figure 7-8; (c) then multiply this ratio by a factor 0.75 to obtain the acceptable *m*-factor.
   6.1. Immediate Occupancy: Primary and secondary components
   The deformation at which permanent, visible damage occurred in the experiments but not greater than 0.50 times the deformation at Point C on the curves.
   6.2. Damage Control: Primary and secondary components
   1.5 times the deformation at which permanent, visible damage occurred in the experiments but not greater than 0.63 times the deformation at Point C on the curves.
   6.3. Primary components:
      6.3.1. Life Safety: 0.75 times the deformation at Point C on the curves, but not greater than 0.55 times the deformation at Point F.
      6.3.2. Limited Safety: 0.88 times the deformation at Point C on the curves, but not greater than 0.65 times the deformation at Point F.
      6.3.3. Collapse Prevention: The deformation at Point C on the curves, but not greater than 0.75 times the deformation at Point F.
   6.4. Secondary components:
      6.4.1. Life Safety: 0.75 times the deformation at Point F.
      6.4.2. Limited Safety: 0.88 times the deformation at Point F.
      6.4.3. Collapse Prevention: 1.0 times the deformation at Point F on the curve.

Figure 7-9b illustrates criteria for linear procedures.

7. Hysteretic parameters defining the expected behavior of the component shall be identified. Specifically, the action shall be identified as having stiffness degradation, strength and stiffness degradation, and whether in-cycle strength degradation is present. In addition, the component action shall be identified as either exhibiting negligible pinching, low pinching, moderate pinching, or significant pinching in accordance with Section 7.4.4.2.5.1.

7.6.4 Modeling Parameters and Acceptance Criteria for Component Actions Based on Experimental Data for Fiber Models Fiber modeling parameters and acceptance criteria for component actions based on experimental data shall meet all of the following criteria:

1. Force–deformation calibration: An idealized force–deformation curve shall be developed from the component experimental data as prescribed in Section 7.6.3. Material stress-deformation relationships and analytical component discretization, including but not limited to element mesh or integration points, in fiber models shall be adjusted such that the enveloped force–deformation relationship of the component or subassemblage is in substantial agreement with the experimental force–deformation curve, and stiffness and strength loss effects of nonmaterial fiber-related mechanisms are captured.
2. Fiber model acceptance criteria: The subassembly experimental force–deformation curve shall be used to classify component behavior as force-controlled or deformation-controlled, and acceptance criteria shall be developed for the subassembly action in accordance with Section 7.6.3. Local deformation acceptance criteria shall be developed for a defined hinge length or otherwise normalized component length. The local deformation acceptance criteria shall be based on the corresponding experimental subassembly force–deformation curve and related acceptance criteria.
3. Cyclic response in nonlinear dynamic procedure: The subassembly action shall be identified as exhibiting negligible pinching, low pinching, moderate pinching, or significant pinching. Cyclic force–deformation envelopes of fiber models shall capture such effects of hysteretic pinching, including unloading and reloading stiffness and energy dissipation for the representative inelastic action in the component. The enveloped force–deformation behavior shall be shown to capture related effects on hysteretic energy dissipation.

7.6.5 Modeling Parameters and Acceptance Criteria for Component Actions Based on Experimental Data for Adaptive Force–Deformation Models in the Mathematical Model The following procedures shall be followed to develop modeling parameters and acceptance criteria for component actions based on experimental data for use in adaptive force–deformation simulation tools. Backbone modeling parameters shall be extracted from data of individual tests as described in Section 7.6.3. In addition to variables related to boundary conditions and intrinsic subassemblage properties, modeling parameters shall be defined based on how the points identified in Section 7.6.3, Item 1.2, change based on loading history. Acceptance criteria shall be defined based on strength degradation thresholds associated with Points C, E, or F as appropriate for the target performance objective and defined in Section 7.6.3, Item 5. It shall be permitted to define acceptance criteria based on data from tests satisfying the loading protocols required in Section 7.6.1 or 7.6.2.

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CHAPTER 8

FOUNDATIONS, SUBSURFACE SOIL, AND GEOLOGIC SITE HAZARDS

8.1 SCOPE

This chapter sets forth general requirements for consideration of foundation and soil load–deformation characteristics, seismic evaluation and retrofit of foundations, and mitigation of geologic site hazards.

Section 8.2 specifies data collection for site characterization and defines geologic site hazards. Section 8.3 outlines procedures for mitigation of geologic site hazards. Sections 8.4 and 8.5 provide soil strength and stiffness parameters for consideration of foundation load–deformation characteristics for shallow and deep foundations, respectively. Section 8.6 specifies procedures for consideration of soil–structure interaction (SSI) effects. Section 8.7 specifies seismic earth pressures on building basement/retaining walls. Section 8.8 specifies requirements for seismic retrofit of foundations.

8.2 SITE CHARACTERIZATION

Site characterization shall include collection of information on the subsurface soil and building foundation as specified in Section 8.2.1 and on seismic–geologic site hazards as specified in Section 8.2.2 as applicable.

8.2.1 Subsurface Soil Foundation Information Information on the foundation and subsurface soil supporting the building, nearby foundation conditions, design foundation loads, and load–deformation characteristics of the foundation soils shall be obtained as specified in Sections 8.2.1.1 through 8.2.1.3 as applicable.

8.2.1.1 Subsurface Soil Conditions Subsurface soil conditions shall be established by field exploration to a depth sufficient to characterize the subsurface soil effects on the seismic performance of the structure. The following shall be documented in a geotechnical report:

1. Unit weight, γ; the effective stress friction angle, ϕ'; the undrained shear strength of clays, s_u; soil compressibility characteristics; small-strain soil shear modulus, $G_{\max}$; and Poisson's ratio, ν, unless it can be demonstrated that the property does not have an effect on the evaluation or retrofit of the structure;
2. The location of the water table and its historic high groundwater level beneath the building if within the depth of influence of the foundation capacity determination; and
3. Each soil layer's susceptibility to liquefaction, seismically induced total and differential settlement, and lateral spreading at each hazard level considered.

8.2.1.2 Foundation Conditions

8.2.1.2.1 Structural Foundation Information The following structural information shall be obtained for the foundation of the building in accordance with the data collection requirements of Section 6.2 and as modified in this chapter as required:

1. Foundation type;
2. Foundation configuration, including dimensions and locations; and
3. Material composition and details of construction.

8.2.1.2.2 Foundation Loads Loads on the foundations and the subsurface soils shall be determined in accordance with the requirements in Chapter 7 and as modified in this chapter.

8.2.1.3 Load–Deformation Characteristics of Subsurface Soil under Seismic Loading Load–deformation characteristics of soil supporting the foundation's vertical, lateral, and overturning actions, when required, shall be obtained from the soil properties as specified in Section 8.2.1.1 and documented in a geotechnical report(s).

8.2.1.4 Soil Shear Modulus and Poisson's Ratio Parameters For shallow foundations, the expected elastic soil properties relevant to dynamic stiffness shall be based on the expected properties of the soil at the soil–footing interface to a depth of two footing widths. If soil properties do not vary more than 50% from the average over this depth, the average value is permitted to be determined at a depth of $D_f + (\sqrt{B_f L_f})/2$,

where

D_f = Depth of the soil–footing interface,
B_f = Width of the footing, and
L_f = Length of the footing.

For deep foundations, soil properties shall be determined at each representative soil layer over the length of the deep foundations.

For seismic loading, Poisson's ratio is permitted to be taken as 0.5 for saturated clay and 0.25 for other soils.

The initial shear modulus, G_0, shall be established via testing as documented in a geotechnical report or, calculated in accordance with one of Equations (8-1) through (8-4). The shear modulus shall be evaluated over the appropriate depth. Equation (8-1) is permitted for all soil types. The shear wave velocity (ν_{s0}) is measured under the expected vertical loads on the footings. Equation (8-2) is permitted for clayey soils, and Equations (8-3) and (8-4) are permitted for sandy soils.

$$G_0 = \frac{\gamma \nu_{s0}^2}{g} \tag{8-1}$$

$$G_0 \cong 120 p_a (N_{60})^{0.77} \quad \text{for clayey soil} \tag{8-2}$$

$$G_0 \cong 435(N_1)_{60}^{1/3}\sqrt{p_a\sigma'_{mp}} \quad \text{for sandy soil} \tag{8-3}$$

$$G_0 \cong \frac{625\sqrt{p_a\sigma'_{mp}}}{0.3+0.7e_v^2} \quad \text{for sandy soil} \tag{8-4}$$

where σ'_{mp} is the mean effective stress $(\sigma'_1+\sigma'_2+\sigma'_3)/3$ averaged over the relevant region below the footing to a depth of two footing widths. For shallow foundations, σ'_{mp} is permitted to be obtained as the larger value from Equations (8-5) and (8-6):

$$\sigma'_{mp} = \frac{1}{6}\left(0.52 - 0.04\frac{L_f}{B_f}\right)\frac{Q_{Gf}}{A_f} \tag{8-5}$$

$$\sigma'_{mp} \geq \sigma'_{vo} = (\gamma_t)(D_f + B_f/2) - u \tag{8-6}$$

where

v_{s0} = Shear wave velocity at low strains at the appropriate depth;
γ = Total unit weight of the soil;
g = Acceleration caused by gravity;
N_{60} = Standard penetration test blow count corrected to an equivalent hammer energy efficiency of 60%;
$(N_1)_{60}$ = Standard penetration test blow count normalized for an effective stress of 1.0 ton/ft^2 (9.76 tonnes/m^2) confining pressure and corrected to an equivalent hammer energy efficiency of 60%;
p_a = Atmospheric pressure;
e_v = Void ratio;
Q_{Gf} = Expected bearing load on footing caused by gravity loads, including load caused by overburden soil above the footing;
L_f = Length of footing;
B_f = Width of footing;
D_f = Depth of the soil–footing interface;
A_f = Area of footing = B_fL_f;
σ'_{vo} = Effective vertical stress at a depth of $D_f + B_f/2$;
γ_t = Average total unit weight of overburden soil; and
u = Pore-water pressure at depth $(D_f + B_f/2)$.

The effective shear modulus, G, for all analysis procedures shall be determined from the effective shear modulus ratio in accordance with Table 8-1. For the nonlinear dynamic procedure (NDP), the effective shear modulus is permitted to be taken as the initial shear modulus G_0 when soil springs account for hysteretic stiffness and strength degradation and slip of the soil under cyclic loads.

8.2.2 Seismic–Geologic Site Hazards Seismic evaluation and retrofit shall include an assessment of earthquake-induced hazards at the site caused by fault rupture, liquefaction, differential settlement, compaction, landsliding, and an assessment of earthquake-induced flooding or inundation in accordance with Sections 8.2.2.1 through 8.2.2.5. The earthquake-induced hazards shall be assessed at Seismic Hazard Levels being considered in the structural and nonstructural evaluation or retrofit of the building.

Where geologic hazards are identified based on published maps, literature research, or by any other assessment, an in situ geotechnical investigation shall be performed to identify the characteristics of that hazard and to determine soil stiffness and strength characteristics.

If the resulting ground movements cause unacceptable performance in the building for the selected performance level, then the hazards shall be mitigated in accordance with Section 8.3.

Table 8-1. Effective Shear Modulus Ratio (G/G_0).

Site Class[a]	Effective Peak Acceleration, $S_{XS}/2.5$[b]			
	0	0.1	0.4	0.8
A	1.00	1.00	1.00	1.00
B	1.00	1.00	0.95	0.90
C	1.00	0.95	0.75	0.60
D	1.00	0.90	0.50	0.10[c]
E	1.00	0.60	0.05	[d]
F	[d]	[d]	[d]	[d]

[a] Values in the table shall be interpolated for intermediate site classes.
[b] Use straight-line interpolation for intermediate values of $S_{XS}/2.5$.
[c] Site-specific geotechnical investigation shall be permitted to determine the G/G_0 ratio in lieu of the value in the table but shall not be taken greater than the prescribed value for the lower seismic hazard.
[d] Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

8.2.2.1 Fault Rupture A geologic fault shall be defined as a plane or zone along which earth materials on opposite sides have moved differentially in response to tectonic forces.

Geologic site information shall be obtained to determine if an active geologic fault is present under the building foundation. If a fault is present, the following information shall be obtained as stated:

1. The degree of activity based on the age of the most recent movement and earthquake rate,
2. The fault type (i.e., strike-slip, normal, reverse, or oblique fault),
3. The width and distribution of the fault-rupture zone,
4. The orientation of slip with respect to building geometry, and
5. Magnitudes of vertical and/or horizontal displacements consistent with the selected Seismic Hazard Level.

8.2.2.2 Liquefaction Liquefaction is defined as a process in which saturated, loose, granular soils lose shear strength and shear stiffness as a result of an increase in pore-water pressure during earthquake shaking or other rapid loading.

Subsurface soil and groundwater information, including soil type, soil plasticity or consistency, soil density, soil stratigraphy, and depth to water table, shall be obtained to determine if liquefiable materials are present under or near the building foundation. If liquefiable soils are present, the following information shall be obtained to perform relevant liquefaction analyses: ground surface slope and proximity of free-face conditions. Relevant liquefaction analyses include lateral spreading, liquefaction-induced settlement, posttriggering slope stability, liquefaction-induced bearing capacity failure, and flotation of buried structures.

A site shall be regarded as nonliquefiable if the site soils meet any of the following criteria:

1. The geologic materials underlying the site are either bedrock or have very low liquefaction susceptibility, according to the liquefaction susceptibility ratings based on the type of deposit and its geologic age, as shown in Table 8-2;
2. The soils underlying the site are stiff to hard clays or clayey silts;

Table 8-2. Estimated Susceptibility to Liquefaction of Surficial Deposits during Strong Ground Shaking.

Type of Deposit	General Distribution of Cohesionless Sediments in Deposits	Likelihood that Cohesionless Sediments, When Saturated, Would Be Susceptible to Liquefaction (by Geologic Age)			
		Modern <500 years	Holocene <11,000 years	Pleistocene <2 million years	Pre-Pleistocene >2 million years
(a) *Continental Deposits*					
River channel	Locally variable	Very high	High	Low	Very low
Floodplain	Locally variable	High	Moderate	Low	Very low
Alluvial fan, plain	Widespread	Moderate	Low	Low	Very low
Marine terrace	Widespread	—	Low	Very low	Very low
Delta, fan delta	Widespread	High	Moderate	Low	Very low
Lacustrine, playa	Variable	High	Moderate	Low	Very low
Collovium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dune	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low
Tephra	Widespread	High	Low	Unknown	Unknown
Residual soils	Rare	Low	Low	Very low	Very low
Sebka	Locally variable	High	Moderate	Low	Very low
(b) *Coastal Zone Deposits*					
Delta	Widespread	Very high	High	Low	Very low
Estuarine	Locally variable	High	Moderate	Low	Very low
Beach, high energy	Widespread	Moderate	Low	Very low	Very low
Beach, low energy	Widespread	High	Moderate	Low	Very low
Lagoon	Locally variable	High	Moderate	Low	Very low
Foreshore	Locally variable	High	Moderate	Low	Very low
(c) *Fill Materials*					
Uncompacted fill	Variable	Very high	—	—	—
Compacted fill	Variable	Low	—	—	—

Source: Adapted from Youd and Perkins (1978).

3. The soils, if fine-grained, are not highly sensitive, based on local experience;
4. The soils are cohesionless with a minimum normalized standard penetration test (SPT) resistance, $(N_1)_{60}$, value greater than 30 blows/ft (30 blows/0.3 m), as defined in ASTM D1586, for depths below the groundwater table;
5. The soils have a water content to liquid limit ratio less than 0.8 and PI greater than 20; or
6. The groundwater table is at least 35 ft (10.7 m) below the deepest foundation depth, or 50 ft (15.2 m) below the ground surface, whichever is shallower, including considerations for seasonal and historic groundwater level rises, and any slopes or free-face conditions in the vicinity do not extend below the groundwater elevation at the site.

If soils susceptible to liquefaction are present at the site, then an evaluation of the triggering of liquefaction and of the effects of liquefaction to the building shall be performed using procedures set forth in Sections 8.2.2.2.1 and 8.2.2.2.2, or any other procedure that sufficiently captures all consequences of site liquefaction.

8.2.2.2.1 Liquefaction-Affected Structural Evaluation To assess the implications of liquefaction on a structure, two seismic analyses of the structure shall be performed. The first analysis shall be in accordance with Chapter 7, assuming that liquefaction has not occurred at the site. The mathematical model of the structure shall assume a flexible foundation condition; fixed-base modeling of the foundation is not permitted. In this analysis, the site response parameters and the foundation stiffness and strength shall not be reduced because of liquefaction.

The second analysis shall be in accordance with Chapter 7, but the seismic hazard parameters, site response spectrum, or acceleration response histories shall be modified based on the effects of soil liquefaction. The mathematical model of the structure shall not assume a fixed foundation condition, and the strength and stiffness parameters for the foundation shall be reduced because of the occurrence of liquefaction under the Seismic Hazard Level being considered in the evaluation or retrofit.

8.2.2.2.2 Postliquefaction Structural Evaluation The structure shall be evaluated for its integrity to accommodate the deformations of the foundation from potential differential settlements and lateral spreading caused by liquefaction. The estimated differential settlement and lateral spread parameters shall be provided for the Seismic Hazard Level under consideration.

A nonlinear mathematical model in accordance with the provisions in Section 7.4.3.2 is to be used for this analysis. The estimated differential settlement and lateral spread displacement shall be applied to the individual foundation elements or to groups of foundation elements in such a manner as to sufficiently account for the various permutations of ground movement under the building. Structural elements shall be checked for their compliance to the acceptance criteria per Section 7.5, or by other rational criteria based on nonlinear response of the elements under consideration.

8.2.2.3 Settlement of Nonliquefiable Soils Soils that do not liquefy during shaking may still generate excess pore-water pressures or experience shaking-induced densification. These settlements occur in addition to settlements associated with liquefaction.

Subsurface soil information shall be obtained to determine if soils susceptible to differential settlement are present under the building foundation.

If a differential settlement hazard is determined to exist at the site, then a more detailed evaluation shall be performed using procedures approved by the Authority Having Jurisdiction.

8.2.2.4 Landsliding A landslide shall be defined as the downslope mass movement of earth resulting from any cause. Subsurface soil information shall be obtained to determine if soils susceptible to a landslide that will cause differential movement of the building foundation are present at the site.

Excluding cases of liquefaction flow failures, slope stability shall be evaluated at sites with the following:

1. Existing slopes exceeding 18 degrees (three horizontal to one vertical), or
2. Prior histories of instability (rotational or translational slides, or rock falls).

Use of pseudo static analyses shall be permitted to determine slope stability if the soils are not susceptible to liquefaction based on Section 8.2.2.2 or are otherwise expected to lose shear strength during deformation. If soils are susceptible to liquefaction based on Section 8.2.2.2 or are otherwise expected to lose shear strength during deformation, dynamic analyses shall be performed to determine slope stability.

Pseudo static analyses shall use a seismic coefficient from an approved procedure at sites associated with the selected Performance Objective or other approved methods. Sites with a static factor of safety equal to or greater than 1.0 shall be judged to have adequate stability and require no further stability analysis.

A sliding-block displacement analysis shall be performed for sites with a static factor of safety of less than 1.0. The displacement analysis shall determine the magnitude of ground movement and its effect upon the performance of the structure.

In addition to the effects of landslides that directly undermine the building foundation, the effects of rock fall or slide debris from adjacent slopes shall be evaluated using approved procedures.

8.2.2.5 Flooding or Inundation For seismic evaluation and retrofit of buildings for performance levels higher than Life Safety, site information shall be obtained to determine if the following sources of earthquake-induced flooding or inundation are present:

1. Dams located upstream, subject to damage by earthquake shaking or fault rupture;
2. Pipelines, aqueducts, and water storage tanks located upstream, subject to damage by fault rupture, earthquake-induced landslides, or strong shaking;
3. Coastal areas within tsunami zones or areas adjacent to bays or lakes, subject to seiche waves; and/or
4. Low-lying areas with shallow groundwater, subject to regional subsidence and surface ponding of water, resulting in inundation of the site.

Damage to buildings from earthquake-induced flooding or inundation shall be evaluated for its effect on the performance of the structure.

In addition to the effects of earthquake-induced flooding or inundation, scour of building foundation soils from swiftly flowing water shall be evaluated using procedures approved by the Authority Having Jurisdiction.

8.3 MITIGATION OF SEISMIC–GEOLOGIC SITE HAZARDS

Mitigation of seismic–geologic hazards identified in Section 8.2 shall be accomplished through modification of the structure, foundation, or soil conditions, or by other methods approved by the Authority Having Jurisdiction. The structure, foundation, and soil for the building shall meet the acceptance criteria for the appropriate chapters of this standard for the selected Performance Objective.

8.4 SHALLOW FOUNDATIONS

Buildings on shallow foundations where the foundations resist gravity and lateral loads by bearing action at the soil–footing interface shall be evaluated in accordance with the requirements of the simplified procedure in Section 8.4.3, the fixed-base procedure in Section 8.4.4, or the flexible-base procedure in Section 8.4.5. Selection of the evaluation procedure shall be determined in accordance with Section 8.4.1, and the expected soil bearing capacities shall be determined in accordance with Section 8.4.2.

8.4.1 Selection of Evaluation Procedures The simplified procedure in Section 8.4.3 shall be permitted to be used to evaluate buildings where

1. The entire building is on a site with a slope less than or equal to 10%.
2. The superstructure is modeled as a fixed base and the foundation evaluated as individual rectangular segments supporting the vertical elements of the seismic-force-resisting system.
3. The axial load generated by the pseudo seismic force as defined in Section 7.4.1.3.1 is less than 20% of the gravity load on the foundation segment. For combined footings with coupled tension and compression seismic action, the pseudo seismic axial load shall be the algebraic sum of the pseudo seismic demands on the foundation segment L_{fs}, as defined in Section 8.4.3.

For buildings not using the simplified procedures, fixed and flexible-base procedures shall be in accordance with the requirements in Section 8.4.4 and Section 8.4.5, respectively. Buildings shall be permitted to be modeled using the fixed-base procedure in accordance with the requirements in Section 8.4.4, where any of the following apply: (1) The seismic-force-resisting system consists only of light-frame wood, cold-formed steel construction, or unreinforced masonry; (2) partial retrofits where effects of foundation flexibility are not considered as part of the Performance Objective; or (3) the seismicity is classified as Very Low, Low, or Moderate from Table 2.6. All other buildings on shallow foundations shall be modeled as a fixed or flexible base considering the degree of fixity provided at the base of the structure, and the sensitivity of the superstructure force distribution and deformation demands to foundation movement.

8.4.2 Expected Soil Bearing Capacities The expected soil bearing capacity of foundation systems shall be determined by prescriptive or site-specific methods, as specified in Sections 8.4.2.1 and 8.4.2.2. Capacities shall be at ultimate levels and based on foundation information obtained as specified in Section 8.2.1. The κ factor applied to the soil bearing capacity in the soil acceptance criteria shall be taken as 1.0.

8.4.2.1 Prescriptive Expected Soil Bearing Capacities Prescriptive expected soil bearing capacities shall be permitted where construction documents or previous geotechnical reports for the existing building are available and provide information on foundation soil design parameters. Prescriptive expected soil bearing capacities shall not be used where geotechnical site investigation is required per Section 6.2 to establish the expected soil bearing capacities, which shall satisfy the requirements of Section 8.2.1.2. Determination of prescriptive expected soil bearing capacities using either Equation (8-7) or (8-8) shall be permitted in the following cases:

1. Where soil bearing capacity is included in available construction documents or geotechnical reports, the prescriptive expected soil bearing capacity, q_c, for a spread footing is calculated using Equation (8-7):

$$q_c = 3q_{\text{allow}} \tag{8-7}$$

   where q_{allow} is the allowable long-term soil bearing pressure specified in available construction documents for the dead load that includes the self-weight of the footing plus the design live load; and
2. Where soil bearing capacity is not included in available construction documents or geotechnical reports, the prescriptive expected soil bearing capacity, q_c, of any foundation is permitted to be calculated using Equation (8-8):

$$q_c = 2.5(D + 0.4L)/A_f \tag{8-8}$$

   where

   D = Calculated dead load including self-weight of the foundation,
   L = Unreduced live load from the original construction period, and
   A_f = Area of the foundation.

Prescriptive expected soil bearing capacities, q_c, from Equation (8-7) or (8-8) shall be permitted to be multiplied by a seismic amplification factor for short-duration seismic loading. The amplified expected soil bearing capacity for short-duration seismic loading, q_{cDA}, is calculated using Equation (8-9):

$$q_{cDA} = 2.0q_c \tag{8-9}$$

Alternatively, the amplified expected soil bearing capacity for short-duration seismic loading q_{cDA} shall be provided in an approved geotechnical report that includes the effects of seismic loading.

8.4.2.2 Site-Specific Capacities For buildings where the methods specified in Section 8.4.2.1 do not apply, a geotechnical site investigation shall be conducted to determine expected soil bearing capacities q_c for gravity loads and q_{cDA} for short-duration seismic loads based on the specific characteristics of the building site.

8.4.3 Simplified Procedure The foundations shall be permitted to be evaluated by the method described in this section when linear analysis procedures are used.

The moment capacity M_{CE} of the foundation segment shall be calculated using Equation (8-10) and shall be calculated considering both directions of loading. The length of the foundation segment L_{fs} beyond the face of the lateral-force-resisting element shall not be taken greater than the lesser of the actual footing dimension, four times the footing depth, and half the clear distance to the next vertical element of the lateral-force-resisting system or gravity column in accordance with Figure 8-1.

$$M_{CE} = \frac{L_{fs}P_{UD}}{2}\left(1 - \frac{q}{q_{cDA}}\right) \tag{8-10}$$

where

$q = \dfrac{P_{UD}}{B_f L_{fs}}$ = Vertical bearing pressure;
P_{UD} = Axial load at the soil–foundation interface determined using Equation (7-36) and includes the footing weight,
B_f = Width of foundation segment (parallel to the axis of overturning action),
L_{fs} = Length of foundation segment in the direction perpendicular to the axis of overturning action, and
q_{cDA} = Short-term expected bearing capacity as defined in Section 8.4.2.1.

The overturning moment demand on each foundation segment shall be the pseudo seismic force M_{UD} from the vertical seismic-force-resisting element. The acceptance criterion for the soil bearing shall satisfy Equation (8-11).

$$\frac{M_{UD}}{mk} \le M_{CE} \tag{8-11}$$

where m is taken from Table 8-4.

Each foundation segment shall be evaluated using the acceptance criteria for the component action from Chapters 9 through 12 corresponding to the foundation material. Foundation demands shall be calculated using a rectangular soil pressure distribution applied at the bottom of the footing from the end of the foundation segment toward the centroid of that segment over a length L_c of magnitude q_c as defined in Section 8.4.2, such that

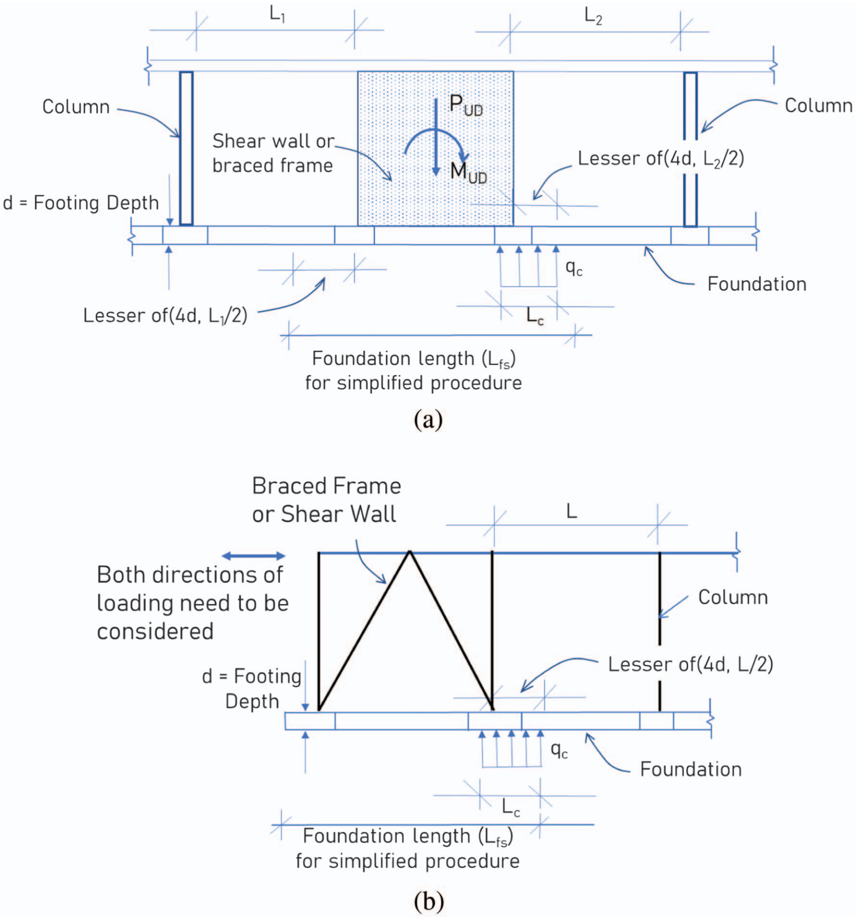
$$L_c = P_{UD}/q_c B_f \tag{8-12}$$

8.4.4 Fixed-Base Procedure For buildings modeled using a fixed-base assumption, base reactions from the fixed-base model shall be used to assess foundation acceptance.

8.4.4.1 Linear Procedures

8.4.4.1.1 Isolated Spread Footings For linear procedures, the moment capacity for isolated spread footings shall be determined in accordance with Section 8.4.4.1.1.1, and acceptance shall be in accordance with Sections 8.4.4.1.1.2 or 8.4.4.1.1.3 as applicable based on axial or moment action on the foundation.

8.4.4.1.1.1 Foundation Overturning Moment Capacity For isolated spread footings, the overturning moment capacity, M_{CE} shall be obtained using the procedures specified in this section. The overturning moment capacity, M_{CE}, is calculated as the product of the axial load, P_U, times the distance from the centroid



(a)

(b)

Figure 8-1. Foundation length for the simplified procedure: (a) intermediate condition, (b) end condition.

of the critical contact area, A_c, to the centroid of the footing. Where, P_U shall be determined from Equation (8-15) and the critical contact area, A_c, defined as the footing area required to support the expected vertical gravity and overturning axial load, shall be calculated using Equation (8-13) as follows:

$$A_c = P_U / q_{cDA} \tag{8-13}$$

For rectangular footings subjected to unidirectional overturning moment, the moment capacity shall be determined in accordance with Section 8.4.4.1.1.1.1. For I-shaped footings subjected to unidirectional moment about its major axis, the moment capacity shall be determined in accordance with Section 8.4.4.1.1.1.2. For all other footings and for overturning moments acting simultaneously about each principal axis, the moment capacity shall be determined in accordance with Section 8.4.4.1.1.1.3.

8.4.4.1.1.1.1 Rectangular Footings For rectangular footings, where the axial load is applied at the centroid of the footing, the overturning moment capacity M_{CE} about the principal axis shall be determined using Equation (8-14):

$$M_{CE} = \frac{L_f P_U}{2}\left(1 - \frac{q}{q_{cDA}}\right) \tag{8-14}$$

where

$q = \dfrac{P_U}{B_f L_f}$ = Vertical bearing pressure;

q_{cDA} = Bearing capacity determined in Section 8.4.2;

B_f = Width of footing (parallel to the axis of overturning action);

L_f = Length of footing in the direction perpendicular to the axis of overturning action; and

P_U = Expected vertical axial load on the soil at the footing interface and is the maximum action that can be developed by gravity and seismic loads based on a limit-state analysis considering the expected strength of the components delivering force to the footing; alternatively, the expected vertical load is permitted to be determined using Equation (8-15):

$$P_U = P_D^{ftg} \pm \frac{P_E}{DCR_{\max}} \tag{8-15}$$

where

P_D^{ftg} = Expected axial gravity load at the soil–footing interface determined as 1.0D;

D = Dead load, which includes the footing weight;

P_E = Q_E as defined in Equation (7-36); and

$DCR_{\max}$ = Maximum demand–capacity ratio of the elements of the lateral-force-resisting system in the direct load path of the footing being evaluated contributing to the seismic axial load as defined in Section 7.3.1.1, which need not be taken as less than C_1C_2 and shall not be taken as greater than $3C_1C_2$; where C_1 and C_2 are as defined in Section 7.4.1.3.1.

8.4.4.1.1.1.2 I-Shaped Footings For I-shaped footings, if L_c is less than the length of the flange, the overturning moment capacity M_{CE} about the major principal axis shall be determined using Equation (8-14), otherwise the moment capacity shall be determined in accordance with Section 8.4.4.1.1.1.3. L_c shall be calculated using Equation (8-16):

$$L_c = \frac{P_U}{q_{cDA} B_{fw}} \tag{8-16}$$

where B_{fw} is the width of the flange.

8.4.4.1.1.1.3 All Other Footings and Footings with Bidirectional Moment The overturning moment capacity, M_{CE}, shall be determined in accordance with Section 8.4.4.1.1.1 and shall consider the effect of overturning moments applied simultaneously about both principal footing axes. It shall be permitted to determine M_{CE} from Equation (8-17) or other rational methods.

If the overturning moment about a principal footing axis is less than $0.2mM_{CE}$, where M_{CE} is calculated for that axis alone, then it shall be permitted to neglect the effects of overturning about that axis.

$$M_{CE} = \sqrt{\left(\frac{M_{UD,x}}{m}\right)^2 + (M_{CE,y})^2} \tag{8-17}$$

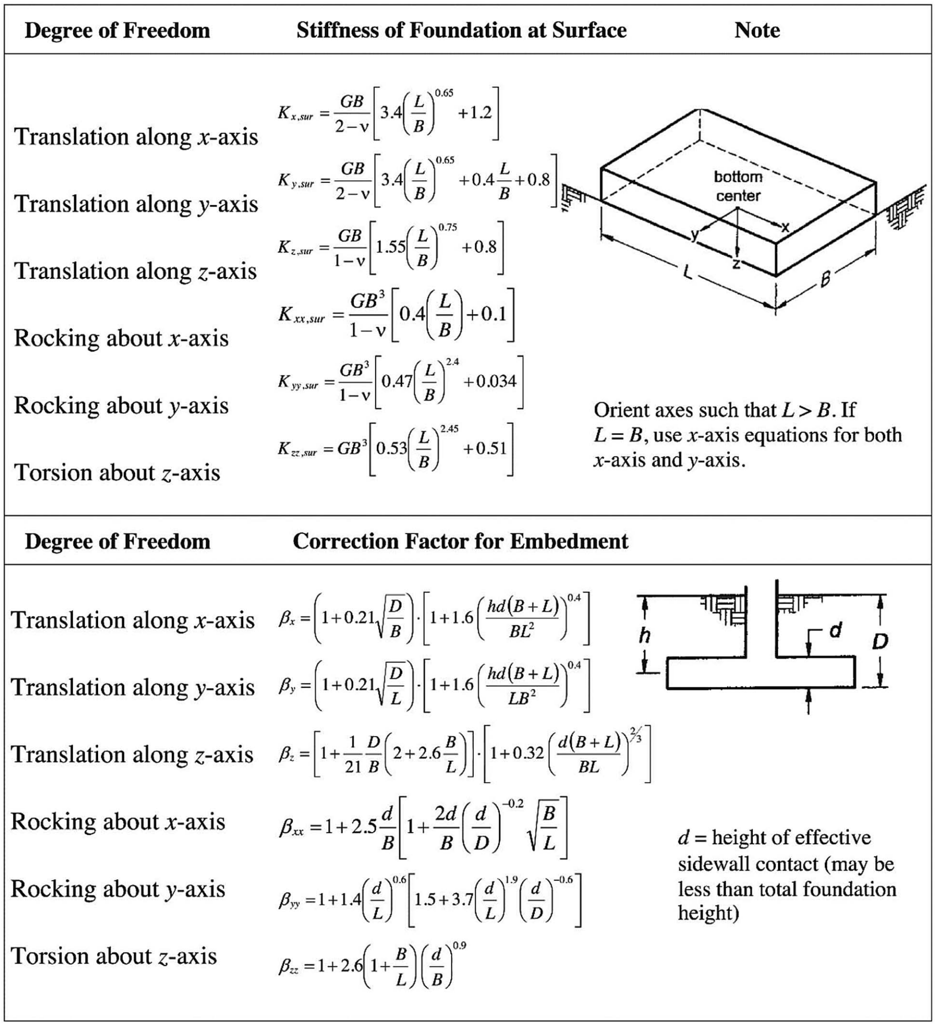
where

$M_{UD,x}$ = Component of overturning moment determined using Equation (7-36) about the x-axis or minor axis of overturning;

$M_{CE,y}$ = Moment capacity of the foundation for rocking about the y-axis (see Figure 8-2 for axis direction), determined as the product of the axial load P_U acting at the centroid of the critical contact area A_c and the x-distance from the centroid of the footing; and

m = Value from Table 8.4 for the required Performance Objective.

8.4.4.1.1.2 Overturning Forming Axial Load Action For isolated spread footings where overturning results in predominantly axial compression or tension action on the footing, and when the moment demands are small relative to its uniaxial capacity such that Equation (8-18) is satisfied, the requirements of this section shall apply, otherwise the requirements of Section 8.4.4.1.1.3 shall apply.



Degree of Freedom	Stiffness of Foundation at Surface	Note
Translation along x-axis	$K_{x,sur} = \frac{GB}{2-\nu}\left[3.4\left(\frac{L}{B}\right)^{0.65} + 1.2\right]$	
Translation along y-axis	$K_{y,sur} = \frac{GB}{2-\nu}\left[3.4\left(\frac{L}{B}\right)^{0.65} + 0.4\frac{L}{B} + 0.8\right]$	
Translation along z-axis	$K_{z,sur} = \frac{GB}{1-\nu}\left[1.55\left(\frac{L}{B}\right)^{0.75} + 0.8\right]$	
Rocking about x-axis	$K_{xx,sur} = \frac{GB^3}{1-\nu}\left[0.4\left(\frac{L}{B}\right) + 0.1\right]$	
Rocking about y-axis	$K_{yy,sur} = \frac{GB^3}{1-\nu}\left[0.47\left(\frac{L}{B}\right)^{2.4} + 0.034\right]$	Orient axes such that $L > B$. If $L = B$, use x-axis equations for both x-axis and y-axis.
Torsion about z-axis	$K_{zz,sur} = GB^3\left[0.53\left(\frac{L}{B}\right)^{2.45} + 0.51\right]$	

Degree of Freedom	Correction Factor for Embedment	
Translation along x-axis	$\beta_x = \left(1 + 0.21\sqrt{\frac{D}{B}}\right) \cdot \left[1 + 1.6\left(\frac{hd(B+L)}{BL^2}\right)^{0.4}\right]$	
Translation along y-axis	$\beta_y = \left(1 + 0.21\sqrt{\frac{D}{L}}\right) \cdot \left[1 + 1.6\left(\frac{hd(B+L)}{LB^2}\right)^{0.4}\right]$	
Translation along z-axis	$\beta_z = \left[1 + \frac{1}{21}\frac{D}{B}\left(2 + 2.6\frac{B}{L}\right)\right] \cdot \left[1 + 0.32\left(\frac{d(B+L)}{BL}\right)^{2/3}\right]$	
Rocking about x-axis	$\beta_{xx} = 1 + 2.5\frac{d}{B}\left[1 + \frac{2d}{B}\left(\frac{d}{D}\right)^{-0.2}\sqrt{\frac{B}{L}}\right]$	d = height of effective sidewall contact (may be less than total foundation height)
Rocking about y-axis	$\beta_{yy} = 1 + 1.4\left(\frac{d}{L}\right)^{0.6}\left[1.5 + 3.7\left(\frac{d}{L}\right)^{1.9}\left(\frac{d}{D}\right)^{-0.6}\right]$	
Torsion about z-axis	$\beta_{zz} = 1 + 2.6\left(1 + \frac{B}{L}\right)\left(\frac{d}{B}\right)^{0.9}$	

Figure 8-2. Elastic solutions for rigid footing spring constraints.

$$M_{Ftg} < 0.2\, m M_{CE\_Ftg} \qquad (8\text{-}18)$$

where

M_{Ftg} = Local moment demand on the footing determined from Equation (7-36);
M_{CE_Ftg} = Moment capacity of the footing, from Equation (8-14) or Equation (8-17) determined using $P_U = 1.0(P_D + 0.25P_L + P_S)$, where P_D, P_L, and P_S are the axial load actions caused by dead, live, and snow loads, respectively, as defined in Section 7.2.3; and m is the value from Table 8-4.

8.4.4.1.1.2.1 Acceptance Criteria for Soil Bearing and Uplift

1. For the controlling load combination where the seismic axial load demand on the foundation is additive to gravity, acceptance criteria for soil bearing shall be evaluated using the axial compression m-factor from Table 8-3 and satisfy Equation (8-19):

$$\frac{P_U}{mk} \le q_{cDA} A_f \qquad (8\text{-}19)$$

   where κ and q_{cDA} are defined in Section 8.4.2, and P_U = Axial compression determined in accordance with Equation (8-15).
2. Where the controlling load combination with seismic axial demand on the foundation results in net tension ($P_U < 0$), the foundation shall be evaluated using the uplift m-factor from Table 8-3 and satisfy Equation (8-20):

$$P_E / m \le P_G \qquad (8\text{-}20)$$

   where

   P_G = Action caused by gravity load at the soil–footing interface determined using Equation (7-2) and includes the weight of the footing, and
   P_E = Seismic axial demand Q_E on the footing determined from Equation (7-36).

8.4.4.1.1.2.2 Acceptance Criteria for the Structural Footing The structural footing shall be evaluated as force controlled or deformation controlled as permitted for that action in Chapters 9 through 12 corresponding to the foundation material using a uniform soil pressure distribution under the footing with axial load demand P_U from Equation (8-15). As an alternate to the $DCR_{\max}$ term in Equation (8-15), it shall be permitted to use the m-factor for axial compression in Table 8-3.

8.4.4.1.1.3 Overturning Induced Moment and Axial Load Actions For isolated spread footings where overturning action results in moment and axial load actions on the footing and Equation (8-18) is not satisfied, the requirements of this section shall be satisfied. The overturning demand $M_{UD} = Q_{UD}$ shall be determined using Equation (7-36).

8.4.4.1.1.3.1 Acceptance Criteria for Soil Bearing and Overturning Acceptance criteria for soil bearing shall satisfy the requirements of Equation (8-21):

$$\frac{M_{UD}}{mk} \le M_{CE} \qquad (8\text{-}21)$$

where

$$M_{UD} = \sqrt{M_{UD,x}^2 + M_{UD,y}^2} \qquad (8\text{-}22)$$

$M_{UD,x}$ = Component of overturning moment determined using Equation (7-36) about the x-axis or minor axis of overturning,
$M_{UD,y}$ = Component of overturning moment determined using Equation (7-36) about the y-axis or major axis of overturning,
M_{CE} = Overturning moment capacity determined in accordance with Equation (8-17),
m = m-Factor values from Table 8-4 for the required performance level, and
κ = Knowledge factor as defined in Section 8.4.2.

For calculation of M_{CE}, the axial load on the footing P_U shall be calculated using Equation (8-15) when gravity loads and seismic axial forces P_E are additive, or shall be the P_U used in the determination of M_{CE_Ftg} in Equation (8-18) when gravity loads and seismic forces are counteracting.

For biaxially symmetric rectangular and I-shaped footings, acceptance criteria for soil bearing shall be permitted to satisfy the requirements of Equation (8-23) using the moment/rotation m-factor values from Table 8-4 and κ as defined in Section 8.4.2.

$$\left(\frac{M_{UD,x}}{m\kappa M_{CE,x\_uniaxial}}\right)^2 + \left(\frac{M_{UD,y}}{m\kappa M_{CE,y\_uniaxial}}\right)^2 \le 1 \qquad (8\text{-}23)$$

where

M_{CE,x_uniaxial} = Moment capacity determined using Equation (8-14) substituting footing length L_f with footing width B_f, and
M_{CE,y_uniaxial} = Moment capacity determined using Equation (8-14).

As an alternative, it shall be permitted to idealize the footing as individual isolated spread footings at each end of the footing under the columns or wall boundary elements and check acceptance in accordance with Section 8.4.4.1.1.2.1.

Table 8-3. *m*-Factors for Overturning from Axial Action.

	Performance Level		
Action	**IO**	**LS**	**CP**
Axial compression	1.25	2	2.5
Axial uplift	4	6	8

Table 8-4. *m*-Factors for Overturning Moment Action.

	Performance Level		
Action	**IO**	**LS**	**CP**
Moment/rotation	2	3	4

Note: Where $A_c/A_f > 0.4$, the m-factors from Table 8-7 shall be used. A_c is the critical contact area calculated using Equation (8-13) and A_f is the area of the footing.

8.4.4.1.1.3.2 Acceptance Criteria for the Structural Footing Rectangular shaped isolated spread footings shall be evaluated in accordance with requirements in Chapters 9 through 12 corresponding to the foundation material for demands generated by an upward rectangular soil pressure distribution q_c as defined in Section 8.4.2 applied at the bottom of the footing starting from the edge toward the centroid over a length L_c such that $L_c = P_U/(q_c B_f)$ and P_U is determined using Equation (8-15).

As an alternative, for all footing geometries and loading, it shall be permitted to evaluate the structural footing as force controlled for the soil pressure distribution using a rectangular soil pressure block for an axial demand P_U and overturning moment M_{UD} divided by m, where P_U, M_{UD}, and m, are the same values used for soil bearing in Section 8.4.4.1.1.2.1 or Section 8.4.4.1.1.3.1 as applicable. The soil pressure distribution shall be obtained using a rational procedure where footing uplift is unrestrained.

8.4.4.1.2 Combined Footings, Mat Foundations, and Isolated Spread Footings In buildings where the foundation is modeled as a fixed base using Section 7.4.1 or 7.4.2, and where the foundation plan consists of combined footings, grade beams resisting bearing pressures, or mat foundations, the foundation evaluation shall comply with the requirements in this section.

8.4.4.1.2.1 Foundations Idealized as Individual Footings Footings interconnected by foundation structural elements such as grade beams are permitted to be discretized as individual footings at the points of contraflexure of the interconnecting structural element and treated as isolated spread footings. Acceptance shall be in accordance with Section 8.4.4.1.1 except the foundation moment capacity M_{CE} shall be determined using principles of mechanics taking the summation of the overturning resistance about the centroid of the soil–footing contact area.

When a two-dimensional analysis is permitted and used, overturning resistance of orthogonal framing is permitted to be included to resist overturning but shall not exceed the allowed flange width of the shear wall, nor the moment and shear capacity of the orthogonal members.

8.4.4.1.2.2 Foundations Evaluated in a Separate Analysis from the Superstructure The foundation soil supports shall be represented by Winkler springs in a separate analysis to that of the superstructure, and the foundation shall be evaluated for superstructure demands from the fixed-base analysis.

8.4.4.1.2.2.1 Soil Stiffness The vertical soil spring stiffness values, expressed as a uniform stiffness or the modulus of subgrade reaction, shall be determined from Section 8.4.5.1 as required for flexible-base procedures or as provided in the approved geotechnical report. Where soil springs resist both tension and compression, the soil spring stiffness shall be multiplied by 0.5.

8.4.4.1.2.2.2 Soil Strength The expected soil bearing capacity for short-duration seismic loading q_{cDA} shall be determined using the procedures specified in Section 8.4.2.

8.4.4.1.2.2.3 Acceptance Criteria for Soil Bearing and Overturning Modeling and acceptance criteria shall comply with the requirements of Procedure 1 or Procedure 2.

Procedure 1: Soil Springs Resist Tension and Compression Pseudo seismic forces from the fixed-base building analysis shall be applied to the structural foundation modeled with elastic properties and supported by discrete springs, representing the soil, which resist both tension and compression. Each spring shall represent a tributary area of contact and be distributed uniformly across the footing–soil interface. The spring stiffness values used in the model shall be the soil stiffness values determined in accordance with Section 8.4.5.2.1.1 for linear procedures.

Acceptance criteria for soil bearing shall be considered satisfied if the maximum rotation demand at the base of the wall, or column(s), or from the bottom of two columns that form a braced-frame is less than the rotation acceptance values in Table 8-8. Section 8.4.5.2.2.1 shall be used to compute the dimensions of an equivalent rectangular footing for use with Table 8-8.

Procedure 2: Soil Springs Are Compression-Only When the structural combined footing or mat foundation is analyzed including springs that cannot resist tension, the seismic demands from the superstructure Q_E determined from Equation (7-36) are permitted to be divided by the applicable m-factor from Table 8-5 prior to the foundation evaluation in conjunction with the gravity load combinations from Section 7.2.3.

Foundation acceptance for soil bearing shall be considered satisfied if the maximum soil bearing pressure at any point under the footing, for load combinations when seismic forces and gravity loads are additive and when seismic forces and gravity loads are counteracting, is less than q_{cDA}.

8.4.4.1.2.2.4 Acceptance Criteria for the Structural Foundation The capacity of the structural components of the combined footing, mat foundation, or isolated spread footing, shall be evaluated based on the provisions of Chapters 9 through 12. Demands used for the acceptance criteria for the structural footing shall be consistent with the methods used for evaluation of soil bearing.

Where the combined footing is evaluated as individual isolated spread footings, the foundation structural component acceptance criteria shall be in accordance with Section 8.4.4.1.1.3.2, except the end of the footing shall originate at the location of the point of contraflexure or a length consistent with that used in the evaluation of soil bearing.

When Procedure 1 is used, the m-factors in Chapters 9 through 12 corresponding to the foundation material are permitted to be applied to component actions classified as deformation controlled.

When Procedure 2 is used, the footing shall be evaluated with pseudo seismic forces reduced by the m-factor from Table 8-5.

8.4.4.2 Nonlinear Procedures When a fixed-base modeling is permitted and used in accordance with Section 7.2 the foundation shall be evaluated in accordance with the requirements in Section 8.4.4.1.2.2. Soil springs used in the foundation analysis model shall be compression-only springs and shall not resist tension. The structural footing shall be evaluated using an m-factor equal to 1.0.

8.4.5 Flexible-Base Procedure For the flexible-base procedure, the superstructure including the structural footing and soil flexibility and strength shall be explicitly modeled in the

Table 8-5. *m*-Factor for Overturning Action.

	Performance Level		
Action	**IO**	**LS**	**CP**
Overturning action	2	3	4

Note: Where $A_c/A_f > 0.4$, the m-factors from Table 8-7 shall be used. A_c is the critical contact area calculated using Equation (8-13) and A_f is the area of the footing.

mathematical model of the building in accordance with Section 7.2.4. The footing supports shall be provided by discrete soil springs that represent a tributary area of contact and be distributed uniformly across the footing–soil interface, or by distributed area springs with a spring coefficient equal to the modulus of subgrade reaction.

8.4.5.1 Soil Stiffness The foundation stiffness per unit area shall be calculated using a unit subgrade spring coefficient, k_{sv}, determined from Equation (8-24) for $L_f/B_f > 3.0$, and $k_{z,sur}$ from Figure 8-2 for $L_f/B_f \leq 3.0$ or provided in an approved geotechnical report based on soil site data from soil properties determined by testing in accordance with Section 8.2.1.

$$k_{sv} = \frac{1.3G}{B_f(1 - \nu)} \tag{8-24}$$

where

G = Effective shear modulus of soil as specified in Section 8.2.1.4;
B_f = Width of footing for isolated footings, or effective width B'_f for mat foundations defined in Section 8.4.5.2.2.1;
L_f = Length of footing or and effective length L'_f for mat foundations defined in Section 8.4.5.2.2.1 and is the larger footing dimension, $L_f \geq B_f$; and
ν = Poisson's ratio.

8.4.5.2 Linear Procedures For linear analysis procedures, the soil spring supports shall be modeled to behave elastically and have the same stiffness and strength in tension or compression, unless the maximum DCR of the primary components of the superstructure determined in accordance with Equation (7-16) is less than or equal to 1.5, in which case nonlinear uplifting foundations are also permitted. The analysis shall be performed using the soil stiffness and strength as defined in this section.

8.4.5.2.1 Isolated Spread Footings

8.4.5.2.1.1 Soil Stiffness The soil stiffness shall be included in the mathematical model using one of the following:

- As single uncoupled axial and rotational springs, as shown in Figure 8-3, with stiffness properties using the equations in Figure 8-2;
- Using a finite-element representation of the foundation modeled as beam-on-elastic vertical springs with subgrade spring coefficient equal to the modulus of subgrade reaction k_{sv} from Equation (8-24);
- The vertical axial stiffness from Figure 8-2 converted to per unit area $k_{z,sur}$; or
- Alternate methods for stiffness distribution shall be permitted when approved by the AHJ.

For use in linear procedures where soil resists both tension and compression, soil stiffness shall be determined in accordance with Section 8.4.5.1 multiplied by 0.5.

8.4.5.2.1.2 Soil Strength The bearing capacity of the subsurface soil shall be determined in accordance with Section 8.4.2.

8.4.5.2.1.3 Acceptance Criteria Where the foundation flexibility is included in the mathematical model and the foundation soil interface is modeled using linear elastic foundation soil representation, the foundation overturning action shall be classified as deformation controlled. Component actions shall be determined by Equation (7-36).

Where seismic overturning resistance on the isolated spread footing is resisted by axial compression action, the foundation shall be evaluated using the axial compression m-factor from Table 8-6 and shall satisfy Equation (8-19).

Where overturning action results in a net axial uplift force demand on the footing, P_U, calculated using Equation (8-15), is less than zero, foundation acceptance shall be evaluated using the uplift m-factor from Table 8-6 and shall satisfy Equation (8-20).

Acceptance for overturning action shall be the same as the acceptance criteria for the fixed-base procedure in Section 8.4.4.1.1.3, except that the m-factors shall be taken from Table 8-7.

The idealized footing configurations and corresponding parameters are defined in Figure 8-4. The parameter b in Table 8-7 is defined as the width of rectangular footings and the flange width of I-shaped footings. The parameter L_c is defined as the length of the contact area and equal to A_c/b. For I-shaped footings, the parameter A_{rect} is equal to the area of the smallest rectangle that covers the footing footprint, and A_f is the actual footing area.

8.4.5.2.2 Combined Footings, Mat Foundations, and Foundations Idealized as Isolated Footings In buildings where the superstructure and foundation soil supports are modeled in the same mathematical model and where the foundation plan consists of combined footings, grade beams resisting bearing pressures, or mat foundations, the foundation evaluation shall comply with the requirements in this section.

8.4.5.2.2.1 Soil Stiffness The vertical soil spring stiffness values expressed as a uniform stiffness, or the modulus of subgrade reaction, shall be determined from Section 8.4.5.1 or as specified in an approved geotechnical report. Where soil springs resist compression and tension, soil stiffness values shall be multiplied by 0.5 and included in the mathematical model using a finite-element representation of the foundation modeled as a beam-on-elastic soil.

For combined footings, or footings interconnected with grade beams, the largest width of the footing supporting the vertical axial load shall be used in the determination of the spring stiffness or modulus of subgrade reaction.

When the entire building or portion thereof is supported by a mat foundation over multiple bays, the modulus of subgrade reaction shall be based on one of the following:

1. The effective width B'_f used to determine the spring stiffness from Section 8.4.5.1 shall be zoned to coincide with

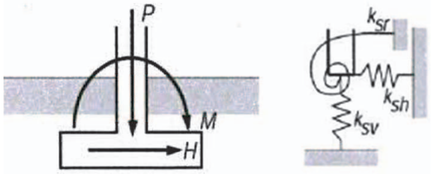


Figure 8-3. Uncoupled spring model for rigid footings with axial and rotational springs.

Table 8-6. *m*-Factors for Axial Actions from Overturning.

	Performance Level		
Actions from Overturning	**IO**	**LS**	**CP**
Axial compression	1.25	2	2.5
Uplift	6	8	10

Table 8-7. Modeling Parameters and Numerical Acceptance Criteria for Linear Procedures.

			m*-Factors		
			Performance Level		
Footing Shape			**IO**	**LS**	**CP**
i. Rectangle					
$\frac{b}{L_c}$	$\frac{A_{\text{rect}} - A_f}{A_{\text{rect}}}$	$\frac{A_c}{A_f}$			
≥10	0	0.20	5	8	10
		0.5	3	5	6
		1	1	1	1
3	0	0.20	4	6	8
		0.5	2	3	4
		1	1	1	1
1	0	0.20	2.5	5	6
		0.5	1.5	2	3
		1	1	1	1
0.3	0	0.20	2	4	5
		0.5	1	1.5	2
		1	1	1	1
ii. I-Shape					
$\frac{b}{L_c}$	$\frac{A_{\text{rect}} - A_f}{A_{\text{rect}}}$	$\frac{A_c}{A_f}$			
$1 \le \frac{b}{L_c} \le 10$	0.3	0.20	3	5	7
		0.5	1.5	2.5	3.5
		1	1	1	1
$1 \le \frac{b}{L_c} \le 10$	0.6	0.20	2.5	4.5	5.5
		0.5	1	2	2
		1	1	1	1
$1 \le \frac{b}{L_c} \le 10$	1	0.20	2	3.5	4.5
		0.5	1	1.5	1.5
		1	1	1	1
$0 \le \frac{b}{L_c} < 1$	1	0.20	2	3.5	4.5
		0.5	1	1.5	1.5
		1	1	1	1

*Linear interpolation between values listed in the table shall be permitted.

the column grid lines and is permitted to be limited by the typical bay width. Widths for end bays shall extend from the edge of the footing and mat to half the distance between the vertical elements at the perimeter of the building and the vertical elements of the first interior bay.

2. Soil spring stiffnesses shall be determined at each location in the mat footing supporting a vertical structural component. At each location, an effective width B'_f is permitted to be used to determine the spring stiffness from Section 8.4.5.1 based on a minimum bearing area $A'_f = B'_f L'_f$ required to support 1.5 times the design dead and live loads using an allowable bearing pressure q_{allow} as defined in Section 8.4.2. The bearing area shall extend equally from all sides of the vertical element supported by the mat unless terminated by the edge of the footing or mat. If the individual bearing areas from multiple vertical elements overlap, the whole area required to support the combined load shall be used in the determination of B'_f.
3. Soil spring stiffnesses shall be determined at each location in the mat footing supporting a vertical structural component. At each location, an effective footing length, L'_f, shall be taken as either the length from points of flexural inflection on either side of the vertical structural component, or from the one-quarter points of the span on either side of the vertical structural component, whichever is greater. L'_f shall not be longer than the actual footing length. The effective footing width shall be taken as four times the footing depth on each side of the vertical element but not more than the actual footing size.
4. Other rational procedures where an equivalent foundation stiffness is determined based on settlement of the mat from finite-element modeling of the soil continuum with applied loads on mat that account for the geometry and rigidity of the mat.

8.4.5.2.2.2 Soil Strength The vertical expected bearing capacity for short-duration seismic loads q_{cDA} of shallow bearing foundations shall be determined using the procedures of Section 8.4.2.

8.4.5.2.2.3 Acceptance Criteria Where the combined footing is idealized as individual footings, acceptance shall be in accordance with Section 8.4.4.1.2.1 using the applicable *m*-factors in Table 8-7 for overturning actions and Table 8-6 for axial actions from overturning. Foundation overturning capacity shall be calculated in accordance with Section 8.4.4.1.1.1.

Where the combined footing is not idealized as individual footings, the requirements in Sections 8.4.4.1.2.2.3 and 8.4.4.1.2.2.4 shall be used for evaluation of the foundation system. Foundation demands shall be obtained from a superstructure analysis model with flexible spring supports and applied to the foundation model.

8.4.5.3 Nonlinear Procedures Expected nonlinear sliding and bearing behavior of foundations shall be represented by a bilinear elastic, perfectly plastic load–deformation relationship unless another approved relationship is available. The initial elastic stiffness shall be calculated using elastic solutions in Figure 8-2 or calculated from Equation (8-24) with expected shear modulus and Poisson's ratio determined according to Section 8.2.1.4. The shear modulus ratio G/G_0 used in the initial stiffness determination need not be taken less than 0.5.

The vertical bearing capacity of the soil springs per unit area of the footing shall be obtained from Section 8.4.2, including the seismic amplification factor represented by $q_{\text{sp,max}} = q_{cDA}$. The tension capacity of the soil springs shall be set at zero.

The foundation structural footing shall be modeled either as linear or nonlinear with the acceptance criteria consistent with the component modeling.

8.4.5.3.1 Modeling Parameters for Nonlinear Static Procedure The axial and shear behavior for foundation springs shall be modeled using the bilinear model shown in Figure 8-5a with $f = 1.0$ (Points F and B are coincident) and the initial elastic stiffness (slope along FA) calculated using conventional elastic solutions from Figure 8-2. The limiting displacement d for axial and shear behavior shall be taken as $L_f/10$, unless analysis shows that larger displacements do not result in loss in soil capacity.

The moment-rotation behavior for foundation springs shall be modeled using the trilinear model shown in Figure 8-5b and modeling parameters in Table 8-8.

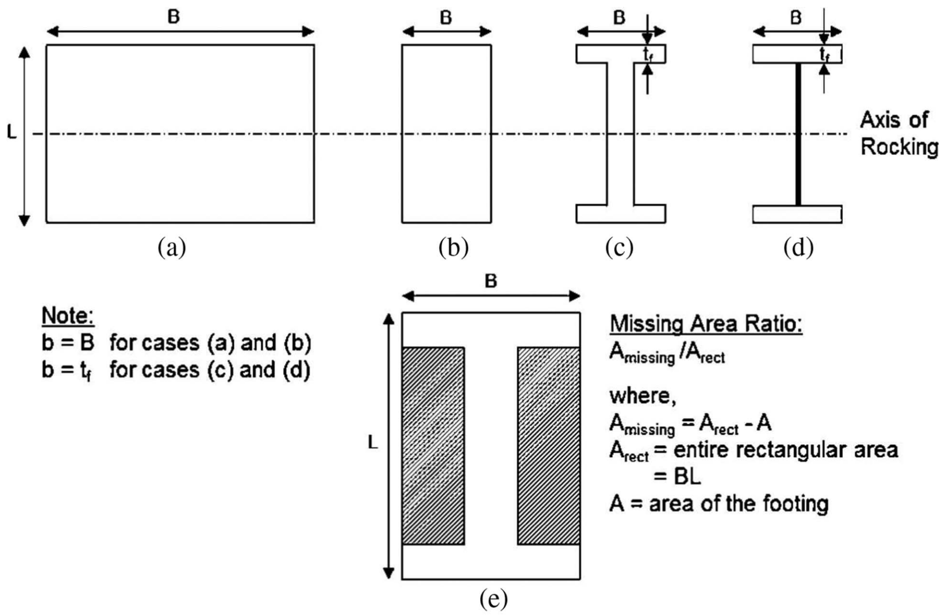


Figure 8-4. Idealized footing configurations and parameter definition.

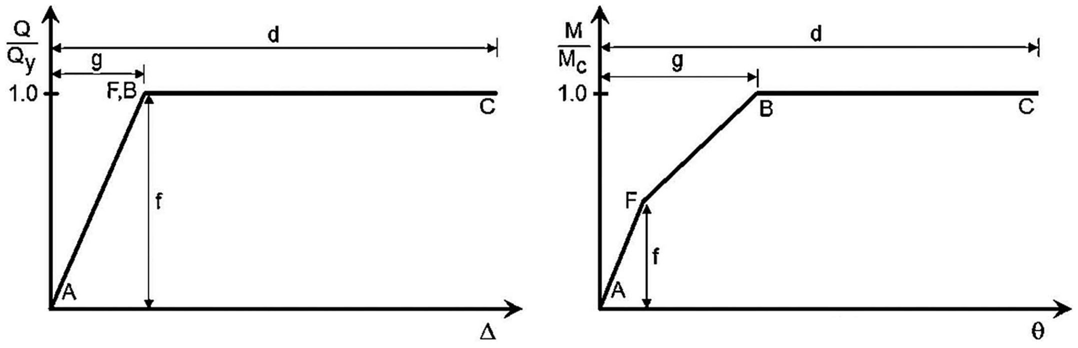


Figure 8-5. Generalized nonlinear force-deformation relations for shallow foundations with (a) bilinear elastoplastic behavior for shear and axial actions, (b) trilinear behavior for rocking or overturning.

Damping caused by energy dissipation from yielding at the soil–footing interface and radiation damping shall only be included in the model where SSI modifications are not used.

8.4.5.3.2 Modeling Parameters for Nonlinear Dynamic Procedure The shape of the soil hysteresis curve shall include self-centering and decentering (gap) effects, pinching of the hysteresis curve, and hysteretic damping, if not included in the ground motion or modeled as part of the system damping. Where explicit modeling for radiation damping effects and hysteretic damping is performed, these effects shall not be included in the determination of ground motions or in the mathematical system (inherent) model damping.

Modification of the acceleration response spectrum caused by kinematic SSI effects shall be permitted when selecting the ground motions for nonlinear dynamic analysis. Damping associated with the localized soil yielding shall be included with hysteretic soil springs, not by reducing the input motion. The expected ground motions, with kinematic effects accounted for, shall be applied to the ends of the hysteretic soil springs. Damping elements with constant radiation damping coefficients are calculated based on $c = \beta(2\sqrt{k_{ie}m})$, where k_{ie} is the initial elastic stiffness of the foundation spring, shall be placed in parallel with the linear component of the foundation spring but shall not be in parallel with the nonlinear components of the foundation springs.

8.4.5.3.3 Acceptance Criteria For the nonlinear static procedure (NSP), where the foundation flexibility and strength are included in the mathematical model and are modeled using nonlinear foundation characteristics, the foundation soil shall be classified as deformation controlled. Acceptability of soil displacements shall be based on the foundation rotation limits in Table 8-8.

Where the structural footing is modeled as linear, it shall be evaluated as force controlled using lower-bound capacity and the demands from the nonlinear static analysis. Where nonlinear properties of the footing are included in the analysis model, the structural footing shall satisfy the acceptance criteria in the material chapters for the component action at the required performance level.

Where the explicit NDP modeling of the foundation occurs and the modeling accurately captures characteristics of settling, soil plasticity, and gapping, the acceptability of soil displacements shall be based on the ability of the structure to accommodate the displacements calculated by the NDP within the acceptance criteria for the selected Performance Objective. If these

Table 8-8. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures.

			Modeling Parameters[a]			Acceptance Criteria[a] Total Footing Rotation Angle, radians[b]		
			Footing Rotation Angle, radians		Elastic Strength Ratio	Performance Level		
Footing Shape			G	d	f	IO	LS	CP
i. Rectangle[a]								
$\frac{b}{L_c}$	$\frac{A_{\text{rect}} - A_t}{A_{\text{rect}}}$	$\frac{A_c}{A_f}$						
≥10	0	0.02	0.009	0.1	0.5	0.02	0.08	0.1
		0.13	0.013	0.1	0.5	0.015	0.08	0.1
		0.5	0.015	0.1	0.5	0.002	0.003	0.004
		1	0.015	0.1	0.5	0.0	0.0	0.0
3	0	0.02	0.009	0.1	0.5	0.02	0.068	0.085
		0.13	0.013	0.1	0.5	0.011	0.06	0.075
		0.5	0.015	0.1	0.5	0.002	0.003	0.004
		1	0.015	0.1	0.5	0.0	0.0	0.0
1	0	0.02	0.009	0.1	0.5	0.02	0.056	0.07
		0.13	0.013	0.1	0.5	0.007	0.04	0.05
		0.5	0.015	0.1	0.5	0.002	0.003	0.004
		1	0.015	0.1	0.5	0.0	0.0	0.0
0.3	0	0.02	0.009	0.1	0.5	0.01	0.04	0.05
		0.13	0.013	0.1	0.5	0.007	0.024	0.03
		0.5	0.015	0.1	0.5	0.001	0.003	0.004
		1	0.015	0.1	0.5	0.0	0.0	0.0
ii. I-Shape								
$\frac{b}{L_c}$	$\frac{A_{\text{rect}} - A_t}{A_{\text{rect}}}$	$\frac{A_c}{A_f}$						
	0.3	0.02	0.009	0.1	0.5	0.02	0.056	0.07
		0.13	0.013	0.1	0.5	0.007	0.04	0.05
		0.5	0.015	0.1	0.5	0.002	0.003	0.004
		1	0.015	0.1	0.5	0.0	0.0	0.0
	0.6	0.02	0.007	0.1	0.5	0.015	0.048	0.06
		0.13	0.010	0.1	0.5	0.007	0.032	0.04
		0.5	0.011	0.1	0.5	0.0015	0.0023	0.003
		1	0.011	0.1	0.5	0.0	0.0	0.0
	1	0.02	0.005	0.1	0.5	0.01	0.04	0.05
		0.13	0.007	0.1	0.5	0.007	0.024	0.03
		0.5	0.008	0.1	0.5	0.001	0.0015	0.002
		1	0.008	0.1	0.5	0.0	0.0	0.0

[a] Linear interpolation between values listed in the table shall be permitted.
[b] Allowable story drift >1%.

characteristics are not adequately captured by the NDP, the acceptability of soil displacements shall be based on the foundation rotation limits in Table 8-8.

8.4.6 Shallow Foundation Lateral Load The lateral capacity of shallow foundations shall be calculated using established principles of soil mechanics and shall include the contributions of traction at the bottom and passive pressure resistance on the leading face. Mobilization of passive pressure shall be calculated using Figure 8-6. Acceptability of soil displacements shall be based on the ability of the structure to accommodate these displacements within the acceptance criteria for the selected Performance Objective.

Alternatively, it shall be acceptable to analyze the response of shallow foundations based on methods that are based on or have been calibrated to test data.

8.5 DEEP FOUNDATIONS

8.5.1 Pile Foundations A pile foundation shall be defined as a deep foundation system composed of one or more driven or cast-in-place piles and a pile cap cast in place over the piles, which together form a pile group supporting one or more load-bearing columns, or a linear sequence of pile groups supporting a shear wall.

Analysis of pile foundations shall be performed using the expected soil capacities in accordance with Section 8.4.2.1 or 8.4.2.2.

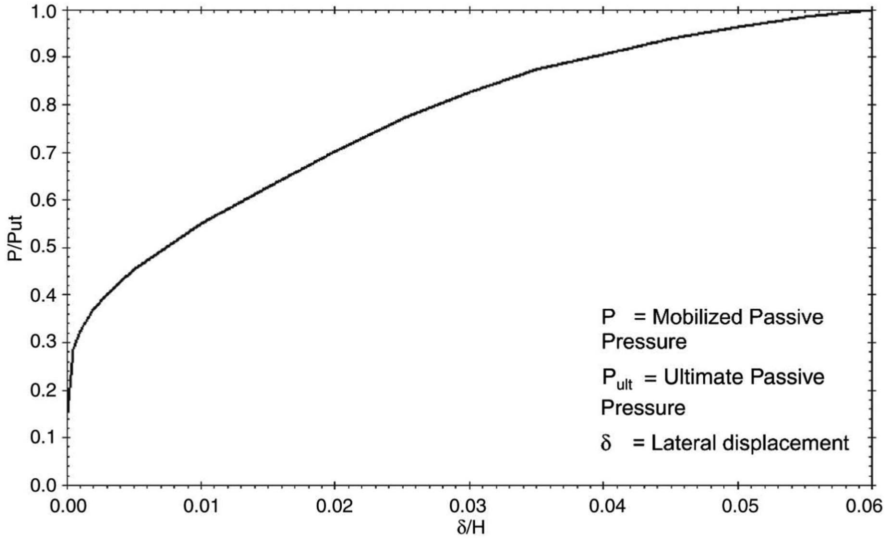


Figure 8-6. Passive pressure mobilization curve.

The requirements of this section shall apply to piles less than or equal to 24 in. (610 mm) in diameter. The stiffness characteristics of single large-diameter piles or drilled shafts larger than 24 in. (610 mm) in diameter shall comply with the requirements of Section 8.5.4.

8.5.1.1 Stiffness Parameters The uncoupled spring model shown in Figure 8-3 shall be used to represent the stiffness of a pile foundation where the footing in the figure represents the pile cap. When calculating the vertical and rocking springs, the contribution of the soil immediately beneath the pile cap shall be neglected. The total lateral stiffness of a pile group shall include the contributions of the piles (with an appropriate modification for group effects) and the passive resistance of the pile cap. The lateral stiffness of piles shall be based on classical methods or on analytical solutions using approved beam–column pile models. The lateral stiffness contribution of the pile cap shall be calculated using the passive pressure mobilization curve in Figure 8-6. Alternatively, it shall be acceptable to analyze the response of pile foundations based on methods that are based on or have been calibrated to test data.

Pile group axial spring stiffness values, k_{sv}, shall be calculated using Equation (8-25):

$$k_{sv} = \sum_{n=1}^{N} \frac{AE}{L} \tag{8-25}$$

where

A = Cross-sectional area of a pile,
E = Modulus of elasticity of piles,
L = Length of piles, and
N = Number of piles in group.

The rocking spring stiffness values about each horizontal pile cap axis shall be computed by modeling each pile axial spring as a discrete Winkler spring. The rotational spring constant, k_{sr} (moment per unit rotation), shall be calculated using Equation (8-26):

$$k_{sr} = \sum_{n=1}^{N} k_{vn} S_n^2 \tag{8-26}$$

where

k_{vn} = Axial stiffness of the nth pile, and
S_n = Distance between nth pile and axis of rotation.

8.5.1.2 Capacity Parameters The expected axial capacity Q_c of piles in compression and tension shall be determined using the procedures in Section 8.4.2 substituting Q_c for q_c in Equation (8-7) or (8-8) as applicable. The expected axial capacity in tension shall not exceed the lower-bound capacity of the foundation structural components.

The moment capacity of a pile group shall be determined assuming a rigid pile cap. Lower-bound moment capacity shall be based on triangular distribution of axial pile loading and lower-bound axial capacity of the piles. Upper-bound moment capacity shall be based on a rectangular distribution of axial pile load using full, upper-bound axial capacity of the piles.

The lateral capacity of a pile group shall include the contributions of the piles (with an appropriate modification for group effects) and the passive resistance of the pile cap. The lateral capacity of the piles shall be calculated using the same method used to calculate the stiffness. The lateral capacity of the pile cap, because of passive pressure, shall be calculated using established principles of soil mechanics. Passive pressure mobilization shall be calculated using Figure 8-6. Alternatively, it shall be acceptable to analyze the response of pile foundations based on methods that are based on or have been calibrated to test data.

8.5.2 Drilled Shafts The stiffness and capacity of drilled shaft foundations and piers of diameter less than or equal to 24 in. (610 mm) shall be calculated using the requirements for pile foundations specified in Section 8.5.1. For drilled shaft foundations and piers of diameter greater than 24 in. (610 mm), the capacity shall be calculated based on the interaction of the soil and shaft where the soil shall be represented using Winkler-type models specified in Section 8.5.1.

8.5.3 Deep Foundation Acceptance Criteria The foundation soil shall comply with the acceptance criteria specified in this section. The structural components of foundations shall meet the appropriate requirements of Chapters 9 through 12. The foundation soil shall be evaluated to support all actions, including vertical loads, moments, and seismic forces applied to the soil by the foundation.

8.5.3.1 Linear Procedures The acceptance criteria for foundation soil analyzed by linear procedures shall be based on the modeling assumptions for the base of the structure specified in Section 8.5.3.1.1 or 8.5.3.1.2.

8.5.3.1.1 Fixed-Base Assumption If the base of the structure is assumed to be completely rigid, the foundation soil at the soil–foundation interface shall be classified as deformation controlled. Component actions shall be determined by Equation (7-36). Acceptance criteria shall be based on Equation (7-39); *m*-factors for foundation soil shall be 2 for Immediate Occupancy, 3 for Life Safety, and 4 for Collapse Prevention, and the use of upper-bound component capacities shall be permitted. A fixed-base assumption shall not be used for buildings being evaluated or retrofitted to the Immediate Occupancy Performance Level that are sensitive to base rotations or other types of foundation movement that would cause the structural components to exceed their acceptance criteria.

8.5.3.1.2 Flexible-Base Assumption If the base of the structure is assumed to be flexible and is modeled using linear foundation soil at the soil–foundation interface, then the foundation soil shall be classified as deformation controlled. Component actions shall be determined by Equation (7-39). Soil strength need not be evaluated. Acceptability of soil displacements shall be based on the ability of the structure to accommodate these displacements within the acceptance criteria for the selected Performance Objective.

8.5.3.2 Nonlinear Procedures The acceptance criteria for foundation soil analyzed by nonlinear procedures shall be based on the modeling assumptions for the base of the structure specified in Sections 8.5.3.2.1 or 8.5.3.2.2.

8.5.3.2.1 Fixed-Base Assumption If the base of the structure is assumed to be completely rigid, then the base reactions for all foundations shall be classified as force controlled, as determined by Equation (7-40), and shall not exceed upper-bound component capacities. A fixed-base assumption shall not be used for buildings being evaluated or retrofitted for the Immediate Occupancy Performance Level that are sensitive to base rotations or other types of foundation movement that would cause the structural components to exceed their acceptance criteria.

8.5.3.2.2 Flexible-Base Assumption If the base of the structure is assumed to be flexible and is modeled using flexible nonlinear foundations, then the foundation soil shall be classified as deformation controlled and the displacements at the base of the structure and foundation shall not exceed the acceptance criteria of this section. For the Life Safety and Collapse Prevention Structural Performance Levels, acceptability of soil displacements shall be based on the ability of the structure and foundation to accommodate these displacements within the acceptance criteria for the selected Performance Objective. For the Immediate Occupancy Structural Performance Level, the permanent, nonrecoverable displacement of the foundation soil at the soil–foundation interface shall be calculated by an approved method based on the maximum total displacement, foundation and soil type, thickness of soil layers, and other pertinent factors. The acceptability of these displacements shall be based on the ability of the structure and foundation to accommodate them within the acceptance criteria for the Immediate Occupancy Structural Performance Level.

8.6 SOIL–STRUCTURE INTERACTION EFFECTS

Where required by Section 7.2.8, soil–structure interaction effects shall be calculated in accordance with Section 8.6.1 for kinematic interaction effects and Section 8.6.2 for foundation damping effects.

8.6.1 Kinematic Interaction Kinematic interaction effects shall be permitted to be calculated directly in the mathematical model or as represented by ratio of response spectra (RRS) factors RRS_{bsa} for base slab averaging, and RRS_e for embedment, which are multiplied by the spectral acceleration ordinates on the response spectrum calculated in accordance with Section 2.4. If kinematic interaction effects are to be included in the analysis of the building, the building's mathematical model must include flexible-base conditions per Section 8.4.5. Reduction of the response spectrum for kinematic interaction effects shall be permitted subject to the limitations in Sections 8.6.1.1 and 8.6.1.2.

The product of $RRS_{bsa} \times RRS_e$ shall not be less than 0.5.

8.6.1.1 Base Slab Averaging The RRS factor for base slab averaging, RRS_{bsa}, shall be determined using Equation (8-27) for each period of interest. RRS_{bsa} shall not be taken as less than the value computed when $T = 0.2$ s. Where base slab averaging is used with the LSP or LDP, in addition to a model with a flexible-base condition, the effective period used to compute RRS_{bsa} shall be assumed to be 1.5 times that obtained from the flexible-base model.

Reductions for base slab averaging shall be permitted when all the following conditions apply:

1. Located on a site with soil conditions characterized as Site Class C, D, or E;
2. Buildings that have structural mats or foundation elements interconnected with structural slabs or that are continuously connected with grade beams or other foundation elements of sufficient lateral stiffness so as not to be characterized as a flexible diaphragm with respect to the vertical elements of the lateral-force-resisting system in the story above based on the definition of flexible diaphragms in Section 1.2; and
3. Foundation elements stronger than the vertical elements of the lateral-force-resisting system.

$$RRS_{bsa} = 0.25 + 0.75 \times \left\{ \frac{1}{b_0^2} \left[1 - \exp(-2b_0^2) \times B_{bsa} \right] \right\}^{1/2} \tag{8-27}$$

where

$$B_{bsa} = \begin{cases} 1 + b_0^2 + b_0^4 + \dfrac{b_0^6}{2} + \dfrac{b_0^8}{4} + \dfrac{b_0^{10}}{12} & b_0 \le 1 \\ \exp(2b_0^2) \left[\dfrac{1}{\sqrt{\pi} b_0} \left(1 - \dfrac{1}{16 b_0^2} \right) \right] & b_0 > 1 \end{cases} \tag{8-28}$$

$$b_0 = 0.0001 \times \left(\frac{2\pi b_e}{T} \right) \tag{8-29}$$

b_e = Effective foundation size in feet;

$$b_e = \sqrt{A_{base}} \le 260\ \text{ft}; \tag{8-30}$$

T = Effective fundamental period of the building, in seconds, computed based on a mathematical model consistent with the requirements of Chapter 7 with a flexible-base condition per Section 8.4.5; and

A_{base} = Area of the foundation footprint if the foundation components are interconnected laterally (ft^2).

8.6.1.2 Embedment The RRS factor for embedment, RRS_e, shall be determined using Equation (8-31) for each period of interest, provided that a minimum of 75% of the foundation footprint is present at the embedment depth. The foundation embedment for buildings located on sloping sites shall be the shallowest embedment. RRS_e shall not be taken as less than the value computed when $T = 0.2$ s. RRS_e shall not be taken as less than the values computed with a maximum embedment of 20 ft (6.1 m). Where embedment effects are used with the LSP and LDP, in addition to a model with a flexible-base condition, the effective period used to compute RRS_e shall be assumed to be 1.5 times that obtained from the flexible-base model.

Reductions for embedment shall be permitted for buildings with the following characteristics:

1. Located on a site with soil conditions characterized as Site Class C, D, or E;
2. Structures that have structural mats or foundation elements interconnected with concrete slabs or that are continuously connected with grade beams or other foundation elements of sufficient lateral stiffness so as not to be characterized as a flexible diaphragm with respect to the vertical elements of the lateral-force-resisting system in the story above based on the definition of flexible diaphragms in Section 1.2; and
3. Foundation elements stronger than the vertical elements of the lateral-force-resisting system.

$$RRS_e = 0.25 + 0.75 \times \cos\left(\frac{2\pi e}{T v_s}\right) \geq 0.50 \tag{8-31}$$

where

e = Foundation embedment depth, in feet;
T = Effective fundamental period of the building, in s, consistent with the requirements of Chapter 7, computed based on a mathematical model with a flexible base per Section 8.4.5;
v_s = Effective shear wave velocity for site soil conditions, taken as average value of velocity over the embedment depth of the foundation (ft/s), or approximated as nv_{s0};
v_{s0} = Shear wave velocity for site soil conditions at low strains, taken as average value of velocity over the embedment depth of the foundation (ft/s);
n = Shear wave velocity reduction factor;

$$n = \sqrt{G/G_o};\ \text{and}$$

G/G_o = Effective shear modulus ratio from Table 8-1.

8.6.2 Foundation Damping Soil–Structure Interaction Effects The effects of foundation damping for nonlinear analyses shall be represented by the effective damping ratio of the structure–foundation system, β_{SSI}, determined in accordance with Equation (8-32). Foundation damping shall be permitted through explicit consideration of damping at the soil–foundation interface in the mathematical model or through modification of the acceleration response spectrum calculated in accordance with Section 2.4, using β_{SSI} in lieu of the effective viscous damping ratio, β, when used with the LSP, LDP, or NSP except where any of the following conditions are present:

1. The foundation system consists of discrete footings that are not interconnected and that are spaced less than the larger dimension of the supported lateral-force-resisting element in the direction under consideration;
2. The foundation system consists of, or includes, deep foundations such as piles or piers;
3. The foundation system consists of structural mats or are interconnected by concrete slabs that are characterized as flexible diaphragms with respect to the vertical elements of the lateral-force-resisting system in the story above based on the definition of flexible diaphragms in Section 1.2, or that are not continuously connected to grade beams or other foundation elements;
4. The foundation elements are weaker than the vertical elements of the lateral-force-resisting system;
5. $v_s T / r_x > 2\pi$ (where v_s = average shear wave velocity to a depth of r_x) and the shear stiffness of foundation soils increases with depth; or
6. The soil profile consists of a soft layer overlying a very stiff material, and the system period is greater that the first-mode period of the layer.

$$\beta_{SSI} = \beta_f + \frac{\beta}{(\tilde{T}/T)^2_{eff}} \leq 0.20 \tag{8-32}$$

where

β_f = Foundation–soil interaction damping ratio, as defined in Equation (8-33);
β = Effective viscous damping ratio of the building;
$\tilde{T}_{eff}/T_{eff}$ = Effective period lengthening ratio, as defined in Equation (8-34);
T = Fundamental period of the building using a mathematical model consistent with the requirements of Chapter 7 with a fixed base, in s;
$\tilde{T}$ = Fundamental period of the building using a mathematical model consistent with the requirements of Chapter 7 with a flexible-base condition per Section 8.4.5, in seconds; and
μ = Expected ductility demand. For nonlinear procedures, μ is the maximum displacement divided by the yield displacement (δ_t/δ_y for NSP). For linear procedures, μ is the maximum base shear divided by the elastic base shear capacity.

The foundation damping caused by radiation damping and soil hysteretic damping, β_f, shall be determined in accordance with Equation (8-33):

$$\beta_f = \left[\frac{(\tilde{T}/T)^2 - 1}{(\tilde{T}/T)^2}\right]\beta_s + \beta_{rd} \tag{8-33}$$

where

β_s = Soil hysteretic damping ratio determined in accordance with Section 19.3.5 of ASCE 7;
β_{rd} = Radiation damping ratio determined in accordance with Section 19.3.3 or Section 19.3.4 of ASCE 7.

$$\frac{\tilde{T}_{eff}}{T_{eff}} = \left\{1 + \frac{1}{\mu}\left[\left(\frac{\tilde{T}}{T}\right)^2 - 1\right]\right\}^{0.5} \tag{8-34}$$

8.6.2.1 Radiation Damping for Rectangular Foundations The effects of radiation damping for structures with a rectangular foundation plan shall be represented by the effective damping ratio of the soil–structure system, β_{rd}, determined in accordance with Equation (8-35):

$$\beta_{rd} = \frac{1}{(\tilde{T}/T_y)^2{}_y} + \frac{1}{(\tilde{T}/T_{xx})^2}\beta_{xx} \qquad (8\text{-}35)$$

$$T_y = 2\pi\sqrt{\frac{M^*}{K_y}} \qquad (8\text{-}36)$$

$$T_{xx} = 2\pi\sqrt{\frac{M^*(h^*)^2}{\alpha_{xx}K_{xx}}} \qquad (8\text{-}37)$$

$$K_y = \frac{GB}{2-\nu}\left[6.8\left(\frac{L}{B}\right)^{0.65} + 0.8\left(\frac{L}{B}\right) + 1.6\right] \qquad (8\text{-}38)$$

$$K_{xx} = \frac{GB^3}{1-\nu}\left[3.2\left(\frac{L}{B}\right) + 0.8\right] \qquad (8\text{-}39)$$

$$\beta_y = \left[\frac{4(L/B)}{(K_y/GB)}\right]\left[\frac{a_0}{2}\right] \qquad (8\text{-}40)$$

$$a_0 = \frac{2\pi B}{\tilde{T}v_s} \qquad (8\text{-}41)$$

$$\beta_{xx} = \left[\frac{(4\psi/3)(L/B)a_0^2}{\left(\frac{K_{xx}}{GB^3}\right)\left[\left(2.2 - \frac{0.4}{(L/B)^3}\right) + a_0^2\right]}\right]\left[\frac{a_0}{2\alpha_{xx}}\right] \qquad (8\text{-}42)$$

$$\psi = \sqrt{\frac{2(1-\nu)}{(1-2\nu)}} \leq 2.5 \qquad (8\text{-}43)$$

$$\alpha_{xx} = 1.0 - \left[\frac{(0.55 + 0.01\sqrt{(L/B) - 1})a_0^2}{\left(2.4 - \frac{0.4}{(L/B)^3}\right) + a_0^2}\right] \qquad (8\text{-}44)$$

where

M^* = Effective modal mass for the fundamental mode of vibration in the direction under consideration;

h^* = Effective structure height taken as the vertical distance from the foundation to the centroid of the first mode shape for multistory structures; as an alternative, h^* is permitted to be approximated as 70% of the total structure height for multistory structures or as the full height of the structure for 1-story structures;

L = Half the larger dimension of the base of the structure;

B = Half the smaller dimension of the base of the structure;

v_s = Average effective shear wave velocity over a depth of B below the base of the structure determined using v_{so} and Table 8-9 or a site-specific study;

v_{so} = Average low-strain shear wave velocity over a depth of B below the base of the structure;

G = Effective shear modulus derived or approximated based on G_0 and Table 8-1;

G_0 = $\gamma v_{so}{}^2/g$ = Average shear modulus for the soils beneath the foundation at small strain levels;

γ = Average unit weight of the soils over a depth of B below the base of the structure; and

ν = Poisson's ratio; it is permitted to use 0.3 for sandy and 0.45 for clayey soils with structure-to-soil stiffness ratio for different aspect ratios.

Table 8-9. Effective Shear Wave Velocity Ratio (v_s/v_{so}).

	Effective Peak Acceleration, $S_{DS}/2.5$[a]			
Site Class	$S_{DS}/2.5 = 0$	$S_{DS}/2.5 = 0.1$	$S_{DS}/2.5 = 0.4$	$S_{DS}/2.5 \geq 0.0$
A	1.00	1.00	1.00	1.00
B	1.00	1.00	0.97	0.95
C	1.00	0.97	0.87	0.77
D	1.00	0.95	0.71	0.32
E	1.00	0.77	0.22	[b]
F	[b]	[b]	[b]	[b]

[a] Use straight-line interpolation for intermediate values of $S_{DS}/2.5$.

[b] Site-specific geotechnical investigation and dynamic site response analyses are to be performed.

8.6.2.2 Soil Hysteretic Damping β_s shall be taken from Table 8-10 or other approved methods. If a site over a depth B or R below the base of the building consists of a relatively uniform layer of depth, D_s overlaying a very stiff layer with a shear wave velocity more than twice that of the surface layer and $4D_S/v_s\tilde{T} < 1$, then the damping values, β_{rd}, in Equation (8-35) shall be replaced by β_s', per Equation (8-45):

$$\beta_s' = \left(\frac{4D_s}{v_s\tilde{T}}\right)^4 \beta_s \qquad (8\text{-}45)$$

8.7 SEISMIC EARTH PRESSURE

Building walls retaining soil shall be evaluated to resist additional earth pressure caused by seismic forces when both of the following are true:

1. Performance Objectives include Immediate Occupancy and/or Damage Control and,
2. Short period spectral response acceleration parameter $S_{xs}/2.5$ for the Seismic Hazard Level exceeds $0.4g$.

The seismic earth pressure shall be added to the unfactored static active earth pressure to obtain the total earth pressure on the wall. The wall is permitted to be evaluated for out-of-plane forces as a deformatio-controlled component for flexural forces, and as

Table 8-10. Soil Hysteretic Damping Ratio, β_s.

	Effective Peak Acceleration, $S_{DS}/2.5$[a]			
Site Class	$S_{DS}/2.5 = 0$	$S_{DS}/2.5 = 0.1$	$S_{DS}/2.5 = 0.4$	$S_{DS}/2.5 \geq 0.8$
C	0.01	0.01	0.03	0.05
D	0.01	0.02	0.07	0.15
E	0.01	0.05	0.20	[b]
F	[b]	[b]	[b]	[b]

[a] Use straight-line interpolation for intermediate values of $S_{DS}/2.5$.

[b] Site-specific geotechnical investigation and dynamic site response analyses are to be performed.

a force-controlled component for shear forces using acceptance criteria based on the type of wall construction and approved methods.

The retaining wall acceptance criteria shall be based on Equation (7-39), using *m*-factors given in Table 8-11 for flexural forces, and Equation (7-40), for shear forces.

Table 8-11. Numerical Acceptance Criteria for Linear Procedures: Reinforced Concrete Retaining Walls Subjected to Seismic Increment and Controlled by Flexure.

		***m*-factors**[a]		
		Performance Level		
Conditions[b]	**ρ**[c]	**IO**	**LS**	**CP**
$\frac{P}{t_w l_w f'_{ce}} < 0.05$	<0.005	3	4.5	6
	>0.015	2	3	4
$\frac{P}{t_w l_w f'_{ce}} = 0.1$	<0.005	2.25	3.25	4.5
	>0.015	1.5	2.25	3.25
$\frac{P}{t_w l_w f'_{ce}} > 0.25$	<0.005	1.0	1.5	2.0
	>0.015	1.0	1.25	1.75

[a] Linear interpolation between values shall be permitted.

[b] P = axial load on the wall, t_w = wall thickness, l_w = wall length, and f'_{ce} = expected concrete strength.

[c] ρ = flexural reinforcing ratio.

8.8 FOUNDATION RETROFIT

Foundation retrofit schemes shall be evaluated in conjunction with any retrofit of the superstructure and according to the general principles and requirements of this standard to ensure that the complete retrofit achieves the selected building performance level for the selected Seismic Hazard Level. Where new retrofit components are used in conjunction with existing components, the effects of differential foundation stiffness on the modified structure shall be demonstrated to meet the acceptance criteria. If existing loads are not redistributed to all the components of the foundation by shoring and/or jacking, the effects of differential strengths and stiffnesses among individual foundation components shall be included in the analysis of the foundation. The effects of a retrofit on stiffness, strength, and deformability shall be taken into account in an analytical model of the building. The compatibility of new and existing components shall be checked at displacements consistent with the performance level chosen.

CHAPTER 9

STEEL AND IRON

9.1 SCOPE

This chapter sets forth requirements for the seismic evaluation and retrofit of structural steel, composite steel–concrete, cast and wrought iron, and cold-formed steel components of the seismic-force-resisting system of an existing building. The requirements of this chapter shall apply to the original components, retrofitted components, and added components of a structural system of an existing building.

Section 9.2 provides the reference standard for structural steel, composite steel–concrete, and cast and wrought iron. Section 9.3 presents modifications of that standard for use with this document. Section 9.4 specifies data collection procedures for obtaining material properties and performing condition assessments of cold-formed steel. Section 9.5 specifies general analysis and design requirements for cold-formed steel components. Sections 9.6 through 9.8 provide modeling procedures, component strengths, acceptance criteria, and retrofit measures for cold-formed steel seismic-force-resisting systems.

9.2 REFERENCE STANDARD FOR STRUCTURAL STEEL, COMPOSITE STEEL–CONCRETE, AND CAST AND WROUGHT IRON

Seismic evaluation and retrofit of structural steel, composite steel–concrete, and cast and wrought iron components of the seismic force-resisting system of an existing building shall be in accordance with the provisions of AISC 342, as modified by Section 9.3.

9.3 MODIFICATION TO THE REFERENCE STANDARD FOR STRUCTURAL STEEL, COMPOSITE STEEL–CONCRETE, AND CAST AND WROUGHT IRON

This section addresses modifications to the reference standard. There are no modifications to the reference standard.

9.4 MATERIAL PROPERTIES AND CONDITION ASSESSMENT FOR COLD-FORMED STEEL

9.4.1 General Material properties for cold-formed steel materials, components, and assemblies shall be based on available construction documents, test reports, manufacturers' data, and as-built conditions for the particular structure as specified in Section 3.2. Where such documentation fails to provide adequate information to quantify material properties, capacities of assemblies, or establish the condition of the structure, such documentation shall be supplemented by material tests, mock-up tests of assemblies, and assessments of existing conditions, as required in Section 6.2.

Material properties of existing cold-formed steel components shall be determined in accordance with Section 9.4.2. A condition assessment shall be conducted in accordance with Section 9.4.3. The extent of materials testing and condition assessment performed shall be used to determine the knowledge factor, κ, as specified in Section 9.4.4.

Use of material properties based on historical information as default values shall be permitted as specified in Section 9.4.2.5.

9.4.2 Properties of In-Place Materials and Components

9.4.2.1 Material Properties The material properties (e.g., base steel thickness, material grade and mechanical properties) of the in-place cold-formed steel (CFS) light-frame components shall be established by one or more of the following methods:

1. Review of construction documents,
2. Inspection of manufacturers' product identification,
3. Examination of samples by an experienced metallurgist,
4. Measurements to establish base steel thickness, and/or
5. Materials testing to establish mechanical properties.

Base steel thickness shall exclude the thickness of any coatings (e.g., zinc or paint).

Where material testing is required by Section 6.2 or this list to establish mechanical properties, tests shall be conducted in accordance with ASTM A370. Samples shall be obtained in a manner that does not compromise the strength or stiffness of the structure. Samples shall be tested in accordance with Section 9.4.2.3.

9.4.2.1.1 Default Mechanical Properties and Nominal or Specified Properties of Cold-Formed Steel Light-Frame Construction

1. **Default Mechanical Properties.** Use of default mechanical properties for CFS light-frame shear walls, diaphragms, components, and connectors shall be permitted in accordance with Section 9.4.2.5. Use of material properties based on historical information for use as default values shall be as specified in Section 9.4.2.5. Other approved values of material properties shall be permitted if they are based on available historical information for a particular type of light-frame construction, prevailing codes, and assessment of existing condition.
2. **Nominal or Specified Properties.** Use of nominal material properties or properties specified in construction documents to compute expected and lower-bound material properties shall be permitted in accordance with Section 9.4.2.5.

9.4.2.2 Component Properties

1. **Elements.** The following component properties, as applicable, shall be determined in accordance with Section 9.4.3:
    1.1. Cross-sectional shape and physical dimensions of the primary components and overall configuration of the structure, including any modifications subsequent to original construction;
    1.2. Configuration of elements, size and thickness of connected materials, base steel thickness and mechanical properties, connection size and spacing, and continuity of load path;
    1.3. Location and dimension of seismic-force-resisting elements, type, materials, and spacing of tie-downs and boundary components; and
    1.4. Current physical condition of components and extent of any deterioration present.
2. **Connections.** The following connection details, as applicable, shall be determined or verified in accordance with Section 9.4.3:
    2.1. Connections between horizontal diaphragms and vertical elements of the seismic-force-resisting system,
    2.2. Size and character of all diaphragm ties,
    2.3. Connections at splices in chord members of horizontal diaphragms,
    2.4. Connections of floor and roof diaphragms to exterior or interior concrete or masonry walls for both in-plane and out-of-plane loads,
    2.5. Connections of shear walls to foundations for transfer of shear and overturning forces, and
    2.6. Method of through-floor transfer of wall shear and overturning forces in multistory buildings.

9.4.2.3 Test Methods to Quantify Mechanical Properties The stiffness and strength of CFS light-frame components and assemblies shall be established through in situ testing or mock-up testing of assemblies in accordance with Section 7.6, unless default values are used in accordance with Section 9.4.2.5. The number of tests required shall be based on Section 9.4.2.4. Expected material properties shall be based on mean values of tests. Lower-bound material properties shall be based on mean values of tests minus one standard deviation.

9.4.2.4 Minimum Number of Tests Materials testing is not required for cold-formed steel if material properties are available from original construction documents that include material test records or material test reports. If such properties differ from default material properties, material properties for evaluation and retrofit shall be selected such that the largest demands on components and connections are generated.

Where required, testing for cold-formed steel shall meet the requirements for usual testing in Section 9.4.2.4.1 or comprehensive testing in Section 9.4.2.4.2.

9.4.2.4.1 Usual Testing for Cold-Formed Steel For cold-formed steel components, the minimum number of tests to quantify expected-strength material properties for usual data collection shall be based on the following criteria:

1. If construction documents containing material property and detailing information for the seismic-force-resisting system are available, at least one element of the seismic-force-resisting system for each story, or for every 100,000 ft^2 (9,290 m^2) of floor area, is to be randomly verified by observation for compliance with the construction documents; and
2. If construction documents are incomplete or not available, at least two locations for each story, or 100,000 ft^2 (9,290 m^2) of floor area, are to be randomly verified by observation or otherwise documented.

9.4.2.4.2 Comprehensive Testing for Cold-Formed Steel For cold-formed steel components, the minimum number of tests necessary to quantify expected-strength properties for comprehensive data collection shall be defined in accordance with the following requirements:

1. If original construction documents exist that define the mechanical properties, at least one location for each story is to be randomly verified by observing product marking for each component type identified as having a different material grade;
2. If original construction documents defining mechanical properties are not complete or do not exist but the date of construction is known and use of a single material grade is confirmed, at least three locations are to be randomly verified—by sampling and testing in accordance with Section 9.4.2.1 or by observing product markings and conditions—for each component type, for every two floors in the building;
3. If no knowledge of the structural system and materials used exists, at least six locations are to be randomly verified—by sampling and testing or by observing product marking and conditions—for each element and component type, for every two floors or 200,000 ft^2 (18,580 m^2) of floor area of construction. If it is determined from testing or observation that more than one material grade exists, additional observations and testing are to be conducted until the extent of use for each material grade has been established;
4. In the absence of construction records defining connector features, the configurations of at least three connectors are to be documented for every floor or 100,000 ft^2 (9,290 m^2) of floor area in the building; and
5. A full-scale mock-up test is to be conducted for archaic assemblies; at least two cyclic tests of each assembly shall be conducted. A third test shall be conducted if the results of the two tests vary by more than 20%.

9.4.2.5 Default Mechanical Properties Use of default mechanical properties to determine component strengths shall be permitted in conjunction with the linear analysis procedures of Chapter 7. Lower-bound material properties shall be based on ASTM standards applicable at the time of construction. If the material grade of steel is not known, then F_y shall be assumed to be no greater than 33 ksi (227 MPa). Default expected-strength material properties shall be permitted to be determined by multiplying the nominal yield stress F_y by R_y and/or the nominal tensile stress F_t by R_t in accordance with Table 9-1.

Default expected-strength values for fasteners and connection hardware used in cold-formed steel light-frame assemblies shall be taken as the average ultimate test values from published reports or from applicable ASTM standards.

Default stiffness values for individual connections between two plies of steel sheet in a range of 33 to 97 mils (0.84 to 2.46 mm) fastened by #8, #10, or #12 fasteners shall be permitted to be based on a deflection of 1/32 in. (0.79 mm) at yield and a deflection of 1/4 in. (6.35 mm) at peak capacity.

Default lower-bound strength values, where required in this chapter and not available per applicable ASTM standards at the time of construction, shall be taken as expected-strength values multiplied by 0.85.

Table 9-1. Multipliers for Expected Yield and Tensile Stress of Sheet and Strip Steel Used in Cold-Formed Steel.

Sheet and Strip Steel	R_y	R_t
$F_y < 37$ kip/in.^{2}	1.5	1.2
37 kip/in.^{2} $< F_y <$ 40 kip/in.^{2}	1.4	1.1
40 kip/in.^{2} $< F_y <$ 50 kip/in.^{2}	1.3	1.1
$F_y > 50$ kip/in.^{2}	1.1	1.1

Source: Sheet and Strip Steel: ASTM A606, A653/653M, A792/A792M, A875, A1003/A1003M, A1008/A1008M, A1011/A1011M.

9.4.3 Condition Assessment

9.4.3.1 General A condition assessment of the existing building and site shall be performed as specified in this section. A condition assessment shall include the following:

1. Examination of the physical condition of primary and secondary components and the documentation of the presence of any degradation;
2. Verification of the presence and configuration of structural elements and components and their connections, and the continuity of load paths among components, elements, and systems; and
3. Identification and documentation of other conditions, including neighboring party walls and buildings, the presence of nonstructural components that influence building performance, and prior remodeling.

9.4.3.2 Scope and Procedures The condition assessment shall include visual inspection of accessible structural elements and components involved in seismic force resistance to verify information shown on available documents.

If coverings or other obstructions exist, either partial visual inspection through use of drilled holes and a fiberscope shall be used, or complete visual inspection shall be performed by local removal of covering materials.

All primary structural components of the gravity and seismic-force-resisting systems shall be included in the condition assessment. The condition assessment shall meet the requirements for visual condition assessment in accordance with Section 9.4.3.2.1 or comprehensive condition assessment in accordance with Section 9.4.3.2.2.

9.4.3.2.1 Visual Condition Assessment of Cold-Formed Steel Components and Connections The dimensions and features of all accessible cold-formed steel components shall be measured and compared with available design information. Similarly, the configuration and condition of all accessible cold-formed connections shall be visually verified, with any deformations or anomalies noted.

9.4.3.2.2 Comprehensive Condition Assessment of Cold-Formed Steel Components and Connections If coverings or other obstructions exist over cold-formed steel components, either partial visual inspection through the use of drilled holes and a fiberscope shall be used, or visual inspection shall be performed by local removal of covering materials in accordance with the following requirements:

1. If construction documents exist, at least three different primary connections are to be exposed for each connection type. If no capacity-reducing deviations from the construction documents exist, the sample is considered representative. If deviations are noted, then all coverings from primary connections of that type are to be removed, unless the connection strength is ignored in the seismic evaluation; and
2. In the absence of construction documents, at least 50% of the top and at least 50% of the base connections for each type of vertical element in the seismic-force-resisting system, as well as collectors, boundary components, and tie-downs, are to be exposed and inspected or inspected fiberscopically. If common detailing is observed, this sample is considered representative. If any details or conditions are observed that result in a discontinuous load path, all primary connections are to be exposed.

9.4.3.3 Basis for the Mathematical Building Model The results of the condition assessment shall be used to quantify the following items needed to create a mathematical building model:

1. Component section properties and dimensions,
2. Component configuration and eccentricities,
3. Interaction of nonstructural components and their involvement in seismic force resistance, and
4. Presence and effects of alterations to the structural system.

If no damage, alteration, or degradation is observed in the condition assessment, component section properties shall be taken from design drawings. If some sectional material loss or deterioration has occurred, the loss shall be quantified by direct measurement and section properties shall be reduced accordingly using principles of structural mechanics. All deviations noted between available construction records and as-built conditions shall be accounted for in the structural analysis.

9.4.4 Knowledge Factor A knowledge factor (κ) for computation of cold-formed steel component capacities and permissible deformations shall be selected in accordance with Chapter 6 with the following additional requirements.

For cold-formed steel components and assemblies, if a comprehensive condition assessment is performed in accordance with Section 9.4.3.2.2, a knowledge factor, $\kappa = 1.0$, shall be permitted in conjunction with default properties of Section 9.4.2.5, and testing in accordance with Section 9.4.2.4 is not required.

9.5 GENERAL ASSUMPTIONS AND REQUIREMENTS FOR COLD-FORMED STEEL

9.5.1 Stiffness Component stiffnesses shall be calculated in accordance with Sections 9.6 through 9.8.

9.5.1.1 Use of Linear Procedures for Cold-Formed Steel Light-Frame Construction Where design actions are determined using the linear procedures of Chapter 7, the stiffnesses for cold-formed steel light-frame materials comprising of individual components shall be based on the material properties determined in accordance with Section 9.4.2.

9.5.1.2 Use of Nonlinear Procedures for Cold-Formed Steel Light-Frame Construction Where design actions are determined using the nonlinear procedures of Chapter 7, component force–deformation response shall be represented by nonlinear force–deformation relations. Linear relations shall be permitted where nonlinear response does not occur in the component. The nonlinear force–deformation relation shall be based on experimental

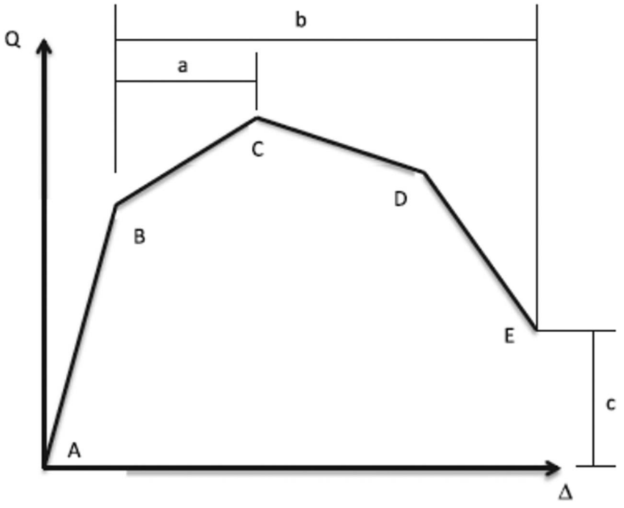


Figure 9-1. Generalized force–deformation relation for cold-formed steel light-frame elements or components.

evidence or parameters derived in accordance with the generalized force–deformation relation shown in Figure 9-1.

9.5.2 Strength and Acceptance Criteria

9.5.2.1 General Actions in a structure shall be classified as being either deformation-controlled or force-controlled, as defined in Section 7.5.1. Design strengths for deformation-controlled and force-controlled actions shall be calculated in accordance with Sections 9.5.2.2 and 9.5.2.3, respectively.

9.5.2.2 Deformation-Controlled Actions Expected strengths for deformation-controlled actions, Q_{CE}, on cold-formed steel light-frame construction shall be taken as the mean maximum strengths obtained experimentally or calculated using accepted principles of mechanics. For components controlled by steel material properties, it is permitted to determine expected strength by using the nominal strength calculations in AISI S100, AISI S240, or AISI S400 and correcting the steel material properties to their expected values based on Section 9.4.2.5. For all other components, unless other procedures are specified in this chapter, expected strengths shall be permitted to be based on 1.5 times the nominal strength calculation in AISI S100, AISI S240, or AISI S400, as appropriate. Acceptance criteria for deformation-controlled actions shall be as specified in Sections 9.6 through 9.8.

9.5.2.3 Force-Controlled Actions Strengths for force-controlled actions, Q_{CL}, on cold-formed steel components shall be taken as lower-bound strengths obtained experimentally or calculated using established principles of mechanics. Where determined by testing, lower-bound strengths for force-controlled actions, Q_{CL}, on cold-formed steel light-frame construction shall be taken as the mean of the maximum strengths obtained experimentally minus one standard deviation. Where calculated using established principles of mechanics or based on load and resistance factor design (LRFD) procedures contained in AISI S100, AISI S240, or AISI S400, the resistance factor, ϕ, shall be taken as 1.0, and default lower-bound material properties determined in accordance with Section 9.4.2.5 shall be used.

Where the force-controlled design actions, Q_{UF}, calculated in accordance with Section 7.5.2.1.2 are based on a limit-state analysis, the expected strength of the components delivering load to the component under consideration shall be taken as not less than 1.5 times the nominal strength.

9.5.2.4 Anchorage to Concrete Connections of cold-formed steel components to concrete components shall comply with the provisions of this chapter and Chapter 10 for determination of strength and classification of actions as deformation-controlled or force-controlled.

The strength of connections between cold-formed steel components and concrete components shall be the lowest value obtained for the limit states of the strength of the cold-formed steel components, strength of the connection plates, and strength of the anchor bolts.

The strength of column baseplates shall be the lowest strength calculated based on the following limit states: expected strength of welds or bolts, expected bearing stress of the concrete, and expected yield strength of the baseplate.

The strength of the anchor bolt connection between the column baseplate and the concrete shall be the lowest strength calculated based on the following limit states: shear or tension yield strength of the anchor bolts, loss of bond between the anchor bolts and the concrete, or failure of the concrete. Anchor bolt strengths for each failure type or limit state shall be calculated in accordance with ACI 318, using $\phi = 1.0$, or other procedures approved by the Authority Having Jurisdiction.

Column base connection limit states controlled by anchor bolt failure modes governed by the concrete shall be considered force controlled.

9.5.3 Connection Requirements in Cold-Formed Steel Light-Frame Construction Unless otherwise specified in this standard, connections between CFS light-frame components of a seismic-force-resisting system shall be considered in accordance with this section. Demands on connectors, including, as applicable, screws and bolts used to link components, shall be considered force-controlled actions. Demands on bodies of connections, and bodies of connection hardware, shall be considered force-controlled actions when associated with fracture limit states and deformation-controlled actions when associated with yielding or bearing limit states.

9.5.4 Components Supporting Discontinuous Shear Walls in Cold-Formed Steel Light-Frame Construction Axial compression on chord studs and flexure and shear on tracks that support discontinuous CFS light-frame shear walls shall be considered force-controlled actions. Lower-bound strengths shall be determined in accordance with Section 9.5.2.3.

9.5.5 Retrofit Measures If portions of a CFS light-frame building structure are deficient for the selected Performance Objective, the structure shall be retrofitted, reinforced, or replaced. If replacement of the element is selected or if new elements are added, the new elements shall satisfy the acceptance criteria of this standard and shall be detailed and constructed in accordance with an approved building code. If reinforcement of the existing framing system is selected, the following factors shall be considered:

1. Degree of degradation in the component from such mechanisms as corrosion, high static or dynamic loading, or other effects;
2. Level of steady-state stress in the components to be reinforced and the potential to temporarily remove this stress, if appropriate;
3. Elastic and inelastic properties of existing components; strain compatibility with any new reinforcement materials shall be provided;

4. Ductility, durability, and suitability of existing connectors between components, and access for reinforcement or modification;
5. Efforts necessary to achieve appropriate fit-up for reinforcing components and connections;
6. Load path and deformation of the components at end connections; and
7. Presence of components manufactured with archaic materials, which can contain material discontinuities, to be examined during the retrofit design to ensure that the selected reinforcement is feasible.

9.6 COLD-FORMED STEEL LIGHT-FRAME CONSTRUCTION, SHEAR WALL SYSTEMS

9.6.1 General Cold-formed steel light-frame construction shear wall systems shall be categorized as primary or secondary components in accordance with Section 7.5.1.

Dissimilar wall sheathing materials on opposite sides of a wall shall be permitted to be combined where there are test data to substantiate the stiffness and strength properties of the combined systems. Otherwise, walls sheathed with dissimilar materials shall be analyzed based on only the wall sheathing with the greatest capacity.

For overturning calculations on shear wall elements, stability shall be evaluated in accordance with Section 7.2.9. Net tension caused by overturning shall be resisted by uplift connections.

The effects of openings in shear walls shall be considered. Where required, reinforcement consisting of chords and collectors shall be added to provide sufficient load capacity around openings to meet the strength requirements for shear walls.

Connections between shear walls and other components, including diaphragm ties, collectors, diaphragms, and foundations, shall be considered in accordance with Section 9.5.3 and shall be designed for forces calculated in accordance with Chapter 7. Components supporting discontinuous shear walls shall be considered in accordance with Section 9.5.4.

Shear wall chord studs, anchorage, and collectors shall be designed for forces calculated in accordance with Chapter 7, including superimposed gravity, Q_G, and earthquake, Q_E, demands. Q_E shall not be less than the demand developed because of the expected strength of the shear wall, Q_{CE}. If the capacity of the chord studs, anchorage, or collectors that are part of the shear wall framing is less than the demand developed because of the expected strength of the shear wall, then the shear wall shall be considered as a force-controlled element.

The expected strength, Q_{CE}, of cold-formed steel light-frame shear walls shall be determined in accordance with Section 9.6.3.

9.6.2 Types of Cold-Formed Steel Light-Frame Construction, Shear Wall Systems

9.6.2.1 Existing Cold-Formed Steel Light-Frame Shear Walls Walls are framed from cold-formed steel members (stud and track) and sheathed with wood structural panels, steel sheet, gypsum board, fiberboard, or plaster on metal lath and connected to the cold-formed steel members.

9.6.2.2 Enhanced Cold-Formed Steel Light-Frame Shear Walls Enhanced cold-formed steel light-frame shear walls shall include existing shear walls retrofitted in accordance with this standard or an approved method.

9.6.2.3 New Cold-Formed Steel Light-Frame Shear Walls New cold-formed steel light-frame shear walls shall include all new wood structural panel, steel sheet, gypsum board, and fiberboard elements added to an existing seismic-force-resisting system. Design of new shear walls shall satisfy the acceptance criteria of this standard. Details of construction for new shear walls, including track anchorage details, tie-down anchor details, fastening details for sheathing, and dimensional limitations for studs and tracks, shall be in accordance with the requirements of AISI S240, AISI S400, or an approved building code.

9.6.3 Stiffness, Strength, Acceptance Criteria, and Connection Design for Cold-Formed Steel Light-Frame Construction Shear Wall Systems

9.6.3.1 Wood Structural Panels

9.6.3.1.1 Stiffness of Wood Structural Panels The deflection of wood structural panel shear walls at yield (Δ_y) shall be determined as 2 times the deflection at 40% of the nominal wall strength determined in accordance with AISI S400. Properties used to compute shear wall deflection and stiffness shall be based on Section 9.4.2.

9.6.3.1.2 Strength of Wood Structural Panels The expected strength of wood structural panel shear walls shall be taken as the mean maximum strength obtained experimentally. Expected strengths of wood structural panel shear walls shall be permitted to be based on strengths determined using LRFD procedures contained in AISI S400, except that the resistance factor, ϕ, shall be taken as 1.0 and expected material properties shall be determined in accordance with Section 9.4.2. The expected-strength values of fasteners shall be calculated in accordance with Section 9.4.2.5, based on approved data. The expected strength of the wood structural panel shear wall shall be permitted to be determined from expected strength of fasteners in accordance with Section 9.4.2.5, where the strength of the shear wall is computed using principles of mechanics.

9.6.3.1.3 Acceptance Criteria for Wood Structural Panels For linear procedures, *m*-factors for use with deformation-controlled actions shall be as specified in Table 9-2. Also, it shall be permitted to derive *m*-factors from experimental data. For nonlinear procedures, the nonlinear force–deformation relation shall be as specified in Table 9-3. Also, it shall be permitted to derive the relation based on experimental evidence or parameters derived in accordance with the generalized force–deformation relation described by Figure 9-1.

9.6.3.1.4 Connections of Wood Structural Panels The connections between parts of the shear wall assembly and other elements of the seismic-force-resisting system shall be considered in accordance with Section 9.6.1.

9.6.3.2 Steel Sheet Sheathing

9.6.3.2.1 Stiffness of Steel Sheet Sheathing The deflection of steel sheet sheathing shear walls at yield (Δ_y) shall be determined as 2 times the deflection at 40% of the nominal wall strength determined in accordance with AISI S240 or AISI S400. Properties used to compute shear wall deflection and stiffness shall be based on Section 9.4.2.

9.6.3.2.2 Strength of Steel Sheet Sheathing The expected strength of steel sheet sheathing shear walls shall be taken as the mean maximum strength obtained experimentally. Expected strengths of steel sheet sheathing shear walls shall be permitted to be based on strengths determined using LRFD procedures contained in AISI S400, except that the resistance factor, ϕ, shall be taken as 1.0. The expected-strength values of fasteners shall be calculated in accordance with Section 9.4.2.5, based on approved

Table 9-2. Numerical Acceptance Factors for Linear Procedures: Cold-Formed Steel Light-Frame Components.

		m-Factors				
			Primary		Secondary	
Component/Action	**Limitation**	**IO**	**LS**	**CP**	**LS**	**CP**
CFS Light-Frame Construction, Shear Wall Systems[a,b]	Height/Width Ratio (h/b)					
Wood Structural Panel						
Structural 1 Plywood	≤2	1.2	1.9	2.4	2.8	3.7
Oriented Strand board (OSB)	≤4	1.7	2.5	3.3	4.2	5.6
Canadian Soft Plywood (CSP)	≤2	1.4	2.1	2.7	3.1	4.1
Canadian Soft Plywood (CSP)	4[c]	1.3	1.9	2.3	2.3	3.1
Douglas Fir Plywood (DFP)	≤2	1.2	1.9	2.4	2.8	3.7
Steel Sheet Sheathing	≤2	1.5	2.2	2.9	5.2	6.9
Steel Sheet Sheathing	4[c]	1.1	1.6	1.9	1.9	2.5
Gypsum Board Panel	≤2	2.3	3.5	4.6	8.3	11.1
Fiberboard Panel	≤2	1.1	1.7	2.3	2.8	3.7
Plaster on metal lath	≤2.0	1.4	2.1	2.8	2.8	3.8
CFS Light-Frame Construction, Strap-Braced Wall Systems[a,b]	Height/Width Ratio (h/b)					
Flat strap	≤2	3.0	4.4	4.9	5.3	7.1
Dogbone strap	≤2	3.8	5.7	6.2	6.2	8.3
Flat strap with 1 or 2 plies of gypsum board	≤2	1.2	1.8	2.4	3.8	5.1
CFS Light-Frame Construction, Diaphragms	Length/Width Ratio (L/b)					
Wood Structural Panel, unblocked, chorded	≤4			[Reserved]		
Wood Structural Panel, blocked, chorded	≤4			[Reserved]		
CFS Members						
CFS Member in Flexure		$0.38\frac{\theta_2}{\theta_y}$	$0.56\frac{\theta_2}{\theta_y}$	$0.75\frac{\theta_2}{\theta_y} \leq 0.56\frac{\theta_4}{\theta_y}$	$0.56\frac{\theta_4}{\theta_y}$	$0.75\frac{\theta_4}{\theta_y}$
CFS Member in Compression				[Reserved]		
CFS Connections	fastener					
Screws—steel to steel (33 to 97 mil sheet)[d]	#8, #10, or #12	2.3	3.4	4.5	15	20
Screws—wood to steel				[Reserved]		
Bolts—steel to steel				[Reserved]		

[a] Components are permitted to be classified as secondary components or nonstructural components, subject to the limitations of Section 7.2.4.3. Acceptance criteria need not be considered for walls classified as secondary or nonstructural.

[b] Components with aspect ratios exceeding maximum listed values are not considered effective in resisting seismic forces.

[c] Linear interpolation between aspect ratios for determination of *m*-factors is permitted.

[d] Median values are provided, and variation across sheet thickness and fastener size and type can be significant.

Note: CFS = Cold-formed steel.

data. The expected strength of the steel sheet sheathing shear wall shall be permitted to be determined using principles of mechanics.

9.6.3.2.3 Acceptance Criteria for Steel Sheet Sheathing For linear procedures, *m*-factors for use with deformation-controlled actions shall be as specified in Table 9-2. Also, it shall be permitted to derive *m*-factors from experimental data. For nonlinear procedures, the nonlinear force–deformation relation shall be as specified in Table 9-3. Also, it shall be permitted to derive the relation based on experimental evidence or parameters derived in accordance with the generalized force–deformation relation described by Figure 9-1.

9.6.3.2.4 Connections of Steel Sheet Sheathing The connections between parts of the shear wall assembly and other elements of the seismic-force-resisting system shall be considered in accordance with Section 9.6.1.

9.6.3.3 Gypsum Board Panel

9.6.3.3.1 Stiffness of Gypsum Board Panel Shear Walls The deflection of gypsum board panel shear walls at yield (Δ_y) shall be determined as 2 times the deflection at 40% of the nominal wall strength determined in accordance with AISI S240. Properties used to compute shear wall deflection and stiffness shall be based on Section 9.4.2.

9.6.3.3.2 Strength of Gypsum Board Panel Shear Walls The expected strength of gypsum board panel shear walls shall be taken as the mean maximum strength obtained experimentally. Expected strengths of gypsum board panel shear walls shall be permitted to be based on strengths determined using LRFD

Table 9-3. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures: Cold-Formed Steel Light-Frame Components.

		Modeling Parameters			Acceptance Criteria		
		Δ/Δ_y		Residual Strength Ratio	Inelastic Deformation ($\Delta_{inelastic}/\Delta_y$)		
		a	b	c	IO	LS	CP
CFS Light-Frame Construction, Shear Wall System[a,b]	Height/ Width Ratio (h/b)						
Wood Structural Panel							
Structural 1 Plywood	≤2	2.3	4.0	0.3	0.7	2.8	4.0
Oriented Strand Board (OSB)	≤4	3.4	6.5	0.3	1.2	4.6	6.5
Canadian Soft Plywood (CSP)	≤2	2.7	4.5	0.3	0.9	3.1	4.5
Canadian Soft Plywood (CSP)	4[c]	2.4	3.2	0.6	0.6	2.2	3.2
Douglas Fir Plywood (DFP)	≤2	2.3	4.0	0.3	0.7	2.8	4.0
Steel Sheet Sheathing	≤2	2.9	8.2	0.6	1.0	5.9	8.2
Steel Sheet Sheathing	4[c]	1.8	2.5	0.8	0.3	1.6	2.5
Gypsum Board Panel	≤2	5.2	13.8	0.6	2.1	10.1	13.8
Fiberboard Panel	≤2	2.0	3.9	0.4	0.5	2.7	3.9
Plaster on Metal Lath	≤2.0			0.2	1.1	3.0	4.0
CFS Light-Frame Construction, Strap-Braced Wall Systems[a,b]	Height/ Width Ratio (h/b)						
Flat strap	≤2	6.9	8.4	0.8	2.5	6.1	8.4
Flat strap	4[c]						
Dogbone strap	≤2	9.2	10.1	0.6	3.2	7.3	10.1
Flat strap with 1 or 2 plies of gypsum board	≤2	2.2	5.8	0.9	0.65	4.1	5.8
CFS Light-Frame Construction Diaphragms	Length/ Width Ratio (L/b)						
Wood Structural Panel, unblocked, chorded	≤4			[Reserved]			
Wood Structural Panel, blocked, chorded	≤4			[Reserved]			
CFS Members							
CFS Member in Flexure		$\frac{\theta_2}{\theta_y}-\frac{\theta_1}{\theta_y}$	$\frac{\theta_4}{\theta_y}-\frac{\theta_1}{\theta_y}$	$\frac{M_4}{M_y}$	$\frac{\theta_2}{\theta_y}\leq 0.67\frac{\theta_4}{\theta_y}$	$0.75\frac{\theta_4}{\theta_y}$	$\frac{\theta_4}{\theta_y}-\frac{\theta_1}{\theta_y}$
CFS Member in Compression				[Reserved]			
CFS Connections							
Screws—steel to steel (33 to 97 mil sheet)		5	25	0.9	2.0	18.5	25
Screws—wood to steel				[Reserved]			
Bolts—steel to steel				[Reserved]			

[a] Components are permitted to be classified as secondary components or nonstructural components, subject to the limitations of Section 7.2.4.3. Acceptance criteria need not be considered for walls classified as secondary or nonstructural.

[b] Components with aspect ratios exceeding maximum listed values are not considered effective in resisting seismic forces.

[c] Linear interpolation between aspect ratios for determination of *m*-factors is permitted.

Notes: Median values are provided, and variation across sheet thickness and fastener size and type can be significant. CFS = Cold-formed steel.

procedures contained in AISI S240, except that the resistance factor, ϕ, shall be taken as 1.0 and expected material properties shall be determined in accordance with Section 9.4.2. The expected-strength values of fasteners shall be calculated in accordance with Section 9.4.2.5, based on approved data. The expected strength of the gypsum board panel shear wall shall be permitted to be determined using principles of mechanics.

9.6.3.3.3 Acceptance Criteria for Gypsum Board Panel Shear Walls For linear procedures, *m*-factors for use with deformation-controlled actions shall be as specified in Table 9-2. Also, it shall be permitted to derive *m*-factors from experimental data. For nonlinear procedures, the nonlinear force–deformation relation shall be as specified in Table 9-3. Also, it shall be permitted to derive the relation based on experimental evidence or parameters derived in accordance with the generalized force–deformation relation described by Figure 9-1.

9.6.3.3.4 Connections of Gypsum Board Panel Shear Walls The connections between parts of the shear wall assembly and other elements of the seismic-force-resisting system shall be considered in accordance with Section 9.6.1.

9.6.3.4 Fiberboard Panels

9.6.3.4.1 Stiffness of Fiberboard Panels The deflection of fiberboard panel shear walls at yield (Δ_y) shall be determined as 2 times the deflection at 40% of the nominal wall strength determined in accordance with AISI S240. Properties used to compute shear wall deflection and stiffness shall be based on Section 9.4.2.

9.6.3.4.2 Strength of Fiberboard Panels The expected strength of fiberboard panel shear walls shall be taken as the mean maximum strength obtained experimentally. Expected strengths of fiberboard panel shear walls shall be permitted to be based on strengths determined using LRFD procedures contained in AISI S240, except that the resistance factor, ϕ, shall be taken as 1.0 and expected material properties shall be determined in accordance with Section 9.4.2. The expected-strength values of fasteners shall be calculated in accordance with Section 9.4.2.5, based on approved data. The expected strength of fiberboard panel shear wall shall be permitted to be determined using principles of mechanics.

9.6.3.4.3 Acceptance Criteria for Fiberboard Panels For linear procedures, *m*-factors for use with deformation-controlled actions shall be as specified in Table 9-2. Also, it shall be permitted to derive *m*-factors from experimental data. For nonlinear procedures, the nonlinear force–deformation relation shall be as specified in Table 9-3. Also, it shall be permitted to derive the relation based on experimental evidence or parameters derived in accordance with the generalized force–deformation relation described by Figure 9-1.

9.6.3.4.4 Connections of Steel Sheet Panels The connections between parts of the shear wall assembly and other elements of the seismic-force-resisting system shall be considered in accordance with Section 9.6.1.

9.6.3.5 Plaster on Metal Lath Shear Walls

9.6.3.5.1 Stiffness of Plaster on Metal Lath Shear Walls The deflection of plaster on metal lath shear walls shall be determined using Equation (9-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 9.4.2.

$$\Delta_y = v_y h / G_d + (h/b) d_a \tag{9-1}$$

where

v_y = Shear at yield in the direction under consideration (in pounds per foot),
h = Shear wall height (in feet),
G_d = Diaphragm shear stiffness = 12,000 lb/in. (82,737 KPa),
b = Shear wall width (in feet), and
d_a = Elongation of anchorage at end of wall determined by anchorage details and load magnitude (in.).

9.6.3.5.2 Strength of Plaster on Metal Lath Shear Walls The expected strength of plaster on metal lath shear walls shall be determined in accordance with Section 9.4.2. The default expected strength is 150 lb/ft (223 kg/m).

9.6.3.5.3 Acceptance Criteria for Plaster on Metal Lath Shear Walls For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 9-2. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 9-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 9-3.

9.6.3.5.4 Connections of Plaster on Metal Lath Shear Wall The presence of connections between parts of the shear wall assembly and other elements of the seismic-force-resisting system shall be verified. If connections are present, they need not be considered in the analysis conducted in accordance with Chapter 7. If connections are absent, they shall be provided in accordance with Section 9.6.1.

9.7 COLD-FORMED STEEL MOMENT-FRAME SYSTEMS

9.7.1 General Cold-formed steel moment-frame systems shall be categorized as primary or secondary components in accordance with Section 7.5.1. Connections between moment-frame systems and other components, including diaphragm ties, collectors, diaphragms, and foundations, shall be considered in accordance with Section 9.5.3 and shall be designed for forces calculated in accordance with Chapter 7. The expected strength, Q_{CE}, of cold-formed steel moment-frame systems shall be determined in accordance with Section 9.7.3. Figure 9-2 illustrates the moment-rotation relation for a cold-formed steel member in bending.

9.7.2 Types of Cold-Formed Steel Moment-Frame Systems

9.7.2.1 Existing Cold-Formed Steel Moment-Frame Systems Assemblages of cold-formed steel members are where the connections are specifically designed to transmit moment.

9.7.2.2 Enhanced Cold-Formed Steel Moment-Frame Systems Enhanced cold-formed steel moment frames shall include existing moment frames retrofitted in accordance with this standard or an approved method.

9.7.2.3 New Cold-Formed Steel Moment-Frame Systems Cold-formed steel moment-frame systems added to an existing seismic-force-resisting system are designated as new. Design of new moment-frame systems shall satisfy the acceptance criteria of this standard.

9.7.3 Stiffness, Strength, Acceptance Criteria, and Connection Design for Cold-Formed Steel Moment-Frame Systems

9.7.3.1 Generic Cold-Formed Steel Moment Connection Requirements for the generic cold-formed steel moment connection shall be in accordance with this section.

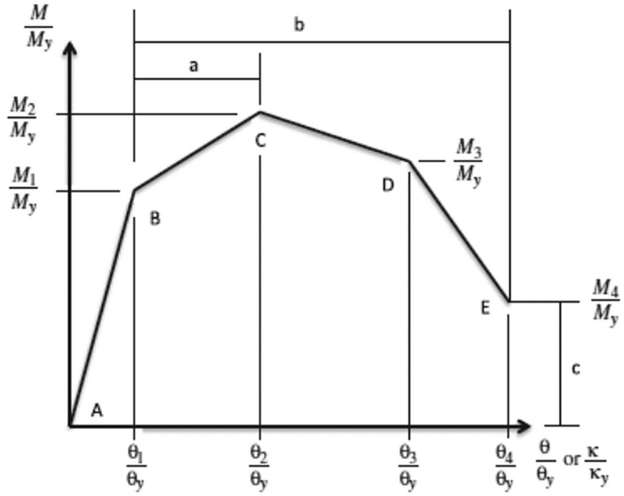


Figure 9-2. Moment-rotation relation for a cold-formed steel member in bending.

9.7.3.1.1 Strength of Generic Cold-Formed Steel Moment Connection The expected strength of a generic cold-formed steel moment-frame system shall be taken as the mean maximum strength obtained experimentally. Expected strengths of cold-formed steel members used in a generic moment-frame system shall be permitted to be based on strengths determined using the procedures of this section. Expected material properties shall be determined in accordance with Section 9.4.2.

For local buckling, the expected moment strengths required for determining acceptance criteria shall be determined in accordance with Equations (9-2) through (9-7):

$$\frac{M_1}{M_y} = \begin{cases} 1 & \text{if } \lambda_l < 0.650 \\ \left(\dfrac{0.650}{\lambda_l^2}\right)^2 & \text{if } \lambda_l \geq 0.650 \end{cases} \tag{9-2}$$

$$\frac{M_2}{M_y} = \begin{cases} 1 + \left(1 - \dfrac{1}{C_{yl}^2}\right)\dfrac{(M_p - M_y)}{M_y} & \text{if } \lambda_l < 0.776 \\ \left(1 - 0.15\left(\dfrac{1}{\lambda_l^2}\right)^{0.4}\right)\left(\dfrac{1}{\lambda_l^2}\right)^{0.4} & \text{if } \lambda_l \geq 0.776 \end{cases} \tag{9-3}$$

$$\frac{M_3}{M_y} = \frac{M_2}{M_y} \tag{9-4}$$

$$\frac{M_4}{M_y} = \frac{M_2}{M_y} - \frac{\Delta M}{M_y} \tag{9-5}$$

where

$$\lambda_l = \sqrt{\frac{M_y}{M_{crl}}} \tag{9-6}$$

$$\frac{\Delta M}{M_y} = 1 - 1/\left(\frac{\lambda_l}{0.776} + 1\right)^{1.1}\frac{M_2}{M_y} \leq 0.5\frac{M_2}{M_y} \tag{9-7}$$

M_y = Yield moment of the gross section;
M_{crl} = Elastic critical local buckling moment; and
M_p = Plastic moment of the gross section.

For sections subject to distortional buckling, the minimum strength between local and distortional buckling controls. Expected distortional buckling moment strengths required for establishing acceptance criteria shall be determined in accordance with Equations (9-8) through (9-13):

$$\frac{M_1}{M_y} = \begin{cases} 1 & \text{if } \lambda_d < 0.60 \\ \left(\dfrac{0.60}{\lambda_d^2}\right)^2 & \text{if } \lambda_d \geq 0.60 \end{cases} \tag{9-8}$$

$$\frac{M_2}{M_y} = \begin{cases} 1 + \left(1 - \dfrac{1}{C_{yd}^2}\right)\dfrac{(M_p - M_y)}{M_y} & \text{if } \lambda_d < 0.673 \\ \left(1 - 0.22\left(\dfrac{1}{\lambda_d^2}\right)^{0.5}\right)\left(\dfrac{1}{\lambda_d^2}\right)^{0.5} & \text{if } \lambda_d \geq 0.673 \end{cases} \tag{9-9}$$

$$\frac{M_3}{M_y} = \frac{M_2}{M_y} \tag{9-10}$$

$$\frac{M_4}{M_y} = \frac{M_2}{M_y} - \frac{\Delta M}{M_y} \tag{9-11}$$

where

$$\lambda_d = \sqrt{\frac{M_y}{M_{crd}}} \tag{9-12}$$

$$\frac{\Delta M}{M_y} = 1 - 1/\left(\frac{\lambda_d}{0.673} + 1\right)^{1.4}\frac{M_2}{M_y} \leq 0.5\frac{M_2}{M_y} \tag{9-13}$$

M_y = Yield moment of the gross section;
M_{crd} = Elastic critical distortional buckling moment; and
M_p = Plastic moment of the gross section.

9.7.3.1.2 Stiffness of Generic Cold-Formed Steel Moment Connection The stiffness of a generic cold-formed steel moment-frame system shall be obtained experimentally. Expected rotations of cold-formed steel members used in a generic cold-formed steel moment-frame system shall be permitted to be based on rotations determined using the procedures of this section. Expected material properties shall be determined in accordance with Section 9.4.2. Rotation of the moment connection shall consider deformations in the connection itself (e.g., local deformations in the cross sections at the connection location or fastener tilting and bearing) in addition to member rotations provided in this section.

For local buckling, the rotations required for establishing acceptance criteria shall be determined in accordance with Equations (9-14) through (9-19):

$$\frac{\theta_1}{\theta_y} = \frac{M_1}{M_y} \tag{9-14}$$

$$\frac{\theta_2}{\theta_y} = \frac{1}{\lambda_l} \geq \frac{M_2}{\theta_y k_e} \tag{9-15}$$

$$\frac{\theta_3}{\theta_y} = \frac{\theta_2}{\theta_y} + \frac{\Delta\theta}{\theta_y} \leq \frac{\theta_4}{\theta_y} \tag{9-16}$$

$$\frac{\theta_4}{\theta_y} = \left\{ \begin{array}{ll} 1.5\dfrac{1}{\lambda_l} & \text{if } \lambda_l > 1 \\ 1.5\left(\dfrac{1}{\lambda_l}\right)^{1/4\lambda_l} & \text{if } \lambda_l \leq 1 \end{array} \right\} \tag{9-17}$$

where

$$\lambda_l = \sqrt{\frac{M_y}{M_{crl}}} \tag{9-18}$$

$$\frac{\Delta\theta}{\theta_y} = \begin{cases} \left(\dfrac{0.776}{\lambda_l}\right) - 1 & \text{if } \lambda_l < 0.776 \\ 0 & \text{if } \lambda_l \geq 0.776 \end{cases} \tag{9-19}$$

θ_y = Rotation at which the gross section would reach the yield moment;
$k_e = \dfrac{M_y}{\theta_y} = \dfrac{M_1}{\theta_1}$ = Elastic rotational stiffness of the gross section;
M_y = Yield moment of the gross section; and
M_{crl} = Elastic critical local buckling moment.

For sections subject to distortional buckling, the minimum strength between local and distortional buckling controls. Expected distortional buckling rotations required for establishing acceptance criteria shall be determined in accordance with Equations (9-20) through (9-25):

$$\frac{\theta_1}{\theta_y} = \frac{M_1}{M_y} \tag{9-20}$$

$$\frac{\theta_2}{\theta_y} = \left(\frac{1}{\lambda_d}\right)^{1.4} \geq \frac{M_2}{\theta_y k_e} \tag{9-21}$$

$$\frac{\theta_3}{\theta_y} = \frac{\theta_2}{\theta_y} + \frac{\Delta\theta}{\theta_y} \leq \frac{\theta_4}{\theta_y} \tag{9-22}$$

$$\frac{\theta_4}{\theta_y} = \left\{ \begin{array}{ll} 1.5\left(\frac{1}{\lambda_d}\right)^{1.4} & \text{if } \lambda_d > 1 \\ 1.5\left(\frac{1}{\lambda_d}\right)^{1.4/\lambda_d} & \text{if } \lambda_d \leq 1 \end{array} \right\} \tag{9-23}$$

where

$$\lambda_d = \sqrt{\frac{M_y}{M_{crd}}} \tag{9-24}$$

$$\frac{\Delta\theta}{\theta_y} = \left\{ \begin{array}{ll} \left(\frac{0.673}{\lambda_d}\right) - 1 & \text{if } \lambda_d < 0.673 \\ 0 & \text{if } \lambda_d \geq 0.673 \end{array} \right. \tag{9-25}$$

$k_e = \frac{M_y}{\theta_y} = \frac{M_1}{\theta_1}$ = Elastic rotational stiffness of the gross section;

θ_y = Rotation at which the gross section would reach the yield moment;

M_y = Yield moment of the gross section; and

M_{crd} = Elastic critical distortional buckling moment.

9.7.3.1.3 Acceptance Criteria for Cold-Formed Steel Generic Moment Connection For linear procedures, *m*-factors for flexural members used with deformation-controlled actions shall be as specified in Table 9-2. Also, it shall be permitted to derive *m*-factors from experimental data. For nonlinear procedures, the nonlinear force–deformation relation for flexural members shall be as specified in Table 9-3. Also, it shall be permitted to derive the relation based on experimental evidence or parameters derived in accordance with the generalized force–deformation relation described by Figure 9-1.

9.7.3.1.4 Connections for Cold-Formed Steel Generic Moment Connection Connection performance in a cold-formed steel moment-frame system shall be established by testing.

9.7.3.2 Cold-Formed Steel Special Bolted Moment Frame Requirements for the cold-formed steel special bolted moment frame shall be in accordance with this section.

9.7.3.2.1 Stiffness of Cold-Formed Steel Special Bolted Moment Frame The deflection of cold-formed steel special bolted moment frames shall be determined in accordance with AISI S400. Deflection of the moment-frame system shall consider deformations in the beam-to-column connection, member beam and column rotations, baseplate, and anchorage deformations. Properties used to compute deflection and stiffness shall be based on Section 9.4.2.

9.7.3.2.2 Strength of Cold-Formed Steel Special Bolted Moment Frame The expected strength of cold-formed steel special bolted moment frames shall be taken as the mean maximum strength obtained experimentally. Expected strengths of cold-formed steel special bolted moment frames shall be permitted to be based on strengths determined using LRFD procedures contained in AISI S400, except that the resistance factor, ϕ, shall be taken as 1.0 and expected material properties shall be determined in accordance with Section 9.4.2.

9.7.3.2.3 Acceptance Criteria for Cold-Formed Steel Special Bolted Moment Frame For linear procedures, *m*-factors shall be derived from experimental data. For nonlinear procedures, the nonlinear force–deformation relation shall be based on experimental evidence or parameters derived in accordance with the generalized force–deformation relation, described by Figure 9-1.

9.7.3.2.4 Connections for Cold-Formed Steel Special Bolted Moment Frame The connections between parts of the cold-formed steel special bolted moment frames assembly and other elements of the seismic-force-resisting system shall be considered in accordance with Section 9.8.1.

9.8 COLD-FORMED STEEL LIGHT-FRAME CONSTRUCTION, STRAP-BRACED WALL SYSTEMS

9.8.1 General Cold-formed steel light-frame constructions with strap-braced walls shall be categorized as primary or secondary components in accordance with Section 7.5.1. Cold-formed steel framed walls with diagonal flat strap bracing shall be permitted to have strap on one or both sides of the wall. For overturning calculations on shear wall elements, stability shall be evaluated in accordance with Section 7.2.9. Net tension caused by overturning shall be resisted by uplift connections. Connections between strap-braced walls and other components, including diaphragm ties, collectors, diaphragms, and foundations, shall be considered in accordance with Section 9.5.3 and shall be designed for forces calculated in accordance with Chapter 7. Components supporting discontinuous strap-braced walls shall be considered in accordance with Section 9.5.4. Chord studs, anchorage, and collectors for the strap-braced wall shall be designed for forces calculated in accordance with Chapter 7 including superposed gravity, Q_G, and earthquake, Q_E, demands. Q_E shall not be less than the demand developed because of the expected strength of the strap-braced walls, Q_{CE}. If the capacity of the chord studs, anchorage, or collectors that are part of the strap-braced wall are less than the demand developed because of the expected strength of the strap-braced wall, then the strap-braced wall shall be considered as a force-controlled element.

The expected strength, Q_{CE}, of cold-formed steel strap-braced walls shall be determined in accordance with Section 9.8.3.

9.8.2 Types of Cold-Formed Steel Light-Frame Construction with Strap-Braced Walls

9.8.2.1 Existing Cold-Formed Steel Light-Frame Construction with Strap-Braced Walls Walls shall be framed from cold-formed steel members (stud and track) and shall have flat steel strap placed diagonally across the wall and connected to the cold-formed steel members.

9.8.2.2 Cold-Formed Steel Light-Frame Construction with Enhanced Strap-Braced Walls Cold-formed steel light-frame construction with enhanced strap-braced walls shall include existing walls retrofitted in accordance with this standard or an approved method.

9.8.2.3 Cold-Formed Steel Light-Frame Construction with New Strap-Braced Walls Cold-formed steel light-frame construction with new strap-braced walls added to an existing seismic-force-resisting system shall be designated as new. Design of new strap-braced walls shall satisfy the acceptance criteria of this standard. Details of construction for new strap-braced walls, including track anchorage details, tie-down anchor details, fastening details for the strap, and dimensional limitations for studs and tracks, shall be in accordance with the requirements of AISI S400 or the approved building code.

9.8.3 Stiffness, Strength, Acceptance Criteria, and Connection Design for Cold-Formed Steel Light-Frame Construction with Strap-Braced Walls

9.8.3.1 Stiffness The deflection of strap-braced walls at yield (Δ_y) shall be determined as the deflection at 80% of the nominal wall strength determined in accordance with AISI S400. Deflection of the wall shall consider deformations in the strap, in the wall members, at connections between the strap and the wall, and at any tie-downs. Properties used to compute wall deflection and stiffness shall be based on Section 9.4.2.

9.8.3.2 Strength The expected strength of strap-braced walls shall be taken as the mean maximum strength obtained experimentally. Expected strengths of strap-braced walls shall be permitted to be based on strengths determined using LRFD procedures contained in AISI S400, except that the resistance factor, ϕ, shall be taken as 1.0 and expected material properties shall be determined in accordance with Section 9.4.2. The expected-strength values of fasteners shall be calculated in accordance with Section 9.4.2.5, based on approved data. The expected strength of strap-braced walls shall be permitted to be determined using principles of mechanics. For strap-braced walls sheathed with one or two plies of gypsum wallboard, the strength shall be determined as the greater of the strength of the gypsum wallboard sheathing or the strap brace.

9.8.3.3 Acceptance Criteria For linear procedures, m-factors for use with deformation-controlled actions shall be as specified in Table 9-2. For nonlinear procedures, the nonlinear force–deformation relation shall be as specified in Table 9-3. Also, it shall be permitted to derive the relation based on experimental evidence or parameters derived in accordance with the generalized force–deformation relation described by Figure 9-1.

9.8.3.4 Connections The connections between parts of the strap-braced wall assembly and other elements of the seismic-force-resisting system shall be considered in accordance with Section 9.8.1.

9.9 COLD-FORMED STEEL DIAPHRAGMS

Seismic evaluation and retrofit of cold-formed steel diaphragms in a building shall be in accordance with the provisions of AISC 342, Chapter G.

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CHAPTER 10
CONCRETE

10.1 SCOPE

This chapter sets forth requirements for the seismic evaluation and retrofit of concrete components of the seismic-force-resisting system of an existing building. The requirements of this chapter apply to existing concrete components of a building system, retrofitted concrete components of a building system, and new concrete components added to an existing building system. Provisions of this chapter do not apply to concrete-encased steel composite components.

10.2 REFERENCE STANDARD

Seismic evaluation and retrofit of concrete components of the seismic-force-resisting system of an existing building shall be in accordance with the provisions of ACI 369.1 as modified by Section 10.3.

10.3 MODIFICATIONS TO THE REFERENCE STANDARD

Modify ACI 369.1 Chapters 3, 7, 12, and 13 and Notation by replacing the referenced sections in ACI 369.1 with the provisions in this section.

10.3.1 General Assumptions and Requirements. ***Replace Section 3.1 of ACI 369.1 with the italicized text as follows***.

ACI 369.1 CHAPTER 3

3.1 MODELING AND DESIGN

3.1.1 General *New components connected to the existing structure shall comply with ACI 318, except as otherwise indicated in this code.*

Table 3.1.2.1. Effective Stiffness Values for Linear Analysis.

Component	***Flexural rigidity***	***Shear rigidity***	***Axial rigidity***
Beams – nonprestressed[a]	$0.3E_{cE}I_g$	$0.4E_{cE}A_w$	$1.0E_{cE}A_g$
Beams – prestressed[a]	$E_{cE}I_g$	$0.4E_{cE}A_w$	$1.0E_{cE}A_g$
Columns with compression caused by design gravity loads $\geq 0.5A_g f'_{cE}$[b]	$0.7E_{cE}I_g$	$0.4E_{cE}A_w$	$1.0E_{cE}A_g$
Columns with compression caused by design gravity loads $\leq 0.1A_g f'_{cE}$ *or with tension*[b]	$0.3E_{cE}I_g$	$0.4E_{cE}A_w$	$1.0E_{cE}A_g$ *(compression)* $1.0E_sA_s$ *(tension)*
Beam-column joints		*Refer to 4.2.2.1*	$1.0E_{cE}A_g$
Flat slabs – nonprestressed	*Refer to 4.4.2*	$0.4E_{cE}A_g$	—
Flat slabs – prestressed	*Refer to 4.4.2*	$0.4E_{cE}A_g$	—
Diaphragms (in-plane) – nonprestressed[e]	$0.25E_{cE}I_g$	$0.25E_{cE}A_g$	$0.25E_{cE}A_g$
Diaphragms (in-plane) – prestressed[e]	$0.5E_{cE}I_g$	$0.4E_{cE}A_g$	$0.5E_{cE}A_g$
Walls – uncracked with compression caused by design gravity loads $\geq 0.3A_g f'_{cE}$[b,c]	$1.0E_{cE}I_g$	$0.3E_{cE}A_w$	$1.0E_{cE}A_g$
Walls – uncracked with compression caused by design gravity loads $\leq 0.05A_g f'_{cE}$ *or with tension*[b,c]	$0.5E_{cE}I_g$	$0.3E_{cE}A_w$	$1.0E_{cE}A_g$
Walls – cracked[c,d]	$0.25E_{cE}I_g$	$0.15E_{cE}A_w$	$1.0E_{cE}A_g$ *(compression)* $1.0E_sA_s$ *(tension)*
Coupling beams with longitudinal or diagonal reinforcement	$0.05\,(l_n/h)\,E_{cE}I_g \leq 0.20E_{cE}I_g$	$0.2E_{cE}A_w$	$1.0E_{cE}A_g$

[a] *For T-beams, I_g can be taken as twice the value of I_g of the web alone. Otherwise, I_g shall be based on the effective width as defined in Section 3.1.3.*

[b] *For columns and walls with axial compression falling between the limits provided, flexural rigidity shall be determined by linear interpolation. If interpolation is not performed, the more conservative effective stiffnesses shall be used. An imposed axial load N_{UG} is permitted to be used for stiffness evaluations.*

[c] *Walls are permitted to be considered cracked due to earthquake demands in flexural actions where flexural demands exceed M_{crE} and/or in shear actions where shear force demands exceed the cracking shear strength defined in Section 7.2.2. It shall be permitted to assume all walls to be cracked.*

[d] *Alternative stiffness values dependent on axial load and boundary longitudinal reinforcement ratio shall be permitted in accordance with Section 7.3.*

[e] *In-plane diaphragm effective stiffness values apply where diaphragm flexibility is considered in accordance with Section 10.2.2.*

Original and retrofitted components of an existing building are not expected to satisfy provisions of ACI 318 but shall be assessed using the provisions of this standard. Brittle or low-ductility failure modes shall be identified as a part of the seismic evaluation.

Evaluation of demands and capacities of reinforced concrete components shall include consideration of locations along the length where seismic force and gravity loads produce maximum effects; where changes in cross section or reinforcement result in reduced strength; and where abrupt changes in cross section or reinforcement, including splices, can produce stress concentrations that result in premature failure.

3.1.2 Stiffness *Component stiffnesses shall be calculated considering shear, flexure, axial behavior, and reinforcement slip deformations. Stress state of the component, cracking extent caused by volumetric changes from temperature and shrinkage, deformation levels under gravity loads, and seismic forces shall be considered. Gravity load effects considered for effective stiffnesses of components shall be determined using ASCE 41, Equation (7-3).*

3.1.2.1 Linear Procedures *Where design actions are determined using the linear procedures of ASCE 41, Chapter 7, effective stiffness values in Table 3.1.2.1 shall be permitted.*

3.1.2.2 Nonlinear Procedures

3.1.2.2.1. Where design actions are determined using the nonlinear procedures of ASCE 41, Chapter 7, component load-deformation response shall be modeled using nonlinear load-deformation relations for each deformation-controlled action. These relations shall include the effective stiffness, expected strength, deformation capacity, and hysteretic response under force or deformation reversals.

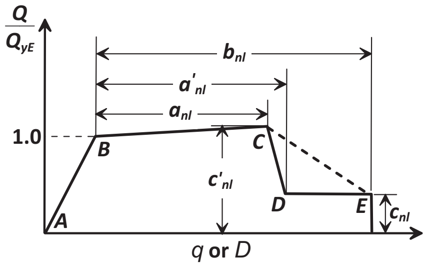
3.1.2.2.2. Linear relations shall be permitted if the lower bound strength of the expected yield strength of the component is not exceeded in each ground motion analysis.

3.1.2.2.3. The nonlinear load-deformation relation shall be based on experimental evidence defined by quantities specified in Chapters 4 through 12. For the nonlinear static procedure (NSP), the generalized load-deformation relation shown in Figure 3.1.2.2.3 shall be used. For the nonlinear dynamic procedure (NDP), the generalized load-deformation relations shown in Figure 3.1.2.2.3 shall be combined with the general hysteresis shapes specified in 3.1.2.2.8.

Alternatively, it shall be permitted to base the nonlinear load-deformation relation derived from laboratory test data for components or subassemblages (1) subjected to gravity load effects and lateral load or deformation histories similar to those expected for building components and (2) exhibiting response modes similar to those expected for building components. Where experimental data sets are used to define nonlinear action-deformation relations, simulated analytical deformation demands shall not exceed the maximum tested deformation imposed on the component or subassemblages used for model calibration.

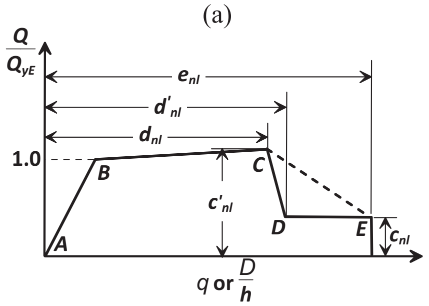
3.1.2.2.4. The slope from Point A to B in Figure 3.1.2.2.3 shall be determined according to Table 3.1.2.2.4 or determined using a secant-to-yield stiffness based on experimental data. Point B shall have an ordinate equal to the expected yield strength of the component.

3.1.2.2.5. Point C in Figure 3.1.2.2.3 shall have an ordinate equal to the expected strength of the component including the effect of strain-hardening, and an abscissa equal to the generalized deformation at which significant strength degradation begins.



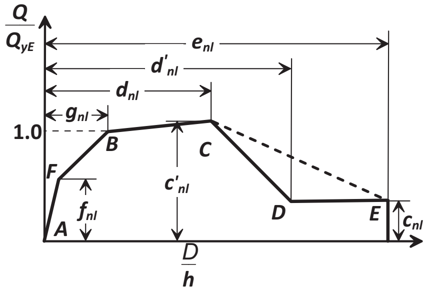
Nonlinear deformations

(a)



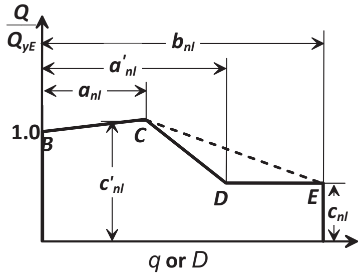
Total deformations

(b)



Bi-linear pre-yield response – total deformations

(c)



Rigid elastic response – nonlinear deformations

(d)

Figure 3.1.2.2.3. Generalized force-deformation relation for concrete elements or components.

3.1.2.2.6. Point D in Figure 3.1.2.2.3 shall have an ordinate equal to the residual strength of the component and an abscissa equal to the generalized deformation at which the residual strength is reached.

Point E in Figure 3.1.2.2.3 shall have an ordinate equal to the residual strength of the component. For actions in which failure results in structural collapse, the abscissa shall be equal to the generalized deformation at which the component loses the ability to sustain gravity loads. For other actions the abscissa shall be equal to the generalized deformation at which the component loses its residual strength. If Point D is unknown, connecting Points C and E by a straight line shall be permitted, as illustrated in Figure 3.1.2.2.3 with a dashed line.

Table 3.1.2.2.4. Effective Stiffness Values for Nonlinear Analysis.[a]

Component	Flexural rigidity	Shear rigidity	Axial rigidity
Beams – nonprestressed[b]	$0.2E_{cE}I_g$	$0.4E_{cE}A_w$	$1.0E_{cE}A_g$
Beams – prestressed[b]	$1.0E_{cE}I_g$	$0.4E_{cE}A_w$	$1.0E_{cE}A_g$
Columns with compression caused by design gravity loads $\geq 0.5A_g f'_{cE}$[c]	$0.7E_{cE}I_g$	$0.4E_{cE}A_w$	$1.0E_{cE}A_g$
Columns with compression caused by design gravity loads $\leq 0.1A_g f'_{cE}$ or with tension[c]	$0.2E_{cE}I_g$	$0.4E_{cE}A_w$	$1.0E_{cE}A_g$ (compression) $1.0E_sA_s$ (tension)
Diaphragms (in-plane) – nonprestressed[d]	$0.25E_{cE}I_g$	$0.25E_{cE}A_g$	$0.25E_{cE}A_g$
Diaphragms (in-plane) – prestressed[d]	$0.5E_{cE}I_g$	$0.4E_{cE}A_g$	$0.5E_{cE}A_g$
Walls-uncracked with compression caused by design gravity loads $\geq 0.3A_g f'_{cE}$[c,e]	$1.0E_{cE}I_g$	$0.3E_{cE}A_w$	$1.0E_{cE}A_g$
Walls-uncracked with compression caused by design gravity loads $\leq 0.05A_g f'_{cE}$ or with tension[c,e]	$0.5E_{cE}I_g$	$0.3E_{cE}A_w$	$1.0E_{cE}A_g$
Walls–cracked[e,f]	$0.25E_{cE}I_g$	[g]	$1.0E_{cE}A_g$ (compression) $1.0E_sA_s$ (tension)
Coupling beams with longitudinal or diagonal reinforcement	$0.05\ (l_n/h)\ E_{cE}I_g \leq 0.20E_{cE}I_g$	$0.2E_{cE}A_w$	$1.0E_{cE}A_g$

[a] Tabulated values for axial, flexural, and shear shall be applied jointly in defining effective stiffness of an element, unless alternative combinations are justified. For other elements not covered in this table, it shall be permitted to use values in Table 3.1.2.1.

[b] For T-beams, I_g can be taken as twice the value of I_g of the web alone. Otherwise, I_g shall be based on the effective width as defined in Section 3.1.3.

[c] For columns and walls with axial compression falling between the limits provided, flexural rigidity shall be determined by linear interpolation. If interpolation is not performed, the more conservative effective stiffnesses shall be used. An imposed axial load N_{UG} is permitted to be used for stiffness evaluations.

[d] In-plane diaphragm effective stiffness values apply where diaphragm flexibility is considered in accordance with Section 10.2.2.

[e] Walls are permitted to be considered cracked due to earthquake demands in flexural actions where flexural demands exceed M_{crE} and/or in shear actions where shear force demands exceed the cracking shear strength defined in Section 7.2.2. It shall be permitted to assume all walls to be cracked.

[f] Alternative stiffness values dependent on axial load and boundary longitudinal reinforcement ratio shall be permitted in accordance with Section 7.3.

[g] When a single value is used to represent shear rigidity and cracking is expected in shear, 0.15 $E_{cE}A_w$ shall be permitted to be used to represent shear rigidity. When a trilinear relationship is used to represent nonlinear shear actions as shown in Figure 3.1.2.2.3c, the initial shear rigidity of the load-deformation relationship from the origin to Point F in Figure 3.1.2.2.3c shall be based on the table value for "walls-uncracked," and the remaining segments of the load-deformation relationship shall be in accordance with Sections 7.4.1.1.2 and 7.4.1.1.3 for shear- and shear-friction-controlled walls, respectively.

3.1.2.2.7. General deformation values for the points identified in Figure 3.1.2.2.3 shall be as specified in Sections 4.2.2.2 and 4.2.2.3 for beams, columns, and joints; Sections 4.3.2.2 and 4.3.2.3 for post-tensioned beams; Sections 4.4.2.2 and 4.4.2.3 for slab-column connections; and Section 7.4.1 for structural walls, and wall segments; and Section 7.6.1 for coupling beams. Other load-deformation relations shall be permitted if justified by experimental evidence.

For analyses using the Nonlinear Dynamic Procedure (NDP), the general hysteretic shape for all components shall be defined in 3.1.2.2.8.

3.1.2.2.8. The following general hysteresis shapes shall be used for analyses with the Nonlinear Dynamic Procedure (NDP):

Type A: Hysteresis shape representing the behavior of the components with low pinching,

Type B: Hysteresis shape representing the behavior of the components with moderate pinching, and

Type C: Hysteresis shape representing the behavior of the components with significant pinching,

3.1.2.2.9. If structural components are modeled using lumped plasticity or distributed plasticity models, the force-deformation or stress-strain relationships shall be adjusted to achieve component expected behavior based on the assumed plastic hinge length or integration length used in the analysis.

3.1.2.2.10. Nonlinear fiber-type section models shall be shown to result in the calculation of load-deformation response that is in substantial agreement with the results of physical tests of reinforced concrete components or subassemblages exhibiting response mechanisms consistent with those expected in the components or subassemblages being modeled.

Where simulation results generated using fiber-type section models cannot be validated using experimental data, the stress-strain relationships or meshing of the fiber elements that compose the fiber-type section models shall be modified such that the predicted response is in substantial agreement with the generalized load-deformation values in Section 3.1.2.2.7.

3.1.2.2.11. For components under combined axial load and bidirectional lateral load, the effect of the combined loading on the strength and deformation capacity of the component shall be considered.

3.1.3 Flanged Construction *In beams consisting of a web and flange that act integrally, the combined stiffness and strength for flexural and axial loading shall be calculated considering a width of effective flange on each side of the web equal to the smallest of*

- *Provided flange width,*
- *Eight times the flange thickness,*
- *Half the distance to the next web, and*
- *One-fifth of the beam span length.*

Where the flange is in compression, the concrete and reinforcement within the effective width shall be considered effective in resisting flexure and axial load. Where the flange is in tension, longitudinal reinforcement within the effective width of the flange and developed beyond the critical section shall be considered fully effective for resisting flexural and axial loads. The portion of the flange extending beyond the width of the web shall be assumed ineffective in resisting shear.

In structural walls, effective flange width should be computed using Chapter 18 of ACI 318.

10.3.2 Concrete Structural Walls. ***Replace Section 7.1 through 7.7 of ACI 369.1 with the italicized text as follows*.**

ACI 369.1 CHAPTER 7

7.1 TYPES OF CONCRETE STRUCTURAL WALLS AND ASSOCIATED COMPONENTS

The provisions of Chapter 7 shall apply to all concrete structural walls and associated components in all types of structural systems that incorporate concrete structural walls. These types include isolated structural walls, structural walls used in wall-frame systems, coupled structural walls, and discontinuous structural walls. Structural walls shall be permitted to be considered as solid walls if they have openings that do not significantly influence the strength or inelastic behavior of the wall. Perforated structural walls shall be defined as walls that have a regular pattern of openings in both horizontal and vertical directions that create a series of wall pier (vertical wall segment) and deep beam components (horizontal wall segment).

For the purposes of evaluating Modeling Parameters and Acceptance Criteria for shear-controlled walls and wall piers, and evaluating shear cracking strength in Section 7.2.2, walls shall be classified as having flanged or rectangular cross-sections. A wall or wall segment shall be considered to have a flanged cross-section when the gross moment of inertia of the wall cross-section including any flanges bounded by the effective flange width defined in Section 3.1.3 (I_{g_flange}), is at least 1.5 times the gross moment of inertia of the rectangular portion of the section bounded by wall length and web thickness (I_{g_rect}).

The provisions of this chapter shall also apply to coupling beams, which shall be exempted from the provisions for beams covered in Chapter 4.

7.1.1 Monolithic Reinforced Concrete Structural Walls and Wall Segments *Monolithic reinforced concrete structural walls shall consist of vertical cast-in-place elements, either uncoupled or coupled, in open or closed shapes. These walls shall have relatively continuous cross sections and reinforcement and shall provide both vertical and lateral force resistance, in contrast with infilled walls defined in Section 6.1.3.*

7.1.2 Reinforced Concrete Columns Supporting Discontinuous Structural Walls *Reinforced concrete columns supporting discontinuous structural walls shall be analyzed in accordance with the requirements of Section 4.2.*

7.1.3 Reinforced Concrete Coupling Beams *Reinforced concrete coupling beams used to link two wall piers together shall be evaluated and retrofitted to comply with the requirements of Sections 7.2 and 7.5.*

7.2 STRENGTH OF REINFORCED CONCRETE STRUCTURAL WALLS, WALL SEGMENTS, AND COUPLING BEAMS

Component strengths shall be computed according to the general requirements of Section 3.2 with the additional requirements of this section. Strength shall be determined considering the potential for a controlling mechanism at any point in flexure, shear, or shear-friction sliding, and effects from inadequate reinforcement development, under combined gravity and lateral load. The controlling actions of components shall be classified as identified in Sections 7.3.2 and 7.4.

Components evaluated in accordance with this section with either a vertical or horizontal reinforcement ratio, ρl *or* ρt, *less than* 0.001 *shall be considered plain concrete, and section strengths shall be calculated in accordance with Chapter 14 of ACI 318.*

For all strength calculations, material strengths shall be determined using lower bound or expected material properties as applicable to force-controlled or deformation-controlled actions, respectively. For the purpose of calculating flexural, shear, and shear-friction strengths of components in this section, expected material properties shall be used, as these actions are classified as deformation-controlled unless noted otherwise. The lower of expected shear strength, $V_{CydWallE}$, and expected shear friction strength, $V_{CyfWallE}$, shall be taken as the controlling expected component strength for comparison against shear actions, V_{CE}, in ASCE 41, Equation (7-39).

7.2.1 Flexural Strength *The flexural yield strength of structural walls or wall segments, M_{Cy}, as represented by Point B in Figure 3.1.2.2.3b, shall be determined using the fundamental principles given in Chapter 22 of ACI 318, using the expected yield strength, f_{yE}, based on experimental testing as prescribed in Section 2.2 or by substituting a 1.1 factor for reinforcing steel yield strength in place of Table 2.2.1.2, and a strength reduction factor equal to 1.0. For calculation of flexural strength, the effective compression and tension flange widths defined in Section 3.1.3 shall be used. When calculating the maximum inelastic flexural strength of the wall, M_{CultE}, as represented by Point C in Figure 3.1.2.2.3b, the effects from strain hardening shall be accounted for by using 1.15 times the expected yield strength of the reinforcement, f_{yE}, by taking M_{CultE} equal to 1.15 M_{CyDE}, or by using experimental material stress-strain data and analysis.*

Splice lengths for primary longitudinal reinforcement shall be evaluated using the procedures given in Section 3.5. Reduced flexural strengths shall be evaluated at locations where splices govern the usable stress in the reinforcement.

7.2.2 Shear Strength *The cracking shear strength of a structural wall or wall segment, $V_{CcrWallE}$, corresponding to Point F in Figure 3.1.2.2.3c shall be evaluated using Equation (7.2.2a) or (7.2.2b). Linear interpolation between Equations (7.2.2a) and (7.2.2b) based on I_{g_flange}/I_{g_rect} shall be permitted for the cracking shear strength of walls and wall segments with $1.0 < I_{g\_flange}/I_{g\_rect} < 1.5$. Alternatively, it shall be permitted to use Equation (7.2.2a) for walls with any cross-section shape.*

$$V_{CcrWallE} = \alpha_c \lambda \sqrt{f'_{cE}} A_{cv} \quad for\ rectangular\ sections \qquad (7.2.2a)$$

$$V_{CcrWallE} = 2\alpha_c \lambda \sqrt{f'_{cE}} A_{cv} \quad for\ flanged\ sections \qquad (7.2.2b)$$

where

$\alpha_c = 3$ *for* $h_w/l_w \leq 1.5$, *and*
$\alpha_c = 2$ *for* $h_w/l_w \geq 2.0$.
α_c *varies linearly between 3 and 2 for* $1.5 < h_w/l_w < 2.0$.

The expected shear strength of a structural wall or wall segment, $V_{CydWallE}$, shall be determined using Equation (7.2.2c).

$$V_{CydWallE} = \left(2.0 - \frac{V_{CWall318E}}{\omega_v V_{MCultE}}\right) V_{CWall318E} \quad \le 1.8 V_{CWall318E} \quad \ge 0.8 V_{CWall318E} \qquad (7.2.2c)$$

where

$$V_{CWall318E} = (\alpha_c \lambda \sqrt{f'_{cE}} + \rho_t f_{ytE}) A_{cv} \qquad (7.2.2d)$$

And the shear amplification factor ω_v need not be applied if V_{MCultE} is obtained from nonlinear analyses procedures. See Section 7.3.2 for ω_v and V_{MCultE}.

Alternatively, it shall be permitted to evaluate $V_{CydWallE}$ using Equation (7.2.2e).

$$V_{CydWallE} = 0.8\, V_{CWall318E} \qquad (7.2.2e)$$

$V_{CydWallE}$ shall be multiplied by 0.85 where ρ_t is less than 0.0015.

The ultimate shear strength, $V_{CultdWallE}$, shall be taken as $V_{CydWallE}$ multiplied by c'_{nl} defined in Section 7.4.1.1.

7.2.3 Shear-Friction Strength *The shear-friction yield strength of a structural wall or wall segment, corresponding to Point B in Figure 3.1.2.2.3d, considering shear transfer across any given plane along wall height shall be determined using Equation (7.2.3) or (7.2.4).*

$$V_{CyfWallE} = \left(2.5 - 2.15 \frac{V_{CyfWallSE}}{\omega_v V_{MCyDE}}\right) V_{CyfWallSE} \quad \le 1.8 V_{CyWallSE} \quad \ge 0.8 V_{CyWallSE} \qquad (7.2.3)$$

$$V_{CyfWallSE} = \mu (A_{vf} f_{yfE} + N_{UG}) \le 0.2 f'_{cE} A_g \qquad (7.2.4)$$

where

μ = 0.7 *for concrete cast monolithically or placed against hardened concrete that is intentionally roughened to a full amplitude of approximately 1/4 in.*

μ = 0.6 *for concrete placed against hardened concrete that is not intentionally roughened.*

f_{yfE} in Equation (7.2.4) shall not be taken greater than 75,000 psi and shall be computed considering reductions with respect to anchorage in accordance with Section 3.5. For flanged wall sections, the reinforcing steel crossing the interface, including the reinforcement within the effective flange width in accordance with Section 3.1.3, shall be included in A_{vf}.

7.3 LINEAR STATIC AND DYNAMIC PROCEDURES FOR REINFORCED CONCRETE STRUCTURAL WALLS AND WALL SEGMENTS

7.3.1 Modeling *The analytical model for a structural wall or wall segment shall represent component stiffness considering axial, flexural, and shear actions. Potential controlling mechanisms in flexure, shear, and reinforcement development at any point in the wall shall be considered. Interaction with other structural and nonstructural components shall be included.*

The effective stiffness of structural walls and wall segments shall satisfy Section 3.1.2. The effective stiffness values given in Table 3.1.2.1 or Table 7.3.1 shall be permitted unless alternative stiffness values are determined by a more detailed analysis. For nonrectangular wall cross-section shapes, such as box-, T-, L-, I-, H-, and C-shaped sections, the effective tension or compression flange widths shall be as specified in 3.1.3 for stiffness evaluations. The calculated strength to be used in assessment shall be in accordance with Section 7.2.

Table 7.3.1. Alternative Effective Stiffness Values for Cracked Structural Walls.

$\frac{N_{UG}}{A_g f'_{cE}}$	ρ_{lb}[a,b]	*Flexural Rigidity*[c]
≤0.05	≤0.01	$0.20 E_{cE} I_g$
	≥0.03	$0.30 E_{cE} I_g$
≥0.50	≤0.01	$0.90 E_{cE} I_g$
	≥0.03	$1.00 E_{cE} I_g$

[a] *ρ_{lb} shall be taken as the longitudinal reinforcement ratio in the tension boundary element in the case of asymmetrical walls about the centerline of the cross-section.*

[b] *It shall be permitted to use a default ρ_{lb} value of 0.02.*

[c] *For walls with axial compression and reinforcement ratios falling between the limits provided, flexural rigidity shall be determined by linear interpolation. If interpolation is not performed, the more conservative effective stiffnesses shall be used.*

Structural walls and wall segments shall be permitted to be modeled as equivalent beam–column elements in which both flexural and shear deformations are simulated but flexural strength and deformation capacity are decoupled from shear response. The flexural strength of the equivalent beam–column elements shall include the interaction of axial load and bending in accordance with 3.3 and shall be calculated based on expected material properties. The rigid connection zone at beam connections to this equivalent beam–column element shall represent the distance from the wall centroid to the edge of the wall at the location where the beam is connected to the wall.

Joints between structural walls and frame elements shall be modeled as stiff components or rigid components, as appropriate. The in-plane and out-of-plane diaphragm behavior of concrete slabs that interconnect structural walls and frame columns shall be represented in the model.

Modification factors used to relate maximum inelastic displacements to linear elastic displacements, C_1 and C_2, shall be calculated in accordance with ASCE 41, Equations (7-22) and (7-24). Table 7-3 in ASCE 41 shall not be used for calculating demands on structural walls and wall segments.

7.3.2 Acceptance Criteria *Design actions (flexure, shear, axial, or force transfer at reinforcing bar anchorages and splices) on components shall be determined as prescribed in Chapter 7 of*

Table 7.3.2a. Structural Wall and Wall Segment Controlling Behavior Classification.

Criteria		*Expected Controlling Behavior*
$V_{CWall318E} \le V_{CyfWallSE}$	$< (w_v V_{MCultE})$	*Shear*
$V_{CyfWallSE} < V_{CWall318E}$	$< (w_v V_{MCultE})$	*Shear-friction*
Otherwise		*Flexure*

Table 7.3.2b. Numerical Acceptance Criteria for Linear Procedures: Reinforced Concrete Structural Walls.

		m-factors[a,b]		
		Performance level		
Controlling Behavior	**Component Type**	**IO**	**LS**	**CP**
Flexure	*Primary*	$\frac{\theta_{yE}+0.1(d_{nl}-\theta_{yE})}{\theta_{yE}}$	$\frac{9}{16}\left(\frac{e_{nl}}{\theta_{yE}}\right)$	$\frac{5}{8}\left(\frac{e_{nl}}{\theta_{yE}}\right)$
	Secondary		$\frac{3}{4}\left(\frac{e_{nl}}{\theta_{yE}}\right)$	$\frac{4}{5}\left(\frac{e_{nl}}{\theta_{yE}}\right)$
Shear	*Primary*	$\frac{g_{nl}+0.1(d_{nl}-g_{nl})}{g_{nl}}$	$\frac{1}{2}\left(\frac{e_{nl}}{g_{nl}}\right)$	$\frac{5}{8}\left(\frac{e_{nl}}{g_{nl}}\right)$
	Secondary		$\frac{3}{5}\left(\frac{e_{nl}}{g_{nl}}\right)$	$\frac{4}{5}\left(\frac{e_{nl}}{g_{nl}}\right)$
Shear-Friction	*Primary*	1.2	$\frac{1}{2}\left(\frac{\frac{a'_{nl}}{h_s}+g_{nl}}{g_{nl}}\right)$	$\frac{5}{8}\left(\frac{\frac{a'_{nl}}{h_s}+g_{nl}}{g_{nl}}\right)$
	Secondary	1.2	$\frac{3}{5}\left(\frac{\frac{a'_{nl}}{h_s}+g_{nl}}{g_{nl}}\right)$	$\frac{4}{5}\left(\frac{\frac{a'_{nl}}{h_s}+g_{nl}}{g_{nl}}\right)$

[a] θ_{yE} shall be calculated per Equation (7.3.2f). It shall be permitted to take θ_{yE} as 0.003 rad in this table, in lieu of using Equation (7.3.2f). The m-factors for LS or CP shall not be smaller than those for IO or LS, respectively. m-factors shall not be smaller than 1.0.

[b] Acceptance criteria for primary members shall not be taken larger that those for secondary members.

[c] g_{nl} shall be taken from Table 7.4.1.1.2.

$$\theta y_E = \frac{M_{CyGE}}{3E_{cE}I_{eff}}\frac{M_{UD}}{V_{UD}} \geq 0.002 \tag{7.3.2f}$$

where $E_{cE}I_{eff}$ is the effective flexural stiffness computed per Section 7.3.1.
Note: IO = Immediate Occupancy, LS = Life Safety, CP = Collapse Prevention.

Table 7.3.2c. Alternative Numerical Acceptance Criteria for Linear Procedures: Reinforced Concrete Structural Walls and Associated Components Controlled by Shear.

Conditions		m-factors*				
		Performance Level				
			Primary		Secondary	
$\frac{N_{UD}}{A_g f'_{cE}}$	$\frac{V_{CWall318E}}{\omega_v V_{MCultDE}}$	**IO**	**LS**	**CP**	**LS**	**CP**
≤0.075	≥1.0	1.3	4.2	4.8	5.6	6.4
	≤0.5	1.1	2.8	3.2	3.8	4.3
≥0.150	≥1.0	1.3	2.1	2.4	2.8	3.2
	≤0.5	1.1	1.4	1.6	1.9	2.1

**Linear interpolation between values listed in the table shall be permitted.*
Note: IO = Immediate Occupancy, LS = Life Safety, CP = Collapse Prevention.

Table 7.3.2d. Alternate Numerical Acceptance Criteria for Linear Procedures: Conforming Reinforced Concrete Structural Walls Controlled by Flexure.

Conditions		m-factors*				
		Performance Level				
			Primary		Secondary	
$\frac{l_w c_{GE}}{b_s^2}$	$\frac{N_{UD}}{A_g f'_{cE}}$	IO	LS	CP	LS	CP
≤ 10	≤ 0.10	1.9	7.5	8.5	10	11
≤ 10	≥ 0.20		6.0	6.8	8.0	9.1
≥ 70	≤ 0.10	1.4	3.8	4.3	5.0	5.7
≥ 70	≥ 0.20		2.6	3.0	3.5	4.0

*Linear interpolation between the values given in the table shall be permitted; however, interpolation between the values specified for Conforming walls (Table 7.3.2d) and Nonconforming walls (Table 7.3.2e) shall not be permitted.

Note: IO = Immediate Occupancy, LS = Life Safety, CP = Collapse Prevention.

Table 7.3.2e. Alternate Numerical Acceptance Criteria for Linear Procedures: Nonconforming Reinforced Concrete Structural Walls Controlled by Flexure.[a,b]

Conditions		m-factors[c]				
		Performance Level				
			Primary		Secondary	
$\frac{l_w c_{GE}}{b_s^2}$	$\frac{N_{UD}}{A_g f'_{cE}}$	IO	LS	CP	LS	CP
≤ 10	≤ 0.10	1.6	6.6	7.4	8.8	9.9
≤ 10	≥ 0.20		3.9	4.5	5.3	6.0
≥ 60	≤ 0.10	1.2	2.8	3.2	3.8	4.3
≥ 60	≥ 0.20		1.9	2.1	2.5	2.8

[a] *This table applies to walls and wall segments with* $\rho_{lw} \geq 0.001$. *For* $0.0025 \geq \rho_{lw} \geq 0.001$ *and* $\frac{l_w c_{DE}}{b_s^2} \leq 20$, *acceptance criteria shall be multiplied by a reduction factor. The reduction factor shall be 0.4 for* $\rho_{lw} = 0.001$ *and* $\frac{l_w c_{DE}}{b_s^2} \leq 10$ *and 1.0 for* $\rho_{lw} = 0.0025$ *and* $\frac{l_w c_{DE}}{b_s^2} = 20$. *Linear interpolation of the reduction factor with respect to* ρ_{lw} *and* $\frac{l_w c_{DE}}{b_s^2}$ *shall be permitted for intermediate values.*

[b] *This table applies to walls with one or multiple curtains of web reinforcement.*

[c] *Linear interpolation between the values given in the table shall be permitted; however, interpolation between the values specified for Conforming walls (Table 7.3.2d) and Nonconforming walls (Table 7.3.2e) shall not be permitted.*

Note: IO = Immediate Occupancy, LS = Life Safety, CP = Collapse Prevention.

ASCE 41. When determining the appropriate value for the design actions, consideration shall be given to gravity loads and to the maximum forces that can be transmitted considering nonlinear action in adjacent components. Design actions for the controlling behavior of structural walls shall be compared with expected strengths in accordance with Section 7.5.2.2 of ASCE 41 and Section 7.2 of this standard.

The controlling behavior for structural walls and wall segments shall be in accordance with Table 7.3.2a and classified as deformation-controlled unless noted otherwise. Structural walls with h_w/l_w *smaller than 1.0 shall be permitted to be classified as either shear or shear-friction controlled. Actions of components or interfaces with either a vertical or horizontal reinforcement ratio,* ρ_l *or* ρ_t, *less than 0.001 shall be classified as force-controlled components.*

The maximum expected flexural strength of a structural wall or wall segment, M_{CultE}, *shall be used to determine the maximum expected shear demand in structural walls and wall segments,* V_{MCultE}, *for the purpose of determining the controlling behavior in accordance with Equation (7.3.2a). For cantilever structural walls, the maximum expected shear demand,* V_{MCultE}, *shall not be less than the magnitude of the lateral force required to develop the maximum expected flexural strength,* M_{CultE}, *of the wall critical section, but need not exceed the maximum load that can be* *delivered to the wall segment. For non-cantilever wall segments, the maximum expected shear demand, V_{MCultE}, shall be equal to the shear corresponding to the development of the positive and negative maximum expected flexural strengths at opposite ends of the wall segment, but need not exceed the maximum load that can be delivered to the wall segment. The dynamic shear amplification factor (ω_v) in Table 7.3.2a shall be determined from Equation (7.3.2b) but shall be permitted to be taken as 1.0 for non-cantilever wall segments where V_{MCultE} corresponds to the development of positive and negative maximum expected flexural strengths at opposite ends of the wall segment or the maximum load that can be delivered to the wall segment.*

$$V_{MCultE} = \frac{M_{CultE}}{M_{UD}} V_{UD} \qquad (7.3.2a)$$

$$\begin{aligned} \omega_v &= 0.9 + \frac{n_s}{10} \quad \text{for } n_s \leq 6 \\ \omega_v &= 1.3 + \frac{n_s}{30} \quad \text{for } n_s > 6 \end{aligned} \qquad (7.3.2b)$$

Table 7.3.2b specifies m-factors for use in Equation (7-39) of ASCE 41 for flexure-, shear-, and shear friction-controlled structural walls and wall segments. Nonlinear modeling parameters, a_{nl}, b_{nl}, d_{nl}, e_{nl}, and g_{nl}, in Table 7.3.2b, where applicable, shall be determined as specified in Table 7.4.1.1.1a and Table 7.4.1.1.1b for flexure-controlled walls, Table 7.4.1.1.2 for shear-controlled walls, and Table 7.4.1.1.3 for shear-friction controlled walls, unless otherwise specified. As an alternative to Table 7.3.2b, it shall be permitted to use m-factors from Table 7.3.2c for walls controlled by shear, and Tables 7.3.2d and 7.3.2e for walls controlled by flexure. Alternate m-factors shall be permitted where justified by experimental evidence and analysis.

Walls that are non-symmetric about a bending axis, in terms of geometry, reinforcement ratio, detailing, and/or applied axial loads shall have their non-symmetric behavior considered in the two directions of loading about that axis. For such non-symmetric wall sections, it shall be permitted to use the moment strength from either direction of loading that results in the largest DCR or lowest m-factor from Table 7.3.2a through e.

For the purpose of determining m-factors from Table 7.3.2d and e, walls shall be considered Conforming where wall detailing complies with (a) through (e); otherwise, the wall shall be considered Nonconforming, if any of the conditions are not met:

(a) *A minimum of two curtains of web vertical and horizontal reinforcement are present,*

(b) *A minimum boundary longitudinal reinforcement ratio based on Equation (7.3.2b) is provided,*

$$\rho_{lb} \geq 6\sqrt{f'_{cE}}/f_{yE} \qquad (7.3.2b)$$

(c) *The minimum ratio of provided area of boundary transverse reinforcement in form of rectilinear hoops and crossties, $A_{sh}/(sb_{core})$, or the ratio of volume of spiral reinforcement to total volume of confined core by spiral or circular hoop, ρ_s, is not less than 0.7 times that computed from Equations (7.3.2c) and (7.3.2d):*

$$\frac{A_{sh}}{sb_{\text{core}}} = 0.3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_{cE}}{f_{yE}} \geq 0.09\frac{f'_{cE}}{f_{yE}} \qquad (7.3.2c)$$
$$\textit{for rectilinear hoops and crossties}$$

$$\rho_s = 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_{cE}}{f_{yE}} \geq 0.12\frac{f'_{cE}}{f_{yE}} \qquad (7.3.2d)$$
$$\textit{for spiral and circular hoops}$$

(d) *The ratio of vertical center-to-center spacing of boundary transverse reinforcement to the diameter of the smallest longitudinal reinforcement,* $\boldsymbol{s/d_b} \leq 8.0$,

(e) *Lap-splice failure of longitudinal reinforcement is precluded.*

7.4 NONLINEAR STATIC AND DYNAMIC PROCEDURES FOR REINFORCED CONCRETE STRUCTURAL WALLS AND WALL SEGMENTS

7.4.1 Modeling *It shall be permitted to simultaneously use decoupled rotational and translational elements to simulate nonlinear flexure and shear behaviors, respectively, for structural walls and wall segments in accordance with Section 7.4.1.1. Deformations at wall critical sections shall be evaluated to verify that the deformation at onset of lateral strength loss including shear and flexural deformations, does not exceed the lower value of modeling parameter d_{nl} of the rotational and translational elements given in Table 7.4.1.1.1a, Table 7.4.1.1.1b, and Table 7.4.1.1.2. Additional deformation acceptance criteria are also required in accordance with Section 7.4.2. Alternatively, it shall be permitted to model structural walls and wall segments using solid elements or layered shell elements that couple flexural and shear behaviors in accordance with Section 7.4.1.2.*

Walls that are nonsymmetric about a bending axis in terms of geometry, reinforcement ratio, detailing and/or applied axial loads shall be modeled considering nonsymmetric behavior in the two directions of loading about that axis. For such nonsymmetric walls, it shall be permitted to use the same wall classification per Table 7.3.2a for both loading directions based on the larger moment strength from either direction. It shall also be permitted to apply to both loading directions the lower deformation capacities based on either direction from Table 7.4.1.1.1a, Table 7.4.1.1b, Table 7.4.1.1.2, and Table 7.4.1.1.3.

The controlling behavior for structural walls and wall segments shall be in accordance with Table 7.3.2a and modeled as deformation controlled, unless noted otherwise. The shear amplification factor ω_v need not be applied where V_{MCultE} is obtained from the nonlinear analysis procedures.

Modification factors used to calculate maximum inelastic displacements in the ASCE 41, Nonlinear Static Procedure, C_1 and C_2, shall be calculated in accordance with ASCE 41, Equations (7-22) and (7-24). ASCE 41, Table 7-3 shall not be permitted for calculating demands on structural walls and wall segments.

Actions of components or interfaces with either a vertical or horizontal reinforcement ratio, ρ_l or ρ_t, less than 0.001 shall be classified as force controlled. Actions not classified as deformation-controlled in this section shall be permitted to be classified as deformation-controlled if the strength and stiffness degradation of the member under the action is modeled explicitly and verified by experimental evidence.

7.4.1.1 Nonlinear Static and Dynamic Procedures Employing Lumped-Plasticity Load-Deformation Models *Nonlinear load-deformation relations for use in analysis by nonlinear static and dynamic procedures shall comply with the requirements of Section 3.1.2. Monotonic and cyclic load-deformation relationships for analytical models that represent structural walls and wall segments shall be in accordance with the generalized relations defined in Section 3.1 and shown in Figure 3.1.2.2.3. Table 3.1.2.2.4 and*

Table 7.4.1.1.1a. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures: Conforming Reinforced Concrete Structural Walls and Associated Components Controlled by Flexure.

Conditions[d]				Acceptance Criteria Performance Level
$\frac{l_w c_{DE}}{b_s^2}$	$\frac{\omega_v V_{MCultDE}}{A_{cv}\sqrt{f'_{cE}}}$ [c]	Overlapping hoops[a] used?	d_{nl}	IO
≤10	≤4	Yes	0.032	$\theta_{yE} + 0.1(d_{nl} - \theta_{yE})$
≤10	≥6	Yes	0.026	
≥70	≤4	Yes	0.018	
≥70	≥6	Yes	0.014	
≤10	≤4	No	0.032	
≤10	≥6	No	0.026	
≥70	≤4	No	0.012	
≥70	≥6	No	0.011	

Conditions[d]						Acceptance Criteria Performance Level	
$\frac{l_w c_{GE}}{b_s^2}$	$\frac{N_{UD}}{A_g f'_{cE}}$	c_{nl}	c'_{nl}	d'_{nl} [b]	e_{nl} [b]	LS	CP
≤10	≤0.10	0.5	1.15	0.036	0.040	0.75 e_{nl}	0.85 e_{nl}
≤10	≥0.20	0.1		0.030	0.032		
≥70	≤0.10	0.0		0.018	0.020		
≥70	≥0.20	0.0		0.014	0.014		

[a] Overlapping hoop definition shall be per ACI 318-19.

[b] Parameters d'_{nl} and e_{nl} shall not be taken smaller than parameter d_{nl}.

[c] The shear amplification factor ω_v need not be applied if V_{MCultE} is obtained from nonlinear analyses procedures.

[d] Linear interpolation between the values given in the table shall be permitted; however, interpolation between the values specified for Conforming walls (Table 7.4.1.1.1a) and Nonconforming walls (Table 7.4.1.1.1b) shall not be permitted.

Note: IO = Immediate Occupancy, LS = Life Safety, CP = Collapse Prevention.

Table 7.3.1 shall be permitted for determining the initial stiffness from Point A to Point B in Figure 3.1.2.2.3. The effective compression and tension flange widths defined in Section 3.1.3 shall be used.

7.4.1.1.1. Structural Walls and Wall Segments Controlled by Flexure For structural walls and wall segments that have inelastic behavior under lateral loading that is controlled by flexure, the following approach shall be permitted. The load-deformation relationship in Figure 3.1.2.2.3b shall be used with the X-axis of the figure taken as the rotation over the plastic hinging region (l_p) at the end of the member, as shown in Figure 7.4.1.1.1. The hinge rotation at Point B in Figure 3.1.2.2.3b corresponds to the yield hinge rotation (θ_{yE}) and shall be calculated in accordance with Equation (7.4.1.1.1).

$$\theta_{yE} = \left(\frac{M_{CyGE}}{E_{cE} I_{eff}}\right) l_p \tag{7.4.1.1.1}$$

where $E_{cE} I_{eff}$ is the effective flexural stiffness computed per Section 7.3.1, M_{CyGE} is computed as described for M_{Cy} in Section 7.2.1 but permitting gravity-only load combinations, and l_p is the assumed plastic hinge length. The value of l_p shall be set equal to the lesser of 0.5 times the total length of the individual wall and the height of the story at the location of the plastic hinge. For wall segments, the value of l_p shall be set equal to the lesser of 0.5 times the effective flexural depth of the member and 0.5 times the element length.

Values for the variables c_{nl}, c'_{nl}, d_{nl}, d'_{nl}, and e_{nl} required to define the location of Points C, D, and E in Figure 3.1.2.2.3b shall be as specified in Table 7.4.1.1.1a and Table 7.4.1.1.1b. A wall shall be considered Conforming or Nonconforming based on the criteria in Section 7.3.2.

For the Nonlinear Dynamic Procedure (NDP), the unloading and reloading stiffnesses, strengths, and any pinching of the load-versus-rotation hysteresis loops shall reflect the behavior experimentally observed for wall elements similar to the one under investigation. Where experimental data are not available to validate complete hysteretic behavior of a component including unloading and reloading stiffness, strength, and pinching of the load-deformation response history, the computational models employed shall define response histories in general agreement with the relationships presented in 3.1 as specified in Table 7.4.1.1.1a and Table 7.4.1.1.1b. Use of the generalized load-deformation relation shown in Figure 3.1.2.2.3b to represent the response envelope for the analysis shall be permitted.

7.4.1.1.2. Structural Walls and Wall Segments Controlled by Shear For structural walls and wall segments whose inelastic response is controlled by shear in accordance with 7.3.2, the following approach shall be permitted. The load-deformation relationship in Figure 3.1.2.2.3c shall be used, with the X-axis of the figure taken as the lateral translation component of the

Table 7.4.1.1.1b. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures: Nonconforming Reinforced Concrete Structural Walls and Associated Components Controlled by Flexure.

Conditions^{d,e}			Acceptance Criteria Performance Level
$\frac{l_w c_{DE}}{b_s^2}$	Detailing^{a,b,c,g}	d_{nl}	IO
≤ 10	$\frac{A_{sh,provided}}{A_{sh,required}} \geq 0.5$ and $\frac{s}{d_b} \leq 9$	0.024	$\theta_{yE} + 0.1(d_{nl} - \theta_{yE})$
≤ 10	$\frac{A_{sh,provided}}{A_{sh,required}} < 0.2$ and $\frac{s}{d_b} > 15$	0.019	
≥ 60	$\frac{A_{sh,provided}}{A_{sh,required}} \geq 0.5$ and $\frac{s}{d_b} \leq 9$	0.010	
≥ 60	$\frac{A_{sh,provided}}{A_{sh,required}} < 0.2$ and $\frac{s}{d_b} > 15$	0.008	

Conditions^{d,e,g}						Acceptance Criteria Performance Level	
$\frac{l_w c_{DE}}{b_s^2}$	$\frac{N_{UD}}{A_g f'_{cE}}$	c_{nl}	c'_{nl}	d'_{nl} ^{f}	e_{nl} ^{f,h}	LS	CP
≤ 10	≤ 0.10	0.4	1.15	0.032	0.035	$0.75\ e_{nl}$	$0.85\ e_{nl}$
≤ 10	≥ 0.20	0.1		0.020	0.021		
≥ 60	≤ 0.10	0.0		0.015	0.015		
≥ 60	≥ 0.20	0.0		0.010	0.010		

[a] $A_{sh,required}$ shall be as calculated per Equation (7.3.2c). In case of boundary elements with transverse reinforcement in the form spiral or circular hoop, the term $A_{sh,provided}/A_{sh,required}$ shall be replaced with $\rho_{s,provided}/\rho_{s,required}$, where $\rho_{s,required}$ is calculated per Equation (7.3.2d).

[b] If values of both $A_{sh,provided}/A_{sh,required}$ and s/d_b fall between the limits given in the table, linear interpolation shall independently be performed for both $A_{sh,provided}/A_{sh,required}$ and s/d_b, and the lower resulting value of parameter d_{nl} shall be taken.

[c] Values of $A_{sh,provided}/A_{sh,required}$ and s/d_b shall be provided over a horizontal distance that extends from extreme compression fiber at least $c_{DE}/3$.

[d] This table applies to walls and wall segments with $\rho_{lw} \geq 0.001$. For $0.0025 \geq \rho_{lw} \geq 0.001$ and $\frac{l_w c_{DE}}{b_s^2} \leq 20$, modeling parameters d_{nl}, d'_{nl} and e_{nl} shall be multiplied by a reduction factor. The reduction factor shall be 0.4 for $\rho_{lw} = 0.001$ and $\frac{l_w c_{DE}}{b_s^2} \leq 10$ and 1.0 for $\rho_{lw} = 0.0025$ and $\frac{l_w c_{DE}}{b_s^2} = 20$. Linear interpolation of the reduction factor with respect to ρ_{lw} and $\frac{l_w c_{DE}}{b_s^2}$ shall be permitted for intermediate values.

[e] This table applies to walls with one or multiple curtains of web reinforcement.

[f] Parameters d'_{nl} and e_{nl} shall not be taken smaller than parameter d_{nl}.

[g] Linear interpolation between the values given in the table shall be permitted; however, interpolation between the values specified for Conforming walls (Table 7.4.1.1.1a) and Nonconforming walls (Table 7.4.1.1.1b) shall not be permitted.

[h] For walls with no boundary transverse reinforcement and $N_{UD} > 0.08\ A_g f'_{cE}$, e_{nl} and d'_{nl} shall be multiplied by 0.8 but shall not be taken less than d_{nl}.

Note: IO = Immediate Occupancy, LS = Life Safety, CP = Collapse Prevention.

lateral drift ratio. Alternatively, the load-deformation relationship in Figure 3.1.2.2.3b shall be permitted, with the X-axis of Figure 3.1.2.2.3b taken as the lateral translation component of the lateral drift ratio. For structural walls, this drift shall be the story drift, as shown in Figure 7.4.1.1.2. For wall segments, Figure 7.4.1.1.2 shall represent the member drift.

Values for the variables f_{nl}, g_{nl}, c_{nl}, c'_{nl}, d_{nl}, d'_{nl}, and e_{nl} required to define Points B, C, D, E, and F in Figure 3.1.2.2.3c shall be as specified in Table 7.4.1.1.2. For nonsymmetric flanged wall sections, flanged or rectangular cross-section designation in Table 7.4.1.1.2 shall apply based on the wall end that is compressed due to moment and shear actions. It shall be permitted to use the modeling parameters and acceptance criteria for rectangular cross-sections for any wall cross-section.

7.4.1.1.3 Structural Walls and Wall Segments Controlled by Shear-Friction For structural walls and wall segments with inelastic behavior under lateral loading that is controlled by shear-friction, in accordance with 7.3.2, the following approaches shall be permitted. The load-deformation relationship in Figure 3.1.2.2.3d shall be used, with the X-axis of the figure taken as the nonlinear sliding displacement of the wall along the sliding plane. Values for the variables a_{nl}, a'_{nl}, b_{nl}, c_{nl}, and c'_{nl} required to define Points C, D, and E in Figure 3.1.2.2.3d shall be as specified in Table 7.4.1.1.3. For walls or wall segments with sustained transverse loading, b_{nl} shall be taken equal to a'_{nl} in Table 7.4.1.1.3.

7.4.1.2 Nonlinear Static and Nonlinear Dynamic Procedures Employing Models Other Than Lumped-Plasticity Load-Deformation Models. *It shall be permitted to model structural walls and wall segments responding primarily in flexure using solid elements, fiber or layered shell elements, or beam-column elements with fiber-type section models in accordance with Section*

Table 7.4.1.1.2. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures: Reinforced Concrete Structural Walls and Associated Components Controlled by Shear.[a]

Condition					Acceptance Criteria Performance Objective
Cross-section shape[b]	$\frac{N_{UD}}{A_g f'_{cE}}$	$\frac{V_{CWall318E}}{\omega_v V_{MCultDE}}$	g_{nl}	d_{nl}[c]	IO
Rectangular	$<= 0.15$	≥ 1.0	0.004	0.015	$g_{nl} + 0.1(d_{nl} - g_{nl})$
	$>= 0.5$	≥ 1.0		θ_{yE}	
	$<= 0.005$	≤ 0.5		0.006	
	$>= 0.5$	≤ 0.5		θ_{yE}	
Flanged	$<= 0.15$	≥ 1.0		0.020	
	$>= 0.5$	≥ 1.0		θ_{yE}	
	$<= 0.005$	≤ 0.5		0.009	
	$>= 0.5$	≤ 0.5		θ_{yE}	

Condition				Acceptance Criteria Performance Objective	
$\frac{N_{UD}}{A_g f'_{cE}}$	$\frac{V_{CWall318E}}{\omega_v V_{MCultDE}}$	d'_{nl}[d]	e_{nl}[d]	LS	CP
≤ 0.075	≥ 1.0	0.025	0.03	0.65 e_{nl}	0.80 e_{nl}
	≤ 0.5	0.015	0.02		
≥ 0.150	≥ 1.0	0.015	0.015		
	≤ 0.5	0.010	0.010		

Condition			
Cross-section shape[b]	$\frac{N_{UD}}{A_g f'_{cE}}$	c_{nl}[e]	c'_{nl}
Rectangular	≤ 0.10	0.25	1.10
	≥ 0.15	0.00	
Flanged	≤ 0.15	0.40	
	≥ 0.20	0.00	

[a] Linear interpolation between values listed in the table shall be permitted.

[b] Linear interpolation between values listed in the table based on I_{g_flange}/I_{g_rect} shall be permitted for walls and wall segments between wall and flanged designations with $1.0 < I_{g\_flange}/I_{g\_rect} < 1.5$

[c] d_{nl} shall be taken as the greater of 0.005 and θ_{yE} when ρ_t and ρ_l are less than 0.0015 and $\frac{V_{CWall318E}}{V_{MCyDE}} \leq 0.5$.

[d] d'_{nl} and e_{nl} shall not be taken less than d_{nl} . . . .

[e] c_{nl} shall be taken as zero where ρ_t is less than 0.0015.

Note: IO = Immediate Occupancy, LS = Life Safety, CP = Collapse Prevention.

Table 7.4.1.1.3. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures: Reinforced Concrete Structural Walls and Associated Components Controlled by Shear-Friction.

Conditions		Sliding Displacements[a] (in.)			Strength Ratios		Acceptance Criteria		
Interface Type	$\frac{V_{CyfWallSE}}{w_v V_{MCyDE}}$[b]	a_{nl}	a'_{nl}	b_{nl}	c'_{nl}	c_{nl}	IO	LS	CP
Monolithic or roughened to ¼ in. amplitude	≥ 1.0	0.65	1.30	3.0 in.	1.10	0.35	0.1 a_{nl}	0.75 b_{nl}	b_{nl}
	≤ 0.5	0.20	0.40						
Other	≥ 1.0	0.80	1.60			0.35			
	≤ 0.5	0.40	0.80						

[a] Linear interpolation between values listed in the table shall be permitted.

[b] Shear amplification factor ω_v need not be applied if V_{MCyDE} is obtained from nonlinear analyses procedures.

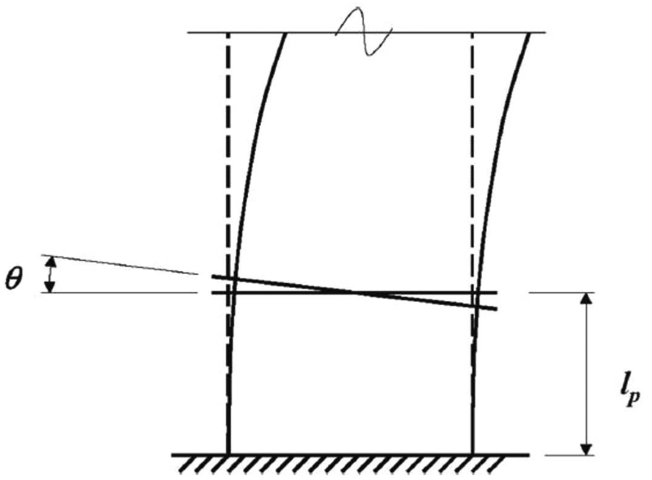


Figure 7.4.1.1.1. Plastic hinge rotation in structural wall where flexure dominates inelastic response.

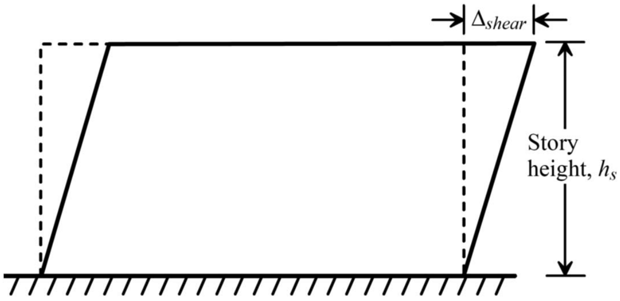


Figure 7.4.1.1.2. Lateral translation component of story drift in a structural wall.

3.1.2. Where simulation results generated using these modeling procedures cannot be validated using experimental data, the stress-strain models that compose these models shall be modified such that predicted response is in general agreement with the generalized load-deformation values in 7.4.1.1.

It shall be permitted to model structural walls and wall segments controlled by shear or shear-friction using shell elements having nonlinear shear force versus shear strain response matching the modeling parameters provided in Table 7.4.1.1.2 and Table 7.4.1.1.3, respectively.

7.4.2 Acceptance Criteria *Components experiencing inelastic response shall satisfy the requirements of Chapter 7, Section 7.5.3.2 of ASCE 41, and the maximum total hinge rotations or drifts shall not exceed the values given in Table 7.4.1.1.1a, Table 7.4.1.1.1b, and Table 7.4.1.1.2 for the selected performance level, regardless of the selected analytical modeling approach in Section 7.4.1. At wall critical sections, employing decoupled shear and flexural analytical models shall be limited to a value corresponding to the lower acceptance criteria permitted in Table 7.4.1.1.1a, Table 7.4.1.1.1b, and Table 7.4.1.1.2 for the applicable Performance Objective.*

It shall be permitted to use Table 7.4.1.1.2 for wall actions controlled by shear-friction at cold joints located at the interface between walls and foundations having a minimum dowel reinforcement ratio of 0.001.

Alternative acceptance criteria based on experimental data shall be permitted in accordance with ASCE 41, Chapter 7, Section 7.6.

7.5 LINEAR STATIC AND DYNAMIC PROCEDURES FOR REINFORCED CONCRETE COUPLING BEAMS

7.5.1 Modeling *A beam element that incorporates both bending and shear deformations shall be used to model coupling beams.*

Table 7.5.2a. Numerical Acceptance Criteria for Linear Procedures Reinforced Concrete Coupling Beams Controlled by Flexure.[a]

Conditions		m-factors[b]				
		Performance Level				
		Component Type				
			Primary		Secondary	
Longitudinal reinforcement and transverse reinforcement[c]	$\frac{V}{hb_w\sqrt{f'_{cE}}}$[d]	**IO**	**LS**	**CP**	**LS**	**CP**
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤3	2	4	6	6	9
	≥6	1.5	3	4	4	7
Conventional longitudinal reinforcement with nonconforming transverse reinforcement	≤3	1.5	3.5	5	5	8
	≥6	1.2	1.8	2.5	2.5	4
Diagonal reinforcement	*N/A*	2	5	7	7	10

[a] *For secondary coupling beams spanning less than 8 ft 0 in., with bottom reinforcement continuous into the supporting walls, secondary values shall be permitted to be doubled.*

[b] *Linear interpolation between values listed in the table shall be permitted.*

[c] *Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the coupling beam. Conforming transverse reinforcement consists of (1) closed stirrups over the entire length of the coupling beam at a spacing $\le d/3$, and (b) strength of closed stirrups $V_s \ge 3/4$ of required shear strength of the coupling beam.*

[d] *V is the design shear force calculated using limit-state analysis procedures in accordance with 7.3.2.*

Table 7.5.2b. Numerical Acceptance Criteria for Linear Procedures: Reinforced Concrete Coupling Beams Controlled by Shear.[a]

Conditions		m-factors[b]				
		Performance level				
		Component type				
			Primary		Secondary	
Longitudinal reinforcement and transverse reinforcement[c]	$\frac{V}{hb_w\sqrt{f'_{cE}}}$ [d]	IO	LS	CP	LS	CP
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤3	1.5	3	4	4	6
	≥6	1.2	2	2.5	2.5	3.5
Conventional longitudinal reinforcement with nonconforming transverse reinforcement	≤3	1.5	2.5	3	3	4
	≥6	1.2	1.2	1.5	1.5	2.5

[a] For secondary coupling beams spanning less than 8 ft 0 in., with bottom reinforcement continuous into the supporting walls, secondary values shall be permitted to be doubled.

[b] Linear interpolation between values listed in the table shall be permitted.

[c] Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the coupling beam. Conforming transverse reinforcement consists of (a) closed stirrups over the entire length of the coupling beam at a spacing ≤ d/3, and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of the coupling beam.

[d] V is the design shear force calculated using limit-state analysis procedures in accordance with 7.3.2.

For coupling beams that have diagonal reinforcement satisfying ACI 318-19 requirements, a beam element representing flexural behavior only shall be permitted. The in-plane and out-of-plane diaphragm behavior of concrete slabs that interconnect structural wall piers and frame columns shall be represented in the model.

The effective stiffness of coupling beams shall satisfy 3.1.2 unless alternative stiffness values are determined by a more detailed analysis. The effective tension or compression flange width for coupling beams cast monolithically with slabs shall be as specified in 3.1.3.

The flexural and shear strengths of coupling beams shall be evaluated using the principles and equations contained in Chapter 18 of ACI 318-19, with the strength reduction factor, ϕ, taken as 1.0. The expected and lower bound strengths of reinforcement and concrete shall be used for deformation- and force-controlled actions, respectively.

7.5.2 Acceptance Criteria *Coupling beams shall be classified as either deformation- or force-controlled, as defined in Section 7.5.1.1 of ASCE 41. In coupling beams, deformation-controlled actions shall be restricted to flexure or shear. All other actions shall be treated as force controlled.*

Design actions shall be compared with strengths in accordance with Section 7.5.2.2 of ASCE 41. Table 7.5.2a and Table 7.5.2b specify m-factors for use in Equation (7-39) of ASCE 41. Alternate m-factors shall be permitted where justified by experimental evidence and analysis.

7.6 NONLINEAR STATIC AND DYNAMIC PROCEDURES FOR REINFORCED CONCRETE COUPLING BEAMS

7.6.1 Modeling *For nonlinear procedures, coupling beams shall be modeled using solid elements, shell elements, or beam-column elements that represent distributed or lumped-plasticity models. The inelastic response shall account for the loss of shear strength and stiffness during reversed cyclic loading to large deformations. Where experimental data are not available to enable validation of models, simulated response shall be in general agreement with the load-deformation relationship in Figure 3.1.2.2.3b, with the X-axis of Figure 3.1.2.2.3b taken as the chord rotation as defined in Figure 7.6.1. Values for the variables d_{nl}, e_{nl}, f_{nl}, g_{nl}, and c_{nl} required to define Points B, C, D, E, and F in Figure 3.1.2.2.3b shall be as specified in Table 7.6.2a and Table 7.6.2b as applicable for the appropriate members.*

7.6.2 Acceptance Criteria *Components experiencing inelastic response shall satisfy the requirements of 7.5.3.2 of ASCE 41, and the maximum chord rotation angles shall not exceed the values given in Table 7.6.2a and Table 7.6.2b for the selected performance level.*

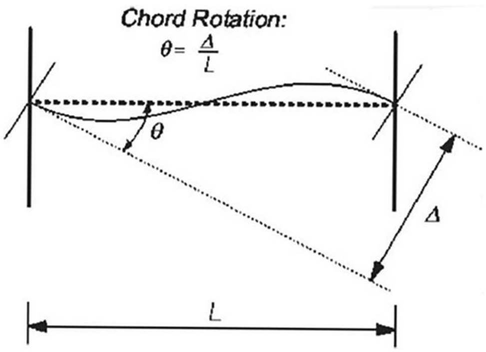


Figure 7.6.1. Chord rotation for concrete coupling beams.

Table 7.6.2a. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures: Reinforced Concrete Coupling Beams Controlled by Flexure.[a]

Conditions		Chord rotation[b], rad		Residual strength ratio[b]	Acceptable Chord rotation[b], rad Performance level		
Longitudinal reinforcement and transverse reinforcement[c]	$\frac{V}{hb_w\sqrt{f'_{cE}}}$	d_{nl}	e_{nl}	c_{nl}	IO	LS	CP
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤3	0.025	0.050	0.75	0.010	0.025	0.050
	≥6	0.020	0.040	0.50	0.005	0.020	0.040
Conventional longitudinal reinforcement with nonconforming transverse reinforcement	≤3	0.020	0.035	0.50	0.006	0.020	0.035
	≥6	0.010	0.025	0.25	0.005	0.010	0.025
Diagonal reinforcement	NA	0.030	0.050	0.80	0.006	0.030	0.050

[a] For coupling beams spanning less than 8 ft, with bottom reinforcement continuous into the supporting walls, acceptance criteria values shall be permitted to be doubled for LS and CP performance.

[b] Linear interpolation between values listed in the table shall be permitted.

[c] Nonprestressed longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the coupling beam. Conforming transverse reinforcement consists of (a) closed stirrups over the entire length of the coupling beam at a spacing less than or equal to d/3; and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of the coupling beam.

Note: IO = Immediate Occupancy, LS = Life Safety, CP = Collapse Prevention.

Table 7.6.2b. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures: Reinforced Concrete Coupling Beams Controlled by Shear[a]

Conditions		Chord rotation[b], rad		Strength ratio[b]	Acceptable chord rotation[b], rad Performance level		
Longitudinal reinforcement and transverse reinforcement[c]	$\frac{V}{hb_w\sqrt{f'_{cE}}}$	d	e	c	IO	LS	CP
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤3	0.020	0.030	0.60	0.006	0.020	0.030
	≥6	0.016	0.024	0.30	0.005	0.016	0.024
Conventional longitudinal reinforcement with nonconforming transverse reinforcement	≤3	0.012	0.025	0.40	0.006	0.010	0.020
	≥6	0.008	0.014	0.20	0.004	0.007	0.012

[a] For coupling beams spanning less than 8 ft 0 in., with bottom reinforcement continuous into the supporting walls, acceptance criteria values shall be permitted to be doubled for LS and CP performance.

[b] Linear interpolation between values listed in the table shall be permitted.

[c] Nonprestressed longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the coupling beam. Conforming transverse reinforcement consists of (a) closed stirrups over the entire length of the coupling beam at a spacing less than or equal to d/3; and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of the coupling beam.

Note: IO = Immediate Occupancy, LS = Life Safety, CP = Collapse Prevention.

7.7 RETROFIT MEASURES FOR REINFORCED CONCRETE STRUCTURAL WALLS, WALL SEGMENTS, AND COUPLING BEAMS

Seismic retrofit measures for reinforced concrete structural walls, wall segments, coupling beams, and columns supporting discontinuous structural walls shall meet the requirements of Section 3.7 and other provisions of this standard.

10.3.3 Concrete Foundations. ***Replace Sections 12.1 through 12.4 of ACI 369.1 with the italicized text as follows.***

ACI 369.1 CHAPTER 12

12.1 TYPES OF CONCRETE FOUNDATIONS

Foundations shall be defined as those components that serve to transmit loads from the vertical structural subsystems, such as columns and walls, of a building to the supporting soil or rock. Concrete foundations for buildings shall be classified as either shallow or deep foundations as defined in ASCE 41 Chapter 8. Requirements of Section 12 shall apply to shallow foundations that include spread or isolated footing, strip or line footing, combination footing, and concrete mat footing and to deep foundations that include pile foundations and cast-in-place piers. Concrete grade beams shall be permitted in both shallow and deep foundation systems and shall comply with the requirements of Section 12.

12.1.1 Shallow Concrete Foundations *Existing spread footings, strip footings, and combination footings may be reinforced or unreinforced. Vertical loads are transmitted by these footings to the soil by direct bearing; seismic forces are transmitted by a combination of friction between the bottom of the footing and the* *soil, and passive pressure of the soil on the vertical face of the footing. Flexure in the footings shall be permitted to be considered deformation-controlled with acceptance criteria based on the provisions of Section 4.4 for slabs, except that the condition for continuity reinforcement shall be replaced with whether the flexural reinforcement ratio meets or exceeds that specified in Section 7.6.1.1 of ACI 318 for tension reinforcement, and corresponding flexure actions not meeting this reinforcement ratio shall be considered force-controlled. All other actions in the footings shall be considered force-controlled.*

Concrete mat footings are reinforced to resist the flexural and shear stresses resulting from the superimposed concentrated and line structural loads and the distributed resisting soil pressure under the footing. Seismic forces are resisted by friction between the soil and the bottom of the footing and by passive pressure developed against foundation walls that are part of the system.

Flexure in the mat shall be considered deformation-controlled with acceptance criteria based on the provisions of Section 4.4 for slabs, except that the condition for continuity reinforcement shall be replaced with whether the flexural reinforcement ratio meets or exceeds that specified in Section 7.6.1.1 of ACI 318 for tension reinforcement, and corresponding flexure actions not meeting this reinforcement ratio shall be considered force-controlled. All other actions in the mat shall be considered force-controlled. An analytical model of the mat-column frame actions, based on the approaches permitted in Section 4.4 for slab-column frames in conjunction with ASCE 41, Section 8.4.5 subgrade stiffness requirements, shall be used to evaluate shear and flexural actions in the mat components.

Force-controlled actions in shallow foundations shall be permitted to be reclassified as Type 3 deformation-controlled actions if it is shown that stability of the foundation system meets acceptance criteria identified in ASCE 41, Section 8.4 with the contribution of the action strength neglected or explicitly degraded in nonlinear analysis.

12.1.2 Deep Concrete Foundations

12.1.2.1 Driven Concrete Pile Foundations Concrete pile foundations shall be composed of a reinforced concrete pile cap supported on driven piles. The piles shall be concrete (with or without prestressing), steel shapes, steel pipes, or composite (concrete in a driven steel shell). Vertical loads are transmitted to the piles by the pile cap. Pile foundation resistance to vertical loads shall be calculated based on the direct bearing of the pile tip in the soil, the skin friction or cohesion of the soil on the surface area of the pile or based on a combination of these mechanisms. Seismic-force resistance shall be calculated based on passive pressure of the soil on the vertical face of the pile cap, in combination with interaction of the piles in bending and passive soil pressure on the pile surface. Actions in concrete piles shall be considered deformation-controlled or force-controlled and have corresponding acceptance criteria based on the provisions for Columns in Section 4.2.4 and additional provisions in Section 12.3. A substructure analysis shall be performed to determine the effective shear span of piles and to relate pile deformations to plastic chord rotations as specified in Section 4.2.4, which considers the nonlinear lateral load-deformation response between the piles and soil. The effective shear span for a pile shall be taken as the distance between maximum positive and negative bending moments when the pile limit state is reached. Actions in the pile cap shall be considered force controlled.

12.1.2.2 Cast-in-Place Concrete Pile Foundations Cast-in-place concrete pile foundations shall consist of reinforced concrete placed in a drilled or excavated shaft. Cast-in-place pile foundation resistance to vertical and seismic forces shall be calculated in the same manner as that of driven pile foundations specified in Section 12.1.2.1.

***12.1.3 Grade Beams** Concrete grade beams shall comprise all the following, if present: reinforced concrete beams below grade, interconnecting footings, pile caps, and piers. Actions in grade beams shall be considered based on the provisions of Section 4.2.4.*

12.2 ANALYSIS OF EXISTING CONCRETE FOUNDATIONS

For concrete buildings, it is permitted to consider components fixed against rotation and translation at the top of the foundation if the connections between components and foundations, the foundations, and supporting soil are shown to be capable of resisting the induced forces and the foundation is rotationally stiff relative to the component stiffness. Where components or foundations are not designed to resist flexural moments, or the connections between components and foundations are not capable of resisting the induced moments, it is permitted to model the components with pinned ends at the top of the foundation. In such cases, the component base shall be evaluated for the ability to accommodate the necessary end rotation of the component. The effects of base stiffness of components shall be taken into account at the point of maximum displacement of the superstructure.

If fixed or pinned boundary elements cannot be justified, a more rigorous analysis procedure shall be used. Appropriate vertical, lateral, and rotational soil springs shall be incorporated in the analytical model as described in ASCE 41, Section 8.4. The spring characteristics shall be as specified in ASCE 41, Chapter 8. Rigorous analysis of structures with deep foundations in soft soils shall be based on special soil–pile interaction studies to determine the probable location of the point of fixity in the foundation and the resulting distribution of forces and displacements in the superstructure. In these analyses, the appropriate representation of the connection of the pile to the pile cap shall be included in the model. Piles with less than 6 in. of embedment without any dowels into the pile cap shall be modeled as being "pinned" to the cap. Unless the pile and pile cap connection detail is identified as otherwise from the available construction documents, the "pinned" connection shall be used in the analytical model.

Where the foundations are included in the analytical model, the responses of the foundation components shall be considered. The reactions of structural components attached at the foundation (axial loads, shears, and moments) shall be used to evaluate the individual components of the foundation system.

12.3 EVALUATION OF EXISTING CONDITION

Allowable soil capacities (subgrade modulus, bearing pressure, and passive pressure) and foundation displacements for the selected Performance Level shall be as prescribed in ASCE 41, Chapter 8 or as established with project-specific data. Actions in all components of existing foundation systems and all new; components, or components required for retrofit shall be evaluated as force-controlled or deformation-controlled in accordance with Section 12.1. Boundary conditions, effective component lengths over which chord rotations and/or displacement ductility demands are determined, and relevant detailing conditions shall *be considered when determining acceptance criteria in Section 12.1. Effective shear span for piles shall be determined based on an evaluation of the distance between points of maximum moment, determined using the results of substructure analyses that include the effects of cyclic load history, inelastic soil-pile interaction, and pile yielding. Alternative approaches or values shall be permitted where justified by experimental evidence and analysis.*

12.4 RETROFIT MEASURES FOR CONCRETE FOUNDATIONS

Seismic retrofit measures for concrete foundations shall meet the requirements of Section 3.7 and other provisions of this standard.

10.3.4 Notation Replace Sections 13.1 through 13.2 of ACI 369.1 with the italicized text as follows.

ACI 369.1 Chapter 13

13.1. NOTATION

A_{ch} = *Cross-sectional area of a member measured to the outside edges of transverse reinforcement, in.*^{2}
A_{cv} = *Gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered in the case of walls, and gross area of concrete section in the case of diaphragms, not to exceed the thickness times the width of the diaphragm, in.*^{2}
A_g = *Gross sectional area of component, in.*^{2}
A_s = *Area of nonprestressed tension reinforcement, in.*^{2}
A_{sh} = *Total cross-sectional area of transverse reinforcement, including crossties, within spacing s and perpendicular to dimension* b_{core}, *in.*^{2}
A_{sl} = *Total area of longitudinal reinforcement in a section, in.*^{2}
A'_s = *Area of compression reinforcement, in.*^{2}
A_{vf} = *Area of shear-friction reinforcement crossing the assumed shear plane, in.*^{2}
A_w = *Area of the web cross section,* = $b_w d$, *in.*^{2}
a_{nl} = *Parameter used to measure deformation capacity in component load-deformation curves, Figure 3.1.2.2.3; same as a in ASCE 41*
a'_{nl} = *Parameter used to measure deformation capacity in component load-deformation curves, Figure 3.1.2.2.3*
b_{core} = *Cross-sectional dimension of member core measured to the outside edges of the transverse reinforcement composing area* A_{sh}, *in.*
b_{nl} = *Parameter used to measure deformation capacity in component load-deformation curves, Figure 3.1.2.2.3; same as b in ASCE 41*
b_s = *Width of flexural compression zone of a wall section, evaluated in accordance with Figure C7.4.1.1, in.*
b_w = *Web width, in.*
c_{DE} = *Section compression neutral axis depth evaluated using expected material properties and axial load* N_{UD}, *in.*
c_{nl} = *Parameter used to measure residual strength, Figure 3.1.2.2.3; same as c in ASCE 41*
c'_{nl} = *Parameter used to measure maximum strength, Figure 3.1.2.2.3*
DCR = *Demand-capacity ratio, computed in accordance with Equation (7-16) in ASCE 41*
d = *Distance from extreme compression fiber to centroid of tension reinforcement, in.; it shall be permitted to assume that* $d = 0.8h$, *where h is the dimension of the column in the direction of shear, in.*
d_b = *Nominal diameter of reinforcing bar, in.*
d_{nl} = *Parameter used to measure deformation capacity, Figure 3.1.2.2.3; same as d in ASCE 41*
d'_{nl} = *Parameter used to measure deformation capacity, Figure 3.1.2.2.3*
E = *Young's modulus of elasticity, psi*
E_{cE} = *Modulus of elasticity of concrete; evaluated using expected material properties, psi*
E_s = *Modulus of elasticity of reinforcement, psi*
$(EI)_{eff}$ = *Effective flexural rigidity of a section, in.*^{2}*-lb*
e_{nl} = *Parameter used to measure deformation capacity, Figure 3.1.2.2.3; same as e in ASCE 41*
f'_c = *Specified compressive strength of concrete, psi*
f'_{cE} = *Expected compressive strength of concrete, psi*
f_{nl} = *Parameter used to measure deformation capacity, Figure 3.1.2.2.3; same as f in ASCE 41*
f_y = *Lower-bound or expected yield strength of reinforcement, as applicable to force-controlled or deformation-controlled actions, respectively, psi*
f_{yf} = *Lower-bound or expected yield strength of* A_{vf}, *as applicable to force-controlled or deformation-controlled actions, respectively, psi*
f_{yE} = *Expected yield strength of steel reinforcement, psi*
f_{yL} = *Lower-bound yield strength of steel reinforcement, psi*
f_{yl} = *Lower-bound or expected yield strength of longitudinal reinforcement, as applicable to force-controlled or deformation-controlled actions, respectively, psi*
f_{ylE} = *Lower-bound yield strength of longitudinal steel reinforcement, psi*
f_{ylL} = *Lower-bound yield strength of longitudinal steel reinforcement, psi*
f_{yt} = *Lower-bound or expected yield strength of transverse reinforcement, as applicable to force-controlled or deformation-controlled actions, respectively, psi*
f_{ytE} = *Expected yield strength of transverse reinforcement, psi*
f_{ytL} = *Lower-bound yield strength of transverse reinforcement, psi*
g_{nl} = *Parameter used to measure deformation capacity, Figure 3.1.2.2.3; same as g in ASCE 41*
h = *Height of member along which deformations are measured*
h = *Overall thickness of member, in.*
h_s = *Height of story at which an action is considered, in.*
h_1 = *Effective height over which bond slip is distributed, taken as the clear height of the wall at the story directly above the anchorage interface*
h_w = *Structural wall height, in.*
I_g = *Moment of inertia of gross concrete or masonry section about centroidal axis, neglecting reinforcement, in.*^{4}
I_{g_flange} = *Gross moment of inertia of the concrete section bounded by the effective flange width defined in 3.1.3 about its centroidal axis, neglecting reinforcement, in.*^{4}

I_{g_rect} = Gross moment of inertia of the rectangular portion of the concrete section about its centroidal axis, neglecting reinforcement, in.^{4}

K_R = Stiffness of rotational spring used to explicitly capture bar slip, in.-lb/rad

l_e = Length of embedment of reinforcement, in.

l_n = Length of clear span measured face-to-face of supports, in.

l_p = Length of plastic hinge used for calculation of inelastic deformation capacity, in.

l_{sp} = Strain penetration depth, in.

l_w = Length of entire wall or a segment of wall considered in the direction of shear force, in.

M_{CultE} = Flexural maximum strength of a component as represented by Point C in Figure 3.1.2.2.3 and determined using reinforcement tensile strength of 1.15 times the yield strength of longitudinal bars and fundamental principles given in Chapter 22 of ACI 318-19, without using a strength reduction factor. M_{CultE} shall be evaluated using expected material properties. M_{CultE} shall be evaluated using applied axial loads in accordance with 3.3, in.-lb

M_{fyGE} = M_{fyE} evaluated using axial load N_{UG}, in.-lb

m = Component demand modification factor to account for expected ductility associated with this action at the selected Structural Performance Level

N_{UD} = Member design axial force evaluated based on Equation (7-34) of ASCE 41 in linear procedures, or 7.5.3.1 in nonlinear procedures, lb

N_{UG} = Member design axial force evaluated based on Gravity Load Combinations in Equation (7-1) through (7-3) of ASCE 41; set to zero for tension force in Equation (4.2.3.1), lb

Q = Generalized force in a component, Figure 3.1.2.2.3

Q_{yE} = Expected yield strength of a component, Figure 3.1.2.2.3

s = Spacing of transverse reinforcement, in.

V = Shear force at section concurrent with moment M, lb

V_{CE} = Expected shear capacity of a member, lowest of $V_{CydWallE}$ and $V_{CyfWallE}$, lb

M_{Cy} = Flexural yield strength of a component as represented by Point B in Figure 3.1.2.2.3 and determined using the fundamental principles given in Chapter 22 of ACI 318-19, without using a strength reduction factor. M_{Cy} shall be evaluated using lower bound or expected material properties as applicable to force-controlled or deformation-controlled actions, respectively. M_{Cy} shall be evaluated using applied axial loads in accordance with 3.3, in.-lb

M_{CyDE} = M_{Cy} evaluated using expected material properties and applied axial loads in accordance with 3.3, in.-lb

M_{CyGE} = M_{Cy} evaluated using expected material properties and applied axial loads N_{UG}, in.-lb

M_{fyE} = Moment of a section at first yield, defined as the moment at which the yield strain of the steel reinforcement is first reached in tension, or a concrete strain of 0.002 is reached in compression; evaluated using expected material properties, in.-lb

V_{Col0E} = Shear strength of concrete columns at a displacement ductility demand not exceeding 2.0, Equation (4.2.3.1); evaluated using expected material properties, lb

$V_{CcrWallE}$ = Shear cracking strength of concrete walls evaluated using expected material properties, lb

V_{CPunE} = Punching shear strength provided by the concrete as defined in ACI 318; evaluated using expected material properties, lb

$V_{CultdWallE}$ = Ultimate shear strength of concrete walls evaluated using expected material properties, lb

$V_{CWall318E}$ = Shear strength of concrete walls from ACI 318-19 evaluated using expected material properties, lb

$V_{CydWall}$ = Shear strength of concrete walls evaluated using lower-bound or expected material properties as applicable to force-controlled or deformation-controlled actions, respectively, lb

$V_{CydWallE}$ = $V_{CydWall}$ evaluated using expected material properties, lb

$V_{CydWallL}$ = $V_{CydWall}$ evaluated using lower bound material properties, lb

$V_{CyfWallE}$ = Shear-friction strength of a structural wall or wall segment considering shear transfer across any given plane; evaluated using expected material properties, lb

$V_{CyfWallS}$ = Simplified shear-friction strength of a structural wall or wall segment considering shear transfer across any given plane; evaluated using lower-bound or expected material properties as applicable to force-controlled or deformation-controlled actions, respectively, lb

V_J = Beam-column joint shear strength calculated using the general procedures of ACI 318-19, as modified by Equation (4.2.3.2), lb

V_{MCyDE} = Shear demand resulting from flexural yielding of the plastic hinges at a moment of M_{CyDE}, lb

V_{MCultE} = Shear demand resulting from flexural yielding of the plastic hinges at a moment of M_{CultE}, calculated per Equation (7.3.2a), lb

V_s = Shear strength provided by shear reinforcement, lb

ϕ_{fyE} = Curvature at section at first yield, defined as the curvature at which the yield strain of the steel reinforcement is first reached in tension, or a concrete strain of 0.002 is reached in compression; evaluated using expected material properties, rad/in.

ϕ_{yE} = Curvature in the effective bilinear moment-curvature relationship associated with M_{yE}; evaluated using expected material properties, rad/in.

α_c = Coefficient defining the relative contribution of concrete strength to shear strength of concrete walls or wall segments

Δ = Calculated deflection of diaphragm, wall, or bracing element; or generalized deformation, Figure 3.1.2.2.3

Δ_{Shear} = Calculated lateral translational component of lateral drift in walls or wall segments, Figure 7.4.1.1.2

λ = Correction factor related to unit weight of concrete as defined in ACI 318

n_s = Number of stories above the critical section and shall not be taken less than 0.007 times the wall height above the critical section measured in inches. This limit is imposed on n_s to account for buildings with large story heights

θ = *Generalized deformation, radians, Figure 3.1.2.2.3*

θ_{yE} = *Yield rotation, radians, Equation (7.4.1.1.1); evaluated using expected material properties*

ρ_l = *Ratio of area of distributed longitudinal reinforcement to gross concrete area perpendicular to that reinforcement*

ρ_{lb} = *Ratio of area of distributed longitudinal reinforcement to gross concrete area perpendicular to that reinforcement in the wall boundary region evaluated per ACI 318*

ρ_{lw} = *Ratio of area of total longitudinal reinforcement to gross concrete area perpendicular to that reinforcement in a wall or wall segment*

ρ_s = *Ratio of volume of spiral reinforcement to total volume of confined core by spiral or circular hoop*

ρ_t = *Ratio of area of distributed transverse reinforcement to gross concrete area perpendicular to that reinforcement* = $A_v/(bs)$

ω_v = *Dynamic shear amplification factor for evaluating maximum wall shear demand in linear analysis procedures*

13.2 DEFINITIONS

Aspect Ratio: *Ratio of full height to length for concrete and masonry shear structural walls; ratio of span to depth for horizontal diaphragms.*

Boundary Component: *A structural component at the boundary of a structural wall or a diaphragm or at an edge of an opening in a structural wall or a diaphragm that possesses tensile or compressive strength to transfer lateral forces to the seismic-force-resisting system.*

Coupling Beam: *A component that ties or couples adjacent structural walls acting in the same plane.*

Element: *An assembly of structural components that act together in resisting forces, including gravity frames, moment-resisting frames, braced frames, structural walls, and diaphragms.*

In-plane Wall: *See* **Structural Wall***.*

Structural wall: *A wall that resists lateral forces applied parallel with its plane; also known as an in-plane wall.*

Wall, Flanged: *A wall or wall segment with gross moment of inertia of the wall cross-section bounded by the effective flange width as defined in Section 3.1.3 is at least 1.5 times the gross moment of inertia of the rectangular portion of the section. Flanged walls include barbell, C-shaped, T-shaped and other non-rectangular shapes.*

CHAPTER 11

MASONRY

11.1 SCOPE

This chapter sets forth requirements for the seismic evaluation and retrofit of masonry components of the seismic-force-resisting system of an existing building. The requirements of this chapter shall apply to existing masonry components of a building system, retrofitted masonry components of a building system, and new masonry components that are added to an existing building system. Adobe and stone masonry are beyond the scope of this chapter.

Section 11.2 specifies data collection procedures for performing condition assessments and obtaining material properties. Section 11.3 provides general analysis and design requirements for masonry components. Section 11.4 provides modeling procedures, component strengths, acceptance criteria, and retrofit measures for masonry infills. Section 11.5 specifies requirements for anchorage to masonry walls. Section 11.6 specifies requirements for masonry foundation elements. Section 11.7 specifies requirements for masonry diaphragms.

Nonstructural components of masonry buildings, including but not limited to parapets, veneer, and masonry partitions that are isolated from the seismic-force-resisting system, are addressed in Chapter 13.

11.2 CONDITION ASSESSMENT AND MATERIAL PROPERTIES

11.2.1 General The procedures for defining masonry structural systems and assessing masonry condition shall be in accordance with the provisions stated in Section 11.2.2.

Mechanical properties for masonry materials and components shall be based on available drawings, specifications, and other documents for the existing construction in accordance with requirements of Section 6.2. Where such documents fail to provide adequate information to quantify masonry material properties or the condition of masonry components of the structure, such information shall be supplemented by materials tests and assessments of existing conditions as required in Section 6.2, and this section.

Material properties of existing masonry components shall be determined in accordance with Section 11.2.3. The extent of materials testing and condition assessment performed shall be used to determine the knowledge factor, as specified in Section 11.2.4.

Use of default material properties shall be permitted in accordance with Section 11.2.3.10.

Other values of material properties shall be permitted if rationally justified, based on available historical information for a particular type of masonry construction, prevailing codes, and assessment of existing conditions.

11.2.2 Condition Assessment A condition assessment of the existing building and site conditions shall be performed as specified in Sections 11.2.2.1 through 11.2.2.3 and 11.5.3.

A condition assessment shall include the following:

1. The physical condition of primary and secondary components shall be examined, and the presence of any degradation shall be noted. The condition of existing masonry shall be evaluated for unit surface or mortar joint deterioration due to weathering caused by freeze–thaw cycles or frequent moisture saturation.
2. The presence and configuration of components and their connections and the continuity of load paths among components, elements, and systems shall be verified or established.
3. Other conditions, including the presence and attachment of veneer, neighboring party walls and buildings, presence of nonstructural components, prior remodeling, and limitations for retrofit that may influence building performance, shall be identified and documented.

The condition of existing masonry shall be classified as good, fair, or poor defined based on visual examination and other approved procedures that consider the nature and extent of damage or deterioration present.

Good condition: Masonry found during condition assessment to have mortar and units intact with no visible cracking, deterioration, or damage.

Fair condition: Masonry found during condition assessment to have mortar and units intact but with minor cracking.

Poor condition: Masonry found during condition assessment to have degraded mortar, degraded masonry units, or significant cracking is limited in use by Section 11.3.1.1.

11.2.2.1 Visual Condition Assessment The size and location of all masonry shear and bearing walls shall be determined by visual examination. The orientation and placement of the walls shall be noted. Overall dimensions of masonry components shall be measured or determined from plans, including wall heights, lengths, and thicknesses. Locations and sizes of window and door openings shall be measured or determined from plans. The distribution of gravity loads to bearing walls shall be estimated where required for the determination of masonry material properties.

Walls shall be classified as reinforced or unreinforced; composite or noncomposite; and grouted, partially grouted, or ungrouted. For reinforced masonry (RM) construction, the size and spacing of horizontal and vertical reinforcement shall be estimated. For multiwythe construction, the number of wythes shall be noted, as well as the distance between wythes and the placement of inter-wythe ties. The condition and attachment of veneer wythes shall be noted. For grouted construction, the quality of grout placement shall be assessed. For partially grouted walls, the locations of grout placement shall be identified.

The type and condition of the mortar and mortar joints shall be determined. Mortar shall be examined for weathering, erosion, and hardness and to identify the condition of any pointing or repointing, including cracks, internal voids, weak components, and/or deteriorated or eroded mortar. Horizontal cracks in bed joints, vertical cracks in head joints and masonry units, and diagonal cracks near openings shall be noted.

Vertical components that are not straight shall be identified. Bulging or undulations in walls shall be observed, as well as separation of exterior wythes, out-of-plumb walls, and leaning parapets or chimneys.

Connections between masonry walls and floors or roofs shall be examined to identify details and condition. If construction drawings are available, a minimum of three connections shall be inspected for each connection type. If no deviations from the drawings are found, the sample shall be considered representative. If drawings are unavailable, or if deviations are noted between the drawings and constructed work, then a random sample of connections shall be inspected until a representative pattern of connections is identified.

11.2.2.2 Comprehensive Condition Assessment Nondestructive tests shall be permitted to quantify and confirm the uniformity of construction quality and the presence and degree of deterioration for comprehensive data collection, including but not limited to the following:

1. Ultrasonic or mechanical pulse velocity to detect variations in the density and modulus of masonry materials and to detect the presence of cracks and discontinuities,
2. Impact-echo tests to confirm whether reinforced walls are grouted,
3. Radiography to confirm location of reinforcing steel,
4. Infrared thermography,
5. Surface penetrating radar, and
6. Borescopic investigations.

11.2.2.3 Supplemental Tests Supplemental tests shall be permitted to enhance the level of confidence in masonry material properties or the assessment of masonry condition for justifying the use of a higher knowledge factor, as specified in Section 11.2.4.

11.2.2.4 Condition Enhancement Where required within the scope of and consistent with the Performance Objective of the seismic evaluation or retrofit, the condition of existing masonry shall be enhanced in accordance with this section. Masonry units with significant surface deterioration shall be replaced. Mortar joint deterioration shall be patched by pointing or repointing of the eroded joint in accordance with Section 11.2.2.5. Existing cracks in unreinforced solid unit and in solid grouted hollow-unit masonry shall be repaired by low-pressure epoxy grout injection.

11.2.2.5 Pointing or Repointing of Unreinforced Masonry Walls Where required within the scope of and consistent with the Performance Objective of the seismic evaluation or retrofit, existing masonry joints shall be pointed or repointed.

11.2.3 Properties of In-Place Materials and Components

11.2.3.1 General The following component and connection material properties shall be obtained for the as-built structure in accordance with the following, and Sections 11.2.3.2 through 11.2.3.9:

1. Masonry compressive strength.
2. Elastic modulus for masonry.
3. Unreinforced and reinforced masonry bed-joint flexural tensile strength.
4. Unreinforced masonry shear strength.
5. Where unreinforced masonry material testing is required by Section 6.2 test methods to quantify masonry strength and stiffness properties shall be determined in accordance with Sections 11.2.3.2 through 11.2.3.7. The minimum number of tests shall comply with the requirements of Section 11.2.3.9.
6. Where reinforced masonry material testing is required by Section 6.2 test methods to quantify strength and stiffness properties shall be determined in accordance with Sections 11.2.3.2 through 11.2.3.5, 11.2.3.7, and 11.2.3.8. The minimum number of tests shall comply with the requirements of Section 11.2.3.9.
7. Expected material properties shall be based on mean values from test data unless specified otherwise. Lower-bound material properties shall be based on mean minus one standard deviation values from test data unless specified otherwise.

11.2.3.2 Nominal or Specified Properties Nominal material properties, or properties specified in construction documents, of clay or concrete units shall be taken as lower-bound material properties. Corresponding expected material properties shall be calculated by multiplying lower-bound values by a factor as specified in Table 11-1 to translate from lower-bound to expected values. Refer to Chapter 10 for properties of reinforcing steel.

11.2.3.3 Masonry Compressive Strength Expected masonry compressive strength, f_{me}, shall be determined using one of the following three methods:

1. Test prisms shall be extracted from an existing wall and tested in accordance with Section 1.4.B.3 of TMS 602;
2. Prisms shall be fabricated from actual extracted masonry units, and a surrogate mortar shall be designed on the basis of a chemical analysis of actual mortar samples; the test prisms shall be tested in accordance with Section 1.4.B.3 of TMS 602; or
3. For solid unreinforced masonry, the strength of the masonry can be estimated using a flat jack test in accordance with ASTM C1197.

For each of the three methods enumerated in this section, the expected compressive strength shall be based on the net mortared area.

11.2.3.4 Masonry Elastic Modulus in Compression Expected values of elastic modulus for masonry in compression, E_{me}, are permitted to be determined as follows:

1. In accordance with TMS 402;
2. Measured from test prisms extracted from an existing wall and tested in compression; or
3. For solid unreinforced masonry, using a flat jack test in accordance with ASTM C1197.

Table 11-1. Factors to Translate Specified Lower-Bound Masonry Strengths to Expected Strengths.

Strength	Factor
Compressive strength (f_{me})	1.3
Flexural tensile strength	1.3
Shear strength	1.3

11.2.3.5 Masonry Flexural Tensile Strength Expected flexural tensile strength, f_{te}, for out-of-plane bending shall be determined for unreinforced masonry using one of the following three methods:

1. Test samples shall be extracted from an existing wall and subjected to minor axis bending using the bond wrench method of ASTM C1072,
2. Test samples shall be tested in situ using the bond wrench method, or
3. Sample wall panels shall be extracted and subjected to minor axis bending in accordance with ASTM E518.

Flexural tensile strength for unreinforced masonry (URM) walls subjected to in-plane seismic forces shall be assumed to be equal to that for out-of-plane bending, unless testing is undertaken to define the expected tensile strength for in-plane bending.

11.2.3.6 Unreinforced Masonry Shear Strength URM masonry may be tested to determine the expected shear strength by one of the following shear tests in Sections 11.2.3.6.1 or 11.2.3.6.2 for each class of URM determined by Section 11.2.3.9.2. These expected shear strengths may be used in lieu of using Tables 11-1, 11-2a, 11-2b and 11-2c.

Lower-bound shear strengths may be determined by using Sections 11.2.3.6.3 or 11.2.3.6.4.

Table 11-2a. Default Lower-Bound Unreinforced Masonry Strengths (in Customary Units).

Material	Solid Units	Hollow Concrete Units
Compressive strength[a]	600 lb/in.^{2}	1,000 lb/in.^{2}
Flexural tensile strength[b]	60 lb/in.^{2}	38 lb/in.^{2c} (95 lb/in.^{2})^d
Shear strength	[e]	[e]

[a] Clay f'_m is based on 2,100 lb/in.^{2} unit compressive strength and Type N mortar. Hollow concrete f'_m is based on 1,900 lb/in.^{2} unit net compressive strength and Type N mortar on face shells only.

[b] Values based on portland cement/lime or mortar cement, Type N mortar.

[c] Ungrouted hollow concrete blocks.

[d] Solid grouting of hollow concrete blocks; may be interpolated for partial grouting based on net area.

[e] Strength shall be taken as 80% of shear strength values determined in accordance with Section 9.2.6 of TMS 402.

Table 11-2b. Default Lower-Bound Reinforced Masonry Strengths (in Customary Units).

Material	Solid Units	Solid Grouted Hollow Concrete Units
Compressive strength[a]	900 lb/in.^{2}	1,500 lb/in.^{2}
Shear strength	[b]	[b]

[a] Clay f'_m is based on 2,100 lb/in.^{2} flatwise unit compressive strength and Type N mortar. Hollow concrete block f'_m is based on 1,900 lb/in.^{2} unit net compressive strength, Type N mortar, and solid grouting.

[b] Strength shall be taken as the shear strength values determined in accordance with Section 9.3.3.1.2 of TMS 402.

Table 11-2c. Default Lower-Bound Strengths for Unreinforced Masonry with Lime Mortar (in Customary Units).

Material	Solid Units
Compressive strength	285 lb/in.^{2}
Flexural tensile strength	5 lb/in.^{2}
Shear strength	*

*Strength shall be taken as 80% of shear strength values determined in accordance with Section 9.2.6 of TMS 402.

11.2.3.6.1 Determination of Expected URM Shear Strength by Testing for Bed-Joint Shear Strength Individual bed-joint shear strength test values, v_{to}, shall be determined in accordance with Equation (11-1) when testing is performed in accordance with ASTM C1531:

$$v_{to} = \frac{V_{\text{test}}}{A_b} - P_{D+L} \tag{11-1}$$

where

V_{test} = Test load at first movement of a masonry unit,
A_b = Sum of net mortared area of bed joints located directly above and below the test unit, and
P_{D+L} = Gravity compressive stress at the test location considering actual unfactored dead plus live loads in place at the time of testing.

The expected URM bed-joint sliding strength, v_{me}, shall be determined from Equation (11-2):

$$v_{me} = \frac{0.75\left(0.75v_{te} + \dfrac{P_D}{A_n}\right)}{1.5} \tag{11-2}$$

where

A_n = Area of net mortared and/or grouted section of a wall or wall pier;
P_D = Superimposed dead load at top of wall or pier under consideration; and
v_{te} = Average of the bed-joint shear strength test values, v_{to}, given in Equation (11-1).

The 0.75 factor on v_{te} shall not be applied for single-wythe masonry walls. The 0.75 factor on v_{te} shall be permitted to be 1.0 if mortar in the collar joint is not present or is in poor condition.

11.2.3.6.2 Alternative Procedures for Determining Expected URM Shear Strength by Testing for Tensile Splitting Strength Wythes (leaves) of solid masonry units may be tested by sampling the masonry by drilled cores of not less than 8 in. (200 mm) in diameter. A bed-joint intersection with a head joint shall be in the center of the core. The tensile splitting strength of these cores should be determined by the standard test method of ASTM C496. The core should be placed in the test apparatus with the bed joint oriented at 45 degrees from the horizontal. The tensile splitting strength should be determined by Equation (11-3):

$$f_{sp} = 2_P/\pi A_n \tag{11-3}$$

Hollow-unit masonry constructed of through-the-wall units may be tested by sampling the masonry by a sawn square prism not less than 18 in. (0.46 m) square. The tensile splitting strength should be determined by the standard test method of ASTM E519. The diagonal axis of the prism should be placed in a vertical position. The tensile splitting strength should be determined by Equation (11-4):

$$f_{sp} = 0.494P/A_n \tag{11-4}$$

where A_n is the diameter of core multiplied by its length or the area of the side of a square prism.

The expected URM shear strength, v_{me}, shall be determined by Equation (11-5):

$$v_{me} = \frac{0.75\left(f_{spe} + \frac{P_D}{A_n}\right)}{1.5} \tag{11-5}$$

where

f_{spe} = Average of the mortar tensile splitting strength values, f_{sp}, given in Equation (11-3) or (11-4); and A_n and P_D are defined in Section 11.2.3.6.1.

11.2.3.6.3 Determination of Lower-Bound URM Shear Strength by Testing for Bed-Joint Shear Strength The lower-bound URM bed-joint sliding strength, v_{mL}, shall be determined from Equation (11-6):

$$v_{mL} = \frac{0.75\left(0.75 v_{tL} + \frac{P_D}{A_n}\right)}{1.5} \tag{11-6}$$

where

v_{tL} = Mean minus one standard deviation of the bed-joint shear strength test values, v_{to}, given in Equation (11-1); and A_n and P_D are defined in Section 11.2.3.6.1.

The 0.75 factor on v_{tL} shall not be applied for single-wythe masonry walls. The 0.75 factor on v_{tL} shall be permitted to be 1.0 if mortar in the collar joint is not present.

11.2.3.6.4 Alternative Procedures for Determining Lower-Bound URM Shear Strength by Testing for Tensile Splitting Strength The lower-bound URM shear strength, v_{mL}, shall be determined by Equation (11-7):

$$v_{mL} = \frac{0.75\left(f_{spL} + \frac{P_D}{A_n}\right)}{1.5} \tag{11-7}$$

where

f_{spL} = Mean minus one standard deviation of the mortar tensile splitting strengths, f_{sp}, given in Equation (11-3) or (11-4); and A_n and P_D are defined in Section 11.2.3.6.1.

11.2.3.7 Masonry Shear Modulus The expected shear modulus of masonry (unreinforced or reinforced), G_{me}, shall be permitted to be taken from Section 4.2.2 of TMS 402.

11.2.3.8 Steel Reinforcement Yield Strength Properties The expected yield strength of reinforcing bars, f_{ye}, shall be based on mill test data or on tension tests of actual reinforcing bars taken from the subject building.

Use of Tables 2.2.1.2, 2.2.5b, and 2.2.5c of ACI 369 shall be permitted for determination of yield strength properties of existing reinforcement.

Where development lengths and lap splices of existing deformed bars meet the provisions of TMS 402, the yield strength does not need to be adjusted. Where development lengths and lap splices of existing deformed bars do not meet the TMS 402 requirements, the yield strength shall be adjusted using Equation (11-8):

$$f_{ye} = 1.25(l_b/l_d) \times f_y \leq 1.25 f_y \tag{11-8}$$

If l_b is less than 12 in. (305 mm), f_{ye} shall be taken as zero.

Prior to application of any m-factor, if the calculated maximum applied bar stress is larger than that determined by Equation (11-8), members shall be deemed to be force controlled because of inadequate splicing. As an alternative, testing may be used to determine the lap splices can perform as deformation controlled.

For RM wall components, existing plain reinforcement development and lap splice lengths shall be taken as twice the values determined per TMS 402 for a deformed bar of equivalent diameter unless other lengths are justified by approved tests or calculations considering only the bond between the bar and the concrete. The allowed length shall not be less than the value for deformed bar per TMS 402.

11.2.3.9 Minimum Number of Tests Materials testing is not required if material properties are available from original construction documents that include material test records or material test reports. Material test records or reports shall be representative of all critical components of the building structure. Otherwise, minimum numbers of tests shall be performed as specified in Sections 11.2.3.9.1 through 11.2.3.9.3, as applicable.

Material samples collection and testing, where required, shall be conducted at locations representative of the material conditions throughout the entire building, taking into account variations in work quality at different levels, variations in weathering of the exterior surfaces, and variations in the condition of the interior surfaces due to deterioration caused by leaks and condensation of water and/or the deleterious effects of other substances contained within the building. The exact test locations shall be determined at the building site by the design professional.

An increased sample size shall be permitted to improve the confidence level. The relation between sample size and confidence shall be as defined in ASTM E122.

11.2.3.9.1 Usual Testing of Reinforced Masonry The minimum number of tests to determine masonry and reinforcing steel material properties for usual data collection shall be based on the following criteria:

1. If the specified design compressive strength of the masonry is known, at least two tests shall be performed on samples of each different masonry compressive strength used in the construction of the building;
2. If the specified design strength of the masonry is not known, at least one test shall be performed on each type of component, with a minimum of six tests performed on the entire building;
3. If the specified design strength of the reinforcing steel is known, use of nominal or specified material properties shall be permitted without additional testing; and
4. If the specified design strength of the reinforcing steel is not known, at least two strength coupons of reinforcing steel shall be removed from a building for testing.

11.2.3.9.2 Usual Testing of Unreinforced Masonry Existing unreinforced masonry shall be categorized into one or more classes based on quality of construction and state of repair, deterioration, and weathering. Classes shall be defined for whole walls, not for small areas within a wall.

The minimum number of tests per class necessary to quantify properties for usual data collection shall be as follows:

1. At each of both the first and top stories, no fewer than two tests per wall or line of wall elements providing a common line of resistance to seismic forces;
2. At each of all other stories, no fewer than one test per wall or line of wall elements providing a common line of resistance to seismic forces;
3. In any case, no fewer than one test per 1,500 ft^2 (139.4 m^2) of wall surface; and
4. No fewer than a total of eight tests.

11.2.3.9.3 Comprehensive Testing of Reinforced and Unreinforced Masonry Existing unreinforced masonry shall be categorized into one or more classes as described in the previous section. In addition to applicable testing in Sections 11.2.3.9.1 and 11.2.3.9.2, the minimum number of tests necessary to quantify properties by in-place testing for comprehensive data collection shall be based on the following criteria:

1. A minimum of three tests shall be performed for each unreinforced masonry class;
2. If original construction documents are available that specify material properties, a minimum of three tests shall be performed for every three floors of construction or 3,000 ft^2 (279 m^2) of wall surface, whichever requires the most testing;
3. If original construction documents are not available, a minimum of six tests shall be performed for every three floors of construction or 3,000 ft^2 (279 m^2) of wall surface, whichever requires the most testing;
4. At least two tests shall be performed for each wall or line of wall elements providing a common resistance to seismic forces;
5. A minimum of eight tests shall be performed for each building; and
6. Additional tests shall be done to estimate material strengths in regions where properties differ. Nondestructive condition assessment tests in accordance with Section 11.2.2.2 shall be used to investigate variations in construction quality and presence and degree of material deterioration.

If the coefficient of variation in test measurements exceeds 25%, the number of tests performed shall be doubled.

11.2.3.10 Default Properties Use of default material properties to determine component strengths shall be permitted with the linear analysis procedures in Chapter 7. Default values as specified in the tables in this section shall only apply to masonry in good or fair condition, as defined in Section 11.2.2.

Default lower-bound values for URM compressive strength, flexural tensile strength, and shear strength are permitted to be as shown in Table 11-2a. Default lower-bound strength for reinforced masonry shall be as shown in Table 11-2b. Default lower-bound values for compressive strength, flexural tensile strength, and shear strength of unreinforced masonry constructed with lime mortar are permitted to be as shown in Table 11-2c. Mortar that is easily scraped away from the joints by hand with a metal tool shall be considered lime mortar. Default expected values for masonry compressive strength, flexural tensile strength, and masonry shear strength shall be determined by multiplying lower-bound strengths by an appropriate factor taken from Table 11-1.

Default lower-bound and expected strength yield stress values for reinforcing bars shall be determined in accordance with Section 2.2 of ACI 369.

11.2.4 Knowledge Factor A knowledge factor, κ, for computation of masonry component capacities and permissible deformations shall be selected in accordance with Section 6.2.3 and with the following additional requirements specific to masonry components. A knowledge factor, κ, equal to 0.75 shall be used if any of the following criteria are met:

1. Components are found to be damaged or deteriorated during assessment, and further testing is not performed to quantify their condition or justify the use of $\kappa = 1.0$;
2. Mechanical properties have a coefficient of variation exceeding 25%; or
3. Components contain archaic or proprietary material and the condition is uncertain.

11.3 MASONRY WALLS

The procedures set forth in this section for determination of stiffness, strength, and deformation of masonry walls shall be applied to building systems made up of any combination of existing masonry walls. Unreinforced or reinforced masonry walls enhanced for seismic retrofit or new walls added to an existing building may be used for seismic retrofit.

Actions in a structure shall be classified as being either deformation controlled or force controlled as defined in Section 7.5.1. Design strengths for deformation-controlled and force-controlled actions shall be calculated in accordance with this section.

Strengths used for deformation-controlled actions are denoted Q_{CE} and shall be taken as equal to expected strengths obtained experimentally, calculated using accepted mechanics principles, or based on default values listed in Section 11.2.3.10 and modified by Table 11-1. Expected strength is defined as the mean maximum resistance expected over the range of deformations to which the component is likely to be subjected. Where calculations are used to define expected strength, expected material properties shall be used. Unless otherwise specified in this standard, use of strength design procedures specified in TMS 402 to calculate expected strengths shall be permitted. The strength reduction factor, ϕ, shall be taken as equal to 1.0.

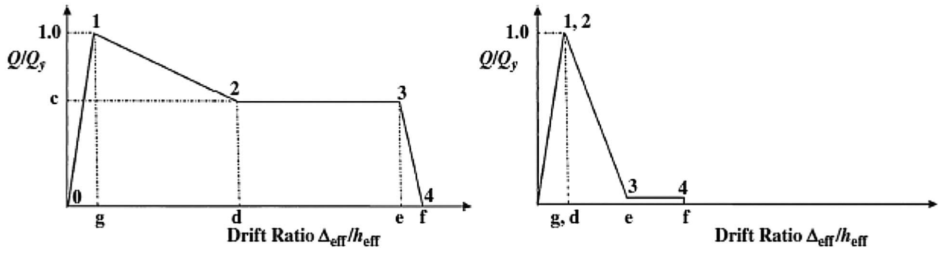
Force-controlled actions shall be as defined in Section 7.5.1. Strengths used in design for force-controlled actions are denoted Q_{CL} and shall be taken as equal to lower-bound strengths obtained experimentally, calculated using established mechanics principles, or based on default values listed in Section 11.2.3.10. Lower-bound strength is defined as the mean minus one standard deviation of resistance over the range of deformations and loading cycles to which the component is subjected. Where calculations are used to define lower-bound strengths, lower-bound material properties shall be used. It shall be permitted to calculate lower-bound properties from expected properties using the conversion factors in Table 11-1. Unless otherwise specified in this standard, use of strength design procedures specified in TMS 402 to calculate lower-bound strengths shall be permitted, except that the strength reduction factor, ϕ, shall be taken as equal to 1.0. Where alternative definitions of design strength are used, they shall be justified by experimental evidence.

Where design actions are determined using the nonlinear procedures of Chapter 7, component force–deformation response shall be represented by nonlinear force–deformation relations. Force–deformation relations shall be based on experimental evidence or the generalized force–deformation relations presented in Sections 11.3.2 and 11.3.4.

11.3.1 Types of Masonry Walls Masonry walls shall be categorized as unreinforced or reinforced; ungrouted, partially grouted, or fully grouted; and composite or noncomposite. Existing, new, or retrofitted masonry walls shall be capable of resisting forces applied parallel to their plane and normal to their plane, as described in Sections 11.3.2 through 11.3.5.

11.3.1.1 Existing Masonry Walls Existing masonry walls shall include all structural walls of a building system that are in place before seismic retrofit.

Existing masonry walls shall be assumed to behave in a manner consistent with new masonry walls, provided that the masonry is in fair or good condition as defined in this standard or has existing damage and weathering degradation repaired in accordance with Section 11.2.2. Masonry with existing damage or deterioration considered in poor condition shall be repaired in accordance with Section 11.2.2 before being considered as a primary or secondary component.



(a) (b)

Figure 11-1. Typical generalized force deformation relationships for unreinforced masonry: (a) deformation-controlled actions; and (b) force-controlled actions.

11.3.1.2 New Masonry Walls New masonry walls shall include all new wall elements added to an existing seismic-force-resisting system. New walls shall be designed in accordance with the requirements set forth in this standard and detailed and constructed in accordance with an approved building code.

11.3.1.3 Retrofitted Masonry Walls Retrofitted masonry walls shall include existing walls that are enhanced by an approved method.

11.3.2 Unreinforced Masonry Walls and Wall Piers Subject to In-Plane Actions Engineering properties of URM walls subjected to seismic forces applied parallel to their plane shall be determined in accordance with this section. Requirements of this section shall apply to cantilevered shear walls that are fixed against rotation at their base and to wall piers between window or door openings that are fixed against rotation top and bottom. Force–deformation relations shall be based on experimental evidence or the generalized force–deformation relations shown in Figure 11-1, with parameters *d*, *e*, and *f* as defined in Table 11-4. Materials that have brittle behavior, as shown in Figure 11-1b, shall be considered force-controlled actions.

URM walls have five primary in-plane actions. Deformation-controlled in-plane actions of URM walls include rocking and bed-joint sliding that includes stair-step cracking through head and bed joints. Force-controlled in-plane actions of URM walls include toe crushing, diagonal tension that causes cracking through the masonry units, and vertical compression.

11.3.2.1 Stiffness of URM Walls and Wall Piers Subject to In-Plane Actions The stiffness of URM walls subjected to seismic in-plane forces shall be determined considering flexural, shear, and axial deformations.

The masonry assemblage of units, mortar, and grout shall be considered to be a homogeneous medium for stiffness computations with an expected elastic modulus in compression, E_{me}, as specified in Section 11.2.3.4.

For linear procedures, the stiffness of a URM wall or wall pier resisting seismic forces parallel to its plane shall be considered to be linear and proportional with the geometrical properties of the uncracked section, excluding veneer wythes.

Story shears in perforated shear walls shall be distributed to wall piers in proportion to the relative lateral uncracked stiffness of each wall pier.

Stiffnesses for existing and retrofitted walls shall be determined using principles of mechanics used for new walls.

The stiffness of URM spandrel beams subjected to seismic in-plane forces shall be determined by accounting for the spandrel shear and flexural flexibility.

11.3.2.2 Strength of URM Walls Subject to In-Plane Actions Expected in-plane strength of URM walls shall be the lesser of rocking strength in Section 11.3.2.2.1 or bed-joint sliding strength in Section 11.3.2.2.2.

Lower-bound in-plane strength of URM walls shall be the lesser of toe-crushing strength in Section 11.3.2.2.3 or diagonal tension strength in Section 11.3.2.2.4. Lower-bound vertical compressive strength shall be determined in Section 11.3.2.2.5.

The latent onset of toe crushing for rocking walls and wall piers subjected to axial force and lateral deformation shall be considered in accordance with Sections 11.3.2.3.1 and 11.3.2.3.2. The effects of wall flanges, spandrels, and the vertical component of seismic loading shall be considered when determining in-plane strength.

11.3.2.2.1 Expected In-Plane Rocking Strength of URM Walls and Wall Piers Expected lateral strength, Q_{CE}, of URM walls or wall pier components shall be the expected rocking strength, calculated in accordance with Equation (11-9):

$$Q_{CE} = V_r = 0.9(\alpha P_D + 0.5P_w)L/h_{\text{eff}} \tag{11-9}$$

where

h_{eff} = Height to resultant of seismic force;
L = Length of wall or wall pier;
P_D = Superimposed dead load at the top of the wall or wall pier under consideration;
P_w = Self-weight of the wall pier;
V_r = Strength of wall or wall pier based on rocking; and
α = Factor equal to 0.5 for fixed-free cantilever wall, or equal to 1.0 for fixed-fixed wall pier.

11.3.2.2.2 Expected In-Plane Bed-Joint Sliding Strength of URM Walls and Wall Piers Expected initial lateral strength, Q_{CE}, of URM walls or pier components shall be calculated in accordance with Equation (11-10):

$$Q_{CE} = V_{bjs1} = v_{me} A_n \tag{11-10}$$

where

A_n = Area of net mortared or grouted section of a wall or wall pier,
v_{me} = Expected bed-joint sliding shear strength in accordance with Section 11.2.3.6, and
V_{bjs1} = Expected initial shear strength of wall or pier based on bed-joint sliding shear strength.

Expected final lateral strength, $Q_{CE,F}$, of URM walls or pier components shall be calculated in accordance with Equation (11-11):

$$Q_{CE,F} = V_{bjs2} = 0.5P_D \tag{11-11}$$

where

P_D = Superimposed dead load at top of the wall or pier under consideration, and
V_{bjs2} = Expected final shear strength of wall or pier based on bed-joint sliding shear strength.

11.3.2.2.3 Lower-Bound In-Plane Toe-Crushing Strength of URM Walls and Wall Piers. Lower-bound lateral strength, Q_{CL}, of URM walls or pier components shall be based on lower-bound toe crushing calculated in accordance with Equation (11-12):

$$Q_{CL} = V_{tc} = (\alpha P_D + 0.5P_W)\left(\frac{L}{h_{\text{eff}}}\right)\left(1 - \frac{f_a}{0.7f'_m}\right) \tag{11-12}$$

where

h_{eff}, L, and α are the same as given for Equation (11-9);
f_a = Axial compression stress caused by gravity loads specified in Equation (7-1);
f'_m = Lower-bound masonry compressive strength determined in accordance with Section 11.2.3.3;
P_D = Superimposed dead load at the top of the wall or wall pier under consideration;
P_W = Self-weight of the wall pier; and
V_{tc} = Lower-bound shear strength based on toe crushing for a wall or wall pier.

11.3.2.2.4 Lower-Bound In-Plane Diagonal Tension Strength of URM Walls and Wall Piers Lower-bound lateral strength, Q_{CL}, of URM walls or pier components shall be based on lower-bound diagonal tension calculated in accordance with Equation (11-13):

$$Q_{CL} = V_{dt} = f'_{dt} A_n \beta \sqrt{1 + \frac{f_a}{f'_{dt}}} \tag{11-13}$$

where

A_n = Area of net mortared and/or grouted section of a wall or wall pier;
β = 0.67 for $L/h_{\text{eff}} < 0.67$, L/h_{eff} when $0.67 \geq L/h_{\text{eff}} \leq 1.0$, and 1.0 when $L/h_{\text{eff}} > 1.0$;
h_{eff} = Height to resultant of seismic force;
L = Length of wall or wall pier;
f_a = Axial compression stress caused by gravity loads specified in Equation (7-1);
f'_{dt} = Lower-bound masonry diagonal tension strength; and
V_{dt} = Lower-bound shear strength based on diagonal tension stress for wall or pier.

Substitution of the lower-bound bed-joint shear strength, v_{mL}, for the diagonal tension strength, f'_{dt} in Equation (11-13) shall be permitted.

11.3.2.2.5 Lower-Bound Vertical Compressive Strength of URM Walls and Wall Piers. Lower-bound vertical compressive strength of URM walls or wall pier components shall be limited by lower-bound masonry compressive stress in accordance with Equation (11-14):

$$Q_{CL} = P_{CL} = 0.80(0.85f'_m A_n) \tag{11-14}$$

where

f'_m = Lower-bound compressive strength determined in accordance with Section 11.2.3.3, and
A_n = Area of net mortared and/or grouted section.

11.3.2.2.6 Expected Strengths of Rectangular URM Wall Spandrels Subject to In-Plane Actions. Expected in-plane strength of URM spandrels shall be the lesser of the flexural strength and shear strength.

In-plane strength of URM spandrels with and without timber, concrete, or steel lintels shall be determined as described in this section.

Peak flexural strength of rectangular URM spandrels with timber lintels shall be calculated in accordance with Equation (11-15) and Figure 11-2:

$$V_{f1} = (f_t + p_{sp})\frac{h_{sp}^2 b_{sp}}{3l_{sp}} \tag{11-15}$$

where

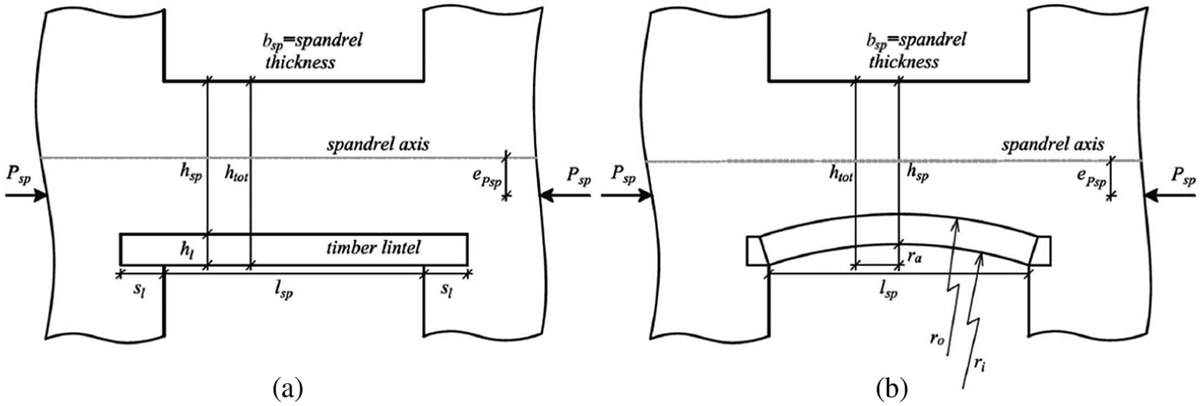
f_t = Equivalent tensile strength of masonry spandrel;
p_{sp} = Axial stress in the spandrel;
h_{sp} = Height of spandrel excluding depth of timber, concrete, or steel lintel, if present;
b_{sp} = Thickness of spandrel; and
l_{sp} = Clear length of spandrel between adjacent wall piers.

The equivalent tensile strength of masonry spandrel, f_t, is calculated by Equation (11-16):

$$f_t = \alpha_s(c_{bj} + 0.5\mu_f p_p) + \frac{c_{hj}}{2\mu_f} \tag{11-16}$$

where

α_s = Bond pattern factor taken as the ratio of the sum of horizontal crack length to the sum of the vertical crack length; for spandrels using common masonry units, α_s can be estimated as follows: running bond: α_s = 1.4; common bond: α_s = 1.2; English bond: α_s = 0.7; and stack bond: α_s = 0.0;



(a) (b)

Figure 11-2. Geometry of (a) spandrels with timber lintel and (b) shallow masonry arch.

Source: Beyer (2012); reproduced with permission from Elsevier.

c_{bj} = Joint shear strength index at zero normal compressive stress, or adhesion stress, derived from ASTM C1531;
μ_f = Masonry coefficient of friction, derived from ASTM C1531 tests;
p_p = Mean axial stress in the adjacent wall pier caused by superimposed dead load; and
c_{hj} = Masonry head joint cohesion, derived from ASTM C1531 tests with adjustments such that axial stress equals zero.

Residual flexural strength of rectangular URM spandrels shall be calculated in accordance with Equation (11-17):

$$V_{fl,r} = \frac{p_{sp} h_{sp}^2 b_{sp}}{l_{sp}} \left(1 - \frac{p_{sp}}{0.85 f_{hm}}\right) \tag{11-17}$$

where

p_{sp} = Axial stress in the spandrel; and
f_{hm} = Compression strength of the masonry in the horizontal direction. In lieu of tests to determine f_{hm}, f_{hm} is permitted to be assumed as $0.5f'_m$.

The peak shear strength, V_s, shall be computed as the lesser of Equation (11-18) or (11-19):

$$V_{s1} = \frac{2}{3}(c_{bj} + \mu_f p_{sp}) h_{sp} b_{sp} \tag{11-18}$$

$$V_{s2} = f'_{dt} \beta_{sp} h_{sp} b_{sp} \sqrt{1 + \frac{p_{sp}}{f'_{dt}}} \tag{11-19}$$

(Note for Public Comment: Line in Equation (11-19) will not be present in final version.)

where

f'_{dt} = Lower-bound masonry diagonal tension strength determined in accordance with Section 11.3.2.2.4; and
β_{sp} = 1.0 for $l_{sp}/h_{\text{sp}} < 1.0$ and 0.67 for $l_{sp}/h_{\text{sp}} > 1.5$, linearly interpolate for intermediate values of l_{sp}/h_{sp}.

Equation (11-18) is the peak shear strength associated with the formation of cracks through head and bed joints over almost the entire height of the spandrel and shall apply when the mortar is weaker than the masonry units. For the case when the mortar is stronger than the masonry units and fracture of the masonry units will occur, Equation (11-19) shall be used.

Residual shear strength of cracked rectangular URM spandrels with lintels shall be the lesser of Equation (11-20) (Figure 11-3) or the shear capacity of the lintel to resist the applied load determined using applicable provisions in the materials chapters.

$$V_{s,r} = \frac{11}{16} p_{sp} \frac{h_{sp}^2 b_{sp}}{l_{sp}} \tag{11-20}$$

When no lintel is present, the residual shear capacity of URM spandrels shall be zero.

11.3.2.2.7 Expected Strengths of URM Wall Spandrels with Shallow Arches Subject to In-Plane Actions. Arches are shallow when the half angle of embrace α_a satisfies Equation (11-21) where r_o, r_i, r_a, and l_{sp} are defined in Figure 11-2(b):

$$\cos \alpha_a \geq \frac{r_i}{r_o} \tag{11-21}$$

where

$$\alpha_a = \tan^{-1}\left(\frac{l_{sp}}{2(r_i - r_a)}\right) \tag{11-22}$$

Expected in-plane strength of URM spandrels shall be the lesser of the flexural strength and shear strength. Peak flexural capacity of a URM spandrel with a shallow arch shall be calculated in accordance with Equation (11-23):

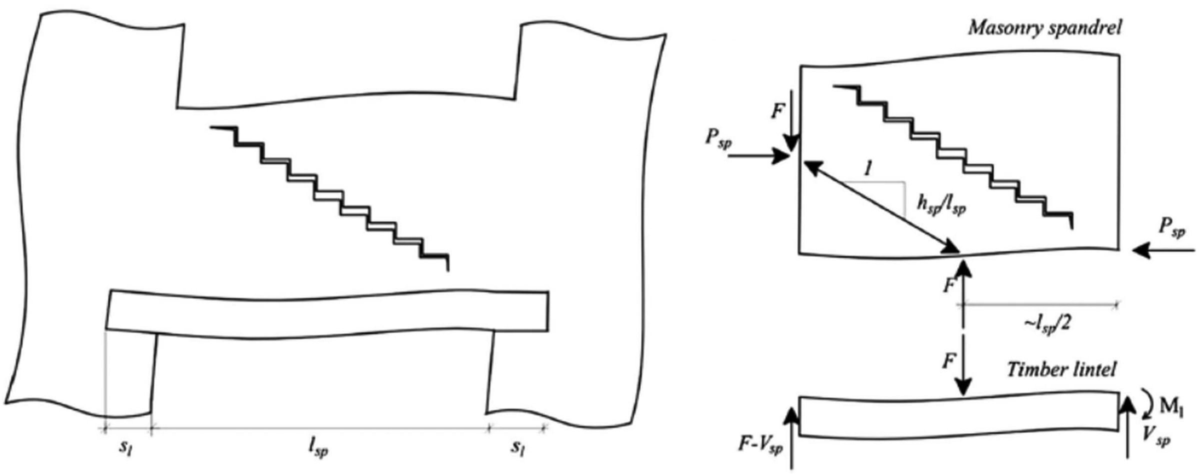
$$V_{fl} = h_{sp} b_{sp} \left(f_t \frac{h_{sp}}{3 l_{sp}} + p_{sp} \tan \alpha_a\right) \tag{11-23}$$

The residual flexural capacity of a URM spandrel with a shallow arch shall be calculated in accordance with Equation (11-24) and Figure 11-4:

$$V_{fl,r} = \frac{p_{sp} h_{sp} h_{\text{tot}} b_{sp}}{l_{sp}} \left(1 - \frac{p_{sp}}{0.85 f_{hm}}\right) \tag{11-24}$$

where dimension h_{tot} is defined in Figure 11-2(b).

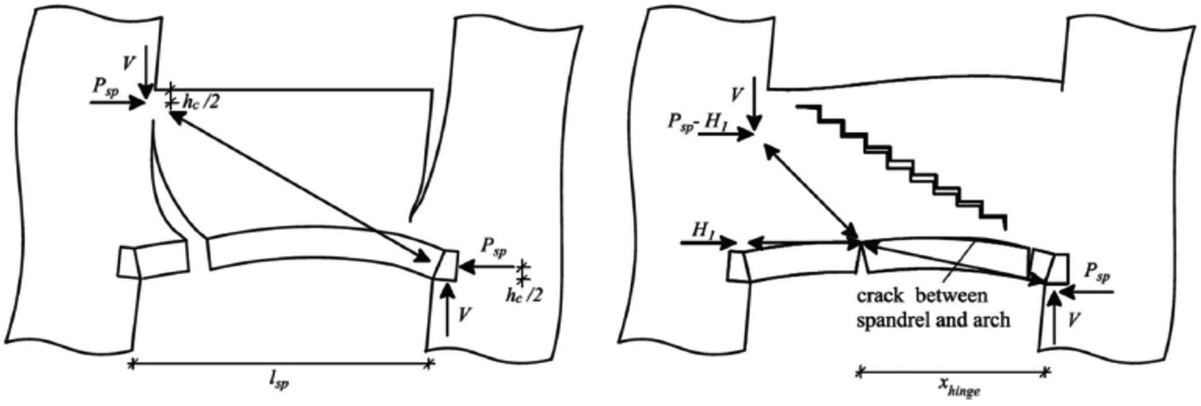
Peak shear strength, V_s, of a URM spandrel with a shallow arch shall be calculated using the lesser of Equation (11-25) or (11-26):



(a) (b)

Figure 11-3. Shear mechanism of URM spandrels with lintels.

Source: Beyer (2012); reproduced with permission from Elsevier.



(a) (b)

Figure 11-4. Spandrel with shallow arch. Assumed load transfer mechanism after (a) flexural and (b) shear cracking.
Source: Beyer (2012); reproduced with permission from Elsevier.

$$V_{s1} = h_{sp} b_{sp} \left[\frac{2}{3} (c_{bj} + \mu_f p_{sp}) + p_{sp} \tan \alpha_a \right] \qquad (11\text{-}25)$$

$$V_{s2} = h_{sp} b_{sp} \left(f'_{dt} \beta_{sp} \sqrt{1 + \frac{p_{sp}}{f'_{dt}}} + p_{sp} \tan \alpha_a \right) \qquad (11\text{-}26)$$

The residual capacity of the lintel shall be calculated in accordance with Equation (11-27):

$$V_{s,r} = h_{sp} b_{sp} p_{sp} \tan \alpha_a \qquad (11\text{-}27)$$

11.3.2.3 Acceptance Criteria for URM In-Plane Actions In-plane lateral shear of unreinforced masonry walls and wall piers in each line of resistance shall be considered a deformation-controlled action if the expected lateral rocking strength or bed-joint sliding strength of each wall or wall pier in the line of resistance, as specified in Sections 11.3.2.2.1 and 11.3.2.2.2, is less than the lower-bound lateral strength of each wall or wall pier limited by diagonal tension or toe crushing, as specified in Sections 11.3.2.2.3 and 11.3.2.2.4. URM walls that do not meet the criteria for deformation-controlled components shall be considered force-controlled components. Expected rocking strength, V_r, as specified in Section 11.3.2.2.1, shall be neglected in lines of resistance not considered deformation controlled. Axial compression on URM wall components shall be considered a force-controlled action.

11.3.2.3.1 Linear Procedures for In-Plane URM Wall Actions. For the linear procedures in Sections 7.4.1 and 7.4.2 component actions shall be compared with capacities in accordance with Section 7.5.2.2. When in-plane URM wall response is governed by bed-joint sliding, V_{bjs1} shall be used when assessing component behavior. The *m*-factors for use with corresponding expected strength shall be obtained from Table 11-3. If v_{tL} is less than 30 lb/in.^{2} (206.8 kPa), the wall or wall pier shall be classified as force controlled or repointed in accordance with Section 11.2.2.5 and retested in accordance with Section 11.2.3.6 to demonstrate that v_{tL} is greater than or equal to 30 lb/in.^{2} (206.8 kPa). Alternatively, *m*-factors for walls or wall piers with v_{tL} less than 30 lb/in.^{2} (206.8 kPa) shall be based on experimentally obtained response characteristics of representative wall subassemblies in accordance with Section 7.6.

For individual lines of resistance where all wall piers are considered deformation controlled for in-plane actions and classified as primary, it shall be permitted to redistribute forces between wall piers within the same line of resistance. Redistribution of forces from or to any individual wall pier shall not exceed 20% for Collapse Prevention, 15% for Life Safety, and 0% for Immediate Occupancy, using absolute values. The total demand resisted by the line of resistance shall remain unchanged. Diaphragms and collectors, including connections, shall be evaluated to transfer the redistributed forces to each wall pier.

11.3.2.3.2 Nonlinear Procedures for In-Plane URM Wall Actions. For the nonlinear static procedure (NSP) in Section 7.4.3 wall, pier, and spandrel components shall meet the requirements of Section 7.4.3.2. For deformation-controlled components, nonlinear deformations shall not exceed the values given in Table 11-4. If v_{tL} is less than 30 lb/in.^{2} (206.8 kPa), the wall or wall pier shall be classified as force controlled or repointed in accordance with Section 11.2.2.5 and retested in accordance with Section 11.2.3.6 to demonstrate that v_{tL} is greater than or equal to 30 lb/in.^{2} (206.8 kPa). Alternatively, *m*-factors for walls or walls piers with v_{tL} less than 30 lb/in.^{2} (206.8 kPa) shall be based on experimentally obtained response characteristics of representative wall subassemblies in accordance with Section 7.6. Variables d, e, and f, representing nonlinear deformation capacities, shall be expressed in terms of drift ratio percentages as defined in Figure 11-1a.

For the nonlinear dynamic procedure (NDP) given in Section 7.4.4 wall, pier, and spandrel components shall meet the requirements of Section 7.4.4.2. Nonlinear force–deflection relations for deformation-controlled wall, pier, and spandrel components shall be established based on the information given in Table 11-4, or an approved procedure based on a comprehensive evaluation of the hysteretic characteristics of those components.

11.3.3 Unreinforced Masonry Walls Subject to Out-of-Plane Actions

As required by Section 7.2.13 out-of-plane stability of URM walls shall be evaluated for out-of-plane inertial forces by considering components to span vertically between diaphragm levels when effective wall-to-diaphragm connections are present, or to span horizontally between intersecting walls, columns, or pilasters, or to span with two-way action. URM walls shall not be analyzed for out-of-plane actions using the linear static procedure (LSP) or NSP prescribed in Chapter 7.

11.3.3.1 Stiffness of URM Walls Subject to Out-of-Plane Actions The out-of-plane stiffness of walls shall be neglected in analytical models that consider the characteristics of the global

Table 11-3. Linear Procedure: *m*-Factors for URM In-Plane Walls, Wall Piers, and Spandrels.

Limiting Behavioral Mode	Performance Level				
		Primary		Secondary	
	IO	LS	CP	LS	CP
Wall and wall pier rocking[a,b,c]					
f_a/f'_m					
$\le 4\%$	$1 \le 1.5\ h_{\text{eff}}/L \le 1.5$	$1.5 \le 3\ h_{\text{eff}}/L \le 3.75$	$2 \le 4\ h_{\text{eff}}/L \le 5$	$2 \le 4\ h_{\text{eff}}/L \le 5$	$3 \le 6\ h_{\text{eff}}/L \le 8$
$\le 8\%$	1	$1 \le 1.5\ h_{\text{eff}}/L \le 1.9$	$1 \le 2\ h_{\text{eff}}/L \le 2.5$	$1 \le 2\ h_{\text{eff}}/L \le 2.5$	$1.5 \le 3\ h_{\text{eff}}/L \le 3$
Wall and wall pier bed-joint sliding	1	3	4	6	8
Spandrels with prismatic lintels	1	1.7	2.2	7.5	10
Spandrels with shallow arch lintels	1	1.7	2.2	4.2	5.6

[a] All rocking-controlled walls and wall piers shall comprise a minimum thickness of 6 in. (152 mm) and, for solid brick masonry, a minimum of two wythes. Multiwythe solid brick masonry walls and wall piers shall be connected with bonded solid headers.
[b] Linear interpolation shall be permitted for f_a/f'_m ratios between 4% and 8%.
[c] Walls and wall piers with f_a/f'_m ratios greater than 8% shall be considered force controlled, unless it can be demonstrated by analysis that toe crushing does not occur at the expected pier drift and the *m*-factor shall equal 1.0 or be substantiated in accordance with Section 7.6. Alternatively, nonlinear procedures and acceptance criteria shall be permitted in accordance with Section 11.3.2.3.2.
Note: IO = Immediate Occupancy, LS = Life Safety, CP = Collapse Prevention.

structural system that include in-plane wall actions in the direction of loading.

11.3.3.2 Strength of URM Walls Subject to Out-of-Plane Actions Unless arching action is considered, flexural cracking shall be limited by the lower-bound tensile stress values given in Section 11.2.3.5 for the Immediate Occupancy Structural Performance Level.

Arching action shall be considered only if surrounding floor, roof, column, or pilaster elements have sufficient stiffness and strength to resist thrusts from arching of a wall panel and a condition assessment has been performed to ensure that there are no gaps between a wall panel and the adjacent structure. The eccentricity of arching action shall be considered when evaluating wall behavior.

The condition of the collar joint shall be considered where estimating the effective thickness of a wall for out-of-plane behavior. The effective void ratio shall be taken as the ratio of the collar joint area without mortar to the total area of the collar joint. Wythes separated by collar joints that are not bonded or that have an effective void ratio greater than 50% shall not be considered part of the effective thickness of the wall for out-of-plane behavior. For cavity walls, the thickness of veneer shall not be considered part of the effective thickness of the wall for out-of-plane behavior, and transfer of out-of-plane forces from veneer to the backing wall shall be ensured by providing properly designed wall ties.

11.3.3.3 Acceptance Criteria for URM Walls Subject to Out-of-Plane Actions For the Immediate Occupancy Structural Performance Level, flexural cracking in URM walls caused by out-of-plane inertial loading shall not be permitted. Bed-joint flexural tensile strength shall be limited in accordance with Section 11.3.3.2 or Table 11-2a. If $v_{tL} \ge 30$ lb/in.^{2} (206.8 kPa), flexural cracking in URM walls caused by out-of-plane inertial loading shall be permitted for the Damage Control, Life Safety, Limited Safety, and Collapse Prevention Structural Performance Levels, provided that cracked wall segments remain stable during dynamic excitation. Equations (11-28a) through (11-28d) shall be used to assess the Structural Performance Levels other than Immediate Occupancy. A wall shall be considered as connected to *stiff diaphragms* if the most flexible diaphragm connected to the wall has a period $T_{\text{DIAPH}} \le 0.2$ s. A wall at a given story shall be considered as connected to *flexible diaphragms* if the most flexible diaphragm connected to the wall has a period $T_{\text{DIAPH}} \ge 0.5$ s. Linear interpolation of $S_{a\text{DIAPH}}$ (1), C_a, C_{pl}, C_{cw}, and C_g in Equation (11-28a) based on the diaphragm period shall be permitted for 0.5 s $> T_{\text{DIAPH}} >$ 0.2 s. Periods of the diaphragms shall be based on diaphragm stiffnesses and Chapter 7. Walls connected to rigid diaphragms shall use the values and equations for stiff diaphragms for Equations (11-28a) through (11-28d). Half the wall height (or any parapet for top-level walls) above and below the diaphragm in question shall be considered in calculation of tributary mass for the diaphragm period. For the purpose of Equation (11-28a), S_{X1} shall be permitted to be taken as the value of S_a at 1 s for the specified Seismic Hazard Level.

A cracked wall shall be considered stable during dynamic excitation if $h/t \le 8$ or

$$S_{X1} \le C_a C_t C_g C_{cw} C_{pl} S_{a\text{DIAPH}}(1) \tag{11-28a}$$

where

$$S_{a\text{DIAPH}}(1) = \begin{cases} \dfrac{4}{h/t} & \text{for stiff diaphragms} \\ \dfrac{1.8}{(h/t)^{0.75}} & \text{for flexible diaphragms} \end{cases} \tag{11-28b}$$

and

C_a = Modification factor for axial loads acting on the wall

Table 11-4. Nonlinear Procedures: Simplified Force–Deflection Relations for URM In-Plane Walls, Wall Piers, and Spandrels with $v_{tL} \geq 30$ lb/in.^{2} (206.8 kPa).

	Modeling Parameters				Acceptance Criteria, Performance Level			
Limiting Behavior Mode	**Residual Strength Ratio**	***d* (%)**	***e* (%)**	***f* (%)**		**IO (%)**	**LS (%)**	**CP (%)**
Wall and Wall Pier Rocking[a,b]	$V_{tc,r}/V_r$	$100\Delta_{tc,r}/h_{eff}$	$100\Delta_{tc,r}/h_{eff}$	$100(\Delta_{tc,r}+\Delta_y)/h_{eff}$	Simplified	0.1	$0.4\ h_{eff}/L$ but not greater than 1.50%[c]	$0.6\ h_{eff}/L$ but not greater than 2.25%[d]
					Comprehensive[d]	0.1	$0.6h_{eff}/L$ but not greater than 2.25%	$100\Delta_{tc,r}/h_{eff}$ but not greater than 2.5%
Wall and Wall Pier Bed-Joint Sliding	V_{bjs2}/V_{bjs1}[e]	0.4	1.0	$1.0+100\Delta_y/h$[f]		0.1	0.75	1.0
Spandrels with Prismatic Lintels	$\text{Min}(V_{fl,r}, V_{s,r})/\text{Min}(V_{fl}, V_s)$	0.3	3.0	3.1		0.1	2.25	3.0
Spandrels with Shallow Arch Lintels	$\text{Min}(V_{fl,r}, V_{s,r})/\text{Min}(V_{fl}, V_s)$	0.3	0.75	0.85		0.1	0.56	0.75

[a] Interpolation for wall piers shall be used between table values.

[b] All rocking walls and wall piers shall comprise a minimum thickness of 6 in. (152 mm) and, for solid brick masonry, a minimum of two wythes. Multiwythe solid brick walls and wall piers shall be connected with bonded solid headers. $V_{tc,r}$ is the seismic shear force associated with the onset of toe crushing after rocking initiates. The axial compressive stress on the toe caused by gravity loads, f_a, shall be based on the strain of the rocking pier and an equivalent compression zone of the effective net section of the rocking pier that is in bearing immediately before the onset of crushing, consistent with Section 9.3.2(g) of TMS 402, or some other analytical approach based on engineering mechanics and the stress–strain response of the materials that compose the pier and its interface with supporting components. $\Delta_{tc,r}$ is the lateral displacement associated with the onset of toe crushing $V_{tc,r}$.

[c] In no case shall the LS acceptance criteria exceed 0.75 times the Collapse Prevention acceptance criteria.

[d] Comprehensive acceptance criteria may be used if an analysis based on moment-curvature is used to explicitly calculate $\Delta_{tc,r}$.

[e] V_{bjs1} and V_{bjs2} shall be calculated in accordance with Section 11.3.2.2.2.

[f] Point F on the force–deformation curve where vertical-load-carrying capacity is diminished shall be based on the drift associated with no greater than one-half the width of the masonry units or units at the spring line of masonry arches, assuming that bed-joint sliding occurs entirely within one bed joint in a wall or pier.

Note: IO = Immediate Occupancy, LS = Life Safety, CP = Collapse Prevention.

$$= \begin{cases} 1 + C'_a(P_D/685) & \text{for } \dfrac{h}{t} < 8 \\ 1 + C'_a(P_D/685)\left(1 - \dfrac{1}{12}\left(\dfrac{h}{t} - 8\right)\right) & \text{for } 8 \leq \dfrac{h}{t} \leq 20; \\ 1 \text{ for } \dfrac{h}{t} > 20 \end{cases} \tag{11-28c}$$

where

$$C'_A = \begin{cases} 0.5 \text{ for stiff diaphragms} \\ 0.2 \text{ for flexible diaphragms} \end{cases};$$

P_D = vertical load acting on the wall in lb/ft (not including the self-weight of the wall at the story under consideration);

$$C_t = \text{Modification factor for thin walls} = 0.2 + \frac{t}{15.7} \leq 1.0 \tag{11-28d}$$

where t is wall thickness (in.);

C_g = Modification factor for ground-level walls;

$$= \begin{cases} 1.0 \text{ for stiff diaphragms} \\ 1.1 \text{ for flexible diaphragms} \end{cases};$$

C_g = 1.0 for walls not at ground level;

C_{cw} = Modification factor for cross walls. Cross walls shall not be spaced more than 40 ft (12.2 m) on center, measured perpendicular to the direction under consideration, and shall extend the full story height between diaphragms. Cross walls shall be located on the interior of the building and have a length-to-height ratio between openings equal to or greater than 1.5;

$$\begin{cases} 1.0 \text{ for stiff diaphragms or bare steel deck diaphragms, with or without cross walls in direction evaluating} \\ 1.0 \text{ for flexible, wood diaphragms with no cross walls in direction evaluating} \\ 1.25 \text{ for flexible, wood diaphragms with at least one cross wall in direction evaluating} \end{cases} \tag{11-28e}$$

and

C_{pl} = Modification factor for Performance Level per Table 11-5.

11.3.4 Reinforced Masonry Walls and Wall Piers In-Plane Actions Provisions in this section shall be applied to both partially and fully grouted RM walls constructed of hollow concrete or clay units unless stated otherwise. Unless supported by experimental evidence, provisions in this section shall not be applied to reinforced clay brick cavity walls.

Design actions (flexure, shear, and axial) on RM wall and wall pier components shall be determined in accordance with Chapter 7 of this standard. The expected flexural strength of an RM wall or wall pier shall be determined based on the strength design method specified in TMS 402 or using the simplified method presented in Section 11.3.4.3. The expected and lower-bound shear strengths of RM wall or wall pier components constructed from units with bed-joint reinforcing or hollow units shall be determined based on strength design procedures specified in TMS 402 Section 9.3. The expected and lower-bound shear strength of an RM wall or wall pier constructed from solid units without bed-joint reinforcing shall be determined based on strength design procedures specified in TMS 402 Section 9.2.6.

The wall component shall be considered flexure-governed if the expected shear strength is greater than the shear required to develop the expected flexural strength. However, if the expected shear strength is greater than the shear required to develop the expected flexural strength but less than 1.4 times the latter, both flexure-governed and shear-governed conditions shall be checked, and the least favorable condition shall govern. Otherwise, the wall component shall be considered shear-governed.

RM wall components shall be considered deformation controlled except for conditions stated in Section 11.3.4.6. The lateral force–deformation relations for flexure- or shear-governed wall components shall be based on experimental data or the generalized force–deformation curves shown in Figure 11-5, where $Q_{\max}$ shall be the expected strength Q_{CE}, h_{eff} is the effective height of a wall component, and Δ_{eff} is the differential displacement between the top and bottom of a wall with an effective height, h_{eff}. The elastic stiffness, and the values of the critical force and deformation parameters for the force–deformation curves shall be determined in accordance with Sections 11.3.4.2 through 11.3.4.4. Veneer wythes shall not be considered in the calculation of wall component properties.

11.3.4.1 Reinforced Masonry Walls and Wall Piers with Flanged Sections Full flange action shall be considered for RM walls and wall piers when the wall intersections are effective in transferring shear with the satisfaction of either Condition 1 or 2, and Condition 3. Otherwise, shear transfer shall be ignored.

Table 11-5. C_{pl}, Performance Modification Factor for URM Subject to Out-of-Plane Actions.

Performance Levels	Stiff Diaphragms	Flexible Diaphragms
Damage Control	0.8	0.8
Life Safety	0.9	0.9
Limited Safety	1.0	1.0
Collapse Prevention	1.15	1.1

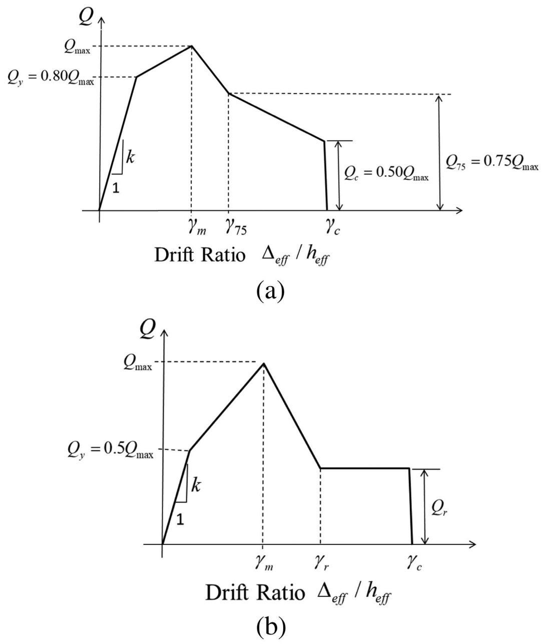


Figure 11-5. Generalized force–deformation relationships for RM walls: (a) flexure-governed walls; and (b) shear-governed walls.

1. The face shells of hollow masonry units at the intersection are removed and the intersection is fully grouted;
2. Units are laid in running bond, and 50% of the masonry units at the intersection are interlocked; and
3. Intersecting reinforced bond beams are provided at a vertical spacing not greater than 48 in. (1.2 m) on center with reinforcement fully developed on each side of the intersection; or the intersecting walls are connected with steel connectors meeting the requirements in TMS 402.

The width of flange considered effective in tension or compression on each side of the web shall be determined in accordance with the provisions in TMS 402.

11.3.4.2 In-Plane Lateral Stiffness of Reinforced Masonry Walls and Wall Piers The elastic lateral stiffness of a wall component, k, as shown in Figure 11-5, shall be calculated based on accepted mechanics principles, taking into account the effect of masonry cracking. The use of Equation (11-29) shall be permitted for the calculation of the stiffness value:

$$k = \frac{1}{\frac{h_{\text{eff}}^2}{3\zeta_f E_m I_g} + \frac{1}{\zeta_v G_m A_v}} \qquad \text{(11-29)}$$

where stiffness is expressed in terms of the lateral force per unit wall drift ratio ($\Delta_{\text{eff}}/h_{\text{eff}}$);

E_m and G_m = Modulus of elasticity and shear modulus of masonry determined in accordance with Sections 11.2.3.4 and 11.2.3.7, respectively, using the expected masonry compressive strength, f_{me}; for partially grouted walls, f_{me} shall be the weighted average of the expected strengths of grouted and ungrouted masonry based on the proportions of grouted and ungrouted masonry areas in the wall section; the expected strength of grouted or ungrouted masonry shall be determined according to Section 11.2.3, except that it shall be based on the gross cross-sectional area of a prism rather than the net area;

ζ_f and ζ_v = Reduction factors for the flexural and shear stiffness terms to account for masonry cracking; their values shall be determined in accordance with Sections 11.3.4.3.1 through 11.3.4.3.3 for flexure-governed walls with rectangular and flanged sections, and Section 11.3.4.4 for shear-governed walls;

I_g = Moment of inertia of gross wall cross section about the strong axis of bending; for flanged walls, it shall be based on the effective flange width determined in accordance with TMS 402; and

A_v = Effective gross cross-sectional area of wall for resisting shear; for rectangular sections, it is equal to 5/6 of the gross cross-sectional area; for flanged sections, it is equal to the gross cross-sectional area of the web.

The gross cross-sectional area of a wall is the total cross-sectional area including the cavities in a partially grouted wall.

If the value of k calculated with Equation (11-29) is less than $Q_{\max}/\gamma_m$, it shall be equal to $Q_{\max}/\gamma_m$.

11.3.4.3 Flexure-Governed In-Plane Actions of Reinforced Masonry Walls and Wall Piers The lateral force–deformation response of a flexure-governed RM wall component shall be determined with a model that accounts for the nonlinear behavior of the materials, including the tensile fracture and compressive crushing of masonry, the yielding of the flexural reinforcement, the possible buckling of the reinforcement in regions where masonry spalling occurs, and the possible fracture of the reinforcement owing to low-cycle fatigue. In lieu of a detailed model that accounts for the aforementioned phenomena, the use of the force–deformation curve shown in Figure 11-5a shall be permitted. The critical drift ratios required to determine the curve shall be calculated using Equations (11-30a) through (11-30c) as follows:

$$\gamma_m = \gamma_{fm} + \gamma_{vm} \tag{11-30a}$$

$$\gamma_{75} = \gamma_{f75} + \gamma_{v75} \tag{11-30b}$$

$$\gamma_c = \gamma_{fc} + \gamma_{vc} \tag{11-30c}$$

The drift ratios γ_m, γ_{75}, and γ_c calculated according to Equation (11-30) shall not exceed the following limits:

$$\gamma_m \le 0.01; \quad \gamma_{75} \le 0.02; \quad \gamma_c \le 0.04$$

The drift ratios contributed by flexure shall be calculated with Equations (11-31a) through (11-31c):

$$\gamma_{fm} = \frac{M_{\max}}{\zeta_f E_m I_g}\frac{h_{eff}}{3} + \left(\phi_m - \frac{M_{\max}}{\zeta_f E_m I_g}\right)\frac{L_p}{h_{eff}}\left(h_{eff} - \frac{L_p}{2}\right) \tag{11-31a}$$

$$\gamma_{f75} = \frac{M_{75}}{\zeta_f E_m I_g}\frac{h_{eff}}{3} + \left(\phi_{75} - \frac{M_{75}}{\zeta_f E_m I_g}\right)\frac{L_p}{h_{eff}}\left(h_{eff} - \frac{L_p}{2}\right) \tag{11-31b}$$

$$\gamma_{fc} = \frac{M_c}{\zeta_f E_m I_g}\frac{h_{eff}}{3} + \left(\phi_c - \frac{M_c}{\zeta_f E_m I_g}\right)\frac{L_p}{h_{eff}}\left(h_{eff} - \frac{L_p}{2}\right) \tag{11-31c}$$

where:

L_p = $0.2h_{eff}$;

$M_{\max}$ = expected moment capacity of the wall section at which the plastic hinge develops; $M_{75} = 0.75M_{\max}$, $M_c = 0.50M_{\max}$; and

ϕ_m, ϕ_{75}, and ϕ_c are wall curvatures at which the moment reaches $M_{\max}$, M_{75}, and M_c, respectively.

The drift ratio contributed by the shear deformation of the wall component shall be calculated with Equations (11-32a) through (11-32c):

$$\gamma_{vm} = \frac{Q_{\max}}{\zeta_v (G_m A_v)} \tag{11-32a}$$

$$\gamma_{v75} = \frac{Q_{75}}{\zeta_v (G_m A_v)} \tag{11-32b}$$

$$\gamma_{vc} = \frac{Q_c}{\zeta_v (G_m A_v)} \tag{11-32c}$$

where the values of $Q_{\max}$, Q_{75}, and Q_c shall be determined from the respective moment capacities $M_{\max}$, M_{75}, and M_c of the wall component based on statics.

The values of ϕ_m, ϕ_{75}, and ϕ_c shall be determined from the nondimensionalized values calculated according to Table 11-6, with $\bar{\omega}$, σ_a, and η_f defined in Equation (11-33a) through (11-33c):

$$\bar{\omega} = \frac{f_{ye}}{f_{me}}\rho_{f,\text{web}} \tag{11-33a}$$

$$\sigma_a = \frac{P}{f_{me} l_w t'} \tag{11-33b}$$

$$\eta_f = \frac{A_{s,\text{flange}}}{A_{s,\text{web}}} \tag{11-33c}$$

where

l_w = Length of the wall;

t' = Effective width of the wall section as defined in Sections 11.3.4.3.1 through 11.3.4.3.3 for walls with rectangular and flanged sections;

$\rho_{f,\text{web}}$ = Ratio of the cross-sectional area of the flexural reinforcement located within the wall length l_w and web width t_w, to the cross-sectional area of the equivalent rectangular section, $l_w \times t'$;

P = Axial compressive force in the wall;

f_{ye} = Expected yield strength of the flexural reinforcement;

f_{me} = Expected compressive strength of the masonry; for partially grouted walls, it shall be the weighted average of the expected strengths of grouted and ungrouted masonry based on the proportions of grouted and ungrouted masonry areas in the wall section as specified in Section 11.3.4.2;

$A_{s,\text{flange}}$ = Area of flexural reinforcement located within the effective width of the flange in tension oustide the web area; and

$A_{s,\text{web}}$ = Area of the flexural reinforcement located within the web.

The value of $M_{\max}$ shall be determined in accordance with the strength design provisions in TMS 402 based on the expected material strengths or shall be calculated with the nondimensionalized moment $M'_{\max}$ determined according to Table 11-6 using Equation (11-34):

$$M_{\max} = f_{me} l_w^2 t' M'_{\max} + Pe \tag{11-34}$$

where e is zero for walls with rectangular sections and is equal to the distance between the centroid of the actual section and the centroid of the equivalent rectangular section for flanged walls, having a positive sign when the edge closer to the centroid of the flanged section is in compression and a negative sign otherwise. For determining e, the actual physical flange width shall be considered.

11.3.4.3.1 Reinforced Masonry Walls and Wall Piers with Rectangular Sections For RM wall components with rectangular

Table 11-6. Calculation of Nondimensionalized Curvature and Moment Capacity Values for Flexure-Governed RM Walls.[a]

$\bar{\alpha} = \bar{\omega}(1 + 3.0\eta_f)$ or $\bar{\bar{\alpha}} = \bar{\omega}(1 + 2.2\eta_f)$	$\sigma_a = \frac{P}{f_{me} l_w t'}$	$\phi_m l_w = a_m \sigma_a + b_m$		$\phi_{75} l_w = a_{75} \sigma_a + b_{75}$		$\phi_c l_w = a_c \sigma_a + b_c$		$M'_{\max} = a_M \sigma_a^2 + b_M \sigma_a + c_M$		
		a_m	b_m	a_{75}	b_{75}	a_c	b_c	a_M	b_M	c_M
0.001–0.05	0.00–0.10	$-0.57 + 3.4\bar{\alpha}$	$0.080 - 0.400\bar{\alpha}$	$-0.47 + 0.50\bar{\alpha}$	$0.100 - 0.160\bar{\alpha}$	$-0.50 + 2.0\bar{\alpha}$	$0.120 - 0.350\bar{\alpha}$	$2.450\bar{\bar{\alpha}}$ -0.5507	$-1.719\bar{\bar{\alpha}}$ $+0.4995$	$0.5694\bar{\bar{\alpha}}$
	0.10–0.15	$-0.09 + 0.28\bar{\alpha}$	$0.030 - 0.070\bar{\alpha}$	$-0.47 + 0.50\bar{\alpha}$	$0.100 - 0.160\bar{\alpha}$	$-0.60 + 2.0\bar{\alpha}$	$0.130 - 0.350\bar{\alpha}$			
	0.15–0.25	$-0.09 + 0.28\bar{\alpha}$	$0.030 - 0.070\bar{\alpha}$	$-0.14 + 0.40\bar{\alpha}$	$0.050 - 0.145\bar{\alpha}$	$-0.20 + 0.05\bar{\alpha}$	$0.070 - 0.057\bar{\alpha}$			
0.05–0.10	0.00–0.10	$-0.77 + 5.9\bar{\alpha}$	$0.100 - 0.650\bar{\alpha}$	$-0.62 + 3.5\bar{\alpha}$	$0.122 - 0.600\bar{\alpha}$	$-0.60 + 4.0\bar{\alpha}$	$0.137 - 0.700\bar{\alpha}$	$0.1247\bar{\bar{\alpha}}$ -0.4344	$-0.5867\bar{\bar{\alpha}}$ $+0.4429$	$0.4127\bar{\bar{\alpha}}$ $+0.0079$
	0.10–0.15	$-0.09 + 0.28\bar{\alpha}$	$0.030 - 0.070\bar{\alpha}$	$-0.62 + 3.5\bar{\alpha}$	$0.122 - 0.600\bar{\alpha}$	$-0.80 + 6.0\bar{\alpha}$	$0.157 - 0.900\bar{\alpha}$			
	0.15–0.25	$-0.09 + 0.28\bar{\alpha}$	$0.030 - 0.070\bar{\alpha}$	$-0.26 + 2.0\bar{\alpha}$	$0.078 - 0.500\bar{\alpha}$	$-0.20 + 0.02\bar{\alpha}$	$0.067 - 0.005\bar{\alpha}$			
0.10–0.20	0.00–0.10	$-0.36 + 1.3\bar{\alpha}$	$0.057 - 0.170\bar{\alpha}$	$-0.29 + 0.25\bar{\alpha}$	$0.065 - 0.040\bar{\alpha}$	$-0.21 + 0.10\bar{\alpha}$	$0.071 - 0.045\bar{\alpha}$	$0.1247\bar{\bar{\alpha}}$ -0.4344	$-0.5867\bar{\bar{\alpha}}$ $+0.4429$	$0.4127\bar{\bar{\alpha}}$ $+0.0079$
	0.10–0.15	$-0.09 + 0.28\bar{\alpha}$	$0.030 - 0.070\bar{\alpha}$	$-0.29 + 0.25\bar{\alpha}$	$0.065 - 0.040\bar{\alpha}$	$-0.21 + 0.10\bar{\alpha}$	$0.071 - 0.045\bar{\alpha}$			
	0.15–0.25	$-0.09 + 0.28\bar{\alpha}$	$0.030 - 0.070\bar{\alpha}$	$-0.13 + 0.40\bar{\alpha}$	$0.045 - 0.100\bar{\alpha}$	$-0.21 + 0.10\bar{\alpha}$	$0.071 - 0.045\bar{\alpha}$			
0.20–0.50	0.00–0.10	$-0.24 + 0.40\bar{\alpha}$	$0.039 - 0.050\bar{\alpha}$	$-0.28 + 0.25\bar{\alpha}$	$0.068 - 0.056\bar{\alpha}$	$-0.20 + 0.05\bar{\alpha}$	$0.065 - 0.015\bar{\alpha}$	-0.4000	$-0.3906\bar{\bar{\alpha}}$ $+0.4038$	$0.3405\bar{\bar{\alpha}}$ $+0.0211$
	0.10–0.15	$-0.045 + 0.05\bar{\alpha}$	$0.019 - 0.014\bar{\alpha}$	$-0.28 + 0.25\bar{\alpha}$	$0.068 - 0.056\bar{\alpha}$	$-0.20 + 0.05\bar{\alpha}$	$0.065 - 0.015\bar{\alpha}$			
	0.15–0.25	$-0.045 + 0.05\bar{\alpha}$	$0.019 - 0.014\bar{\alpha}$	$-0.06 + 0.04\bar{\alpha}$	$0.032 - 0.017\bar{\alpha}$	$-0.20 + 0.05\bar{\alpha}$	$0.065 - 0.015\bar{\alpha}$			
0.50–1.00	0.00–0.10	$-0.065 + 0.045\bar{\alpha}$	$0.016 - 0.006\bar{\alpha}$	$-0.24 + 0.20\bar{\alpha}$	$0.053 - 0.034\bar{\alpha}$	$-0.28 + 0.21\bar{\alpha}$	$0.087 - 0.060\bar{\alpha}$	$-2.627\bar{\bar{\alpha}}$ $+0.9559$	0.2000	$0.2793\bar{\bar{\alpha}}$ $+0.0498$
	0.10–0.15	$-0.022 + 0.010\bar{\alpha}$	$0.012 - 0.003\bar{\alpha}$	$-0.24 + 0.20\bar{\alpha}$	$0.053 - 0.034\bar{\alpha}$	$-0.28 + 0.21\bar{\alpha}$	$0.087 - 0.060\bar{\alpha}$			
	0.15–0.25	$-0.022 + 0.010\bar{\alpha}$	$0.012 - 0.003\bar{\alpha}$	$-0.04 + 0.01\bar{\alpha}$	$0.027 - 0.011\bar{\alpha}$	$-0.28 + 0.21\bar{\alpha}$	$0.087 - 0.060\bar{\alpha}$			
1.00–1.50	0.00–0.10	$-0.025 + 0.012\bar{\alpha}$	$0.012 - 0.003\bar{\alpha}$	$-0.07 + 0.04\bar{\alpha}$	$0.027 - 0.010\bar{\alpha}$	$-0.15 + 0.08\bar{\alpha}$	$0.047 - 0.020\bar{\alpha}$	0.0000	$-0.0695\bar{\bar{\alpha}}$ -0.2236	$0.0870\bar{\bar{\alpha}}$ $+0.2474$
	0.10–0.15	$-0.025 + 0.012\bar{\alpha}$	$0.012 - 0.003\bar{\alpha}$	$-0.07 + 0.04\bar{\alpha}$	$0.027 - 0.010\bar{\alpha}$	$-0.15 + 0.08\bar{\alpha}$	$0.047 - 0.020\bar{\alpha}$			
	0.15–0.25	$-0.025 + 0.012\bar{\alpha}$	$0.012 - 0.003\bar{\alpha}$	$-0.07 + 0.04\bar{\alpha}$	$0.027 - 0.010\bar{\alpha}$	$-0.15 + 0.08\bar{\alpha}$	$0.047 - 0.020\bar{\alpha}$			
1.50–2.00	0.00–0.10	$-0.025 + 0.012\bar{\alpha}$	$0.012 - 0.003\bar{\alpha}$	$-0.02 + 0.006\bar{\alpha}$	$0.014 - 0.002\bar{\alpha}$	$-0.06 + 0.02\bar{\alpha}$	$0.020 - 0.002\bar{\alpha}$	0.0000	$-0.0695\bar{\bar{\alpha}}$ -0.2236	$0.0870\bar{\bar{\alpha}}$ $+0.2474$
	0.10–0.15	$-0.025 + 0.012\bar{\alpha}$	$0.012 - 0.003\bar{\alpha}$	$-0.02 + 0.006\bar{\alpha}$	$0.014 - 0.002\bar{\alpha}$	$-0.06 + 0.02\bar{\alpha}$	$0.020 - 0.002\bar{\alpha}$			
	0.15–0.25	$-0.025 + 0.012\bar{\alpha}$	$0.012 - 0.003\bar{\alpha}$	$-0.02 + 0.006\bar{\alpha}$	$0.014 - 0.002\bar{\alpha}$	$-0.06 + 0.02\bar{\alpha}$	$0.020 - 0.002\bar{\alpha}$			

[a] $\bar{\omega} = \frac{f_{ye}}{f_{me}} \rho_{f,\text{web}}$; $\eta_f = \frac{A_{s,\text{flange}}}{A_{s,\text{web}}}$; $\rho_{f;\text{web}} = \frac{A_{s,\text{web}}}{l_w t'}$

sections, the effective width $t' = t_w$ and $\eta_f = 0$; and it shall be permitted that ζ_f and ζ_v in Equations (11-29), (11-31), and (11-32) be both equal to 0.25.

11.3.4.3.2 Reinforced Masonry Walls and Wall Piers with One Flange For RM walls with flange at one end only and the wall intersection meeting the conditions in Section 11.3.4.1, the force-displacement relation shall be determined as follows.

For the loading direction in which the extreme edge of the flange is in compression, it shall be permitted that ζ_f and ζ_v in Equations (11-29), (11-31), and (11-32) be both equal to 0.25.

For the loading direction in which the flange is in tension, it shall be permitted that ζ_f and ζ_v be equal to 0.5 and 0.25, respectively.

The values of ϕ_m, ϕ_{75}, ϕ_c, and $M'_{\max}$ shall be calculated according to Table 11-6 with the values of $\bar{\omega}$ and σ_a calculated with Equations (11-33a) and (11-33b) based on equivalent rectangular sections of length l_w and effective width t' as follows:

1. For the loading direction in which the extreme edge of the flange is in compression, t' = the effective flange width determined in accordance with TMS 402, including the web width, t_w, and the value of η_f shall be 0. If the effective flange width is less than five times t_w, then $t' = t_w$.
2. For the loading direction in which the extreme edge of the flange is in tension, $t' = t_w$, and η_f shall be determined with Equation (11-33c).

11.3.4.3.3 Reinforced Masonry Walls and Wall Piers with Two Flanges For RM walls with flanges at both ends and the wall intersections meeting the conditions in Section 11.3.4.1, it shall be permitted that ζ_f and ζ_v in Equations (11-29), (11-31), and (11-32) be equal to 0.50 and 0.25, respectively.

The values of ϕ_m, ϕ_{75}, ϕ_c, and $M'_{\max}$ shall be calculated according to Table 11-6 with the values of $\bar{\omega}$, and σ_a calculated with Equations (11-33a) and (11-33b) based on equivalent rectangular sections of length l_w and width t', where t' is the the total effective width of the flange in compression determined in accordance with Section 11.3.4.1, including the width of the web, t_w. The value of η_f shall be determined with Equation (11-33c). If the total effective flange width in compression is less than five times t_w, $t' = t_w$.

Table 11-7. Critical Drift Ratios for Shear-Governed RM Walls.

	γ_m	γ_r	γ_c
Fully grouted walls	0.005	0.01	0.02
Partially grouted walls	0.002	0.004	0.008

11.3.4.4 Shear-Governed In-Plane Actions of Reinforced Masonry Walls and Wall Piers For RM walls and wall piers with behavior governed by shear, the force–deformation response shall be determined by the generalized backbone curve shown in Figure 11-5b. The elastic lateral stiffness, k, shall be determined in accordance with Section 11.3.4.2. It shall be permitted that ζ_f and ζ_v in Equations (11-29), (11-31), and (11-32) be equal to 0.5 and 0.25, respectively. The shear strength, $Q_{\max}$, and residual strength, Q_r, of a wall component shall be equal to the total shear strength and the strength provided by the shear reinforcement, respectively, determined in accordance with the strength design provisions in TMS 402 using the expected strengths of the masonry and shear reinforcement, without the strength reduction factor. $Q_{\max}$ shall be equal to Q_r if the wall component is subjected to tension. The critical drift ratios shall have the values shown in Table 11-7.

11.3.4.5 Vertical Compressive Strength of Walls and Wall Piers Lower-bound vertical compressive strength of existing RM wall or wall pier components shall be determined based on strength design provisions in TMS 402.

11.3.4.6 Acceptance Criteria for In-Plane Actions of Reinforced Masonry Walls and Wall Piers For RM wall components governed by flexure, flexural actions shall be considered deformation controlled, except for the conditions specified in this section. For RM components governed by shear, shear actions shall be considered deformation controlled, except for the conditions specified in this section. Axial compression on RM wall or wall pier components shall be considered a force-controlled action.

11.3.4.6.1 Linear Procedures for In-Plane Actions of Reinforced Masonry Walls For the linear procedures of Sections 7.4.1 and 7.4.2, component actions shall be compared with capacities in accordance with Section 7.5.2.2. The m-factors for use in Equation (7-39) for flexure-governed wall components shall be calculated with the equations in Table 11-8. Flexure-governed walls with $(3\sigma_a + \bar{\omega}) > 0.6$ shall be considered force controlled.

The values of E_m, I_g, ϕ_m, ϕ_{75}, ϕ_c, and $M_{\max}$ in Table 11-8 shall be determined in accordance with Sections 11.3.4.2 and 11.3.4.3.

The m-factors for shear-governed wall components are given in Table 11-9. For walls governed by shear, the axial stress on the

Table 11-8. *m*-Factors for In-Plane Flexure-Governed RM Walls.

Performance Levels			
	IO	LS	CP
Primary Components	$1.0 \le m = \dfrac{0.05\phi_m E_m I_g}{M_{\max}} \le 2.0$	$1.5 \le m = \dfrac{0.10\phi_{75} E_m I_g}{M_{\max}} \le 4.5$	$1.5 \le m = \dfrac{0.10\phi_c E_m I_g}{M_{\max}} \le 6.0$
Secondary Components		$1.5 \le m = \dfrac{0.10\phi_{75} E_m I_g}{M_{\max}} \le 6.0$	$1.5 \le m = \dfrac{0.10\phi_c E_m I_g}{M_{\max}} \le 8.0$

Note: IO = Immediate Occupancy, LS = Life Safety, CP = Collapse Prevention.

Table 11-9. *m*-Factors for In-Plane Shear-Governed RM Walls.

	Primary and Secondary Components Performance Levels		
	IO	LS	CP
Fully grouted walls	1.0	2.0	3.0
Partially grouted walls	1.0	1.5	2.0

Note: IO = Immediate Occupancy, LS = Life Safety, CP = Collapse Prevention.

member shall be less than or equal to $0.15f_{me}$ when the actual height-to-length ratio of the wall is greater than 0.50, and be less than or equal to $0.3f_{me}$ when the actual height-to-length ratio of the wall is less than or equal to 0.50; otherwise, the component shall be treated as force controlled.

11.3.4.6.2 Nonlinear Procedures for In-Plane Actions of Reinforced Masonry Walls For the NSP of Section 7.4.3 wall and wall pier components shall meet the requirements of Section 7.5.3.2. For the NDP of Section 7.4.4, wall and wall pier components shall meet the requirements of Section 7.5.3.2.

For flexure-governed wall components, the nonlinear lateral force–deformation relations shall be established using the curve given in Figure 11-5a in accordance with the procedures given in Section 11.3.4.3, or an approved procedure based on comprehensive evaluation of the hysteretic characteristics of those components. In-plane drift ratios of flexure-governed wall components shall not exceed the limits given in Table 11-10 where the values of γ_m, γ_{75}, and γ_c shall be determined in accordance with Section 11.3.4.3.

Flexure-governed walls with $(3\sigma_a + \bar{\omega}) > 0.6$ shall be considered force controlled.

For shear-governed wall components, the nonlinear lateral force–deflection relations shall be established using the curve given in Figure 11-5b in accordance with the procedures given in Section 11.3.4.4, or an approved procedure based on comprehensive evaluation of the hysteretic characteristics of those components. In-plane drift ratios of shear-governed wall components shall not exceed the limits given in Table 11-11, where the values of γ_m, γ_r, and γ_c shall be determined in accordance with Section 11.3.4.4.

For wall components governed by shear, the axial stress on the member shall be less than or equal to $0.15f_{me}$ when the actual height-to-length ratio of the wall is greater than 0.50, and be less than or equal to $0.3f_{me}$ when the actual height-to-length ratio of the wall is less than or equal to 0.50; otherwise, the component shall be treated as force controlled.

Table 11-10. Acceptable Drift Ratios for In-Plane Flexure-Governed RM Walls.

Acceptable Drift Ratios for Different Performance Levels		
IO	LS	CP
γ_m	γ_{75}	γ_c

Note: IO = Immediate Occupancy, LS = Life Safety, CP = Collapse Prevention.

Table 11-11. Drift Ratio Limits for In-Plane Shear-Governed RM Walls.

Acceptable Drift Ratios for Different Performance Levels		
IO	LS	CP
$0.80\gamma_m$	$0.5(\gamma_m + \gamma_r)$	γ_c

Note: IO = Immediate Occupancy, LS = Life Safety, CP = Collapse Prevention.

11.3.5 Reinforced Masonry Wall Out-of-Plane Actions RM walls shall be capable of resisting out-of-plane inertial forces as isolated components spanning between floor levels and/or spanning horizontally between columns or pilasters. Walls shall not be analyzed out of plane with the LSP or NSP prescribed in Chapter 7, but they shall be capable of resisting out-of-plane inertial forces as given in Section 7.2.13 or shall be capable of responding to earthquake motions as determined using the NDP, while satisfying the deflection criteria given in Section 11.3.5.3.

11.3.5.1 Stiffness: Reinforced Masonry Wall Out-of-Plane Actions RM walls shall be considered local elements spanning out of plane between individual story levels.

The out-of-plane stiffness of walls shall be neglected in analytical models of the global structural system.

Stiffness shall be based on the net mortared or grouted area of the uncracked section, provided that net flexural tensile stress does not exceed the expected tensile strength, f_{te}, in accordance with Section 11.2.3.5.

Stiffness shall be based on the cracked section for a wall where the net flexural tensile stress exceeds the expected tensile strength.

Stiffnesses for existing and new reinforced out-of-plane walls shall be assumed to be the same.

11.3.5.2 Strength: Reinforced Masonry Wall Out-of-Plane Actions Expected flexural strength shall be determined based on strength design provisions in TMS 402. For walls with an h/t ratio exceeding 20, second-order moment effects caused by out-of-plane deflections shall be considered.

11.3.5.3 Acceptance Criteria for Reinforced Masonry Wall Out-of-Plane Actions Out-of-plane forces on RM walls shall be considered force-controlled actions. Out-of-plane RM walls shall be sufficiently strong in flexure to resist the out-of-plane loads prescribed in Section 7.2.13.

If the NDP is used, the following performance criteria shall be based on the maximum out-of-plane deflection normal to the plane of a wall:

1. For the Immediate Occupancy Structural Performance Level, the out-of-plane story drift ratio shall be equal to or less than 2%;
2. For the Life Safety Structural Performance Level, the out-of-plane story drift ratio shall be equal to or less than 3%; and
3. For the Collapse Prevention Structural Performance Level, the out-of-plane story drift ratio shall be equal to or less than 5%.

11.4 MASONRY INFILLS

The requirements of this section shall apply to masonry infill panels composed of any combination of existing panels, panels enhanced for seismic retrofit, and new panels added to an existing building for seismic retrofit. The procedures for determination of stiffness, strength, and deformation of masonry infills shall be based on this section and used with the analytical methods and acceptance criteria prescribed in Chapter 7, unless noted otherwise.

Masonry infill panels shall be considered primary elements of a seismic-force-resisting system. For the Collapse Prevention Structural Performance Level, if the analysis shows that the surrounding frame remains stable after the loss of in-plane strength of an infill panel without infill falling out of plane, such infill panels not meeting the acceptance criteria of this section shall be permitted.

11.4.1 Types of Masonry Infills Infills shall include masonry panels built partially or fully within the plane of structural steel or concrete frames and bounded by beams and columns.

Infill panel types considered in this standard include masonry consisting of solid and/or hollow clay and concrete units. Infills made of stone or glass block are not addressed in this standard.

Infill panels shall be considered isolated from the surrounding frame when there are gaps at the top and two sides that accommodate maximum expected lateral frame deflections. Isolated panels shall be restrained in the transverse direction to ensure stability under out-of-plane forces. For panels in full contact with the frame elements on all four sides, the forces exerted on the bounding frame members and connections caused by the frame–infill interaction shall be evaluated.

11.4.1.1 Existing Masonry Infills Existing masonry infills considered in this section shall include all structural infills of a building system that are in place before seismic retrofit. Infill types included in this section consist of unreinforced panels and composite or noncomposite panels. For existing infill panels, the seismic forces applied within their plane shall be considered separately as described in Section 11.4.2 from the forces normal to their plane, as described in Section 11.4.3.

Existing masonry infills shall be assumed to behave the same as new masonry infills, provided that the masonry is in good or fair condition as defined in this standard.

11.4.1.2 New Masonry Infills New masonry infills shall include all new panels added to an existing seismic-force-resisting system for structural retrofit. New elements shall be designed in accordance with this standard and detailed and constructed in accordance with an approved building code.

11.4.1.3 Retrofitted Masonry Infills Retrofitted masonry infill panels shall include existing infills that are enhanced by an approved method.

11.4.2 Masonry Infill In-Plane Actions The calculation of masonry infill in-plane stiffness and strength based on nonlinear finite-element analysis of a composite frame substructure with infill panels that account for the presence of openings, postyield cracking, and cyclic degradation of masonry shall be permitted. The use of simplified numerical models with diagonal struts to simulate the effect of the infill shall be permitted to model infilled frames. Because of the complexity of the seismic behavior of the structural system caused by the frame–infill interaction, finite-element and strut models shall be validated by considering published or project-specific experimental data from cyclic quasistatic or dynamic tests. Alternatively, the methods of Sections 11.4.2.1 and 11.4.2.2 shall be used.

11.4.2.1 Stiffness: Masonry Infill In-Plane Actions The initial in-plane stiffness of an uncracked infilled frame with a solid unreinforced masonry infill panel without openings, K_{un}^{solid}, shall be estimated for each bay in each story using Equation (11-35), assuming the structure is a composite cantilever column, with the columns being the flanges and the masonry wall, the web of the column:

$$K_{un}^{\text{solid}} = \frac{1}{\dfrac{1}{K_{fl}} + \dfrac{1}{K_{sh}}} \tag{11-35}$$

where

K_{fl} = Flexural stiffness of the equivalent composite cantilever column, and

K_{sh} = Shear stiffness of the equivalent composite cantilever column.

For the flexural stiffness, K_{fl}, the equivalent properties of the composite column shall be considered, although for the shear stiffness only the contribution of the wall can be considered. The flexural stiffness shall be calculated from Equation (11-36):

$$K_{fl} = \frac{3E_c I_{ce}}{h_{\text{inf}}^3} \tag{11-36}$$

where

h_{inf} = Clear height of the infill wall for an individual bay in one story,

E_c = Modulus of elasticity of column, and

I_{ce} = Equivalent moment of inertia of the transformed section.

The cracked moment of inertia, I_{ce}, depends on the ratio of elastic moduli of concrete or structural steel and masonry, as well as the geometry of the cross section. Alternatively, the modulus of elasticity of masonry can be used in Equation (11-36) if the composite cross section is transformed to an equivalent masonry cross section. Assuming that the shear stress is uniform across the wall, the shear stiffness shall be calculated from Equation (11-37):

$$K_{sh} = \frac{A_w G_{me}}{h_{\text{inf}}} \tag{11-37}$$

where

A_w = Cross-sectional area of infill masonry wall,

G_{me} = Shear modulus of masonry in accordance with Section 11.2.3.7, and

h_{inf} = Height of infill wall.

Only the wythes in full contact with the frame elements shall be considered when computing the in-plane stiffness, unless anchorage capable of transmitting in-plane forces from frame members to all masonry wythes is provided on all sides of the walls.

11.4.2.2 Stiffness: Masonry Infill with Openings In-Plane Actions The initial in-plane stiffness of an uncracked infilled frame with an unreinforced masonry infill panel with one opening with an area not exceeding 40% of the total infill panel area, K_{un}^{op}, shall be estimated, based on the stiffness of the frame with a solid panel, K_{un}^{solid}, obtained from Equation (11-35) using Equation (11-38):

$$K_{un}^{op} = \left(1 - 2\frac{A_{op}}{A_{\text{inf}}}\right) K_{un}^{\text{solid}} \tag{11-38}$$

where

A_{op} = Opening area; and

A_{inf} = Total area of a frame bay infilled with masonry, including openings in the infill wall.

Table 11-12. Classification of Infilled Reinforced Concrete Frames.

	Infill	
Frame	**Relatively Stiff Infill**	**Relatively Flexible Panel**
Nonductile	$K_{\text{inf}}/K_c > 125\frac{V_n}{V_p} \le 1$	$K_{\text{inf}}/K_c \le 125\frac{V_n}{V_p} \le 1$
Ductile	$K_{\text{inf}}/K_c > 125\frac{V_n}{V_p} > 1$	$K_{\text{inf}}/K_c \le 125\frac{V_n}{V_p} > 1$

11.4.2.3 Strength: Infilled Reinforced Concrete Frames In-Plane Actions The masonry panel of an infilled reinforced concrete frame shall be classified as strong or weak and the reinforced concrete frame shall be classified as ductile or nonductile according to Table 11-12. K_{inf} is the infill lateral stiffness determined from Equation (11-39):

$$K_{\text{inf}} = \frac{1}{\frac{1}{K_{\mathit{inff}}} + \frac{1}{K_{\mathit{infs}}}} \qquad (11\text{-}39)$$

where

K_{inff} = Infill flexural stiffness determined from Equation (11-40);

$$K_{\mathit{inff}} = \frac{3E_{me}I_{\text{inf}}}{h_{\text{inf}}^3} \qquad (11\text{-}40)$$

K_{infs} = Infill shear stiffness determined from Equation (11-41);

$$K_{\mathit{infs}} = \frac{A_W G_{me}}{h_{\text{inf}}} \qquad (11\text{-}41)$$

K_c = Column flexural stiffness determined from Equation (11-42);

$$K_c = \frac{3E_c I_c}{h_{\text{inf}}^3} \qquad (11\text{-}42)$$

V_p = Column shear force corresponding to the development of plastic hinges over the column at a distance hp. The shear force is determined from Equation (11-43);

$$V_p = \frac{2M_p}{h_p} \qquad (11\text{-}43)$$

and

A_W = Horizontal cross-sectional area of an infill panel $t_{\text{inf}}\ L_{\text{inf}}$. In case of an infill with no more than one opening in each panel and where the opening's area does not exceed 40% of the total infill panel area, the length of the opening shall be subtracted such that $A_w = t_{\text{inf}}\ (L_{\text{inf}} - L_o)$;
E_c = Modulus of elasticity of the column;
E_{me} = Modulus of elasticity of masonry;
G_{me} = Shear modulus of masonry;
h_{inf} = Height of the infill panel;
h_p = Distance between plastic hinges in a column. As an alternative to more detailed analysis, h_p is permitted to be taken as equal to infill height divided by 2 for solid infills;
I_{inf} = Effective moment of inertia of infill panel;
I_c = Effective moment of inertia of a column;
L_{inf} = Length of the infill panel;
L_o = Horizontal length of the opening in an infill panel;
M_p = Column plastic moment capacity in accordance with Chapter 10;
t_{inf} = Thickness of infill panel; and
V_n = Column shear strength in accordance with Chapter 10.

The peak strength, Q_{CE}, of an infilled frame bay with a solid masonry infill shall be determined from Equation (11-37) and shall not be less than the frictional resistance of the infill increased by the shear resistance of the leeward column determined from Equation (11-38).

The term *windward* is used for the column to which the application of the lateral force introduces tension. The term *leeward* is used for the columns to which the application of the lateral forces introduces compression.

$$V_{\text{max}} = P_{\text{inf}}^{\text{grav}} \times \mu + A_w \times C \qquad (11\text{-}44)$$

$$V_{\text{max}} = P_{\text{inf}}^{\text{max}}\mu + V_{lc}^{\text{max}} \qquad (11\text{-}45)$$

The yield strength, V_y shall be determined from Equation (11-46):

$$V_y = 0.67V_{\text{max}} \qquad (11\text{-}46)$$

The residual strength shall be determined from Equation (11-47):

$$V_{\text{res}} = P_{\text{inf}}^{\text{res}}\mu_{\text{res}} + V_{lc}^{\text{res}} \qquad (11\text{-}47)$$

where

$P_{\text{inf}}^{\text{max}}$ = Total axial load supported by the infill at a distance equal to half of the column depth from the bottom of the infill when the maximum strength is reached;
V_{lc}^{max} = Shear strength of the leeward column governed by the minimum shear or the flexural capacity of the column. It shall be assumed equal to the strength of the column to shear failure, V_n for nonductile frames and equal to the shear strength caused by plastic hinge formation, V_p for ductile frames;
$P_{\text{inf}}^{\text{grav}}$ = Axial load supported by the infill caused by gravity distributed between the infill and the columns based on their relative axial stiffnesses assuming full contact between the infill and the beams. If there is a gap between the infill and the beam that will not close under lateral deformations, $P_{\text{inf}}^{\text{grav}}$ shall be taken as equal to zero;
$P_{\text{inf}}^{\text{res}}$ = Total axial load applied on the infill when the residual strength is reached;
μ = Infill initial friction coefficient, which shall be measured in accordance with ASTM C1531;
μ_{res} = Infill residual friction coefficient, which shall be measured in accordance with ASTM C1531;
C = Cohesion of the brick–mortar interface, which is equal to the shear strength when no axial stress is applied and shall be measured in accordance with Section 11.2.3.6;
V_{lc}^{res} = Residual resistance of the leeward column. For nonductile frames, it shall be assumed to be equal to V_s, the resistance of the shear reinforcement after the opening of a diagonal shear crack in the column. For ductile frames, it shall be taken as equal to the shear force, V_p, determined by Equation (11-43); and
Vs = Column shear strength accounting for the resistance of transverse reinforcement only in accordance with Chapter 10.

The peak strength, Q_{CE}, of an infilled frame bay with an infill with one opening with area less than 20% of the total infill area shall be determined as 80% of the strength of the same infilled frame bay with a solid panel as determined from Equations (11-44) and (11-45). Similarly, the residual strength of an infilled frame bay with one opening with area less than 20% of the

Table 11-13. Axial Force Supported by Infill According to the Frame–Infill Classification.

	Infill	
Frame	**Relatively Stiff Infill**	**Relatively Flexible Panel**
Nonductile	$P_{\text{inf}}^{\max} = P_{\text{inf}}^{\text{grav}}$ $P_{\text{inf}}^{\text{res}} = P_T + P_1$ where $P_1 = P_2 + P_{\text{inf}}^{\text{grav}} - P_T \le A_s f F_y$ $P_2 = \dfrac{V_{\text{inf}} h_{\text{inf}} + P_T \dfrac{L_{\text{inf}}}{2} - P_{\text{inf}}^{\text{grav}} \alpha L_{\text{inf}}}{L_{\text{inf}}}$ and For $AR \ge 0.77$, $\alpha = 0.88$ For $AR < 0.77$, $\alpha = 1.05 - 0.13/AR$	$P_{\text{inf}}^{\max} = \dfrac{\dfrac{V_{lc}^{\max}}{(1+\alpha) L_{\text{inf}}/h_{\text{inf}}} + \dfrac{P_T}{2(1+\alpha)}}{1 - \dfrac{h_{\text{inf}}}{(1+\alpha) L_{\text{inf}}}}$ $P_{\text{inf}}^{\text{res}} = P_{\text{inf}}^{\max}$ $AR \ge 0.67$, $\alpha = 0.7$ $AR < 0.67$, $\alpha = 0.5$
Ductile	$P_{\text{inf}}^{\max} = \dfrac{V_{\text{inf}} h_{\text{inf}} + P_T \dfrac{L_{\text{inf}}}{2}}{(1+\alpha) L_{\text{inf}}}$ $P_{\text{inf}}^{\text{res}} = P_{\text{inf}}^{\max}$ For $AR \ge 0.77$, $\alpha = 0.88$ For $AR < 0.77$, $\alpha = 1.05 - 0.13/AR$	

total infill area shall be taken as equal to 80% of the residual strength of the same frame infilled with a solid panel as determined from Equation (11-47).

The axial loads on the wall at the point of peak shear resistance, $P_{\text{inf}}^{\max}$, and at the onset of the residual shear resistance, $P_{\text{inf}}^{\text{res}}$, shall be determined according to Table 11-13.

where

A_s = Area of nonprestressed longitudinal reinforcement in column of an infilled frame, in.^{2};

AR = Infill height to infill length ratio $h_{\text{inf}}/L_{\text{inf}}$;

F_y = Yield stress of reinforcing steel;

$P_{\text{inf}}^{\text{grav}}$ = Axial load applied on the infill because of gravity distributed between the infill and the columns based on relative axial stiffnesses assuming contact between the infill and the beam. If there is a gap between the infill and the beam that will not close under lateral deformation, $P_{\text{inf}}^{\text{grav}}$ shall be assumed to be zero;

P_T = Total axial load applied on the frame because of gravity;

P_1 = Total tension load applied on the windward column at a distance of $d/2$ from the bottom of the infill when the maximum strength is reached;

P_2 = Total compression load applied on the leeward column at a distance of $d/2$ from the bottom of the infill when the maximum strength is reached;

V_{inf} = Sliding strength of the masonry infill determined using Equation (11-48):

$$V_{\text{inf}} = P_{\text{inf}}^{\text{grav}} \times \mu + A_w \times C \qquad (11\text{-}48)$$

α = Coefficient that when multiplied by the distance between column centerlines gives the infill axial force resultant position measured from the windward column centerline when the maximum strength is reached.

The infilled frame is permitted to be considered elastic until the development of the separation cracks between the infill and the surrounding frame that occurs at approximately 60% of the peak strength, Q_{CE}. Therefore, the resistance at Point 1 in Figure 11-1a that onsets the nonlinear region of the force-versus-deformation curve shall be defined using the stiffness determined by Equation (11-35) or (11-38).

11.4.2.4 Strength: Infilled Steel Frames In-Plane Actions In lieu of detailed nonlinear finite-element analysis, the strength of an infilled steel frame shall be permitted to be determined according to Section 11.4.2.3, using the ductile frame provisions described therein if in the inspection or evaluation, the steel frame is found to be continuous with sufficiently strong connections. In this case, steel beam, column, and connection capacities, as applicable to Section 11.4.2.3, shall be determined in accordance with Chapter 9. In case the inspection or evaluation identifies components, including force-controlled actions, that prevent the steel frame from developing plastic hinges in the columns over the distance h_p, as defined by Section 11.4.2.3, the nonductile frame provisions in Section 11.4.2.3 shall be used to estimate the strength of the infilled steel frame.

The shear capacity of the steel frame, V_n, shall be determined as the minimum of the shear capacity of the steel column and the shear capacity of the beam–column connection determined in accordance with Chapter 9. The effects of concurrent moments and axial load, in conjunction with capacity limitations of any connections and splices, shall also be considered in the evaluation of steel column capacity.

11.4.2.5 Drift: Infill Wall In-Plane Actions The drift at which the peak strength of an infilled frame is reached shall be determined according to Table 11-14.

The drift at which the residual strength is reached shall be determined according to Table 11-15.

11.4.2.6 Strut Model for Infill In-Plane Actions The envelope curve of an infilled frame that shall be determined according to Sections 11.4.2.2 to 11.4.2.5 shall be used to calibrate the diagonal struts to represent the masonry infill. Assuming that the material properties and dimensions of the concrete members are known, the bare concrete frame shall be modeled directly. The difference between the envelope curve of the infilled frame

Table 11-14. Drift at Peak Strength for an Infilled Frame Bay According to the Infilled Frame and Masonry Infill Classification of Table 11-8.

	Infill	
Frame	**Relatively Flexible Panel**	**Relatively Stiff Infill**
Nonductile	$\Delta_{\text{peak}} = 0.35$	Both $\begin{cases} AR > 0.50 \\ \Delta_{\text{peak}} = 0.82 - \dfrac{1}{(3\,AR)} \\ AR \le 0.50 \\ \Delta_{\text{peak}} = 0.15 \end{cases}$
Ductile	$AR > 0.77$: $\Delta_{\text{peak}} = 0.6 - \dfrac{0.23}{AR}$ $AR \le 0.77$: $\Delta_{\text{peak}} = 0.30$	

Table 11-15. Drift at the Onset of Residual Strength for an Infilled Frame Bay According to the Infilled Frame and Masonry Infill Classification of Table 11-8.

	Infill	
Frame	**Relatively Stiff Infill**	**Relatively Flexible Panel**
Nonductile	$\Delta_{\text{res}} = 1.6 \times \Delta_{\text{peak}}$	$\Delta_{\text{res}} = 0.55$
Ductile	$\Delta_{\text{res}} = 1.6 \times \Delta_{\text{peak}}$	$\Delta_{\text{res}} = 1.0$

and the curve of the bare frame shall be attributed to the struts representing the infill.

11.4.2.7 Acceptance Criteria for Infill Wall In-Plane Actions The acceptance criteria for linear and nonlinear procedures shall be in accordance with this section.

11.4.2.7.1 Required Strength of Column Members Adjacent to Infill Panels To demonstrate compliance, the expected flexural and shear strengths of column members adjacent to an infill panel shall exceed the forces resulting from one of the following conditions:

1. The application of the horizontal component of the expected infill strut force at the column using the shear strength of the column with zero axial load in accordance with Chapter 10 for concrete columns and Chapter 9 for structural steel columns; or
2. The shear force resulting from development of expected column flexural strengths at the top and bottom of a column; in this case, a reduced column height, l_{ceff}, equal to the distance between the flexural hinges, shall be considered.

The reduced column length, l_{ceff}, shall be equal to the clear height of openings in infilled walls for a column supported by a partial height infill.

The requirements of this section shall be waived if the lower-bound masonry shear strength, v_{mL}, as measured in accordance with test procedures of Section 11.2.3.6, is less than 20 lb/in.^{2} (138 kPa).

In addition, the strength of reinforced concrete beam–column joints shall be determined to exceed the expected infill diagonal tension forces acting on the joints, considering the reinforcement, development, degree of confinement, and load paths of the joints. Similarly, the strength of structural steel beam–column joints shall be determined to exceed the expected infill diagonal tension forces, considering the load paths through the joints.

11.4.2.7.2 Acceptance Criteria for Linear Procedures for Infill Wall In-Plane Actions Actions on masonry infills are permitted to be considered deformation controlled. For the linear procedures of Sections 7.4.1 and 7.4.2 component actions shall be compared with capacities in accordance with Section 7.5.2.2. *m*-factors for use in Equation (7-36) shall be as specified in Table 11-16. For an infill panel, the seismic-force action, Q_E, shall be the horizontal component of the unreduced axial force in the equivalent strut member.

For determination of *m*-factors in accordance with Table 11-16, the ratio of frame to infill strengths, β, shall be determined considering the expected lateral strength of each component. V_{fre} is the expected story shear strength of the bare frame taken as the shear capacity of the column, V_n, and V_{infe} is the expected shear strength of the infill panel determined using Equation (11-48).

11.4.2.7.3 Acceptance Criteria for Nonlinear Procedures for Infill Wall In-Plane Actions For the NSP given in Section 7.4.3, infill panels shall meet the requirements of Section 7.5.3.2.

Table 11-16. Linear Procedure: *m*-Factors for Masonry Infill Panels.

		m-Factors				
			Primary		Secondary	
$\beta = \dfrac{V_{fre}}{V_{infe}}$	$h_{\text{inf}} / L_{\text{inf}}$	**IO**	**LS**	**CP**	**LS**	**CP**
$\beta < 0.7$	2.0	1.0	$0.56\Delta_{\text{peak}}/\Delta_y$	$0.75\Delta_{\text{peak}}/\Delta_y$	$0.56\Delta_{\text{res}}/\Delta_y$	$0.75\Delta_{\text{res}}/\Delta_y$
	1.0	1.0	$0.56\Delta_{\text{peak}}/\Delta_y$	$0.75\Delta_{\text{peak}}/\Delta_y$	$0.56\Delta_{\text{res}}/\Delta_y$	$0.75\Delta_{\text{res}}/\Delta_y$
	0.5	1.0	$0.56\Delta_{\text{peak}}/\Delta_y$	$0.75\Delta_{\text{peak}}/\Delta_y$	$0.56\Delta_{\text{res}}/\Delta_y$	$0.75\Delta_{\text{res}}/\Delta_y$
$0.7 \le \beta < 1.3$	2.0	1.5	$0.56\Delta_{\text{peak}}/\Delta_y$	$0.75\Delta_{\text{peak}}/\Delta_y$	$0.56\Delta_{\text{res}}/\Delta_y$	$0.75\Delta_{\text{res}}/\Delta_y$
	1.0	1.2	$0.56\Delta_{\text{peak}}/\Delta_y$	$0.75\Delta_{\text{peak}}/\Delta_y$	$0.56\Delta_{\text{res}}/\Delta_y$	$0.75\Delta_{\text{res}}/\Delta_y$
	0.5	1.0	$0.56\Delta_{\text{peak}}/\Delta_y$	$0.75\Delta_{\text{peak}}/\Delta_y$	$0.56\Delta_{\text{res}}/\Delta_y$	$0.75\Delta_{\text{res}}/\Delta_y$
$\beta \ge 1.3$	2.0	1.5	$0.56\Delta_{\text{peak}}/\Delta_y$	$0.75\Delta_{\text{peak}}/\Delta_y$	$0.56\Delta_{\text{res}}/\Delta_y$	$0.75\Delta_{\text{res}}/\Delta_y$
	1.0	1.2	$0.56\Delta_{\text{peak}}/\Delta_y$	$0.75\Delta_{\text{peak}}/\Delta_y$	$0.56\Delta_{\text{res}}/\Delta_y$	$0.75\Delta_{\text{res}}/\Delta_y$
	0.5	1.0	$0.56\Delta_{\text{peak}}/\Delta_y$	$0.75\Delta_{\text{peak}}/\Delta_y$	$0.56\Delta_{\text{res}}/\Delta_y$	$0.75\Delta_{\text{res}}/\Delta_y$

Note: Interpolation shall be used between table values.

Table 11-17. Nonlinear Procedure: Simplified Force–Deflection Relations for Masonry Infill Panels.

$\beta = = \frac{V_{fre}}{V_{infe}}$	h_{inf}/L_{inf}	Residual Strength Ratio c	d (%)	e^b (%)	Acceptance Criteria		
					IO (%)	LS (%)	CP (%)
$\beta < 0.7$	2.0	V_{res}/V_{max}	Δ_{res}	1.0	Δ_y	0.75 Δ_{res}	Δ_{res}
	1.0	V_{res}/V_{max}	Δ_{res}	1.0	Δ_y	0.75 Δ_{res}	Δ_{res}
	0.5	V_{res}/V_{max}	Δ_{res}	1.0	Δ_y	0.75 Δ_{res}	Δ_{res}
$0.7 \leq \beta < 1.3$	2.0	V_{res}/V_{max}	Δ_{res}	1.0	$2.0\Delta_y$	0.75 Δ_{res}	Δ_{res}
	1.0	V_{res}/V_{max}	Δ_{res}	1.0	$1.6\Delta_y$	0.75 Δ_{res}	Δ_{res}
	0.5	V_{res}/V_{max}	Δ_{res}	1.0	Δy	0.75 Δ_{res}	Δ_{res}
$\beta \geq 1.3$	2.0	V_{res}/V_{max}	Δ_{res}	1.0	$2.0\Delta_y$	0.75 Δ_{res}	Δ_{res}
	1.0	V_{res}/V_{max}	Δ_{res}	1.0	$1.6\Delta_y$	0.75 Δ_{res}	Δ_{res}
	0.5	V_{res}/V_{max}	Δ_{res}	1.0	Δ_y	0.75 Δ_{res}	Δ_{res}

Note: Interpolation shall be used between table values.

Nonlinear lateral drifts shall not exceed the values given in Table 11-17.

For determination of acceptable drift levels using Table 11-17, the ratio of frame to infill strengths, β, shall be determined considering the expected lateral strength of each component.

For the NDP given in Section 7.4.4, infill panels shall meet the requirements of Section 7.5.3.2. Nonlinear force–deflection relations for infill panels shall be established based on the information given in Table 11-17 or on an approved procedure based on a comprehensive evaluation of the hysteretic characteristics of those components.

11.4.3 Masonry Infill Wall Out-of-Plane Actions Unreinforced infill panels with h_{inf}/t_{inf} ratios less than those given in Table 11-18, and meeting the requirements for arching action given in the following section, need not be analyzed for out-of-plane seismic forces.

11.4.3.1 Stiffness: Infill Wall Out-of-Plane Actions Infill panels shall be considered local elements spanning out-of-plane vertically between floor levels or horizontally across bays of frames.

The out-of-plane stiffness of infill panels shall be neglected in analytical models of the global structural system in the orthogonal direction.

Flexural stiffness for uncracked masonry infills subjected to transverse forces shall be based on the minimum net sections of mortared and grouted masonry. Flexural stiffness for unreinforced, cracked infills subjected to transverse forces shall be assumed to be equal to zero unless arching action is considered.

Arching action is permitted to be considered only if all the following conditions exist:

1. The panel is in full contact with the surrounding frame components;
2. The product of the elastic modulus of the frame material, E_{fe}, times the moment of inertia, if the most flexible frame component in the direction of arching action exceeds a value of 3.6×10^9 lb in.^{2} (10,331 kN m^2);
3. The frame components have sufficient strength to resist thrusts from arching of an infill panel; and
4. The $h_{\text{inf}}/t_{\text{inf}}$ ratio is less than or equal to 35.

Table 11-18. Maximum h_{inf}/t_{inf} Ratios.

Performance Level	Very Low and Low Seismicity	Moderate Seismicity	High Seismicity
IO	14	13	8
LS	15	14	9
CP	16	15	10

11.4.3.2 Strength: Infill Wall Out-of-Plane Actions Where arching action is not considered, the lower-bound strength of a URM infill panel shall be limited by the lower-bound masonry flexural tension strength, f_t, which shall be taken as 0.7 times the expected tensile strength, f_{te}, as determined in accordance with Section 11.2.3.5.

If arching action is considered, the lower-bound out-of-plane strength of a solid infill panel in lb/ft^2, $q^{\text{solid}}_{\text{inf},oop}$, shall be determined using Equation (11-49):

$$q^{\text{solid}}_{\text{inf},oop} = \frac{0.3 f'_m R_1 R_2 e^{-0.0985\left(\frac{h_{\text{inf}}}{t_{\text{inf}}}\right)}}{\left(\frac{h_{\text{inf}}}{t_{\text{inf}}}\right)} \times 144 \qquad (11\text{-}49)$$

where

f'_m = Lower bound of masonry compressive strength, in lb/in.^{2}, determined in accordance with Section 11.2.3.3; and

R_1 = Factor to account for the effect of damage due to in-plane loading. R_1 is assumed to be equal to 1.0 if the engineer incorporates in-plane and out-of-plane interactions into their analysis in accordance with Section 11.4.3.3. Otherwise, R_1 can be assumed to equal 0.6.

$$R_2 = 0.35 + 7.14 \times 10^{-11} E_{fe} I_f \leq 1.0 \qquad (11\text{-}50)$$

where

R_2 = Factor to account for flexibility in the bounding frame; and

$E_{fe}I_f$ = Product of the elastic modulus of the frame material, E_{fe}, times the moment of inertia, I_f, of the most flexible frame component or the frame component at the discontinuous panel edge in the direction of arching action, in lb in.^{2}.

The out-of-plane strength of an infill panel with openings in, lb/ft^2, $q_{\text{inf},oop}$, shall be determined using Equation (11-51):

$$Q_{CL} = q_{\text{inf},oop} = \left(1 - \frac{A_{op}}{A_{W\text{tot}}}\right) q_{\text{inf},oop}^{\text{solid}} \qquad (11\text{-}51)$$

where

$q_{\text{inf},oop}^{\text{solid}}$ = Uniformly distributed lateral load capacity of an equivalent infill panel with no openings determined using Equation (11-49),

A_{op} = Total area of the openings in the infill panel, and

$A_{W\text{tot}}$ = Gross area of an equivalent infill panel with no openings.

11.4.3.3 Strength: Infill Wall In-Plane and Out-of-Plane Interaction Infill wall in-plane and out-of-plane interaction shall be considered in conjunction with Section 7.2.6. If consideration of concurrent multidirectional seismic effects is not required according to this section, out-of-plane actions shall be permitted to be considered according to Sections 11.4.3, 11.4.3.1, and 11.4.3.2. Otherwise, infill walls shall be evaluated considering in-plane and out-of-plane interaction according to Section 11.4.3.3.

The strength of infill walls under in-plane and out-of-plane actions acting concurrently shall be permitted to be evaluated using Equation (11-52) when analyzed with LSP and LDP:

$$\left(\frac{Q_{IP}/m_{IP}}{Q_{IP0}}\right)^{3/2} + \left(\frac{Q_{OOP}/1.5}{Q_{OOP0}}\right)^{3/2} \leq 1.0 \qquad (11\text{-}52)$$

where

Q_{IP0} = Infill wall in-plane strength without out-of-plane force, determined according to Section 11.4.2.3;

Q_{OOP0} = Infill wall out-of-plane strength without in-plane force determined using Equation (11-51);

Q_{IP} = Infill wall in-plane force demand in LSP and the response spectrum method of LDP; and the maximum in-plane force demand in the response history method of LDP;

Q_{OOP} = Infill wall out-of-plane force demand in LSP and the response spectrum method of LDP; and the maximum out-of-plane force demand in the response history method of LDP. Alternatively, Equations (7-13) and (7-14) shall be permitted to calculate the out-of-plane force demand; and

m_{IP} = m-Factor for masonry infill panel in-plane action, as determined from Table 11-16.

When analyzed with NSP, Equation (11-53) shall be used to consider the in-plane, out-of-plane interaction:

$$\left(\frac{Q_{IPE}}{Q_{IP0}}\right)^{3/2} + \left(\frac{Q_{OOP}}{Q_{OOP0}}\right)^{3/2} \leq 1.0 \qquad (11\text{-}53)$$

where

Q_{IP0} and Q_{OOP0} are as defined previously;

Q_{OOP} = Infill wall out-of-plane force demand caused by the out-of-plane inertial loading. Top and bottom story accelerations shall be permitted to be used for the calculation of inertial loading. Story accelerations shall be determined as the maximum external story force in NSP divided by the story mass; and

Q_{IPE} = Infill wall in-plane strength in the presence of out-of-plane force, to be used in NSP.

The strength of infill walls under in-plane and out-of-plane actions acting concurrently shall be permitted to be characterized using Equation (11-54) when analyzed with NDP:

$$\left(\frac{Q_{IP}}{Q_{IP0}}\right)^{3/2} + \left(\frac{Q_{OOP}}{Q_{OOP0}}\right)^{3/2} \leq 1.0 \qquad (11\text{-}54)$$

where

Q_{IP0} and Q_{OOP0} are as defined previously,

Q_{IP} = Infill wall in-plane force demand at an integration step in NDP, and

Q_{OOP} = Infill wall out-of-plane force demand at an integration step in NDP.

11.4.3.4 Acceptance Criteria: Infill Wall Out-of-Plane Actions Infill panels loaded out-of-plane shall not be analyzed with the LSP or NSP prescribed in Chapter 7.

The lower-bound transverse strength of URM infill panels shall exceed normal pressures as prescribed in Section 7.2.13.

The Immediate Occupancy Structural Performance Level is assumed to be reached when flexural cracking caused by out-of-plane inertial loading occurs. The Collapse Prevention Structural Performance Level of an unreinforced masonry infill is assumed to be reached at the out-of-plane strength estimated using Equations (11-53) and (11-54).

11.5 ANCHORAGE TO MASONRY WALLS

11.5.1 Types of Anchors Anchors considered shall include through-bolts with bearing plates, headed anchors, bent bar anchors, and approved adhesive anchors embedded into masonry. Anchors in hollow-unit masonry shall be embedded in grout or shall be embedded in approved adhesives within approved anchoring devices.

Tension and shear strength of anchors, except for through-bolts with bearing plates, shall be verified by approved test procedures.

11.5.2 Analysis of Anchors Anchors embedded into existing or new masonry walls shall be analyzed in accordance with applicable sections of Chapter 13, TMS 402, and ACI 318. Lower-bound values for strengths of embedded anchors with respect to pullout, shear, and combinations of pullout and shear shall be as specified using load and resistance factor design (LRFD) procedures taking $\phi = 1.0$.

The minimum effective embedment length or edge distance for considerations of pullout and shear strength of embedded anchors shall be used. Shear strength of anchors with edge distances equal to or less than 1 in. (25 mm) shall be taken as zero.

Adhesive anchors in concrete masonry units are permitted to be analyzed using anchor provisions in Chapter 10 and ACI 318.

11.5.3 Quality Assurance for Anchors in Masonry Walls When required by the Authority Having Jurisdiction, the design professional shall provide a quality assurance plan for new and existing anchors that are part of the seismic-force-resisting system and that provide connections to masonry walls. The plan shall include the following:

1. In unreinforced masonry walls, tests to determine bed-joint shear strengths adjacent to anchor locations per Section 11.2.3.6.3 or 11.2.3.6.4 to comply with lower-bound bed-joint shear strengths specified in the plan;
2. In reinforced grouted masonry walls, tests to determine lower-bound reinforced masonry strengths, per Section 11.2.3.1;
3. In fully and partially grouted masonry walls, the use of a borescope or other direct methods to determine the presence, absence, and quality of grout in reinforced masonry, hollow-unit walls, or cavity walls at and adjacent to anchor locations;
4. In URM walls, proof load of at least 25% of each type and diameter of existing anchors in tension to a load

corresponding to the design allowable load in accordance with ASTM E488 and the *International Existing Building Code*, Appendix Chapter A1, or equivalent;
5. In grouted masonry walls, proof load of at least 10% of installed new anchors in confined tension in accordance with ASTM E488 to at least twice the lower-bound tension load, accounting for edge distances or 80% of the yield strength of the anchor, whichever is less;
6. For adhesive anchors in concrete masonry units, tests using anchor provisions in Chapter 10 and ACI 318.
7. Quality control provisions and documentation for the installation of new anchors and the condition of adjacent mortar joint and masonry units;
8. Visual inspection and documentation of the condition of existing anchors, adjacent mortar joints, and masonry units;
9. Special inspections for adhesive anchors by qualified special inspectors; and
10. For proprietary anchors, provisions for the verification of the qualifications of the installers of anchors for the specific types of anchors and masonry materials based on experience and training, as specified by the anchor manufacturer.

The plan shall be consistent with strengths; numbers of tests; procedures for tests; quality control; and, where applicable, inspection requirements specified by manufacturers' published installation instructions, acceptance criteria established by an independent evaluation services agency for proprietary anchors, or that specified by the design professional for generic anchors and existing anchors. The plan shall also include provisions for increasing the percentage of anchors to be tested to address conditions where failures are reported during initial tension and shear testing. The plan is permitted to include exemptions for tension and shear tests for anchors that extend through the entire wall thickness and bear on plates on the opposite wall face.

11.6 MASONRY FOUNDATION ELEMENTS

11.6.1 Types of Masonry Foundations Masonry foundations of all types shall be evaluated or retrofitted in accordance with this section.

11.6.2 Seismic Evaluation of Existing Masonry Foundations The deformability of the masonry footings and the flexibility of the soil under them shall be considered in the seismic-force analysis of the building system. The strength and stiffness of the soil shall be determined in accordance with the requirements of Chapter 8.

Masonry retaining walls shall be evaluated to resist static and seismic soil pressures in accordance with Section 8.7. Stiffness, strength, and acceptability criteria for masonry retaining walls shall be the same as those for other masonry walls subjected to out-of-plane loadings, as specified in Sections 11.3.3 and 11.3.5.

11.6.3 Foundation Retrofit Measures Seismic retrofit measures for masonry foundations shall meet the requirements of Chapter 8.

11.7 MASONRY DIAPHRAGMS

11.7.1 General Masonry diaphragms are those consisting of shallow brick arches that span between steel floor beams, with or without concrete fill or a topping slab. The arches are packed tightly between the beams to provide the necessary resistance to thrust forces. Masonry diaphragms shall be evaluated or retrofitted in accordance with this section.

11.7.2 Seismic Evaluation of Masonry Diaphragms Masonry diaphragms shall be considered force controlled. A rational analysis procedure shall be used to evaluate the actions, stiffness, and strength of the system, including connected components. Diaphragm deformations and displacements shall not lead to a loss of bearing support for the elements of the arches. In addition, for performance levels higher than Life Safety, the deformation caused by diagonal tension shall not result in the loss of the load transfer mechanism. Deformations shall remain below the threshold of deflections that cause damage to other elements, either structural or nonstructural, at specified performance levels. These values shall be established in conjunction with those for steel framing in accordance with Chapter 9, and in accordance with Chapter 10 where concrete fill or a topping slab exists.

11.7.3 Retrofit Measures for Masonry Diaphragms Seismic retrofit measures for masonry diaphragms shall meet the requirements of the appropriate material chapters and other provisions of this standard.

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CHAPTER 12
WOOD

12.1 SCOPE

This chapter sets forth requirements for the seismic evaluation and retrofit of wood components of an existing building. The requirements of this chapter shall apply to existing wood components of a building system, retrofitted wood components of a building system, and new wood components that are added to an existing building system.

Section 12.2 specifies data collection procedures for obtaining material properties and performing condition assessments. Section 12.3 specifies general assumptions and requirements. Sections 12.4 and 12.5 provide modeling procedures, component strengths, acceptance criteria, and retrofit measures for wood shear walls and wood diaphragms, respectively. Section 12.6 specifies requirements for wood foundations. Section 12.7 specifies requirements for other wood components, including but not limited to knee-braced frames, rod-braced frames, and braced horizontal diaphragms.

12.2 MATERIAL PROPERTIES AND CONDITION ASSESSMENT

12.2.1 General Mechanical properties for wood materials, components, and assemblies shall be based on available construction documents, test reports, manufacturers' data, and as-built conditions for the particular structure. Where such information fails to provide adequate information to quantify material properties, capacities of assemblies, or condition of the structure, such information shall be supplemented by materials tests, mock-up tests of assemblies, and assessments of existing conditions as required in Section 6.2.

Material properties of existing wood components and assemblies shall be determined in accordance with Section 12.2.2. A condition assessment shall be conducted in accordance with Section 12.2.3. The extent of materials testing and condition assessment performed shall be used to determine the knowledge factor, κ, as specified in Section 12.2.4.

12.2.2 Properties of In-Place Materials and Components

12.2.2.1 Material Properties

12.2.2.1.1 Wood Construction The species and grade of wood shall be established by one of the following methods:

1. Review of construction documents,
2. Inspection to identify grade by viewing grade stamps or comparing grading rules, or
3. Examination of samples by an experienced wood pathologist to establish the species.

Where materials testing is required by Section 6.2 grading shall be performed using the ASTM D245 (ASTM 2022a) grading methodology or an approved grading handbook for the assumed wood species and application. Samples shall be obtained in a manner that does not compromise the strength or stiffness of the structure. Samples shall be tested in accordance with Section 12.2.2.3.

12.2.2.1.2 Use of Default Properties Use of default properties for wood shear walls, wood diaphragms, components, and connectors shall be permitted in accordance with Section 12.2.2.5. Use of material properties based on historical information for use as default values shall be as specified in Section 12.2.2.5. Other approved values of material properties shall be permitted if they are based on available historical information for a particular type of wood construction, prevailing codes, and assessment of existing condition. For wood construction materials comprising individual components, the use of default properties shall be permitted where the species and grade of wood have been determined. Use of default properties for connectors in wood construction shall be permitted where the species and grade of the connected members have been determined.

12.2.2.1.3 Nominal or Specified Properties Use of nominal material properties or properties specified in construction documents to compute expected and lower-bound material properties shall be permitted in accordance with Section 12.2.2.5.

12.2.2.2 Component Properties

12.2.2.2.1 Elements The following component properties, as applicable, shall be determined in accordance with Section 12.2.3:

1. Cross-sectional shape and physical dimensions of the primary components and overall configuration of the structure, including any modifications subsequent to original construction;
2. Configuration of elements, size and thickness of connected materials, lumber grade, connection size and spacing, and continuity of load path;
3. Location and dimension of seismic-force-resisting elements, type, materials, and spacing of tie-downs and boundary components; and
4. Current physical condition of components and extent of any deterioration present.

12.2.2.2.2 Connections The following connection details, as applicable, shall be determined or verified in accordance with Section 12.2.3:

1. Connections between horizontal diaphragms and vertical elements of the seismic-force-resisting system;
2. Size and character of all diaphragm ties, including splice connections;
3. Connections at splices in chord members of horizontal diaphragms;

4. Connections of floor and roof diaphragms to exterior or interior concrete or masonry walls for both in-plane and out-of-plane loads;
5. Connections of cross-tie members for concrete or masonry buildings;
6. Connections of shear walls to foundations for transfer of shear and overturning forces; and
7. Method of through-floor transfer of wall shear and overturning forces in multistory buildings.

12.2.2.3 Test Methods to Quantify Material Properties The stiffness and strength of wood components and assemblies shall be established through in situ testing or mock-up testing of assemblies in accordance with Section 7.6 unless default values are used in accordance with Section 12.2.2.5. The number of tests required shall be based on Section 12.2.2.4. Expected material properties shall be based on mean values of tests. Lower-bound material properties shall be based on mean values of tests minus one standard deviation.

12.2.2.4 Minimum Number of Tests Where required, testing shall meet the requirements for usual testing in Section 12.2.2.4.1 or comprehensive testing in Section 12.2.2.4.2.

12.2.2.4.1 Usual Testing The minimum number of tests to quantify expected strength material properties for usual data collection shall be based on the following criteria:

1. If construction documents containing material property and detailing information for the seismic-force-resisting system are available, at least one element of the seismic-force-resisting system for each story, or for every 100,000 ft^2 (9,290 m^2) of floor area, is to be randomly verified by observation for compliance with the construction documents; and
2. If construction documents are incomplete or not available, at least two locations for each story, or 100,000 ft^2 (9,290 m^2) of floor area, are to be randomly verified by observation or otherwise documented.

12.2.2.4.2 Comprehensive Testing The minimum number of tests necessary to quantify expected strength properties for comprehensive data collection shall be defined in accordance with the following requirements:

1. If original construction documents exist that define the grade and mechanical properties, at least one location for each story is to be randomly verified by observing product marking or by compliance with wood grading rules for each component type identified as having a different material grade;
2. If original construction documents defining properties are not complete or do not exist but the date of construction is known and single material use is confirmed, at least three locations are to be randomly verified—by sampling and testing or by observing grade stamps and conditions—for each component type, for every two floors in the building;
3. If no knowledge of the structural system and materials used exists, at least six locations are to be randomly verified—by sampling and testing or by observing product marking and conditions—for each element and component type, for every two floors or 200,000 ft^2 (18,580 m^2) of floor area of construction. If it is determined from testing or observation that more than one material grade exists, additional observations and testing are to be conducted until the extent of use for each grade has been established;
4. In the absence of construction records defining connector features present, the configuration of at least three connectors are to be documented for every floor or 100,000 ft^2 (9,290 m^2) of floor area in the building; and
5. A full-scale mock-up test is to be conducted for archaic assemblies; at least two cyclic tests of each assembly shall be conducted. A third test shall be conducted if the results of the two tests vary by more than 20%.

12.2.2.5 Default Properties Use of default properties to determine component strengths shall be permitted in conjunction with the linear analysis procedures of Chapter 7.

Default expected strength and stiffness values for existing wood shear wall assemblies shall be taken from Table 12-1. The shear wall type shall be as defined in Section 12.4.2 for wood construction. Default expected strength and stiffness values for wood diaphragm assemblies shall be taken from Table 12-2.

The estimated deformation of any hardware, including allowance for poor fit or oversized holes, shall be summed to obtain the total deformation of the connection.

Default expected-strength values for connection hardware shall be taken as the average ultimate test values from published reports.

Default lower-bound strength values, where required in this chapter, shall be taken as expected-strength values multiplied by 0.85.

12.2.2.5.1 Wood Construction Default Properties Default expected-strength values for wood materials comprising individual components shall be based on design resistance values associated with the American Wood Council (AWC) National Design Specification (NDS) for Wood Construction as determined in accordance with ASTM D5457. All adjustment factors, including the time-effect factor, that are applicable in accordance with AWC NDS shall be considered. The resistance factor, ϕ, shall be taken as 1.0. If components are damaged, reductions in capacity and stiffness shall be applied, considering the position and size of the ineffective cross section.

Default expected-strength values for connectors shall be based on design resistance values associated with AWC NDS, as determined in accordance with ASTM D5457. All adjustment factors, including the time-effect factor, that are applicable in accordance with AWC NDS shall be considered. The resistance factor, ϕ, shall be taken as 1.0.

Default expected-strength values shall be permitted to be directly computed from allowable stress values listed in an approved code using the method contained in ASTM D5457.

Default deformations at yield of connectors shall be taken as the following:

1. 0.03 in. (0.76 mm) for wood-to-wood and 0.02 in. (0.51 mm) for wood-to-metal nailed connections,
2. 0.04 in. (1.02 mm) for wood-to-wood and 0.03 in. (0.76 mm) for wood-to-steel screw connections,
3. 0.04 in. (1.02 mm) for wood-to-wood and 0.027 in. (0.69 mm) for wood-to-steel lag bolt connections, and
4. 0.045 in. (1.14 mm) for wood-to-wood and 0.03 in. (0.76 mm) for wood-to-steel bolted connections.

12.2.3 Condition Assessment

12.2.3.1 General A condition assessment of the existing building and site shall be performed as specified in this section.

A condition assessment shall include the following:

1. The physical condition of primary and secondary components is to be examined, and the presence of degradation is to be noted;

Table 12-1. Default Expected Strength Values for Wood: Shear Walls.

	Property	
Shear Wall Type[a]	**Shear Stiffness (G_d) lb/in. (N/mm)**	**Expected Strength (Q_{CE}) lb/ft (N/mm)**
Single-layer horizontal lumber sheathing or siding	2,000 (350)	80 (1.17)
Single-layer diagonal lumber sheathing	8,000 (1,401)	700 (10.22)
Double-layer diagonal lumber sheathing	18,000 (3,152)	1,300 (18.97)
Vertical wood siding	1,000 (175)	70 (1.02)
Wood siding over horizontal lumber sheathing	4,000 (701)	500 (7.30)
Wood siding over diagonal lumber sheathing	11,000 (1,926)	1,100 (16.05)
Wood structural panel sheathing[b]	—	—
Stucco on studs, sheathing, or fiberboard	14,000 (2,452)	350 (5.11)
Gypsum plaster on wood lath	8,000 (1,401)	400 (5.84)
Gypsum plaster on gypsum lath	10,000 (1,751)	80 (1.17)
Gypsum wallboard	8,000 (1,401)	100 (1.46)
Gypsum sheathing	8,000 (1,401)	100 (1.46)
Plaster on metal lath	12,000 (2,102)	150 (2.19)
Horizontal lumber sheathing with cut-in braces or diagonal blocking	2,000 (350)	80 (1.17)
Fiberboard or particleboard sheathing	6,000 (1,051)	100 (1.46)

[a] As defined in Section 12.4.

[b] See Section 12.4.3.6 for shear stiffness and expected strength of wood structural panel walls.

Table 12-2. Default Expected Strength Values for Wood Diaphragms.

		Property	
Diaphragm Type[a]		**Shear Stiffness (G_d) lb/in. (N/mm)**	**Expected Strength (Q_{CE}) lb/ft (N/mm)**
Single-layer straight lumber sheathing[b] chorded and unchorded		2,000 (350)	120 (1.75)
Double-layer straight lumber sheathing	Chorded	15,000 (2,627)	600 (8.76)
	Unchorded	7,000 (1,226)	400 (5.84)
Single-layer diagonal lumber sheathing	Chorded	8,000 (1,401)	600 (8.76)
	Unchorded	4,000 (701)	420 (6.13)
Diagonal lumber sheathing with straight lumber sheathing or flooring above	Chorded	18,000 (3,152)	900 (13.13)
	Unchorded	9,000 (1,576)	625 (9.12)
Double-layer diagonal lumber sheathing	Chorded	18,000 (3,152)	900 (13.13)
	Unchorded	9,000 (1,576)	625 (9.12)
Wood structural panel sheathing[c]	Unblocked, Chorded	8,000 (1,401)	—
	Unblocked, Unchorded	4,000 (701)	—
Wood structural panel overlays on (a) straight or diagonal lumber sheathing[d] or (b) existing wood structural panel sheathing[e]	Unblocked, Chorded	9,000 (1,576)	450 (6.57)
	Unblocked, Unchorded	5,000 (876)	300 (4.38)
	Blocked, Chorded	18,000 (3,152)	—
	Blocked, Unchorded	7,000 (1,226)	—

[a] As defined in Section 12.5.

[b] For single-layer straight lumber sheathing, expected strength shall be multiplied by 1.5 where built-up roofing is present. The value for stiffness shall not be changed.

[c] See Section 12.5.3.6 for shear stiffness and expected strength of wood structural panel diaphragms.

[d] See Section 12.5.3.7 for expected strength of wood structural panel overlays on straight or diagonal lumber sheathing.

[e] See Section 12.5.3.8 for expected strength of wood structural panel overlays on existing wood structural panel sheathing.

2. The presence and configuration of components and their connections, and the continuity of load paths among components, elements, and systems is to be verified or established; and
3. Other conditions, including neighboring party walls and buildings, presence of nonstructural components, and prior remodeling are to be reviewed and documented.

12.2.3.2 Scope and Procedures for Condition Assessment Condition assessment shall meet the requirements for visual condition assessment in accordance with Section 12.2.3.2.1 or comprehensive condition assessment in accordance with Section 12.2.3.2.2. All primary structural components of the gravity- and seismic-force-resistance system shall be included in the condition assessment.

12.2.3.2.1 Visual Condition Assessment The dimensions and features of all accessible components shall be measured and compared with available design information. Similarly, the configuration and condition of all accessible connections shall be visually verified, with any deformations or anomalies noted.

12.2.3.2.2 Comprehensive Condition Assessment If coverings or other obstructions exist, either partial visual inspection through the use of drilled holes and a fiberscope shall be used or visual inspection shall be performed by local removal of covering materials in accordance with the following requirements:

1. If construction documents exist, at least three different primary connections are to be exposed for each connection type. If no capacity-reducing deviations from the construction documents exist, the sample is considered representative. If deviations are noted, then all coverings from primary connections of that type are to be removed, unless the connection strength is ignored in the seismic evaluation; and
2. In the absence of construction documents, at least 50% of the top and at least 50% of the base connections for each type of vertical element in the seismic-force-resisting system, as well as collectors, boundary components, and tie-downs, are to be exposed and inspected or inspected fiberscopically. If common detailing is observed, this sample is considered representative. If any details or conditions are observed that result in a discontinuous load path, all primary connections are to be exposed.

12.2.3.3 Basis for the Mathematical Building Model The results of the condition assessment shall be used to quantify the following items needed to create the mathematical building model:

1. Component section properties and dimensions,
2. Component configuration and eccentricities,
3. Interaction of nonstructural components and their involvement in seismic-force resistance, and
4. Presence and effects of alterations to the structural system.

All deviations noted between available construction records and as-built conditions shall be accounted for in the structural analysis.

12.2.4 Knowledge Factor A knowledge factor, κ, for computation of wood component capacities and permissible deformations shall be selected in accordance with Section 6.2.3.1.

12.2.4.1 Wood Components and Assemblies If a comprehensive condition assessment is performed in accordance with Section 12.2.3.2.2, a knowledge factor, κ = 1.0, shall be permitted in conjunction with default properties of Section 12.2.2.5, and testing in accordance with Section 12.2.2.4 is not required.

12.3 GENERAL ASSUMPTIONS AND REQUIREMENTS

12.3.1 Stiffness Component stiffnesses shall be calculated in accordance with Sections 12.4 through 12.7.

12.3.1.1 Use of Linear Procedures Where design actions are determined using the linear procedures of Chapter 7, the stiffnesses for wood materials comprising individual components shall be based on the material properties determined in accordance with Section 12.2.2.

12.3.1.2 Use of Nonlinear Procedures for Wood Construction Where design actions are determined using the nonlinear procedures of Chapter 7, component force–deformation response shall be represented by nonlinear force–deformation relations. Linear relations shall be permitted where nonlinear response does not occur in the component. The nonlinear force–deformation relation shall be either based on experimental evidence or on the generalized force–deformation relation shown in Figure 12-1, with parameters c, d, and e as defined in Table 12-4 for wood components and assemblies. Distance d is considered the maximum deflection at the point of first loss of strength. Distance e is the maximum deflection at a strength or capacity equal to value c. Where the yield strength is not determined by testing in accordance with Section 7.6, the yield strength at Point B shall be taken as the expected strength at Point C divided by 1.5.

12.3.2 Strength and Acceptance Criteria

12.3.2.1 General Actions in a structure shall be classified as being either deformation controlled or force controlled, as defined in Section 7.5.1. Design strengths for deformation-controlled and force-controlled actions shall be calculated in accordance with Sections 12.3.2.2 and Sections 12.3.2.3, respectively.

12.3.2.2 Deformation-Controlled Actions The requirements for deformation-controlled actions shall be in accordance with Section 12.3.2.2.1 for wood construction.

12.3.2.2.1 Wood Construction Expected strengths for deformation-controlled actions, Q_{CE}, shall be taken as the mean maximum strengths obtained experimentally or calculated using accepted principles of mechanics. Unless other procedures are specified in this chapter, expected strengths shall be permitted to be based on 1.5 times the yield strengths. Yield strengths shall be determined using load and resistance factor design (LRFD) procedures contained in NDS, except that the resistance factor, ϕ, shall be taken as 1.0 and expected material properties shall be determined in accordance with Section 12.2.2. Acceptance criteria for deformation-controlled actions shall be as specified in Sections 12.4 through 12.7.

12.3.2.3 Force-Controlled Actions The requirements for force-controlled actions shall be in accordance with Section 12.3.2.3.1 for wood construction.

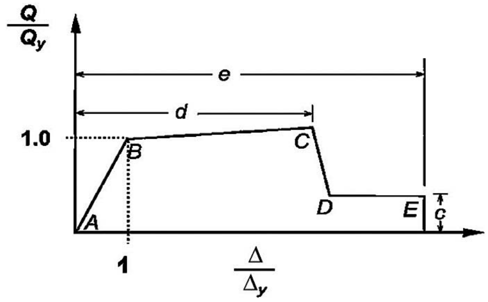

Figure 12-1. Generalized force–deformation relation for wood elements or components.

12.3.2.3.1 Wood Construction Where determined by testing, lower-bound strengths for force-controlled actions, Q_{CL}, shall be taken as mean minus one standard deviation of the maximum strengths obtained experimentally. Where calculated using established principles of mechanics or based on LRFD procedures contained in NDS, the resistance factor, ϕ, shall be taken as 1.0, and default lower-bound material properties determined in accordance with Section 12.2.2.5 shall be used.

Where the force-controlled design actions, Q_{UF}, calculated in accordance with Section 7.5.2.1.2, are based on a limit-state analysis, the expected strength of the components delivering load to the component under consideration shall be taken as not less than 1.5 times the yield strength.

12.3.3 Connection Requirements The connections between wood components shall be addressed in accordance with Section 12.3.3.1.

12.3.3.1 Wood Construction Unless otherwise specified in this standard, connections between wood components of a seismic-force-resisting system shall be considered in accordance with this section. Demands on connectors, including nails, screws, lags, bolts, split rings, and shear plates used to connect wood components to other wood or metal components shall be considered deformation-controlled actions. Demands on bodies of connected wood elements and bodies of connection hardware shall be considered force-controlled actions.

12.3.4 Components Supporting Discontinuous Shear Walls

12.3.4.1 Wood Construction Axial compression on wood posts and flexure and shear on wood beams that support discontinuous shear walls shall be considered force-controlled actions. Lower-bound strengths shall be determined in accordance with Section 12.3.2.3.

12.3.5 Retrofit Measures Retrofit measures for wood construction shall be in accordance with Section 12.3.5.1.

12.3.5.1 Wood Construction If portions of a wood building structure are deficient for the selected Performance Objective, the structure shall be rehabilitated, reinforced, or replaced. If replacement of the element is selected or if new elements are added, the new elements shall satisfy the acceptance criteria of this standard and shall be detailed and constructed in accordance with an approved building code. If reinforcement of the existing framing system is selected, the following factors shall be considered:

1. Degree of degradation in the component from such mechanisms as biological attack, creep, high static or dynamic loading, moisture, or other effects;
2. Level of steady-state stress in the components to be reinforced and the potential to temporarily remove this stress, if appropriate;
3. Elastic and inelastic properties of existing components; strain compatibility with any new reinforcement materials shall be provided;
4. Ductility, durability, and suitability of existing connectors between components, and access for reinforcement or modification;
5. Efforts necessary to achieve appropriate fit-up for reinforcing components and connections;
6. Load path and deformation of the components at end connections; and
7. Presence of components manufactured with archaic materials, which may contain material discontinuities, to be examined during the retrofit design to ensure that the selected reinforcement is feasible.

12.4 WOOD SHEAR WALLS

12.4.1 General Wood shear walls shall be categorized as primary or secondary components in accordance with Section 7.5.1.

Dissimilar wall sheathing materials on opposite sides of a wall shall be permitted to be combined where there are test data to substantiate the stiffness and strength properties of the combined systems. Otherwise, walls sheathed with dissimilar materials shall be analyzed based on only the wall sheathing with the greatest capacity.

For overturning calculations on shear wall elements, stability shall be evaluated in accordance with Section 7.2.9. Net tension caused by overturning shall be resisted by uplift connections.

The effects of openings in shear walls shall be considered. Where required, reinforcement consisting of chords and collectors shall be added to provide sufficient load capacity around openings to meet the strength requirements for shear walls.

Connections between shear walls and other components, including diaphragm ties, collectors, diaphragms, posts, and foundations, shall be considered in accordance with Section 12.3.3 and shall be designed for forces calculated in accordance with Chapter 7. Components supporting discontinuous shear walls shall be considered in accordance with Section 12.3.4.

The expected strength, Q_{CE}, of wood shear wall assemblies shall be determined in accordance with Section 12.4.3.

12.4.2 Types of Wood Shear Walls

12.4.2.1 Existing Wood Shear Walls

12.4.2.1.1 Single-Layer Horizontal Lumber Sheathing or Siding Single-layer horizontal lumber sheathing or siding shall include horizontal sheathing or siding applied directly to studs or horizontal boards nailed to studs 2 in. (50.8 mm) nominal or wider.

12.4.2.1.2 Diagonal Lumber Sheathing Diagonal lumber sheathing shall include sheathing applied at approximately a 45-degree angle to the studs in a single or double layer with three or more nails per stud, sill, and top plates.

12.4.2.1.3 Vertical Wood Siding Only Vertical wood siding shall include vertical boards nailed directly to studs and blocking 2 in. (50.8 mm) nominal or wider.

12.4.2.1.4 Wood Siding over Horizontal Lumber Sheathing Wood siding over horizontal sheathing shall include siding connected to horizontal lumber sheathing with nails that go through the sheathing to the studs.

12.4.2.1.5 Wood Siding over Diagonal Lumber Sheathing Wood siding over diagonal lumber sheathing shall include siding connected to diagonal lumber sheathing with nails that go through the lumber sheathing to the studs.

12.4.2.1.6 Wood Structural Panel Sheathing or Siding Wood structural panel sheathing or siding shall include wood structural panels oriented vertically or horizontally and nailed to studs 2 in. (50.8 mm) nominal or wider.

12.4.2.1.7 Stucco on Studs Stucco on studs (over sheathing or wire-backed building paper) shall include portland cement plaster applied to wire lath or expanded metal lath. Wire lath or expanded metal lath shall be nailed to the studs.

12.4.2.1.8 Gypsum Plaster on Wood Lath Gypsum plaster on wood lath shall include gypsum plaster keyed onto spaced wood lath that is nailed to the studs.

12.4.2.1.9 Gypsum Plaster on Gypsum Lath Gypsum plaster on gypsum lath shall include plaster that is glued or keyed to gypsum lath nailed to studs.

12.4.2.1.10 Gypsum Wallboard Gypsum wallboard shall include manufactured panels with a paper facing and gypsum core that are oriented horizontally or vertically and nailed to studs or blocking in a single layer or multiple layers.

12.4.2.1.11 Gypsum Sheathing Gypsum sheathing shall include manufactured gypsum panels that are oriented horizontally or vertically and nailed to studs or blocking.

12.4.2.1.12 Plaster on Metal Lath Plaster on metal lath shall include gypsum plaster applied to expanded wire lath that is nailed to the studs.

12.4.2.1.13 Horizontal Lumber Sheathing with Cut-In Braces or Diagonal Blocking Horizontal lumber sheathing with cut-in braces or diagonal blocking shall include nominal 1 in. (25.4 mm) wide horizontal lumber sheathing or siding applied directly to studs or nominal 1 in. × 4 in. (25.4 mm × 101.6 mm) to 1 in. × 12 in. (304.8 mm) horizontal boards nailed to studs 2 in. (50.8 mm) nominal or wider. The wall shall be braced with diagonal cut-in braces or blocking extending from corner to corner.

12.4.2.1.14 Fiberboard or Particleboard Sheathing Fiberboard or particleboard sheathing walls shall include fiberboard or particleboard panels that are applied directly to the studs with nails.

12.4.2.2 Enhanced Wood Shear Walls Enhanced wood shear walls shall include existing shear walls retrofitted in accordance with this standard or an approved method. Enhanced wood shear walls consisting of wood structural panel sheathing added to unfinished wood stud walls or wood structural panel sheathing overlay on existing wood stud shear walls shall be evaluated in accordance with Section 12.4.3.6. Where wood structural panel sheathing is applied over existing sheathing, the expected strength shall be based on the expected strength of the overlaid material only and reduced by 20% unless a different value is substantiated by testing.

12.4.2.3 New Wood Shear Walls New wood shear walls shall include all new wood structural panel shear walls added to an existing seismic-force-resisting system. Design of new wood shear walls shall satisfy the acceptance criteria of this standard. Details of construction for new wood shear walls shall be in accordance with the requirements of the AWC SDPWS or an approved building code.

12.4.3 Stiffness, Strength, Acceptance Criteria, and Connection Design for Wood Shear Walls

12.4.3.1 Single-Layer Horizontal Lumber Sheathing or Siding Shear Walls

12.4.3.1.1 Stiffness of Single-Layer Horizontal Lumber Sheathing or Siding Shear Walls The deflection of single-layer horizontal lumber sheathing or siding shear walls shall be calculated in accordance with Equation (12-1):

$$\Delta_y = v_y h/G_d + (h/b)d_a \tag{12-1}$$

where

v_y = Shear per unit length at yield in the direction under consideration, in lb/ft (N/mm);
h = Shear wall height, in feet (meter);
G_d = Diaphragm shear stiffness from Table 12-2, in lb/in. (N/mm);
b = Shear wall width, in feet (meter);
d_a = Elongation of anchorage at end of wall determined by anchorage details and load magnitude, in inches (millimeter); and
Δ_y = Calculated shear wall deflection at yield, in in. (mm).

Properties used to compute shear wall deflection and stiffness shall be based on Section 12.2.2.

12.4.3.1.2 Strength of Single-Layer Horizontal Lumber Sheathing or Siding Shear Walls The expected strength of horizontal lumber sheathing or siding shall be determined in accordance with Section 12.2.2.

12.4.3.1.3 Acceptance Criteria for Single-Layer Horizontal Lumber Sheathing or Siding Shear Walls For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relations, described by Figure 12-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 12-4.

12.4.3.1.4 Connections of Single-Layer Horizontal Lumber Sheathing or Siding Shear Walls The connections between parts of the shear wall assembly and other elements of the seismic-force-resisting system shall be considered in accordance with Section 12.4.1.

12.4.3.2 Diagonal Lumber Sheathing Shear Walls

12.4.3.2.1 Stiffness of Diagonal Lumber Sheathing Shear Walls The deflection of diagonal lumber sheathing shear walls shall be determined using Equation (12-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 12.2.2.

12.4.3.2.2 Strength of Diagonal Lumber Sheathing Shear Walls The expected strength of diagonal lumber sheathing shear walls shall be determined in accordance with Section 12.2.2.

12.4.3.2.3 Acceptance Criteria for Diagonal Lumber Sheathing Shear Walls For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria for primary and secondary components, shall be taken from Table 12-4.

12.4.3.2.4 Connections for Diagonal Lumber Sheathing Shear Walls The connections between parts of the shear wall assembly and other elements of the seismic-force-resisting system shall be considered in accordance with Section 12.4.1.

12.4.3.3 Vertical Wood Siding Shear Walls

12.4.3.3.1 Stiffness of Vertical Wood Siding Shear Walls The deflection of vertical wood siding shear walls shall be determined using Equation (12-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 12.2.2.

12.4.3.3.2 Strength of Vertical Wood Siding Shear Walls The expected strength of vertical wood siding shear walls shall be determined in accordance with Section 12.2.2.

12.4.3.3.3 Acceptance Criteria for Vertical Wood Siding Shear Walls For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 12-4.

Table 12-3. Numerical Acceptance Factors for Linear Procedures: Wood Components.

		m-Factors				
			Primary		Secondary	
	Height/Width Ratio (*h*/*b*)	IO	LS	CP	LS	CP
Wood Shear Walls[a,b]						
Horizontal lumber sheathing	≤1.0	1.6	3.4	4.0	4.0	5.0
Wood siding over horizontal lumber sheathing	≤1.5	1.3	2.3	2.6	2.8	3.0
Diagonal lumber sheathing	≤1.5	1.4	2.7	3.1	3.1	3.6
Wood siding over diagonal lumber sheathing	≤2.0	1.3	2.0	2.3	2.5	2.8
Double-layer diagonal lumber sheathing	≤2.0	1.2	1.7	1.9	2.0	2.5
Wood structural panel sheathing or siding	≤3.5	1.7	3.8	4.5	4.5	5.5
Stucco on studs[c]	≤1.0	1.5	3.1	3.6	3.6	4.0
	2.0	1.3	2.2	2.5	5.0	6.0
Stucco over 1 in. × horizontal lumber sheathing	≤2.0	1.5	3.0	3.5	3.5	4.0
Gypsum plaster on wood lath	≤2.0	1.7	3.9	4.6	4.6	5.1
Gypsum plaster on gypsum lath	≤2.0	1.8	4.2	5.0	4.2	5.5
Gypsum plaster on metal lath	≤2.0	1.7	3.7	4.4	3.7	5.0
Gypsum sheathing	≤2.0	1.9	4.7	5.7	4.7	6.0
Gypsum wallboard[c]	≤1.0	1.9	4.7	5.7	4.7	6.0
	2.0	1.6	3.4	4.0	3.8	4.5
Horizontal 1 in. × 6 in. (25.4 mm × 152.4 mm) lumber sheathing with cut-in braces or diagonal blocking	≤1.0	1.7	3.7	4.4	4.2	4.8
Fiberboard or particleboard sheathing	≤1.5	1.6	3.2	3.8	3.8	5.0
	Length/Width Ratio (*L*/*b*)					
Diaphragms[d]						
Single-layer straight lumber sheathing, chorded	≤3.0	1	2.0	2.5	2.4	3.1
Single-layer straight lumber sheathing, unchorded	≤3.0	1	1.5	2.0	1.8	2.5
Double-layer straight lumber sheathing, chorded	≤3.0	1.25	2.0	2.5	2.3	2.8
Double-layer straight lumber sheathing, unchorded	≤3.0	1	1.5	2.0	1.8	2.3
Single-layer diagonal lumber sheathing, chorded	≤3.0	1.25	2.0	2.5	2.3	2.9
Single-layer diagonal lumber sheathing, unchorded	≤3.0	1	1.5	2.0	1.8	2.5
Straight lumber sheathing over diagonal lumber sheathing, chorded	≤3.0	1.5	2.5	3.0	2.8	3.5
Straight lumber sheathing over diagonal lumber sheathing, unchorded	≤3.0	1.25	2.0	2.5	2.3	3.0
Double-layer diagonal lumber sheathing, chorded	≤3.5	1.5	2.5	3.0	2.9	3.5
Double-layer diagonal lumber sheathing, unchorded	≤3.5	1.25	2.0	2.5	2.4	3.1
Wood structural panel, blocked, chorded[c]	≤3.0	1.5	3.0	4.0	3.0	4.5
	4	1.5	2.5	3.0	2.8	3.5
Wood structural panel, unblocked, chorded[c]	≤3	1.5	2.5	3.0	2.9	4.0
	4	1.5	2.0	2.5	2.6	3.2
Wood structural panel, blocked, unchorded[c]	≤2.5	1.25	2.5	3.0	2.9	4.0
	3.5	1.25	2.0	2.5	2.6	3.2
Wood structural panel, unblocked, unchorded[c]	≤2.5	1.25	2.0	2.5	2.4	3.0
	3.5	1.0	1.5	2.0	2.0	2.6
Wood structural panel overlay on sheathing, chorded[c]	≤3	1.5	2.5	3.0	2.9	4.0
	4	1.5	2.0	2.5	2.6	3.2
Wood structural panel overlay on sheathing, unchorded[c]	≤2.5	1.25	2.0	2.5	2.4	3.0
	3.5	1.0	1.5	2.0	1.9	2.6
Components/Elements						
Frame components subject to axial tension and/or bending		1.0	2.5	3.0	2.5	4.0
Frame components subject to axial compression		Force controlled				
Wood piles, bending and axial		1.2	2.5	3.0	—	—

continues

Table 12-3 *(Continued)*. Numerical Acceptance Factors for Linear Procedures: Wood Components.

	Height/Width Ratio (*h*/*b*)	*m*-Factors				
			Primary		Secondary	
		IO	LS	CP	LS	CP
Cantilever pole structures, bending and axial		1.2	3.0	3.5	—	—
Pole structures with diagonal bracing		1.0	2.5	3.0	—	—
Connectors[e]						
Nails—8d and larger—wood to wood		2.0	6.0	8.0	8.0	9.0
Nails—8d and larger—metal to wood		2.0	4.0	6.0	5.0	7.0
Screws—wood to wood		1.2	2.0	2.2	2.0	2.5
Screws—metal to wood		1.1	1.8	2.0	1.8	2.3
Lag bolts—wood to wood		1.4	2.5	3.0	2.5	3.3
Lag bolts—metal to wood		1.3	2.3	2.5	2.4	3.0
Machine bolts—wood to wood		1.3	3.0	3.5	3.3	3.9
Machine bolts—metal to wood		1.4	2.8	3.3	3.1	3.7
Split rings and shear plates		1.3	2.2	2.5	2.3	2.7

[a] Shear walls shall be permitted to be classified as secondary components or nonstructural components, subject to the limitations of Section 7.2.4.3. Acceptance criteria need not be considered for walls classified as secondary or nonstructural.

[b] Shear wall components with aspect ratios exceeding maximum listed values shall not be considered effective in resisting seismic forces.

[c] Linear interpolation shall be permitted for intermediate values of aspect ratio.

[d] For diaphragm components with aspect ratios between maximum listed values and 4.0, *m*-factors shall be decreased by linear interpolation between the listed values and 1.0. Diaphragm components with aspect ratios exceeding 4.0 shall not be considered effective in resisting seismic forces.

[e] Actions on connectors not listed in this table shall be considered force controlled.

Note: IO = Immediate Occupancy; LS = Life Safety, CP = Collapse Prevention

Table 12-4. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures: Wood Components.

		Modeling Parameters			Acceptance Criteria		
				Residual Strength Ratio	Acceptable Deformation Ratio Δ/Δ_y		
		Δ/Δ_y			Performance Level		
		d	e	c	IO	LS	CP
	Height/Width Ratio (h/b)						
Wood Shear Walls[a]							
Horizontal lumber sheathing	≤1.0	4	5	0.3	1.8	4	5
Wood siding over horizontal lumber sheathing	≤1.5	2.6	3.6	0.2	1.4	2.6	3.6
Diagonal lumber sheathing	≤1.5	3.1	4	0.2	1.5	3.1	4
Wood siding over diagonal lumber sheathing	≤2.0	2.3	3	0.2	1.3	2.3	3
Double-layer diagonal lumber sheathing	≤2.0	2	2.5	0.2	1.3	2	2.5
Vertical 1 in. × 10 in. (25.4 mm × 254 mm) lumber sheathing	≤1.0	3.6	4	0.3	1.7	3.6	4
Wood structural panel sheathing or siding	≤3.5	4.5	5.5	0.3	1.9	4.5	5.5
Stucco on studs[b]	≤1.0	3.6	4	0.2	1.7	3.6	4
	2.0	2.5	3	0.2	1.4	2.5	3
Stucco over 1 in. × horizontal lumber sheathing	≤2.0	3.5	4	0.2	1.6	3.5	4
Gypsum plaster on wood lath	≤2.0	4.6	5	0.2	1.9	4.6	5
Gypsum plaster on gypsum lath	≤2.0	5	6	0.2	2	5	6
Gypsum plaster on metal lath	≤2.0	4.4	5	0.2	1.9	4.4	5
Gypsum sheathing	≤2.0	5.7	6.3	0.2	2.2	5.7	6.3
Gypsum wallboard[b]	≤1.0	5.7	6.3	0.2	2.2	5.7	6.3
	2.0	4	5	0.2	1.8	4	5

continues

Table 12-4 *(Continued)*. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures: Wood Components.

		Modeling Parameters			Acceptance Criteria		
		Δ/Δ_y		Residual Strength Ratio	Acceptable Deformation Ratio Δ/Δ_y		
					Performance Level		
		d	e	c	IO	LS	CP
Horizontal 1 in. × 6 in. (25.4 mm × 152.4 mm) lumber sheathing with cut-in braces or diagonal blocking	≤1.0	4.4	5	0.2	1.9	4.4	5
Fiberboard or particleboard sheathing	≤1.5	3.8	4	0.2	1.7	3.8	4
	Length/Width Ratio (L/b)						
Diaphragms[c]							
Single-layer straight lumber sheathing, chorded	≤2.0	2.5	3.5	0.2	1.4	2.5	3.5
Single-layer straight lumber sheathing, unchorded	≤2.0	2	3	0.3	1.3	2	3
Double-layer straight lumber sheathing, chorded	≤2.0	2.5	3.5	0.2	1.4	2.5	3.5
Double-layer straight lumber sheathing, unchorded	≤2.0	2	3	0.3	1.3	2	3
Single-layer diagonal lumber sheathing, chorded	≤2.0	2.5	3.5	0.2	1.4	2.5	3.5
Single-layer diagonal lumber sheathing, unchorded	≤2.0	2	3	0.3	1.3	2	3
Straight lumber sheathing over diagonal lumber sheathing, chorded	≤2.0	3	4	0.2	1.5	3	4
Straight lumber sheathing over diagonal lumber sheathing, unchorded	≤2.0	2.5	3.5	0.3	1.4	2.5	3.5
Double-layer diagonal lumber sheathing, chorded	≤2.0	3	4	0.2	1.5	3	4
Double-layer diagonal lumber sheathing, unchorded	≤2.0	2.5	3.5	0.2	1.4	2.5	3.5
Wood structural panel, blocked, chorded[b]	≤3	4	5	0.3	1.8	4	5
	4	3	4	0.3	1.5	3	4
Wood structural panel, unblocked, chorded[b]	≤3	3	4	0.3	1.5	3	4
	4	2.5	3.5	0.3	1.4	2.5	3.5
Wood structural panel, blocked, unchorded[b]	≤2.5	3	4	0.3	1.5	3	4
	3.5	2.5	3.5	0.3	1.4	2.5	3.5
Wood structural panel, unblocked, unchorded[b]	≤2.5	2.5	3.5	0.4	1.4	2.5	3.5
	3.5	2	3	0.4	1.3	2	3
Wood structural panel overlay on sheathing, chorded[b]	≤3	3	4	0.3	1.5	3	4
	4	2.5	3.5	0.3	1.4	2.5	3.5
Wood structural panel overlay on sheathing, unchorded[b]	≤2.5	2.5	3.5	0.4	1.4	2.5	3.5
	3.5	2	3	0.4	1.3	2	3
Connectors[d]							
Nails—wood to wood		7	8	0.2	2.5	7	8
Nails—metal to wood		5.5	7	0.2	2.1	5.5	7
Screws—Wood to wood		2.5	3	0.2	1.4	2.5	3
Screws—Wood to metal		2.3	2.8	0.2	1.3	2.3	2.8
Lag bolts—Wood to wood		2.8	3.2	0.2	1.5	2.8	3.2
Lag bolts—Metal to wood		2.5	3	0.2	1.4	2.5	3
Bolts—Wood to wood		3	3.5	0.2	1.5	3	3.5
Bolts—Metal to wood		2.8	3.3	0.2	1.5	2.8	3.3

[a] Shear wall components with aspect ratios exceeding maximum listed values shall not be considered effective in resisting seismic forces.

[b] Linear interpolation shall be permitted for intermediate values of aspect ratio.

[c] For diaphragm components with aspect ratios between maximum listed values and 4.0, deformation ratios shall be decreased by linear interpolation between the listed values and 1.0. Diaphragm components with aspect ratios exceeding 4.0 shall not be considered effective in resisting seismic forces.

[d] Actions on connectors not listed in this table shall be considered force controlled.

Note: IO = Immediate Occupancy, LS = Life Safety, CP = Collapse Prevention.

12.4.3.3.4 Connections of Vertical Wood Siding Shear Walls The presence of connections between parts of the vertical wood siding shear wall assembly and other elements of the seismic-force-resisting system shall be verified. If connections are present, they need not be considered in the analysis conducted in accordance with Chapter 7. In the absence of connections, connections shall be provided in accordance with Section 12.4.1.

12.4.3.4 Wood Siding over Horizontal Lumber Sheathing Shear Walls

12.4.3.4.1 Stiffness of Wood Siding over Horizontal Lumber Sheathing Shear Walls The deflection of wood siding over horizontal lumber sheathing shear walls shall be determined using Equation (12-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 12.2.2.

12.4.3.4.2 Strength of Wood Siding over Horizontal Lumber Sheathing Shear Walls The expected strength of wood siding over horizontal lumber sheathing shall be determined in accordance with Section 12.2.2.

12.4.3.4.3 Acceptance Criteria for Wood Siding over Horizontal Lumber Sheathing Shear Walls For linear procedures, m-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 12-4.

12.4.3.4.4 Connections of Wood Siding over Horizontal Lumber Sheathing Shear Walls The connections between parts of the shear wall assembly and other elements of the seismic-force-resisting system shall be considered in accordance with Section 12.4.1.

12.4.3.5 Wood Siding over Diagonal Lumber Sheathing Shear Walls

12.4.3.5.1 Stiffness of Wood Siding over Diagonal Lumber Sheathing Shear Walls The deflection of these shear walls shall be calculated in accordance with Equation (12-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 12.2.2.

12.4.3.5.2 Strength of Wood Siding over Diagonal Lumber Sheathing Shear Walls The expected strength of wood siding over diagonal lumber sheathing shall be determined in accordance with Section 12.2.2.

12.4.3.5.3 Acceptance Criteria for Wood Siding over Diagonal Lumber Sheathing Shear Walls For linear procedures, m-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 12-4.

12.4.3.5.4 Connections of Wood Siding over Diagonal Lumber Sheathing Shear Walls The connections between parts of the shear wall assembly and other elements of the seismic-force-resisting system shall be considered in accordance with Section 12.4.1.

12.4.3.6 Wood Structural Panel Sheathing or Siding Shear Walls

12.4.3.6.1 Stiffness of Wood Structural Panel Sheathing or Siding Shear Walls The deflection of wood structural panel shear walls at yield shall be determined using Equation (12-2):

$$\Delta_y = 8v_y h^3/(EAb) + v_y h/(G_v t_v) + 0.75he_n + (h/b)d_a \qquad (12\text{-}2)$$

$$\Delta_y = 2v_y h^3/(3EAb) + v_y h/(G_v t_v) + he_n/406 + (h/b)d_a \qquad (12\text{-}2.\text{SI})$$

where

v_y = Shear per unit length at yield in the direction under consideration, in lb/ft (N/mm);
h = Shear wall height, in feet (millimeters);
E = Modulus of elasticity of boundary member, in lb/in.^{2} (N/mm^2);
A = Area of boundary member cross section, in in.^{2} (mm^2);
b = Shear wall width, in feet (millimeters);
$G_v t_v$ = Shear stiffness of wood structural panel, in lb/in. of depth (N/mm of depth);
d_a = Deflection of anchorage at end of wall determined by anchorage details and load magnitude, in in. (mm);
e_n = Nail deformation at yield load per nail, in inches (millimeters) (values listed are for Structural I panels; multiply by 1.2 for all other panel grades);
= 0.13 in. (3.3 mm) for 6d nails at yield;
= 0.08 in. (2.0 mm) for 8d nails at yield;
= 0.08 in. (2.0 mm) for 10d nails at yield; and
Δ_y = Calculated shear wall deflection at yield, in inches (millimeters).

Properties used to compute shear wall deflection and stiffness shall be based on Section 12.2.2.

12.4.3.6.2 Strength of Wood Structural Panel Sheathing or Siding Shear Walls The expected strength of wood structural panel shear walls shall be taken as mean maximum strengths obtained experimentally. Expected strengths of wood structural panel shear walls shall be permitted to be based on 1.07 times nominal strengths. Yield strengths of wood structural panel shear walls shall be permitted to be based on nominal strengths divided by 1.4. Nominal strengths shall be determined using LRFD procedures contained in AWC SDPWS, except that the resistance factor, ϕ, shall be taken as 1.0 and expected material properties shall be determined in accordance with Section 12.2.2.

Approved allowable stress values for fasteners shall be permitted to be converted in accordance with Section 12.2.2.5.1, where the strength of a shear wall is computed using principles of mechanics.

For existing wood structural panel shear walls framed with 2 in. (50.8 mm) nominal framing at adjoining panel edges where 3 in. (76.2 mm) nominal framing is required per AWC SDPWS, the expected strength shall not be taken as greater than 0.90 times the expected strength associated with use of 3 in. (76.2 mm) nominal framing at adjoining panel edges.

12.4.3.6.3 Acceptance Criteria for Wood Structural Panel Sheathing or Siding Shear Walls For linear procedures, m-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described in Equation (12-1), and deformation acceptance criteria for primary and secondary components shall be taken from Table 12-4.

12.4.3.6.4 Connections of Wood Structural Panel Sheathing or Siding Shear Walls The connections between parts of the shear wall assembly and other elements of the seismic-force-resisting system shall be considered in accordance with Section 12.4.1.

12.4.3.7 Stucco on Studs, Sheathing, or Fiberboard Shear Walls

12.4.3.7.1 Stiffness of Stucco on Studs, Sheathing, or Fiberboard Shear Walls The deflection of stucco on studs, sheathing, or fiberboard shear walls shall be determined using Equation (12-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 12.2.2.

12.4.3.7.2 Strength of Stucco on Studs, Sheathing, or Fiberboard Shear Walls The expected strength of stucco on studs, sheathing, or fiberboard shall be determined in accordance with Section 12.2.2.

12.4.3.7.3 Acceptance Criteria for Stucco on Studs, Sheathing, or Fiberboard Shear Walls For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 12-4.

12.4.3.7.4 Connections of Stucco on Studs, Sheathing, or Fiberboard Shear Walls The connection between the stucco mesh and the wood framing shall be investigated. The connections between the shear wall and foundation, and between the shear wall and other elements of the seismic-force-resisting system, shall be considered in accordance with Section 12.4.1.

12.4.3.8 Gypsum Plaster on Wood Lath Shear Walls

12.4.3.8.1 Stiffness of Gypsum Plaster on Wood Lath Shear Walls The deflection of gypsum plaster on wood lath shear walls shall be determined using Equation (12-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 12.2.2.

12.4.3.8.2 Strength of Gypsum Plaster on Wood Lath Shear Walls The expected strength of gypsum plaster shall be determined in accordance with Section 12.2.2.

12.4.3.8.3 Acceptance Criteria for Gypsum Plaster on Wood Lath Shear Walls For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 12-4.

12.4.3.8.4 Connections of Gypsum Plaster on Wood Lath Shear Walls The presence of connections between parts of the shear wall assembly and other elements of the seismic-force-resisting system shall be verified. If connections are absent, they shall be provided in accordance with Section 12.4.1.

12.4.3.9 Gypsum Plaster on Gypsum Lath Shear Walls

12.4.3.9.1 Stiffness of Gypsum Plaster on Gypsum Lath Shear Walls The deflection of gypsum plaster on gypsum lath shear walls shall be determined using Equation (12-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 12.2.2.

12.4.3.9.2 Strength of Gypsum Plaster on Gypsum Lath Shear Walls The expected strength of gypsum plaster on gypsum lath shear walls shall be determined in accordance with Section 12.2.2.

12.4.3.9.3 Acceptance Criteria for Gypsum Plaster on Gypsum Lath Shear Walls For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 12-4.

12.4.3.9.4 Connections of Gypsum Plaster on Gypsum Lath Shear Walls The presence of connections between parts of the shear wall assembly and other elements of the seismic-force-resisting system shall be verified. If connections are present, they need not be considered in the analysis conducted in accordance with Chapter 7. If connections are absent, they shall be provided in accordance with Section 12.4.1.

12.4.3.10 Gypsum Wallboard Shear Walls

12.4.3.10.1 Stiffness of Gypsum Wallboard Shear Walls The deflection of gypsum wallboard shear walls shall be determined using Equation (12-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 12.2.2.

12.4.3.10.2 Strength of Gypsum Wallboard Shear Walls The expected strength of gypsum wallboard shear walls shall be determined in accordance with Section 12.2.2.

12.4.3.10.3 Acceptance Criteria for Gypsum Wallboard Shear Walls For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 12-4.

12.4.3.10.4 Connections of Gypsum Wallboard Shear Walls The connections between parts of the shear wall assembly and other elements of the seismic-force-resisting system shall be considered in accordance with Section 12.4.1.

12.4.3.11 Gypsum Sheathing Shear Walls

12.4.3.11.1 Stiffness of Gypsum Sheathing Shear Walls The deflection of gypsum sheathed shear walls shall be determined using Equation (12-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 12.2.2.

12.4.3.11.2 Strength of Gypsum Sheathing Shear Walls The expected strength of gypsum wallboard shear walls shall be determined in accordance with Section 12.2.2.

12.4.3.11.3 Acceptance Criteria for Gypsum Sheathing Shear Walls For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 12-4.

12.4.3.11.4 Connections of Gypsum Sheathing Shear Walls The connections between parts of the shear wall assembly and other elements of the seismic-force-resisting system shall be considered in accordance with Section 12.4.1.

12.4.3.12 Plaster on Metal Lath Shear Walls

12.4.3.12.1 Stiffness of Plaster on Metal Lath Shear Walls The deflection of plaster on metal lath shear walls shall be determined using Equation (12-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 12.2.2.

12.4.3.12.2 Strength of Plaster on Metal Lath Shear Walls The expected strength of plaster on metal lath shear walls shall be determined in accordance with Section 12.2.2.

12.4.3.12.3 Acceptance Criteria for Plaster on Metal Lath Shear Walls For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 12-4.

12.4.3.12.4 Connections of Plaster on Metal Lath Shear Walls The presence of connections between parts of the shear wall assembly and other elements of the seismic-force-resisting system shall be verified. If connections are present, they need not be considered in the analysis conducted in accordance with Chapter 7. If connections are absent, they shall be provided in accordance with Section 12.4.1.

12.4.3.13 Horizontal Lumber Sheathing with Cut-In Braces or Diagonal Blocking Shear Walls

12.4.3.13.1 Stiffness of Horizontal Lumber Sheathing with Cut-In Braces or Diagonal Blocking Shear Walls The deflection of horizontal lumber sheathing with cut-in braces or diagonal blocking shear walls shall be calculated using Equation (12-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 12.2.2.

12.4.3.13.2 Strength of Horizontal Lumber Sheathing Shear Walls with Cut-In Braces or Diagonal Blocking The expected strength of horizontal lumber sheathing or siding shall be determined in accordance with Section 12.2.2.

12.4.3.13.3 Acceptance Criteria for Horizontal Lumber Sheathing with Cut-In Braces or Diagonal Blocking Shear Walls For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 12-4.

12.4.3.13.4 Connections of Horizontal Lumber Sheathing with Cut-In Braces or Diagonal Blocking Shear Walls The connections between the parts of the shear wall assembly and other elements of the seismic-force-resisting system shall be considered in accordance with Section 12.4.1.

12.4.3.14 Fiberboard or Particleboard Sheathing Shear Walls

12.4.3.14.1 Stiffness of Fiberboard or Particleboard Sheathing Shear Walls For structural particleboard sheathing, see Section 12.4.3.6. The deflection of shear walls sheathed in nonstructural particleboard shall be determined using Equation (12-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 12.2.2. Fiberboard sheathing shall not be considered a structural element for resisting seismic loads.

12.4.3.14.2 Strength of Fiberboard or Particleboard Sheathing Shear Walls The expected strength of structural particleboard shall be based on Section 12.4.3.6. The strength of nonstructural fiberboard or particleboard sheathed walls shall be determined in accordance with Section 12.2.2.

12.4.3.14.3 Acceptance Criteria for Fiberboard or Particleboard Sheathing Shear Walls For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 12-4.

12.4.3.14.4 Connections of Fiberboard or Particleboard Sheathing Shear Walls The connections between parts of structural particleboard shear wall assemblies and other elements of the seismic-force-resisting system shall be considered in accordance with Section 12.4.1.

The presence of connections between parts of nonstructural particleboard shear wall assemblies and other elements of the seismic-force-resisting system shall be verified. If connections are present, they need not be considered in the analysis conducted in accordance with Chapter 7. If connections are absent, they shall be provided in accordance with Section 12.4.1.

12.5 WOOD DIAPHRAGMS

12.5.1 General The expected strength of wood diaphragm assemblies, Q_{CE}, shall be determined in accordance with Sections 12.5.3.1 to 12.5.3.8. The expected strength, Q_{CE}, of braced horizontal diaphragm systems shall be determined in accordance with Section 12.5.3.9.

The effects of openings in wood diaphragms shall be considered. Chords and collectors shall be added to provide sufficient load capacity around openings to meet the strength requirements for the diaphragm or analysis performed to demonstrate adequacy of the diaphragm without chords and collectors.

Connections between diaphragms and other components, including shear walls, diaphragm ties, collectors, cross ties, and out-of-plane anchors, shall be considered in accordance with Section 12.3.3 and shall be designed for forces calculated in accordance with Chapter 7.

12.5.2 Types of Wood Diaphragms

12.5.2.1 Existing Wood Diaphragms

12.5.2.1.1 Single-Layer Straight Lumber Sheathing Single-layer straight lumber sheathing diaphragms shall include diaphragms with lumber sheathing laid perpendicular to the framing members.

12.5.2.1.2 Double-Layer Straight Lumber Sheathing Double-layer straight lumber sheathing diaphragms shall include diaphragms with one layer of lumber sheathing laid perpendicular to the framing members and a second layer of lumber sheathing laid either perpendicular or parallel to the first layer, where both layers of lumber sheathing are fastened to the framing members.

12.5.2.1.3 Single-Layer Diagonal Lumber Sheathing Single-layer diagonal lumber sheathing diaphragms shall include diaphragms with lumber sheathing laid at approximately a 45-degree angle and connected to the framing members.

12.5.2.1.4 Diagonal Lumber Sheathing with Straight Lumber Sheathing or Flooring Above Diagonal lumber sheathing with straight lumber sheathing or flooring above shall include diaphragms with sheathing laid at a 45-degree angle to the framing members, with a second layer of straight lumber sheathing or wood flooring laid on top of the diagonal lumber sheathing at a 90-degree angle to the framing members.

12.5.2.1.5 Double-Layer Diagonal Lumber Sheathing Double-layer diagonal lumber sheathing diaphragms shall include diaphragms with one layer of lumber sheathing laid at a 45-degree angle to the framing members and a second layer of sheathing laid at a 90-degree angle to the first layer.

12.5.2.1.6 Wood Structural Panel Sheathing Wood structural panel sheathing diaphragms shall include diaphragms with wood structural panels, or other wood structural panels as defined in this standard, fastened to the framing members.

12.5.2.1.7 Braced Horizontal Diaphragms Braced horizontal diaphragms shall include diaphragms with a horizontal truss system at the floor or roof level of the building.

12.5.2.2 Enhanced Wood Diaphragms Enhanced wood diaphragms shall include existing diaphragms retrofitted in accordance with the standard or by an approved method.

12.5.2.3 New Wood Diaphragms

12.5.2.3.1 New Wood Structural Panel Sheathing New wood structural panel sheathing diaphragms shall include new wood structural panels connected to new framing members or connected to existing framing members after existing sheathing has been removed.

12.5.2.3.2 New Single-Diagonal Sheathing New single-layer diagonal lumber sheathing wood diaphragms shall include new lumber sheathing laid at approximately a 45-degree angle and connected to the existing framing members.

12.5.2.3.3 New Double-Diagonal Sheathing New double-layer diagonal lumber sheathing wood diaphragms shall include diaphragms with new lumber sheathing laid at approximately a 45-degree angle to the existing framing members with a second layer of lumber sheathing laid at approximately a 90-degree angle to the first layer, where both layers shall be connected to the framing members.

12.5.2.3.4 New Braced Horizontal Diaphragms New braced horizontal diaphragms shall include a new horizontal truss system attached to the existing framing at the floor or roof level of the building.

12.5.3 Stiffness, Strength, Acceptance Criteria, and Connection Design for Wood Diaphragms

12.5.3.1 Single-Layer Straight Lumber Sheathing Diaphragms

12.5.3.1.1 Stiffness of Single-Layer Straight Lumber Sheathing Diaphragms The midspan deflection of single span single-layer straight lumber sheathing diaphragms with uniformly distributed load shall be calculated using Equation (12-3):

$$\Delta_y = v_y L/(4G_d) \tag{12-3}$$

$$\Delta_y = 5v_y L^3/(96EAb) + v_y L/(4G_d) + \Sigma(\Delta_c X)/(2b) \qquad \text{(12-3.SI)}$$

where

L = Diaphragm span, distance between shear walls or collectors, in feet (meters);
v_y = Shear per unit length at yield in the direction under consideration, in lb/ft (N/m); and
Δ_y = Calculated diaphragm deflection at yield, in inches (millimeters).

Properties used to compute diaphragm deflection and stiffness shall be based on Section 12.2.2.

12.5.3.1.2 Strength of Single-Layer Straight Lumber Sheathing Diaphragms The expected strength of single-layer straight lumber sheathing diaphragms shall be determined in accordance with Section 12.2.2.

12.5.3.1.3 Acceptance Criteria for Single-Layer Straight Lumber Sheathing Diaphragms For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria shall be taken from Table 12-4.

12.5.3.1.4 Connections of Single-Layer Straight Lumber Sheathing Diaphragms Connections between diaphragms and shear walls and other vertical elements shall be considered in accordance with Section 12.5.1.

12.5.3.2 Double-Layer Straight Lumber Sheathing Diaphragms

12.5.3.2.1 Stiffness of Double-Layer Straight Sheathing Diaphragms The midspan deflection of single span double-layer straight lumber sheathing diaphragms with uniformly distributed load shall be calculated using Equation (12-4):

$$\Delta_y = 5v_y L^3/(8EAb) + v_y L/(4G_d) + \Sigma(\Delta_c X)/(2b) \tag{12-4}$$

$$\Delta_y = 5v_y L^3/(96EAb) + v_y L/(4G_d) + \Sigma(\Delta_c X)/(2b) \qquad \text{(12-4.SI)}$$

where

A = Area of diaphragm chord cross section, in in.^{2} (mm^2);
b = Diaphragm width, in feet (meters);
E = Modulus of elasticity of diaphragm chord, in lb/in.^{2} (N/mm^2);
$\Sigma(\Delta_c X)$ = Sum of individual chord-splice slip values, in in. (mm), on both sides of the diaphragm, each multiplied by its distance to the nearest support, in ft (mm);
G_d = Diaphragm shear stiffness from Table 12-2, in lb/in. (N/mm);
L = Diaphragm span, distance between shear walls or collectors, in feet (meters);
v_y = Shear per unit length at yield in the direction under consideration, in lb/ft (N/m); and
Δ_y = Calculated diaphragm deflection at yield, in inches (meters).

Properties used to compute diaphragm deflection and stiffness shall be based on Section 12.2.2.

12.5.3.2.2 Strength of Double-Layer Straight Lumber Sheathing Diaphragms The expected strength of double-layer straight lumber sheathing diaphragms shall be determined in accordance with Section 12.2.2.

12.5.3.2.3 Acceptance Criteria for Double-Layer Straight Lumber Sheathing Diaphragms For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria shall be taken from Table 12-4.

12.5.3.2.4 Connections of Double-Layer Straight Lumber Sheathing Diaphragms Connections between diaphragms and shear walls and other vertical elements shall be considered in accordance with Section 12.5.1.

12.5.3.3 Single-Layer Diagonal Lumber Sheathing Diaphragms

12.5.3.3.1 Stiffness of Single-Diagonal Sheathing Diaphragms The midspan deflection of single span single-layer diagonal lumber sheathing diaphragms with uniformly distributed load shall be calculated using Equation (12-4). Properties used to compute diaphragm deflection and stiffness shall be based on Section 12.2.2.

12.5.3.3.2 Strength of Single-Layer Diagonal Lumber Sheathing Diaphragms The expected strength for single-layer diagonal lumber sheathing diaphragms with chords shall be determined in accordance with Section 12.2.2.

12.5.3.3.3 Acceptance Criteria for Single-Layer Diagonal Lumber Sheathing Diaphragms For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria shall be taken from Table 12-4.

12.5.3.3.4 Connections of Single-Layer Diagonal Lumber Sheathing Diaphragms Connections between diaphragms and shear walls and other vertical elements shall be considered in accordance with Section 12.5.1.

12.5.3.4 Diagonal Lumber Sheathing with Straight Lumber Sheathing or Flooring above Diaphragms

12.5.3.4.1 Stiffness of Diagonal Lumber Sheathing with Straight Lumber Sheathing or Flooring above Diaphragms The midspan deflection of single span diagonal lumber sheathing diaphragms with uniformly distributed load and with straight lumber sheathing or flooring above shall be calculated using Equation (12-4). Properties used to compute diaphragm deflection and stiffness shall be based on Section 12.2.2.

12.5.3.4.2 Strength of Diagonal Lumber Sheathing with Straight Lumber Sheathing or Flooring above Diaphragms The expected strength of diagonal lumber sheathing diaphragms with straight lumber sheathing or flooring above shall be determined in accordance with Section 12.2.2.

12.5.3.4.3 Acceptance Criteria for Diagonal Lumber Sheathing with Straight Lumber Sheathing or Flooring above Diaphragms For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria shall be taken from Table 12-4.

12.5.3.4.4 Connections of Diagonal Lumber Sheathing with Straight Lumber Sheathing or Flooring above Diaphragms Connections between diaphragms and shear walls and other vertical elements shall be considered in accordance with Section 12.5.1.

12.5.3.5 Double-Layer Diagonal Lumber Sheathing Diaphragms

12.5.3.5.1 Stiffness of Double-Layer Diagonal Lumber Sheathing Diaphragms The midspan deflection of single span double-layer diagonal lumber sheathing diaphragms with uniformly distributed load shall be calculated using Equation (12-4). Properties used to compute diaphragm deflection and stiffness shall be based on Section 12.2.2.

12.5.3.5.2 Strength of Double-Layer Diagonal Lumber Sheathing Diaphragms The expected strength of double-layer diagonal lumber sheathing diaphragms shall be determined in accordance with Section 12.2.2.

12.5.3.5.3 Acceptance Criteria for Double-Layer Diagonal Lumber Sheathing Diaphragms For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria shall be taken from Table 12-4.

12.5.3.5.4 Connections of Double-Layer Diagonal Lumber Sheathing Diaphragms Connections between diaphragms and shear walls and other vertical elements shall be considered in accordance with Section 12.5.1.

12.5.3.6 Wood Structural Panel Sheathing Diaphragm

12.5.3.6.1 Stiffness of Wood Structural Panel Sheathing Diaphragms The midspan deflection of single span blocked and chorded wood structural panel diaphragms with uniformly distributed load and with constant nailing across the diaphragm length shall be determined using Equation (12-5):

$$\Delta_y = 5v_yL^3/(8EAb) + v_yL/(4G_vt_v) + 0.188Le_n + \Sigma(\Delta_cX)/(2b) \tag{12-5}$$

$$\Delta_y = 5v_yL^3/(96EAb) + v_yL/(4G_vt_v) + Le_n/1{,}621 + \Sigma(\Delta_cX)/(2b) \tag{12-5.SI}$$

where

A = Area of diaphragm chord cross section, in in.^{2} (mm^2);
b = Diaphragm width, in feet (meters);
E = Modulus of elasticity of diaphragm chord, in lb/in.^{2} (N/mm^2);
e_n = Nail deformation at yield load per nail, in inches (millimeters); values listed are for Structural I panels; multiply by 1.2 for all other panel grades;
= 0.13 in. (3.3 mm) for 6d nails at yield;
= 0.08 in. (2.0 mm) for 8d nails at yield;
= 0.08 in. (2.0 mm) for 10d nails at yield;
G_vt_v = Shear stiffness of wood structural panels, in lb/in. of depth (N/mm of depth);
L = Diaphragm span, distance between shear walls or collectors, in fet (meters);
v_y = Shear per unit length at yield in the direction under consideration, in lb/ft (N/mm);
$\Sigma(\Delta_cX)$ = Sum of individual chord-splice slip values, in inches (millimeters) on both sides of the diaphragm, each multiplied by its distance to the nearest support, in feet (meters); and
Δ_y = Calculated deflection of diaphragm at yield, in inches (millimeters).

Alternatively, a more rigorous calculation of diaphragm deflection based on rational engineering principles shall be permitted.

The midspan deflection of single span blocked and chorded wood structural panel diaphragms with uniformly distributed load and with variable nailing across the diaphragm length shall be determined using Equation (12-6):

$$\Delta_y = 5v_yL^3/(8EAb) + v_yL/(4G_vt_v) + 0.376Le_n + \Sigma(\Delta_cX)/(2b) \tag{12-6}$$

$$\Delta_y = 5v_yL^3/(96EAb) + v_yL/(4G_vt_v) + Le_n/811 + \Sigma(\Delta_cX)/(2b) \tag{12-6.SI}$$

Alternatively, a more rigorous calculation of diaphragm deflection based on rational engineering principles shall be permitted.

The midspan deflection of single span unblocked diaphragms with uniformly distributed load shall be calculated using Equation (12-4). Properties used to compute diaphragm deflection and stiffness shall be based on Section 12.2.2.

12.5.3.6.2 Strength of Wood Structural Panel Sheathing Diaphragms The expected strength of wood structural panel diaphragms shall be taken as mean maximum strengths obtained experimentally. Expected strengths shall be permitted to be based on 1.07 times nominal strengths of wood structural panel diaphragms. Yield strengths of wood structural panel diaphragms shall be permitted to be based on nominal strengths divided by 1.4. Nominal strengths shall be determined using LRFD procedures contained in AWC SDPWS, except that the resistance factor, ϕ, shall be taken as 1.0 and expected material properties shall be determined in accordance with Section 12.2.2.

For existing wood structural panel diaphragms framed with 2 in. (50.8 mm) nominal framing at adjoining panel edges where 3 in. (76.2 mm) nominal framing is required per AWC SDPWS, the expected strength shall not be taken as greater than 0.80 times the expected strength associated with use of 3 in. (76.2 mm) nominal framing at adjoining panel edges.

Approved allowable stress values for fasteners shall be permitted to be converted in accordance with Section 12.2.2.5.1 where the strength of a diaphragm is computed using principles of mechanics.

The expected shear capacity of unchorded diaphragms shall be calculated by multiplying the values given for chorded diaphragms by 0.60.

12.5.3.6.3 Acceptance Criteria for Wood Structural Panel Sheathing Diaphragms For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria shall be taken from Table 12-4.

12.5.3.6.4 Connections of Wood Structural Panel Sheathing Diaphragms Connections between diaphragms and shear walls and other vertical elements shall be considered in accordance with Section 12.5.1.

12.5.3.7 Wood Structural Panel Overlays on Straight or Diagonal Lumber Sheathing Diaphragms

12.5.3.7.1 Stiffness of Wood Structural Panel Overlays on Straight or Diagonal Lumber Sheathing Diaphragms Placement of the new wood structural panel overlay shall be consistent with Section 12.5.2.2.

The midspan deflection of single span wood structural panel overlays on straight or diagonal lumber sheathing diaphragms with uniformly distributed load shall be calculated using Equation (12-4).

12.5.3.7.2 Strength of Wood Structural Panel Overlays on Straight or Diagonal Lumber Sheathing Diaphragms Strength of wood structural panel overlays shall be determined in accordance with Section 12.3.2.2. It shall be permitted to take the expected strength of wood structural panel overlays as the value for the corresponding wood structural panel diaphragm without the existing sheathing below, computed in accordance with Section 12.5.3.6.2.

12.5.3.7.3 Acceptance Criteria for Wood Structural Panel Overlays on Straight or Diagonal Lumber Sheathing Diaphragms For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria shall be taken from Table 12-4.

12.5.3.7.4 Connections of Wood Structural Panel Overlays on Straight or Diagonal Lumber Sheathing Diaphragms Connections between diaphragms and shear walls and other vertical elements shall be considered in accordance with Section 12.5.1.

12.5.3.8 Wood Structural Panel Overlays on Existing Wood Structural Panel Sheathing Diaphragms

12.5.3.8.1 Stiffness of Wood Structural Panel Overlays on Existing Wood Structural Panel Sheathing Diaphragms Diaphragm deflection shall be calculated in accordance with Equation (12-4) or using accepted principles of mechanics. Nails in the upper layer of the wood structural panel shall have sufficient embedment in the framing to meet the requirements of AWC SDPWS.

12.5.3.8.2 Strength of Wood Structural Panel Overlays on Existing Wood Structural Panel Sheathing Diaphragms Expected strength shall be calculated based on the combined two layers of wood structural panel sheathing, with the strength of the overlay limited to 75% of the values calculated in accordance with Section 12.5.3.6.2.

12.5.3.8.3 Acceptance Criteria for Wood Structural Panel Overlays on Existing Wood Structural Panel Sheathing Diaphragms For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria shall be taken from Table 12-4.

12.5.3.8.4 Connections of Wood Structural Panel Overlays on Existing Wood Structural Panel Sheathing Diaphragms Connections between diaphragms and shear walls and other vertical elements shall be considered in accordance with Section 12.5.1.

12.5.3.9 Braced Horizontal Diaphragms Braced horizontal diaphragms shall be considered in accordance with Section 12.7.1.

Connections between members of the horizontal bracing system and shear walls or other vertical elements shall be considered in accordance with Section 12.5.1.

12.6 WOOD FOUNDATIONS

12.6.1 Types of Wood Foundations Types of wood foundations include wood piling, wood footings, and pole structures. Wood piling shall include friction or end-bearing piles that resist only vertical loads.

12.6.2 Analysis, Strength, and Acceptance Criteria for Wood Foundations The expected strength of wood piles shall be computed in accordance with Section 12.3.2.2. Lateral deflection of piles under seismic loads shall be calculated based on an assumed point of fixity. Unless rigidly connected to the pile cap, wood piles shall be taken as pinned at the top.

Flexure and axial loads in wood piles shall be considered deformation controlled. The *m*-factors shall be taken from Table 12-3.

Wood footings shall be investigated for the presence of deterioration. Acceptability of soils below wood footings shall be determined in accordance with Chapter 8.

Component and connection strength of pole structures shall be based on Section 12.2. Pole structures shall be modeled as cantilever elements and analyzed in accordance with Chapter 7.

Flexure and axial loads in pole structures shall be considered deformation controlled. The *m*-factors shall be taken from Table 12-3. Where concentrically braced diagonals are added to enhance the capacity of the pole structure, reduced *m*-factors taken from Table 12-3 shall be used.

12.6.3 Retrofit Measures for Wood Foundations Seismic retrofit measures for wood foundations shall meet the requirements of Section 12.3.5 and other provisions of this standard.

Wood foundations showing signs of deterioration shall be retrofitted or replaced as required to satisfy the selected Performance Objective.

12.7 OTHER WOOD ELEMENTS AND COMPONENTS

12.7.1 General Wood elements and components that are not addressed by Tables 12.1 through 12.4 or Section 12.6, “Wood

Foundations," shall be considered in accordance with this section. Where an assembly includes wood components and steel rods, the rods shall be considered in accordance with applicable provisions of Chapter 9.

12.7.1.1 Stiffness of Other Wood Elements and Components The stiffness and deflection of wood elements that are not addressed by Tables 12.1 through 12.4 or Section 12.6, "Wood Foundations" shall be determined based on a mathematical model or by a test program for the assembly, considering the configuration, stiffness, and interconnection of the individual components approved by the Authority Having Jurisdiction.

12.7.1.2 Strength of Other Wood Elements and Components The capacities of individual components, including connections, shall be determined in accordance with Section 12.3.2.

12.7.1.3 Acceptance Criteria for Other Wood Elements and Components For linear procedures, design actions shall be compared with design capacities in accordance with Section 7.5.2.2. Connections shall be considered in accordance with Section 12.3.3. Axial tension and axial tension with bending shall be considered deformation controlled. Axial compression and connections between steel rods and wood components shall be considered force controlled. The m-factors for deformation-controlled actions shall be taken from Table 12-3 for component actions listed. The m-factors for deformation-controlled component actions not included in Table 12-3 shall be established in accordance with Section 7.6. For nonlinear procedures, coordinates of the generalized force–deformation relation, described by Figure 12-1, and deformation acceptance criteria shall be taken from Table 12-4.

CHAPTER 13

ARCHITECTURAL, MECHANICAL, AND ELECTRICAL COMPONENTS

13.1 SCOPE

This chapter sets forth requirements for the seismic evaluation and retrofit of existing architectural, mechanical, and electrical components and systems that are permanently installed in, or are an integral part of, a building system. Nonstructural components shall be evaluated to achieve the Performance Objective selected in accordance with Section 2.2, and the Performance Levels for nonstructural components as defined in Section 2.2.2. Requirements of this section apply to nonstructural components that are evaluated or retrofitted to the Hazards Reduced, Position Retention, Life Safety, and Operational Nonstructural Performance Levels. Components evaluated for Hazards Reduced Performance Level shall use the Life Safety acceptance criteria. The requirements for Operational Nonstructural Performance shall be consistent with ASCE 7, Chapter 13, requirements for the case where I_p, as defined in ASCE 7, is set equal to 1.5 and as stipulated herein or through the use of other approved methods.

Buildings in regions of very low seismicity, unless specifically required in Chapter 4, 5, or 16, or buildings where the target building Performance Level includes Nonstructural Performance Level Not Considered need not comply with the provisions of this chapter.

Sections 13.2 and 13.3 provide requirements for condition assessment and component evaluation. Section 13.4 specifies procedures for determining forces and deformations on nonstructural components. Section 13.5 identifies retrofit methods. Sections 13.6, 13.7, and 13.8 specify evaluation and acceptance criteria for architectural components; mechanical, electrical, and plumbing (MEP) systems; and furnishings and interior equipment, respectively.

Nonstructural components shall be included in the mathematical model of the building in accordance with the requirements of Section 7.2.4.3. Nonstructural components included in the mathematical model of the building shall be evaluated for forces and deformations imposed by the structure, computed in accordance with Chapter 7.

In structures incorporating seismic isolation systems, nonstructural components located at or above the isolation interface shall comply with the requirements in Section 14.5.2 and the requirements of this chapter. Nonstructural components that cross the isolation interface shall comply with the requirements of Section 14.5.3. Nonstructural components located below the isolation interface shall comply with the requirements of this chapter.

New nonstructural components installed in existing buildings shall conform to the requirements of this standard. New nonstructural components designed to the Life Safety Performance Level are permitted to be designed using the requirements of similar components for new buildings.

13.2 EVALUATION AND RETROFIT PROCEDURE FOR NONSTRUCTURAL COMPONENTS

Nonstructural components shall be evaluated and retrofitted by completing the following steps:

1. The Performance Objective shall be established in accordance with Section 2.2, which includes selection of a Nonstructural Performance Level and a Seismic Hazard Level. The Level of Seismicity shall be determined in accordance with Section 2.5.
2. A walk-through and condition assessment shall be performed in accordance with Section 13.3, and testing of anchorage for nonstructural components into existing concrete and masonry shall be performed in accordance with Section 13.3.2.
3. Evaluation, and retrofit requirements for the selected Nonstructural Performance Level and appropriate Level of Seismicity shall be determined for nonstructural components using Table 13-1. "Yes" indicates that retrofit shall be required if the component does not meet applicable acceptance criteria specified in Sections 13.6 through 13.8.
4. Interaction between structural and nonstructural components shall be considered in accordance with Section 7.2.4.3, Section 13.4, and Section 14.6.
5. The classification of each type of nonstructural component shall be determined in accordance with Section 13.2.1.
6. Evaluation shall be conducted in accordance with Section 13.4. Seismic forces shall be calculated in accordance with Section 13.4.4, and seismic deformations shall be calculated in accordance with Section 13.4.5. The acceptability of bracing components and connections between nonstructural components and the structure shall be determined in accordance with Section 13.4.1. Existing anchors for nonstructural components shall be evaluated in accordance with Section 13.3.
7. Nonstructural components not meeting the requirements of the selected Nonstructural Performance Level shall be retrofitted in accordance with Section 13.5.

13.2.1 Classification of Components Nonstructural components shall be classified based on their response sensitivity in each primary orthogonal horizontal direction as follows:

1. Nonstructural components that are sensitive to and subject to damage from inertial loading shall be classified as acceleration-sensitive components,
2. Nonstructural components that are sensitive and subject to damage imposed by drift or deformation of the structure shall be classified as deformation sensitive, and

Table 13-1. Nonstructural Components: Applicability of Hazards Reduced, Life Safety, and Position Retention Requirements.

	Seismicity								
	High Seismicity			Moderate Seismicity			Low Seismicity		
	PR	LS	HR	PR	LS	HR	PR	LS	HR
Component Type	Performance Level								
Architectural (Section 13.6)									
1. **Exterior Envelope Components (Section 13.6.1)**									
Adhered veneer	Yes	Yes	No	Yes	Yes	No	Yes	Yes	No
Anchored veneer	Yes	Yes	Yes[a]	Yes	Yes	No	Yes	Yes	No
Glass blocks and other nonstructural masonry walls	Yes	Yes	Yes[a]	Yes	Yes	Yes[a]	Yes	Yes	Yes[a]
Prefabricated panels	Yes	Yes	Yes[a]	Yes	Yes	Yes[a]	No	No	Yes[a]
Glazed exterior wall systems	Yes	Yes	No	Yes	Yes	No	No	No	No
2. **Partitions (Section 13.6.2)**									
Heavy, URM, or hollow clay tile	Yes	Yes	Yes[a]	Yes	Yes	Yes[a]	Yes	Yes	Yes[a]
Light	Yes	No	No	Yes	No	No	No	No	No
Glazed	Yes	Yes	No	Yes	Yes	No	No	No	No
3. **Interior Veneers (Section 13.6.3)**									
Stone, including marble	Yes	Yes	Yes[a]	Yes	Yes	No	No	No	No
4. **Ceilings (Section 13.6.4)**									
Directly applied to structure	Yes	No	No	Yes	No	No	No	No	No
Dropped furred gypsum board	Yes	Yes	No	No	No	No	No	No	No
Suspended lath and plaster	Yes	Yes	Yes[a]	Yes	Yes	No	Yes	No	No
Suspended integrated ceiling	Yes	No	No	Yes	No	No	No	No	No
5. **Parapets and Cornices (Section 13.6.5)**									
Unreinforced masonry	Yes	Yes	Yes[a]	Yes	Yes	Yes[a]	Yes	Yes	Yes[a]
Concrete and reinforced masonry	Yes	Yes	Yes[a]	Yes	Yes	Yes[a]	Yes	No	No
Other	Yes	Yes	No	Yes	Yes	No	Yes	No	No
6. **Architectural Appendages and Marquees (Section 13.6.6)**	Yes	Yes	Yes[a]	Yes	Yes	Yes[a]	Yes	Yes	Yes[a]
7. **Penthouses (Section 13.6.7)**	Yes	Yes	No	Yes	No	No	Yes	No	No
8. **Clay Tile Roofs (Section 13.6.8)**	Yes	Yes	Yes[a]	Yes	Yes	Yes[a]	Yes	Yes	Yes[a]
7. **Chimneys and Stacks (Section 13.6.9)**	Yes	Yes	Yes[a]	Yes	Yes	Yes[a]	Yes	Yes	Yes[a]
8. **Stairs and Ramps (Section 13.6.10)**	Yes	Yes	No	Yes	Yes	No	Yes	Yes	No
9. **Doors Required for Emergency Services Egress (Section 13.6.11)**	Yes	Yes	No	Yes	No	No	No	No	No
10. **Computer Access Floor (Section 13.6.12)**	Yes	No	No	No	No	No	No	No	No
Mechanical Equipment (Section 13.7)									
1. **Mechanical Equipment (Section 13.7.1)**									
Boilers, furnaces, pumps, and chillers	Yes	No	No	No	No	No	No	No	No
General manufacturing and process machinery	Yes	No	No	No	No	No	No	No	No
Hazardous material equipment	Yes	Yes	Yes[a]	Yes	Yes	Yes[a]	Yes	Yes	Yes[a]
Fire suppression equipment	Yes	Yes	No	Yes	Yes	No	Yes	Yes	No
HVAC equipment, vibration isolated	Yes	Yes	No	Yes	No	No	No	No	No
HVAC equipment, nonvibration isolated	Yes	Yes	No	Yes	No	No	No	No	No
HVAC equipment, mounted in line with ductwork	Yes	Yes	No	Yes	No	No	No	No	No
2. **Storage Vessels and Water Heaters (Section 13.7.2)**									
Structurally supported vessels (Category 1)	Yes	Yes	No	Yes	No	No	No	No	No
Flat-bottom vessels (Category 2)	Yes	Yes	No	Yes	No	No	No	No	No
Fire water storage tanks and reservoirs	Yes	Yes	No	Yes	Yes	No	Yes	Yes	No
3. **Pressure Piping (Section 13.7.3)**	Yes	Yes	No	Yes	No	No	No	No	No
4. **Fire Suppression Piping (Section 13.7.4)**	Yes	Yes	No	Yes	Yes	No	Yes	Yes	No

continues

Table 13-1 *(Continued)*. Nonstructural Components: Applicability of Hazards Reduced, Life Safety, and Position Retention Requirements.

	Seismicity								
	High Seismicity			Moderate Seismicity			Low Seismicity		
	PR	LS	HR	PR	LS	HR	PR	LS	HR
Component Type	Performance Level								
5. **Fluid Piping, Oher Than Fire Suppression (Section 13.7.5)**	Yes	No	No	No	No	No	No	No	No
Hazardous materials	Yes	Yes	No	Yes	Yes	No	Yes	Yes	No
Nonhazardous materials	Yes	No	No	Yes	No	No	No	No	No
6. **Ductwork (Section 13.7.6)**									
Stair and smoke ducts	Yes	Yes	No	Yes	Yes	No	Yes	Yes	No
Hazardous material ducts	Yes	Yes	No	Yes	Yes	No	Yes	Yes	No
Other HVAC ducts	Yes	No[b]	No	No[b]	No[b]	No	No[b]	No	No
Electrical and Communications (Section 13.7)									
1. **Electrical and Communications Equipment (Section 13.7.7)**	Yes	Yes	No	Yes	No	No	No	No	No
2. **Electrical and Communications Distribution Equipment (Section 13.7.8)**									
Emergency power equipment	Yes	Yes	No	Yes	Yes	No	Yes	Yes	No
Other	Yes	No	No	Yes	No	No	No	No	No
3. **Light Fixtures (Section 13.7.9)**									
Recessed	Yes	No	No	No	No	No	No	No	No
Surface mounted	Yes	No	No	No	No	No	No	No	No
Integrated ceiling	Yes	No	No	Yes	No	No	No	No	No
Pendant	Yes	Yes	No	Yes	No	No	No	No	No
Emergency lighting	Yes	No	No	Yes	No	No	Yes	No	No
4. **Elevators (Section 13.7.11)**	Yes	Yes	No	Yes	No	No	No	No	No
5. **Conveyors (Section 13.7.12)**	Yes	No	No	No	No	No	No	No	No
Furnishings and Interior Equipment (Section 13.8)									
1. **Storage Racks (Section 13.8.1)**	Yes	Yes	Yes[a]	Yes	Yes	Yes[a]	No	No	No
2. **Contents (Section 13.8.2)**									
Tall and narrow	Yes	Yes	No	Yes	No	No	No	No	No
Fall prone	Yes	Yes	No	No	No	No	No	No	No
Suspended contents	Yes	No	No	Yes	No	No	No	No	No
3. **Hazardous Materials Storage (Section 13.8.3)**	Yes	Yes	Yes[a]	Yes	Yes	Yes[a]	Yes	Yes	Yes[a]
4. **Computer and Communication Racks (Section 13.8.4)**	Yes	No	No	No	No	No	No	No	No

[a] If it can be demonstrated that the component does not pose a threat of serious injury to many people due to falling or failing under the Seismic Hazard Level being considered, the component need not be considered in the Hazards Reduced Nonstructural Performance Level.

[b] Ductwork that exceeds 6 ft^2 (0.56 m^2) in cross-sectional area, or is suspended more than 12 in. (305 mm) from the top of the duct to the supporting structure at any support point, shall meet the requirements of the selected Performance Objective.

Note: PR = Position Retention Nonstructural Performance Level, LS = Life Safety Nonstructural Performance Level, HR = Hazards Reduced Nonstructural Performance Level.

3. Nonstructural components that are sensitive to both inertial loading and drift and deformation of the structure shall be classified as both acceleration and deformation sensitive.

13.3 COMPONENT CONDITION ASSESSMENT AND ANCHORAGE TESTING

13.3.1 Condition Assessment A condition assessment of nonstructural components shall be performed as part of the nonstructural evaluation and retrofit process. As a minimum, this assessment shall determine the following:

1. The presence and configuration of each type of nonstructural component and its attachment to the structure,
2. The physical condition of each type of nonstructural component and whether or not degradation is present,
3. The presence of nonstructural components that potentially influence overall building performance, and
4. The presence of other nonstructural components whose failure could affect the performance of the nonstructural component being considered.

Direct visual inspection shall be performed on each type of nonstructural component in the building as follows:

1. If detailed drawings are available, at least one sample of each type, but not less than 5% of the total, of nonstructural component shall be observed. If no deviations from the drawings exist, the sample shall be considered representative of installed conditions. If deviations are observed, then at least 10% of all occurrences of the component shall be observed.
2. If detailed drawings are not available, at least three occurrences of each type of nonstructural component, but not less than 10%, shall be observed. If no deviations among the three occurrences are observed, the sample shall be considered representative of installed conditions. If deviations are observed, at least 20% of all occurrences of the component shall be observed.

13.3.2 Testing Requirements for Evaluating the Performance of Existing Anchorage for Nonstructural Components

Nonstructural components evaluated to achieve a desired Performance Objective selected in accordance with Section 2.2, and where capacities of the attachment to the existing concrete or masonry structure are unknown, shall have a testing program instituted to establish the acceptance of the existing anchorage system. Where approved by the Authority Having Jurisdiction, a reduction in the testing frequency of no more than 50% is permitted where the capacity of the attachment can be calculated based on available documentation of the size, configuration, and material properties of the anchors and bracing and where the capacities are based on calculated lower-bound properties of the base material. The force requirements for testing shall be determined in accordance with Section 13.4.4 at the strength design level. The overstrength factor Ω_0 need not apply in determination of force demands when anchors are tested in accordance with this section, unless explicitly required by this section.

Out-of-plane wall anchorage into concrete shall be evaluated in accordance with the requirements in Chapter 10 and wall anchorage into masonry shall be evaluated in accordance with the requirements of Chapter 11. Anchorage for components that are exempt from the requirements of Section 13.1.4 of ASCE 7 need not be tested and are deemed to comply with the requirements of this section.

13.3.2.1 Components Evaluated to the Operational Performance Level

13.3.2.1.1 Concrete or Masonry Anchors Used for Distributed Systems For anchors into concrete or masonry used in the seismic bracing of distributed systems such as pipes, ducts, or conduit, and repetitively installed architectural components, such as ceilings, cladding, and partitions, the following shall apply:

1. Twenty percent of the anchors of a given size, embedment, and type (e.g., cast-in-place, adhesive, wedge, or shell and sleeve for expansion bolts) at each level of the structure shall be tension tested in a random sample to twice the calculated force requirements but not less than 500 lb (2.2 kN). Where a system has fewer than 100 anchors of a given size, embedment, and type, one of every five anchors of the same size, embedment, and type shall be tested.
2. Where a nonstructural component is anchored with four or more anchors, those anchors are defined as a bolt group. A minimum of one anchor in any bolt group shall be tested assuming an equal distribution of the calculated force to the bolt group. Anchors of diameter 1/4 in. (6.4 mm) need not be tested.

EXCEPTION: Internally threaded anchors, such as shell-type anchors, shall be tested to two and one-half times the maximum calculated force requirements. Attachment hardware shall be shimmed or removed before testing so that it does not prevent the possible withdrawal of the anchor.

If an anchor fails the tension test, all anchors up to a maximum of 20, installed by the same trade, in the immediate vicinity of the failed anchor shall be tested before resuming to the 20% sampling rate for testing.

13.3.2.1.2 Concrete or Masonry Anchors Used in the Attachment of Equipment and Other Components For anchors into concrete and masonry used in the attachment of mechanical and electrical equipment and other components, the following shall apply:

1. For each piece of equipment or other nonstructural component anchored with four or more anchors, the anchors for that component are defined as a bolt group. A minimum of one anchor in any bolt group shall be tested assuming an equal distribution of the calculated force to the bolt group. Where one or more anchors in the bolt group have a higher calculated tension force, one of the bolts with the higher calculated tension force shall be tested.
2. The tension test load shall be twice the maximum tension force calculated for an anchor in the attachment group using the calculated force requirements or 500 lb (2.2 kN) minimum. Anchors of diameter 1/4 in. (6.4 mm) need not be tested.

   EXCEPTION: Internally threaded anchors, such as shell-type anchors, shall be tested to two and one-half times the maximum calculated design loads. Attachment hardware shall be shimmed or removed before testing so that it does not prevent the possible withdrawal of the anchor.
3. If a single anchor fails, all anchors in the attachment group shall be tested. If two or more anchors fail, the component anchorage shall be retrofitted for the forces as for new construction.

13.3.2.2 Components Evaluated to the Position Retention or Life Safety Performance Level

13.3.2.2.1 Concrete or Masonry Anchors Used in the Seismic Bracing of Distributed Systems For anchors into concrete or masonry used in the seismic bracing of distributed systems, such as pipes, ducts, or conduit, and repetitively installed architectural components, such as ceilings, cladding, and partitions, the following shall apply:

1. Ten percent of the anchors of a given size, embedment, and type (e.g., cast-in-place, adhesive, wedge, or shell and sleeve for expansion bolts) at each level of the structure shall be tension tested in a random sample to twice the calculated force requirements but not less than 400 lb (1.8 kN). Where a system has fewer than 100 anchors of a given size, embedment, and type, one of every 10 anchors of the same size, embedment, and type shall be tested.
2. Where a nonstructural component is anchored with four or more anchors, those anchors are defined as a bolt group. A minimum of one anchor in any bolt group shall be tested assuming an equal distribution of the calculated force to the bolt group. Anchors of diameter 1/4 in. (6.4 mm) need not be tested.

   EXCEPTION: Internally threaded anchors, such as shell-type anchors, shall be tested to two and one-half times the maximum calculated force requirements. Attachment hardware shall be shimmed or removed before testing so that it does not prevent the possible withdrawal of the anchor.
3. If an anchor fails the tension test, all anchors, 10 maximum, installed by the same trade, in the immediate vicinity of the

failed anchor shall be tested before resuming to the 10% sampling rate for testing.

13.3.2.2.2 Concrete or Masonry Anchors Used in the Attachment of Equipment and Other Nonstructural Components For anchors into concrete or masonry used in the attachment of mechanical and electrical equipment and other components, the following shall apply:

1. For each piece of equipment or other nonstructural component anchored with four or more anchors, the anchors for that component are defined as a bolt group. A minimum of one anchor in any bolt group shall be tested assuming an equal distribution of the calculated force to the bolt group. Where one or more anchors in the bolt group have a higher calculated tension force, one of the bolts with the higher calculated tension force shall be tested.
2. The tension test load shall be twice the maximum tension force calculated for an anchor in the attachment group using the calculated force requirements or 400 lb (1.8 kN) minimum. Anchors of diameter 1/4 in. (6.4 mm) need not be tested.

   EXCEPTION: Internally threaded anchors, such as shell-type anchors, shall be tested to two and one-half times the maximum calculated design loads. Attachment hardware shall be shimmed or removed before testing so that it does not prevent the possible withdrawal of the anchor.
3. If a single anchor fails, all anchors in the attachment group shall be tested. If two or more anchors fail, the component anchorage shall be retrofitted for the forces as for new construction.

13.3.2.3 Tension Testing Procedure Testing of concrete or masonry anchors shall be accomplished by the application of externally applied direct tension force to the anchor. The test load is permitted to be applied by any method that will effectively measure the tension in the anchor. The testing apparatus shall not restrict the probable shear cone failure surface of the concrete or masonry.

Torque testing shall not be permitted in lieu of tension testing unless specifically allowed in these provisions.

A failure shall be defined when the tension load on the anchor produces a slip of 1/8 in. (3 mm), a shear cone failure in the concrete or masonry, concrete splitting, or fracture of the steel anchor itself before attaining the test load value.

EXCEPTION: For internally threaded anchors, the allowable slip shall not exceed 1/16 in. (1.5 mm).

Anchors that are not subject to net tension loads using the seismic force and load combination requirements of Section 13.4.4 shall be evaluated for shear strength in accordance with Section 13.4.6.6.

13.3.2.4 Torque Testing Procedure Torque testing procedures are only permitted when specifically allowed in accordance with Section 13.3.2.6 for anchors where there is no net tension and only to establish adequate installation to evaluate shear capacity. Anchors shall be tested with a calibrated torque wrench and shall attain the required torque within one half turn of the nut after the nut is seated on the attachment, per Table 13-2.

13.3.2.5 Alternate Test Criteria In lieu of testing in accordance with Section 13.3.2.1 or 13.3.2.2, a test load is permitted to be established by the evaluating engineer with the condition that the strength design load that the anchor can resist shall be determined by dividing the test load by two and one-half for internally threaded anchors or two for all other anchors.

13.3.2.6 Shear Capacity of Existing Anchors Where the force requirements of Section 13.4.4 result in no net tension loads on the concrete or masonry anchors or attachments for the nonstructural components, the shear capacity of the anchors shall be determined in accordance with Chapter 17 of ACI 318 using a ϕ factor of 1.0 using a maximum embedment depth of 2 in. (51 mm), unless the anchor embedment is specified on the original construction drawings, and a concrete compressive strength of 2,500 lb/in.^{2} (18 MPa) or the concrete strength determined by tests. The maximum shear demand on the critical anchor or group shall be amplified by Ω_0. The anchor edge distance shall be considered in the calculation of the anchor shear capacity.

Table 13-2. Required Torque.

Anchor Diameter	Wedge (ft lb/Nm)	Sleeve (ft lb/Nm)
5/16 in. (8 mm)	—	5 (6.78 Nm)
3/8 in. (10 mm)	25 (33.90 Nm)	10 (13.56 Nm)
1/2 in. (13 mm)	50 (67.79 Nm)	20 (27.12 Nm)
5/8 in. (16 mm)	80 (108.47 Nm)	45 (61.0 Nm)
3/4 in. (19 mm)	150 (203.37 Nm)	90 (122.02 Nm)
1 in. (25 mm)	250 (338.95 Nm)	N/A

Alternatively, the testing requirements in Section 13.3.2.1.2 or 13.3.2.2.2 shall consist of torque testing for post-installed anchors, except for adhesive and shell anchors, in accordance with Section 13.3.2.4 or application of a 400 lb (1.8 kN) minimum tension test load to the critical anchors with minimum edge distance. The shear capacity shall be permitted to use the allowable loads from corresponding ICC-ES or other evaluation reports enforced at the time for the size of anchor and minimum embedment depth when the anchor was installed. Larger embedment depths are permitted to be used if justified by removal of unused or redundant anchors. Shear demands need not be amplified by Ω_0.

Where net tension exists and the anchors are tested to twice the calculated maximum tension force, the anchor shall be deemed to be adequate for shear demands to the component.

13.4 EVALUATION PROCEDURES

Nonstructural components shall be evaluated using the analytical procedure in Section 13.4.2 or the prescriptive procedure in Section 13.4.3 as permitted by Sections 13.6 through 13.8. Where anchorage of nonstructural components for seismic forces is required by Table 13-1 and Sections 13.6 through 13.8, component attachments shall be bolted, welded, anchored, or otherwise positively fastened without consideration of frictional resistance produced by the effects of gravity.

13.4.1 Acceptance Criteria Acceptance criteria for nonstructural components being evaluated to the Hazards Reduced, Life Safety, Position Retention, and Operational Nonstructural Performance Levels shall be based on criteria listed in Sections 13.6 through 13.8. Forces on bracing and connections for nonstructural components calculated in accordance with Section 13.4.2 shall be compared with capacities using strength design procedures. Deformations for evaluating deformation sensitive components shall be determined in accordance with Section 13.4.5.

13.4.2 Analytical Procedure Where the prescriptive procedure is not permitted based on Section 13.6 through 13.8, forces and deformations on nonstructural components shall be calculated as follows:

1. For acceleration-sensitive components, seismic forces shall be calculated in accordance with Section 13.4.4 using load combinations in Section 13.4.4.3;

2. For deformation sensitive components, drift ratios or relative displacements shall be calculated in accordance with Section 13.4.5;
3. For components that are both acceleration-sensitive and deformation sensitive, seismic forces shall be calculated in accordance with Section 13.4.4, and drift ratios or relative displacements shall be calculated in accordance with Section 13.4.5; and
4. Alternatively, the calculation of seismic forces and deformations in accordance with Section 13.4.6 shall be permitted.

13.4.3 Prescriptive Procedure Where the prescriptive procedure is permitted in Sections 13.6 through 13.8, the characteristics of the nonstructural component shall be compared with characteristics as specified in referenced codes and standards or other approved procedures.

13.4.4 Force Analysis: General Equations

13.4.4.1 Horizontal Seismic Forces Horizontal seismic forces on nonstructural components shall be determined in accordance with Equation (13-1):

$$F_p = \frac{0.4a_p S_{XS} W_p \left(1 + \frac{2x}{h}\right)}{\left(\frac{R_p}{I_p}\right)} \tag{13-1}$$

F_p calculated in accordance with Equation (13-1) shall be based on the stiffness of the component and the ductility of its bracing and anchorage, but it need not exceed the default value of F_p calculated in accordance with Equation (13-2) and shall not be less than F_p computed in accordance with Equation (13-3):

$$F_p(\text{maximum}) = 1.6 S_{XS} I_p W_p \tag{13-2}$$

$$F_p(\text{minimum}) = 0.3 S_{XS} I_p W_p \tag{13-3}$$

where

W_p = Component operating weight;
a_p = Component amplification factor from Table 13-3 or 13-4;
F_p = Component seismic force applied horizontally at the center of gravity of the component and distributed according to the mass distribution of the component;
S_{XS} = Spectral response acceleration parameter at short periods for the Seismic Hazard Level associated with the Structural Performance Level for the building determined in accordance with Section 2.4.1.6 or 2.4.2.1;
h = Average roof elevation of structure, relative to grade elevation;
R_p = Component response modification factor from Table 13-3 or 13-4;
x = Elevation in structure of the average point of attachment of the component to the structure; for items attached at or below grade, the value of x shall be taken as 0, and the value of x shall never exceed h; and
I_p = Component importance factor, as set forth in Sections 13.6 through 13.8.

The fundamental period of vibration of the nonstructural component (T_p) in each direction shall be estimated using Equation (13-4):

$$T_p = 2\pi\sqrt{\frac{W_p}{K_p g}} \tag{13-4}$$

where

T_p = Component fundamental period;
W_p = Component operating weight;
g = Gravitational acceleration; and
K_p = Approximate stiffness of the support system of the component, its bracing, and its attachment, determined in terms of load per unit deflection at the center of gravity of the component.

In lieu of Equation (13-1), nonstructural seismic forces shall be permitted to be calculated based on the provisions of Section 13.3.1.5, of ASCE 7, with the exception that the seismic response history analysis used as the basis for the floor response spectrum shall be based on the procedures in Chapter 14 for seismic isolation systems, Chapter 15 for damping systems, and Section 7.4, for all other building types.

13.4.4.2 Vertical Seismic Forces Where specifically required by Sections 13.6, 13.7, and 13.8, vertical seismic forces on nonstructural components shall be determined in accordance with Equation (13-5).

F_{pv} shall be calculated in accordance with Equation (13-5) but need not exceed F_p calculated in accordance with Equation (13-2):

$$F_{pv} = 0.2 S_{XS} W_p \tag{13-5}$$

where F_{pv} is the component seismic force applied vertically at the center of gravity of the component or distributed according to the mass distribution of the component.

All other terms in Equation (13-5) shall be as defined in Section 13.4.3.1.

13.4.4.3 Load Combinations The nonstructural forces for components and associated bracing elements shall be determined based on following load combinations [Equation (13-6)]:

$$Q_{UF} = 1.2W_p + F_{pv} \pm F_p \tag{13-6a}$$

$$Q_{UF} = 0.9W_p - F_{pv} \pm F_p \tag{13-6b}$$

The nonstructural forces for anchorage of components to concrete and masonry shall be determined based on the following load combinations [Equation (13-7)]:

$$Q_{UF} = 1.2W_p + F_{pv} \pm \Omega_{0p} F_p \tag{13-7a}$$

$$Q_{UF} = 0.9W_p + F_{pv} \pm \Omega_{0p} F_p \tag{13-7b}$$

where Ω_0 is the overstrength factor for the component from Tables 13-3 or 13-4.

13.4.4.4 Nonstructural Support Capacity

13.4.4.4.1 Existing Components The capacity of existing nonstructural components, including bracing and anchorage, being evaluated shall be determined using design standards approved by the design professional. Where strength design procedures are used, the strength reduction factor, φ, shall be permitted to be taken as 1.0.

13.4.4.4.2 New Components The capacity of new nonstructural components, including bracing, and anchorage shall be determined using design standards approved by the design professional.

Table 13-3. Coefficients for Architectural Components.

Architectural Component	a_p[a]	R_p	Ω_0[b]
Interior nonstructural walls and partitions[c]			
Plain (unreinforced) masonry walls	1	1½	1½
All other walls and partitions	1	2½	2
Cantilever elements (unbraced or braced to structural frame below its center of mass)			
Parapets and cantilever interior nonstructural walls	2½	2½	2
Chimneys where laterally braced or supported by the structural frame	2½	2½	2
Cantilever elements (braced to structural frame above its center of mass)			
Parapets	1	2½	2
Chimneys	1	2½	2
Exterior nonstructural walls[c]	1[b]	2½	2
Exterior nonstructural wall elements and connections[b]			
Wall element	1	2½	NA
Body of wall panel connections	1	2½	NA
Fasteners of the connecting system	1¼	1	1
Veneer			
Limited-deformability elements and attachments	1	2½	2
Low-deformability elements and attachments	1	1½	2
Penthouses (except where framed by an extension of the building frame)	2½	3½	2
Ceilings			
All	1	2½	2
Cabinets			
Permanent floor-supported storage cabinets more than 6 ft (1,829 mm) tall, including contents	1	2½	2
Permanent floor-supported library shelving, book stacks, and bookshelves more than 6 ft (1,829 mm) tall, including contents	1	2½	2
Laboratory equipment	1	2½	2
Access floors			
Special access floors (designed in accordance with Section 13.5.7.2)	1	2½	2
All other	1	1½	1½
Appendages and ornamentations	2½	2½	2
Signs and billboards	2½	3	2
Other rigid components			
High-deformability elements and attachments	1	3½	2
Limited-deformability elements and attachments	1	2½	2
Low-deformability materials and attachments	1	1½	1½
Other flexible components			
High-deformability elements and attachments	2½	3½	2½
Limited-deformability elements and attachments	2½	2½	2½
Low-deformability materials and attachments	2½	1½	1½
Egress stairways not part of the building seismic-force-resisting system	1	2½	2
Egress stairs and ramp fasteners and attachments	2½	2½	2½

[a] A lower value for a_p shall not be used unless justified by detailed dynamic analysis. The value for a_p shall not be less than 1. The value of $a_p = 1$ is for rigid components and rigidly attached components. The value of $a_p = 2½$ is for flexible components and flexibly attached components.

[b] Overstrength, where required for nonductile anchorage to concrete and masonry. See Section 13.4.3 for seismic load effects, including overstrength.

[c] Where flexible diaphragms provide lateral support for concrete or masonry walls and partitions, the design forces for anchorage to the diaphragm shall be as specified in Section 12.11.2.

Table 13-4. Seismic Coefficients for Mechanical and Electrical Components.

Components	a_p[a]	R_p[b]	Ω_0[c]
MECHANICAL AND ELECTRICAL COMPONENTS			
Air-side HVACR, fans, air handlers, air conditioning units, cabinet heaters, air distribution boxes, and other mechanical components constructed of sheet metal framing	2½	6	2
Wet-side HVACR, boilers, furnaces, atmospheric tanks and bins, chillers, water heaters, heat exchangers, evaporators, air separators, manufacturing or process equipment, and other mechanical components constructed of high-deformability materials	1	2½	2
Air coolers (fin fans), air-cooled heat exchangers, condensing units, dry coolers, remote radiators, and other mechanical components elevated on integral structural steel or sheet metal supports	2½	3	1½
Engines, turbines, pumps, compressors, and pressure vessels not supported on skirts and not within the scope of Chapter 15	1	2½	2
Skirt-supported pressure vessels not within the scope of Chapter 15	2½	2½	2
Elevator and escalator components	1	2½	2
Generators, batteries, inverters, motors, transformers, and other electrical components constructed of high-deformability materials	1	2½	2
Motor control centers, panel boards, switch gear, instrumentation cabinets, and other components constructed of sheet metal framing	2½	6	2
Communication equipment, computers, instrumentation, and controls	1	2½	2
Roof-mounted stacks, and cooling and electrical towers laterally braced below their center of mass	2½	3	2
Roof-mounted stacks and, cooling and electrical towers laterally braced above their center of mass	1	2½	2
Lighting fixtures	1	1½	2
Other mechanical or electrical components	1	1½	2
VIBRATION-ISOLATED COMPONENTS AND SYSTEMS[b]			
Components and systems isolated using neoprene elements and neoprene isolated floors with built-in or separate elastomeric snubbing devices or resilient perimeter stops	2½	2½	2
Spring-isolated components and systems and vibration-isolated floors closely restrained using built-in or separate elastomeric snubbing devices or resilient perimeter stops	2½	2	2
Internally isolated components and systems	2½	2	2
Suspended vibration-isolated equipment including in-line duct devices and suspended internally isolated components	2½		2
DISTRIBUTION SYSTEMS			
Piping in accordance with ASME B31 (2001, 2002, 2008, and 2010), including in-line components with joints made by welding or brazing	2½	12	2
Piping in accordance with ASME B31, including in-line components, constructed of high- or limited-deformability materials, with joints made by threading, bonding, compression couplings, or grooved couplings	2½	6	2
Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high-deformability materials, with joints made by welding or brazing	2½	9	2
Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high- or limited-deformability materials, with joints made by threading, bonding, compression couplings, or grooved couplings	2½	4½	2
Piping and tubing constructed of low-deformability materials, such as cast iron, glass, and nonductile plastics	2½	3	2
Ductwork, including in-line components, constructed of high-deformability materials, with joints made by welding or brazing	2½	9	2
Ductwork, including in-line components, constructed of high- or limited-deformability materials with joints made by means other than welding or brazing	2½	6	2
Ductwork, including in-line components, constructed of low-deformability materials, such as cast iron, glass, and nonductile plastics	2½	3	2
Electrical conduit and cable trays	2½	6	2
Bus ducts	1	2½	2
Plumbing	1	2½	2
Pneumatic tube transport systems	2½	6	2

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[a] A lower value for a_p is permitted where justified by detailed dynamic analyses. The value for a_p shall not be less than 1. The value of a_p equal to 1 is for rigid components and rigidly attached components. The value of a_p equal to 2½ is for flexible components and flexibly attached components.

[b] Components mounted on vibration isolators shall have a bumper restraint or snubber in each horizontal direction. The design force shall be taken as $2F_p$ if the nominal clearance (air gap) between the equipment support frame and restraint is greater than 0.25 in. (6 mm). If the nominal clearance specified on the construction documents is not greater than 0.25 in. (6 mm), the design force is permitted to be taken as F_p.

[c] Overstrength, as required for anchorage to concrete and masonry. See Section 13.4.3 for seismic load effects, including overstrength.

13.4.5 Deformation Analysis Where nonstructural components are anchored by connection points at different levels, x and y, on the same building or structural system, drift ratios (D_r) shall be calculated in accordance with Equation (13-8):

$$D_r = (\delta_{xA} - \delta_{yA})/(X - Y) \tag{13-8}$$

Where nonstructural components are anchored by connection points on separate buildings or structural systems at the same level x, relative displacements (D_p) shall be calculated in accordance with Equation (13-9). Deformations shall be determined in accordance with Chapter 7 using the seismic hazard corresponding to the Performance Objective for which the building is being evaluated.

$$D_p = |\delta_{xA}| + |\delta_{xB}| \tag{13-9}$$

where

D_p = Relative seismic displacement;
D_r = Drift ratio;
X = Height of upper support attachment at level x as measured from grade;
Y = Height of lower support attachment at level y as measured from grade;
δ_{xA} = Deflection at level x of Building A;
δ_{yA} = Deflection at level y of Building A; and
δ_{xB} = Deflection at level x of Building B or equal to 0.03 times the height, X, of level x above grade or as determined using other approved approximate procedures.

The effects of seismic displacements shall be considered in combination with displacements caused by other loads that are present.

13.4.6 Component Testing As an alternative to the analytical requirements of Section 13.4. 4, testing shall be deemed an acceptable method to verify the seismic performance of components and their supports and attachments for the Position Retention Nonstructural Performance Level. Seismic qualification by testing based on a nationally recognized testing procedure, such as ICC-ES AC-156, acceptable to the Authority Having Jurisdiction, shall be deemed to satisfy the evaluation or retrofit requirements, provided that the substantiated seismic capacities equal or exceed the seismic demands determined in accordance with Section 13.4.4.1.

13.4.7 Overturning Evaluation Where permitted by Section 13.7, unanchored equipment weighing more than 100 lb (0.45 kN) supported directly on structural floor framing shall be evaluated for resistance to overturning. The spectral acceleration associated with the overturning resistance of a component (S_{aO}) shall be calculated as in Equations (13-10), (13-11), and (13-12):

$$S_{aO} = \frac{2B}{\alpha} \tag{13-10}$$

where

$$\propto\, = \arctan\left(\frac{b_2}{h_{cq}}\right) \tag{13-11}$$

$$B = \cos(\propto) + \frac{b_2}{h_{cq}}\sin(\propto) - 1 \tag{13-12}$$

h_{cg} = Height to the center of gravity of the component, and
b_2 = One-half the least width of the component.

The ratio of S_{aO} to the calculated floor acceleration shall be greater than 1.5. The calculated floor acceleration shall be taken as F_p/W_p.

13.5 RETROFIT APPROACHES

Nonstructural retrofit shall be accomplished by approved methods based on the classification of the nonstructural component and the Performance Level desired for the nonstructural component:

1. For the retrofit of nonstructural components that are acceleration sensitive, the retrofit approach shall provide appropriate anchorage or bracing for the anchorage to resist the forces calculated by Section 13.4.4. If the nonstructural component itself is deficient, the component shall be strengthened or replaced.
2. For the retrofit of nonstructural components that are deformation sensitive, the retrofit approach shall provide for sufficient deformation capability to allow the nonstructural component to undergo the calculated deformation while maintaining position. Alternately, structural retrofitting of the building shall be designed to reduce the deformation demand for the nonstructural components to meet the deformation acceptance criteria in Sections 13.6 through 13.8.

13.6 ARCHITECTURAL COMPONENTS: DEFINITION, BEHAVIOR, AND ACCEPTANCE CRITERIA

13.6.1 Exterior Wall Components

13.6.1.1 Adhered Veneer

13.6.1.1.1 Definition and Scope Adhered veneer shall include the following types of exterior finish materials secured to a backing material, which shall be masonry, concrete, cement plaster, or a structural framework material, by adhesives:

1. Tile, masonry, stone, terra-cotta, or other similar materials;
2. Glass mosaic units;
3. Ceramic tile; and
4. Exterior plaster (stucco).

13.6.1.1.2 Component Behavior and Retrofit Methods Adhered veneer shall be considered acceleration sensitive in its out-of-plane direction and deformation sensitive in plane.

Adhered veneer not conforming to the acceptance criteria of Section 13.6.1.1.3 shall be retrofitted in accordance with Section 13.5.

13.6.1.1.3 Acceptance Criteria Acceptance criteria for adhered veneer specified in this section shall be applied in accordance with Section 13.3.

1. **Life Safety and Position Retention Nonstructural Performance Level:** The veneer and backup system shall conform to the requirements of Section 13.5.3, of ASCE 7. The backup system shall be adequately attached to resist both in-plane and out-of-plane seismic forces computed in accordance with Section 13.4.4 of this standard using a component importance factor, I_p, of 1.0 and shall be attached to accommodate the drifts computed in accordance with Section 13.4.5 without failure of the backup system or dislodging of the veneer. The adequacy of the backup system to accommodate the calculated drifts shall be demonstrated by analysis or testing. In lieu of testing or analysis, the drift ratio computed in accordance with Section 13.5, of this standard, with ($\delta_{xA} - \delta_{yA}$) not less than 0.5 in. (13 mm), shall be limited to 0.02.
2. **Operational Nonstructural Performance Level:** The veneer and backup system shall conform to the

requirements of Section 13.5.3 of ASCE 7. The backup system shall be adequately attached to resist both in-plane and out-of-plane seismic forces computed in accordance with Section 4 of this standard using a component importance factor, I_p, of 1.5 and shall be attached to accommodate the drifts computed in accordance with Section 5 without yielding of the backup system or dislodging of the veneer or sealant joints. The adequacy of the backup system to accommodate the calculated drifts shall be demonstrated by analysis or testing. In lieu of testing or analysis, the drift ratio computed in accordance with Section 13.4.5, with $(\delta_{xA} - \delta_{yA})$ not less than 0.5 in. (13 mm), shall be limited to 0.01.

13.6.1.1.4 Evaluation Requirements Adhered veneer shall be evaluated by visual observation and tapping to discern looseness or cracking.

13.6.1.2 Anchored Veneer

13.6.1.2.1 Definition and Scope Anchored veneer shall include the following types of masonry or stone units that are attached to the supporting structure by mechanical means:

1. Masonry units,
2. Stone units, and
3. Stone slab units.

The provisions of this section shall apply to units that are more than 48 in. (1.2 m) above the ground or the adjacent exterior area.

13.6.1.2.2 Component Behavior and Retrofit Methods Anchored veneer shall be considered acceleration sensitive in the out-of-plane direction and deformation sensitive in plane.

Anchored veneer and connections not conforming to the acceptance criteria of Section 13.6.1.2.3 shall be retrofitted in accordance with Section 13.5.

13.6.1.2.3 Acceptance Criteria Acceptance criteria for anchored veneer specified in this section shall be applied in accordance with Section 13.3.

1. **Life Safety and Position Retention Nonstructural Performance Level:** The veneer and backup system shall conform to the requirements of Section 13.5.3 and Section 14.4.6 of ASCE 7. The backup systems and the veneer's anchorage to the backup system shall be adequately attached to resist both in-plane and out-of-plane seismic forces computed in accordance with Section 4 using a component importance factor, I_p, of 1.0 and shall be attached to accommodate the drifts computed in accordance with Section 5 without failure of the backup system or dislodging of the veneer. The adequacy of the backup system to accommodate the calculated drifts shall be demonstrated by analysis or testing. In lieu of testing or analysis, the drift ratio computed in accordance with Section 5, with $(\delta_{xA} - \delta_{yA})$ not less than 0.5 in. (13 mm), shall be limited to 0.02.
2. **Operational Nonstructural Performance Level:** The veneer and backup system shall conform to the requirements of Section 13.5.3 and Section 14.4.6, of ASCE 7. The backup systems and the veneer's anchorage to the backup system shall be adequately attached to resist both in-plane and out-of-plane seismic forces computed in accordance with Section 4 using a component importance factor, I_p, of 1.5 and shall be attached to accommodate the drifts computed in accordance with Section 5 without yielding of the backup system or dislodging of the veneer and sealant joints. The adequacy of the backup system to accommodate the calculated drifts shall be demonstrated by analysis or testing. In lieu of testing or analysis, the drift ratio computed in accordance with Section 5, with $(\delta_{xA} - \delta_{yA})$ not less than 0.5 in. (13 mm), shall be limited to 0.01.

13.6.1.2.4 Evaluation Requirements Veneer units shall have adequate stability, joint detailing, and maintenance to prevent moisture penetration from weather that could destroy the anchors. There shall be sufficient support for the veneer over openings. There shall be sufficient connection of the veneer to the backup system over weakened planes. The anchors shall be visually inspected and tested to determine capacity if any signs of deterioration are visible.

13.6.1.3 Glass Block Units and Other Nonstructural Masonry

13.6.1.3.1 Definition and Scope Glass block and other units that are self-supporting for static vertical loads, held together by mortar, and structurally detached from the surrounding structure shall be retrofitted in accordance with this section.

13.6.1.3.2 Component Behavior and Retrofit Methods Glass block units and other nonstructural masonry shall be considered both acceleration and deformation sensitive.

Retrofit of individual walls less than 144 ft^2 (13.4 m^2) or 15 ft (4.6 m) in any dimension using prescriptive procedures based on Section 2110 of IBC shall be permitted. For walls larger than 144 ft^2 (13.4 m^2) or 15 ft (4.6 m) in any dimension, the analytical procedure shall be used.

Glass block units and other nonstructural masonry not conforming with the requirements of Section 13.6.1.3.3 shall be retrofitted in accordance with Section 13.5.

13.6.1.3.3 Acceptance Criteria Acceptance criteria for glass block units and other nonstructural masonry specified in this section shall be applied in accordance with Section 13.3.

1. **Life Safety and Position Retention Nonstructural Performance Level:** The glass block and other nonstructural masonry wall systems shall conform to the requirements of Section 13.5.3 of ASCE 7. Glass block and other nonstructural masonry walls and their enclosing framing shall be capable of resisting both in-plane and out-of-plane forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0 and shall be attached to accommodate the drifts computed in accordance with Section 5 without failure of the backup system or dislodging of the masonry. The adequacy of the masonry walls and their enclosing frames to accommodate the calculated drifts shall be demonstrated by analysis or testing. In lieu of testing or analysis, the drift ratio calculated in accordance with Section 13.4.5, with $(\delta_{xA}–\delta_{yA})$ not less than 0.5 in. (13 mm), shall be limited to 0.02.
2. **Operational Nonstructural Performance Level:** The glass block and other nonstructural masonry wall systems shall conform to the requirements of Section 13.5.3 of ASCE 7. Glass block and other nonstructural masonry walls and their enclosing framing shall be capable of resisting both in-plane and out-of-plane forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5 and shall be attached to accommodate the drifts computed in accordance with Section 5 without yielding of the backup system or dislodging of the masonry and sealant joints. The adequacy of

the backup system to accommodate the calculated drifts shall be demonstrated by analysis or testing. In lieu of testing or analysis, the drift ratio calculated in accordance with Section 13.4.5, with (δ_{xA}–δ_{yA}) not less than 0.5 in. (13 mm), shall be limited to 0.01.

13.6.1.3.4 Evaluation Requirements Glass block units and other nonstructural masonry shall be evaluated based on the criteria of Section 2110 of IBC.

13.6.1.4 Prefabricated Panels

13.6.1.4.1 Definition and Scope The following types of prefabricated panels designed to resist wind, seismic, and other applied forces shall be retrofitted in accordance with this section:

1. Precast concrete and concrete panels with facing (in general, stone) laminated or mechanically attached;
2. Laminated metal-faced insulated panels; and
3. Steel strong-back panels with insulated, water-resistant facing, or mechanically attached metal or stone facing.

13.6.1.4.2 Component Behavior and Retrofit Methods Prefabricated panels shall be considered acceleration sensitive in the out-of-plane direction and deformation sensitive in plane.

Prefabricated panels not conforming to the acceptance criteria of Section 13.6.1.4.3 shall be retrofitted in accordance with Section 13.5.

13.6.1.4.3 Acceptance Criteria Acceptance criteria for prefabricated panels specified in this section shall be applied in accordance with Section 13.3.

1. **Life Safety and Position Retention Nonstructural Performance Level:** Prefabricated panels and connections shall conform to the requirements of Section 13.5.3 of ASCE 7. Prefabricated panels and connections shall be capable of resisting both in-plane and out-of-plane forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0 and shall be attached to accommodate the drifts computed in accordance with Section 5 without failure of the backup system or dislodging of the panels. The adequacy of the panels and their connections to accommodate the calculated drifts shall be demonstrated by analysis or testing. In lieu of testing or analysis, the drift ratio computed in accordance with Section 13.4.5, with (δ_{xA}–δ_{yA}) not less than 0.5 in. (13 mm), shall be limited to 0.02.
2. **Operational Nonstructural Performance Level:** Prefabricated panels and connections shall conform to the requirements of Section 13.5.3 of ASCE 7. Prefabricated panels and connections shall be capable of resisting both in-plane and out-of-plane forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5 and shall be attached to accommodate the drifts computed in accordance with Section 13.4.5 without yielding of the panels and connections or dislodging of the sealant joints. The adequacy of the backup system to accommodate the calculated drifts shall be demonstrated by analysis or testing. In lieu of testing or analysis, the drift ratio computed in accordance with Section 5, with (δ_{xA}–δ_{yA}) not less than 0.5 in. (13 mm), shall be limited to 0.01.

13.6.1.4.4 Evaluation Requirements Connections shall be visually inspected and tested to determine capacity if any signs of deterioration or displacement are visible. Anchors shall be tested in accordance with Section 13.4.6.

13.6.1.5 Glazed Exterior Wall Systems

13.6.1.5.1 Definition and Scope Glazed exterior wall systems shall include the following types of assemblies:

1. Glazed curtain wall systems that extend beyond the edges of structural floor slabs and are assembled from prefabricated units (e.g., “unitized” curtain wall systems) or assembled on site (e.g., “stick” curtain wall systems),
2. Glazed storefront systems that are installed between structural floor slabs and are prefabricated or assembled on site, and
3. Structural silicone glazing in which silicone sealant is used for the structural transfer of loads from the glass to its perimeter support system and for the retention of the glass in the opening.

13.6.1.5.2 Component Behavior and Retrofit Methods Glazed exterior wall systems shall be considered both deformation sensitive and acceleration sensitive.

Glazed exterior wall systems not conforming to the acceptance criteria of Section 13.6.1.5.3 shall be retrofitted in accordance with Section 13.5.

13.6.1.5.3 Acceptance Criteria Acceptance criteria for glazed exterior wall systems specified in this section shall be applied in accordance with Section 13.3.

1. **Life Safety and Position Retention Nonstructural Performance Level.** Glazed exterior wall systems and their supporting structure shall be capable of resisting the in-plane and out-of-plane seismic forces computed in accordance with Section 4 and shall be attached to accommodate the drifts computed in accordance with Section 5 without failure of the backup system or dislodging of the panels. Glass components meeting any of the following criteria need not be retrofitted for the Life Safety Nonstructural Performance Level:
   1.1. Any glass component shall have sufficient clearance from the frame such that physical contact between the glass and the frame does not occur at the relative seismic displacement that the component shall be capable of accommodating, as demonstrated by Equation (13-13) and (13-14):

$$D_{\text{clear}} \geq 1.25 D_p \tag{13-13}$$

$$D_{\text{clear}} = 2c_1\left(1 + \frac{h_p c_2}{b_p c_1}\right); \tag{13-14}$$

   where

   h_p = Height of rectangular glass,
   b_p = Width of rectangular glass,
   c_1 = Clearance (gap) between vertical glass edges and the frame,
   c_2 = Clearance (gap) between horizontal glass edges and the frame, and
   D_p = Relative seismic displacement that the component shall be capable of accommodating. D_p shall be determined similar to Equation (13-8) over the height of the glass component under consideration.

   1.2. Fully tempered monolithic glass that is located no more than 10 ft (3.1 m) above a walking surface.
   1.3. Annealed or heat-strengthened laminated glass in single thickness with interlayer no less than 0.03 in. (0.8 mm) that is captured mechanically in a wall system glazing pocket, and whose perimeter is secured

to the wall system frame by a wet-glazed perimeter bead of 0.5 in. (13 mm) minimum glass contact width, or other approved anchorage system.

1.4. Any glass component that meets the relative displacement requirement of Equation (13-15):

$$\Delta_{\text{fallout}} \geq 1.25 D_p \tag{13-15}$$

or 0.5 in. (13 mm), whichever is greater, where

D_p = Relative seismic displacement that the component shall be capable of accommodating; and

D_{fallout} = Relative seismic displacement (drift) causing glass fallout from the curtain wall, storefront, or partition, as determined in accordance with an approved engineering analysis method.

2. **Operational Nonstructural Performance Level.** Glazed exterior wall systems and their supporting structure shall be capable of resisting both in-plane and out-of-plane seismic forces computed in accordance with Section 4 and shall be attached to accommodate the drifts computed in accordance with Section 5 without yielding of the wall systems and connections or dislodging of the glazing and sealant joints. Glass components meeting any of the following criteria need not be retrofitted for Performance Levels higher than the Position Retention Nonstructural Performance Level:

2.1. Any glass component with sufficient clearance from the frame such that physical contact between the glass and the frame does not occur at the relative seismic displacement that the component shall be capable of accommodating, as demonstrated by Equation (13-13).

2.2. Annealed or heat-strengthened laminated glass in single thickness with interlayer no less than 0.03 in. (0.8 mm) that is captured mechanically in a wall system glazing pocket, and whose perimeter is secured to the wall system frame by a wet-glazed perimeter bead of 0.5 in. (13 mm) minimum glass contact width, or other approved anchorage system.

2.3. Any glass component that meets the relative displacement requirement of Equation (13-16):

$$\Delta_{\text{fallout}} \geq 1.5 \times 1.25 D_p \tag{13-16}$$

or 0.5 in. (13 mm), whichever is greater.

13.6.1.5.4 Evaluation Requirements To establish compliance with Criteria 1.1, 1.2, 1.3, 2.1, or 2.2 in Section 13.6.1.5.3, glazed exterior wall systems shall be evaluated visually to determine glass type, support details, mullion configuration, sealant type, and anchors. To establish compliance with Criterion 1.4 or 2.3, an approved analysis shall be used.

Δ_{fallout}, which is used in Equation (13-15), shall be determined using AAMA 501.6 or by engineering analysis.

13.6.2 Partitions

13.6.2.1 Definition and Scope Partitions shall include vertical non-load-bearing interior components that provide space division.

Heavy partitions shall include, but are not limited to, partitions constructed of masonry materials or assemblies.

Light partitions shall include, but are not limited to, partitions constructed of metal or wood studs surfaced with lath and plaster, gypsum board, wood, or other facing materials.

13.6.2.2 Component Behavior and Retrofit Methods Partitions shall be considered both acceleration and deformation sensitive.

Glazed partitions that span from floor to ceiling or to the underside of the floor or roof above shall conform to the requirements of Section 13.6.1.5.

Light and heavy partitions shall be evaluated based on the provisions in Section 13.6.2.3.

Partitions not meeting the acceptance criteria of Section 13.6.2.3 shall be retrofitted in accordance with Section 13.5.

13.6.2.3 Acceptance Criteria Acceptance criteria for partitions specified in this section shall be applied in accordance with Section 13.3.

13.6.2.3.1 Life Safety Nonstructural Performance Level

1. **Heavy Partitions.** Nonstructural heavy partitions shall be capable of resisting both in-plane and out-of-plane forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0 and shall be attached to accommodate the drifts computed in accordance with Section 13.4.5 without failure of the partition or connections. The adequacy of the partitions and their connections to accommodate the calculated drifts shall be demonstrated by analysis or testing. In lieu of testing or analysis, the drift ratio computed in accordance with Section 13.4.5 shall be limited to 0.01.
2. **Light Partitions.** Nonstructural light partitions need not be evaluated for the Life Safety Nonstructural Performance Level.

13.6.2.3.2 Position Retention Nonstructural Performance Level

1. **Heavy Partitions.** Nonstructural heavy partitions and their connections shall conform to the requirements of Section 13.5.8 of ASCE 7 and shall be capable of resisting both in-plane and out-of-plane forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0 and shall be attached to accommodate the drifts computed in accordance with Section 13.4.5 without failure of the partition or connections. The adequacy of the partitions and their connections to accommodate the calculated drifts shall be demonstrated by analysis or testing. In lieu of testing or analysis, the drift ratio computed in accordance with Section 13.4.5 shall be limited to 0.01.
2. **Light Partitions.** Nonstructural light partitions shall conform to the requirements of Section 13.5.8 of ASCE 7 and shall be capable of resisting the out-of-plane forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0. The drift ratio computed in accordance with Section 13.4.5 shall be limited to 0.02.

13.6.2.3.3 Operational Nonstructural Performance Level

1. **Heavy Partitions.** Nonstructural heavy partitions and their connections shall conform to the requirements of Section 13.5.8 of ASCE 7 and shall be capable of resisting both in-plane and out-of-plane forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5 and shall be attached to accommodate the drifts computed in accordance with Section 13.4.5 without failure of the partition or connections. The adequacy of the partitions and their connections to accommodate the calculated drifts shall be demonstrated by analysis or testing. In lieu of testing or analysis, the drift ratio computed in accordance with Section 13.4.5 shall be limited to 0.005.
2. **Light Partitions.** Nonstructural light partitions and their connections shall conform to the requirements of Section 13.5.8 of ASCE 7 and shall be capable of resisting both

in-plane and out-of-plane forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5 and shall be attached to accommodate the drifts computed in accordance with Section 13.4.5 without failure of the partition or connections. The adequacy of the partitions and their connections to accommodate the calculated drifts shall be demonstrated by analysis or testing. In lieu of testing or analysis, the drift ratio computed in accordance with Section 13.4.5 shall be limited to 0.01.

13.6.2.4 Evaluation Requirements Partitions shall be evaluated to ascertain the type of material.

13.6.3 Interior Veneers

13.6.3.1 Definition and Scope Interior veneers shall include decorative finish materials applied to interior walls and partitions. These provisions of this section shall apply to veneers mounted 4 ft (1.2 m) or more above the floor.

13.6.3.2 Component Behavior and Retrofit Methods Interior veneers shall be considered acceleration sensitive and deformation sensitive.

Interior veneers not conforming to the acceptance criteria of Section 13.6.3.3 shall be retrofitted in accordance with Section 13.5.

13.6.3.3 Acceptance Criteria Acceptance criteria for interior veneers specified in this section shall be applied in accordance with Section 13.3.

13.6.3.3.1 Life Safety and Position Retention Nonstructural Performance Level Backing shall be adequately attached to resist both in-plane and out-of-plane seismic forces computed in accordance with Section 13.4.3 using a component importance factor, I_p, of 1.0 and shall be attached to accommodate the drifts computed in accordance with Section 13.4.4 without failure of the backup system or dislodging of the veneer. The adequacy of the backup system to accommodate the calculated drifts shall be demonstrated by analysis or testing. In lieu of testing or analysis, the drift ratio computed in accordance with Section 13.4.4 shall be limited to 0.02.

13.6.3.3.2 Operational Nonstructural Performance Level Backing shall be adequately attached to resist both in-plane and out-of-plane seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5 and shall be attached to accommodate the drifts computed in accordance with Section 13.4.5 without yielding of the backup system or dislodging of the veneer and sealant. The adequacy of the backup system to accommodate the calculated drifts shall be demonstrated by analysis or testing. In lieu of testing or analysis, the drift ratio computed in accordance with Section 13.4.5 shall be limited to 0.01.

13.6.3.4 Evaluation Requirements Backup walls or other supports and the attachments to that support shall be evaluated, as well as the condition of the veneer itself.

13.6.4 Ceilings

13.6.4.1 Definition and Scope Ceilings shall be categorized as one of the following types:

1. **Category A:** Surface-applied or furred with materials that are applied directly to wood joists, concrete slabs, or steel decking with mechanical fasteners or adhesives;
2. **Category B:** Short-dropped gypsum board sections [less than 2 ft (610 mm) drop] attached to wood or metal furring supported by carrier members;
3. **Category C:** Dropped gypsum board sections greater than 2 ft (610 mm) and suspended metal lath and plaster; and
4. **Category D:** Suspended acoustical board inserted within T-bars, together with lighting fixtures and mechanical items, to form an integrated ceiling system.

13.6.4.2 Component Behavior and Retrofit Methods Ceiling systems shall be considered both acceleration and deformation sensitive. Forces for ceilings shall be calculated in accordance with Section 13.4.4. Deformations shall be determined in accordance with Section 13.4.5.

Ceilings not conforming to the acceptance criteria of Section 13.6.4.3 shall be retrofitted in accordance with Section 13.5.

Where retrofit is required for ceilings in Category A or B, they shall be strengthened to resist seismic forces computed in accordance with Section 13.4.4. Where retrofit is required for ceilings in Category D, they shall be retrofitted by the prescriptive procedures of Section 13.5.6.2 of ASCE 7.

13.6.4.3 Acceptance Criteria Acceptance criteria for ceilings shall be as specified in this section and Section 13.4.

13.6.4.3.1 Life Safety Nonstructural Performance Level Ceilings in Category A, B, or D need not be evaluated for the Life Safety Performance Level except that ceilings in Category A over 10 ft^2 (0.93 m^2) in area shall meet the Position Retention Nonstructural Performance Level. Ceilings in Category C shall be capable of accommodating the relative displacement computed in accordance with Section 13.4.5.

13.6.4.3.2 Position Retention Nonstructural Performance Level Ceilings in Category A or B shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0. Ceilings in Category C shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0 and accommodating the relative displacement computed in accordance with Section 13.4.5. Ceilings in Category D shall be evaluated in accordance with the prescriptive procedures of Section 13.5.6.2 of ASCE 7.

13.6.4.3.3 Operational Nonstructural Performance Level Ceilings in Category A, B, or D shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5. Ceilings in Category C shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0 and accommodating the relative displacement computed in accordance with Section 13.4.5.

13.6.4.4 Evaluation Requirements The condition of the ceiling finish material, its attachment to the ceiling support system, the attachment and bracing of the ceiling support system to the structure, and the potential seismic impacts of other nonstructural systems on the ceiling system shall be evaluated.

13.6.5 Parapets and Cornices

13.6.5.1 Definition and Scope Parapets and cornices shall include exterior nonstructural features that project above or away from the building. The following parapets and appendages shall be evaluated and retrofitted in accordance with this section:

1. Unreinforced masonry parapets with an aspect ratio greater than 1.5;

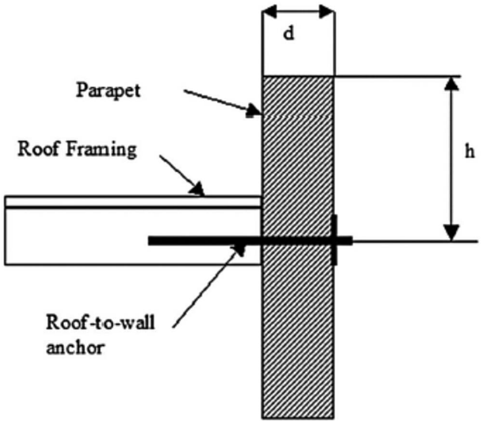

Figure 13-1. Parapet aspect ratio.

2. Reinforced masonry or reinforced concrete parapets with an aspect ratio greater than 3.0;
3. Cornices or ledges constructed of stone, terra-cotta, or brick, unless supported by a steel or reinforced concrete structure; and
4. Sculptures and ornamental features constructed of stone, terra-cotta, masonry, or concrete with an aspect ratio greater than 1.5.

The aspect ratio of parapets and appendages shall be defined as the height of the component above the level of anchorage (h) divided by the width of the component d, as shown in Figure 13-1. For horizontal projecting appendages, the aspect ratio shall be defined as the ratio of the horizontal projection beyond the vertical support of the building to the perpendicular dimension.

13.6.5.2 Component Behavior and Retrofit Methods Parapets and appendages shall be considered acceleration sensitive in the out-of-plane direction. Forces for parapets and cornices shall be calculated in accordance with Section 13.4.4.

Parapets and appendages not conforming to the requirements of Section 13.6.5.3 shall be retrofitted in accordance with Section 13.5.

Exception: Retrofit of unreinforced masonry parapets not more than 4 ft (1.2 m) high shall be permitted to be designed and detailed to conform to approved prescriptive procedures.

13.6.5.3 Acceptance Criteria Acceptance criteria for parapets and cornices specified in this section shall be applied in accordance with Section 13.3.

13.6.5.3.1 Life Safety and Position Retention Nonstructural Performance Level Parapets and appendages shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0.

13.6.5.3.2 Operational Nonstructural Performance Level Parapets and appendages shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5.

13.6.5.4 Evaluation Requirements The condition of mortar and masonry, connection to supports, type and stability of the supporting structure, and horizontal continuity of the parapet coping shall be considered in the evaluation.

13.6.6 Architectural Appendages and Marquees

13.6.6.1 Definition and Scope Architectural appendages shall include projections from an exterior wall that are extensions of the horizontal building structure or independent structures that are tied to the building. They shall also include sculptures and other ornamental features. Marquees shall include freestanding structures.

Canvas or other fabric projections need not be retrofitted in accordance with this section.

13.6.6.2 Component Behavior and Retrofit Methods Canopies and marquees shall be considered acceleration sensitive.

Canopies and marquees not conforming to the acceptance criteria of Section 13.6.6.3 shall be retrofitted in accordance with Section 13.5.

13.6.6.3 Acceptance Criteria Acceptance criteria for architectural appendages and marquees specified in this section shall be applied in accordance with Section 13.3.

13.6.6.3.1 Life Safety and Position Retention Nonstructural Performance Level Canopies, marquees, and other appendages shall be capable of resisting both horizontal and vertical seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0.

13.6.6.3.2 Operational Nonstructural Performance Level Canopies, marquees, and other appendages shall be capable of resisting both horizontal and vertical seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5.

13.6.6.4 Evaluation Requirements Buckling in bracing, connection to supports, and type and stability of the supporting structure shall be considered in the evaluation.

13.6.7 Penthouses

13.6.7.1 Definition and Scope Penthouse structures that are enclosed, unoccupied rooftop structures that are intended to shelter mechanical and electrical equipment with occasional access for maintenance shall be evaluated in accordance with this section. Rooftop structures intended for occupancy or with an aggregate area that exceeds one-third of the area of the supporting roof, an aggregate effective weight greater than 25% of the effective weight of the supporting story, or an aggregate effective weight greater an 10% of the effective weight of the entire structure shall be considered as part of the structure. Unenclosed rooftop structures, such as screens used to shield mechanical and electrical equipment, shall be evaluated as an appendage.

13.6.7.2 Component Behavior and Retrofit Methods Penthouses shall be considered acceleration sensitive.

Penthouses not conforming to the acceptance criteria of Section 13.6.7.3 shall be retrofitted in accordance with Section 13.5.

13.6.7.3 Acceptance Criteria Acceptance criteria for the lateral-force-resisting systems of penthouses as specified in this section shall be applied in accordance with Section 13.3.

13.6.7.3.1 Life Safety and Position Retention Nonstructural Performance Level Penthouse structures shall be capable of resisting seismic forces in each orthogonal direction computed in accordance with Section 13.4.3 using a component importance factor, I_p, of 1.0.

13.6.7.3.2 Operational Nonstructural Performance Level Penthouse structures shall be capable of resisting seismic forces in each orthogonal direction computed in accordance with Section 13.4.3 using a component importance factor, I_p, of 1.5.

13.6.8 Tile Roofs

13.6.8.1 Definition and Scope Heavy tiles on sloped roofs shall be evaluated in accordance with this section. Tiles weighing less than 4 lb/ft^2 need not be evaluated.

13.6.8.2 Component Behavior and Retrofit Methods Roof tiles on sloped roofs shall be considered acceleration sensitive.

Fasteners for tiles shall be prescriptively evaluated per the requirements of Chapter 15 of the 2018 *International Existing Building Code* (IEBC). Tiles not conforming to the prescriptive requirements and not meeting the acceptance criteria of Section 13.6.8.3 shall be retrofitted in accordance with Section 13.5.

13.6.8.3 Acceptance Criteria Acceptance criteria for tile roofs as specified in this section shall be applied in accordance with Section 13.3.

13.6.8.3.1 Life Safety and Position Retention Nonstructural Performance Level Roof tiles shall be capable of resisting seismic forces computed in accordance with Section 13.4.3 using a component importance factor, I_p, of 1.0.

13.6.8.3.2 Operational Nonstructural Performance Level Individual tiles shall be independently anchored to the structural roof substrate with a connection capable of resisting seismic forces computed in accordance with Section 13.4.3 using a component importance factor, I_p, of 1.5.

13.6.9 Chimneys and Stacks

13.6.9.1 Definition and Scope Chimneys and stacks that are cantilevered above building roofs shall be evaluated in accordance with this section. Light metal residential chimneys need not comply with the provisions of this document.

13.6.9.2 Component Behavior and Retrofit Methods Chimneys and stacks shall be considered acceleration sensitive. Forces for chimneys and stacks shall be calculated in accordance with Section 13.4.4.

Chimneys and stacks not conforming to the acceptance criteria of Section 13.6.7.3 shall be retrofitted in accordance with Section 13.5.

Exception: Retrofit of residential masonry chimneys using approved prescriptive procedures shall be permitted.

13.6.9.3 Acceptance Criteria Acceptance criteria for chimneys and stacks specified in this section shall be applied in accordance with Section 13.3.

13.6.9.3.1 Life Safety and Position Retention Nonstructural Performance Level Chimneys and stacks shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0.

13.6.9.3.2 Operational Nonstructural Performance Level Chimneys and stacks shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5.

13.6.9.4 Evaluation Requirements The condition of the mortar and masonry, connection to adjacent structure, and type and stability of foundations shall be considered in the evaluation.

Concrete shall be evaluated for spalling and exposed reinforcement. Steel shall be evaluated for corrosion.

13.6.10 Stairs and Ramps

13.6.10.1 Definition and Scope Stairs shall include the treads, risers, guardrails, and landings that make up passageways between floors. Ramps shall include sloped elevated walkways that provide access between floor levels, as well as the guardrails.

Stairs and ramps without sliding or ductile connections that can accommodate seismic relative displacements shall be considered part of the seismic-force-resisting system, and their strength and stiffness shall be included in the building structural model in accordance with Section 7.2.4.3. The stiffness and strength of such stairs or ramps shall be included in the building structural model, and their components shall be evaluated using the provisions for the applicable materials in Chapters 9 through 12.

13.6.10.2 Component Behavior and Retrofit Methods Each of the separate components of the stairs shall be defined as either acceleration or deformation sensitive, depending on the predominant behavior. Components of stairs that are attached to adjacent floors or floor framing shall be considered deformation sensitive. All other stair components shall be considered acceleration sensitive.

Stairs not conforming to the acceptance criteria of Section 13.6.10.3 shall be retrofitted in accordance with Section 13.5.

13.6.10.3 Acceptance Criteria Acceptance criteria for stairs and ramps specified in this section shall be applied in accordance with Section 13.3.

13.6.10.3.1 Life Safety and Position Retention Nonstructural Performance Level Stairs and ramps shall be capable of resisting the seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0 and shall be capable of accommodating the expected relative displacement of the structure at the location of the stair computed in accordance with Section 13.4.5. The net relative displacement shall be assumed to occur in any horizontal direction. Such elements shall be supported by means of positive and direct structural supports or by mechanical connections and fasteners in accordance with the following requirements:

1. Sliding connections with slotted or oversize holes, sliding bearing supports with keeper assemblies or end stops, and connections that permit movement by deformation of metal attachments shall accommodate the drifts computed in accordance with Section 13.4.5, but not less than 0.5 in. (13 mm), without loss of vertical support or inducement of displacement-related compression forces in the stair.
2. Sliding bearing supports without keeper assemblies or end stops shall be designed to accommodate a displacement equal to 1.5 times the drifts computed in accordance with Section 13.4.5, but not less than 1.0 in. (25 mm) without loss of vertical support. Breakaway restraints are permitted if their failure does not lead to loss of vertical support.
3. Metal supports shall be designed with rotation capacity to accommodate seismic relative displacements as defined in Item 2. The strength of such metal supports shall not be limited by bolt shear, weld fracture, or other brittle modes.
4. All fasteners and attachments such as bolts, inserts, welds, dowels, and anchors shall be designed for the seismic design forces determined in accordance with Section 13.4.4.1, with R_p, a_p, and Ω_0 as given in Table 13.5-1, of ASCE 7.

13.6.10.3.2 Operational Nonstructural Performance Level Stairs and ramps shall be capable of resisting the seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5 and shall be capable of accommodating the expected relative displacement of the structure at the location of the stair computed in accordance with Section 13.4.5. The net relative displacement shall be assumed to occur in any horizontal direction. Such elements shall be supported by means of positive and direct structural supports or by mechanical connections and fasteners in accordance with the following requirements:

1. Sliding connections with slotted or oversize holes, sliding bearing supports with keeper assemblies or end stops, and connections that permit movement by deformation of metal

attachments shall accommodate the drifts computed in accordance with Section 13.4.5, but not less than 1.0 in. (25 mm), without loss of vertical support or inducement of displacement-related compression forces in the stair.
2. Sliding bearing supports without keeper assemblies or end stops shall be capable of accommodating a displacement equal to 1.5 times the drifts computed in accordance with Section 13.4.5, but not less than 2.0 in. (51 mm) without loss of vertical support. Breakaway restraints are permitted if their failure does not lead to loss of vertical support.
3. Metal supports shall have rotation capacity to accommodate seismic relative displacements as defined in Item 2. The strength of such metal supports shall not be limited by bolt shear, weld fracture, or other brittle modes.
4. All fasteners and attachments such as bolts, inserts, welds, dowels, and anchors shall be evaluated for the seismic design forces determined in accordance with Section 13.4.4.1 with R_p, a_p, and Ω_0 as given in Table 13.5-1 of ASCE 7.

13.6.10.4 Evaluation Requirements The materials and condition of stair members and their connections to supports and the types and stability of supporting and adjacent walls, windows, and other portions of the stair shaft system shall be considered in the evaluation. Anchors shall be tested in accordance with Section 13.3.2.

13.6.11 Doors Required for Emergency Services Egress in Essential Facilities

13.6.11.1 Definition and Scope Doors shall include the apparatuses of the garage door systems, their connections to fire stations, and other door systems and connections that are critical for egress of emergency services from buildings immediately after earthquakes.

13.6.11.2 Component Behavior and Retrofit Methods Each of the separate components of the door systems shall be defined as either acceleration or deformation sensitive, depending on the predominant behavior. Door jambs, vertical and horizontal tracks, rollers, and their connections shall be considered deformation sensitive.

13.6.11.3 Acceptance Criteria Acceptance criteria shall be applied in accordance with Section 13.3.

13.6.11.3.1 Life Safety and Position Retention Nonstructural Performance Levels Door systems shall be capable of resisting the seismic forces computed in accordance with Section 13.4.4. Doors and connections shall be capable of accommodating a drift ratio of 0.01 computed in accordance with Section 13.4.5. A deformation compatibility analysis shall demonstrate that door systems can accommodate the drifts such that the door can be manually opened or closed without binding.

13.6.11.3.2 Operational Nonstructural Performance Level Door systems shall be capable of resisting the seismic forces computed in accordance with Section 13.4.4. Doors and connections shall be capable of accommodating a drift ratio of 0.005 computed in accordance with Section 13.4.5. A deformation compatibility analysis shall demonstrate that door systems can accommodate the drifts such that the door can be manually opened or closed without binding.

13.6.11.4 Evaluation Requirements The components of door systems, their connections to supports, and gaps and tolerances between the components shall be considered in the evaluation.

13.6.12 Computer Access Floors

13.6.12.1 Definition and Scope Computer access floors shall include panelized, elevated floor systems designed to facilitate access to wiring, fiber optics, and other services associated with computers and other electronic components.

13.6.12.2 Component Behavior and Retrofit Methods Computer access floors shall be considered both acceleration sensitive and deformation sensitive.

Computer access floors not conforming to the acceptance criteria of Section 13.6.12.3 shall be retrofitted in accordance with Section 13.5.

13.6.12.3 Acceptance Criteria Acceptance criteria shall be applied in accordance with Section 13.3.

13.6.12.3.1 Life Safety Nonstructural Performance Level Computer access floors need not be evaluated for the Life Safety Nonstructural Performance Level.

13.6.12.3.2 Position Retention Nonstructural Performance Level Computer access floors shall conform to the requirements of Section 13.5.7 of ASCE 7.

If the prescriptive procedure is selected, prescriptive requirements of Section 13.4.3 shall be met. If the analytical procedure is selected, computer access floors shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0.

13.6.12.3.3 Operational Nonstructural Performance Level Computer access floors shall conform to the requirements of Section 13.5.7 of ASCE 7 and shall be special access floors per the requirements of Section 13.5.7.2 of ASCE 7.

If the prescriptive procedure is selected, prescriptive requirements of Section 13.4.3 shall be met. If the analytical procedure is selected, computer access floors shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5.

13.6.12.4 Evaluation Requirements Buckling and racking of access floor supports, connection to the support structure, and the effects of mounted equipment shall be considered in the evaluation. Anchors shall be tested in accordance with Section 13.4.6.

13.7 MECHANICAL, ELECTRICAL, AND PLUMBING COMPONENTS: DEFINITION, BEHAVIOR, AND ACCEPTANCE CRITERIA

13.7.1 Mechanical Equipment

13.7.1.1 Definition and Scope Equipment and distribution systems that are in a building, are supported by a building, or are permanently connected to the mechanical or electrical system of a building that meets one or more of the following types shall be evaluated for seismic anchorage in accordance with this section for the nonstructural performance level:

1. All equipment weighing more than 400 lb (1.8 kN);
2. All equipment containing hazardous materials;
3. Discrete mechanical equipment weighing more than 20 lb (0.9 kN) that is attached to floor, ceiling, wall, or other support with a center of mass more than 4 ft (1.2 m) above the floor; and
4. Distribution systems, including the following:
    4.1. Piping and tubing systems, including fire suppression piping;
    4.2. Conveyors (nonpersonnel);

4.3. Conduits, cable trays, and raceways;
4.4. Distribution systems conveying hazardous materials; and
4.5. HVAC ductwork.

For Hazards Reduced, Life Safety, and Position Retention Performance levels, all other equipment and distribution systems shall be positively fastened to the building or shall have an overturning resistance in accordance with Section 13.4.7.

Exceptions: (1) Category 1 PV arrays designed in accordance with section 13.7.10; and (2) unanchored equipment weighing less than 100 lb (0.45 kN).

For the Operational Performance Level, all equipment in regions of high and moderate seismicity shall be evaluated in accordance with this section.

13.7.1.2 Component Behavior and Retrofit Methods Mechanical equipment shall be considered acceleration sensitive. Forces for equipment shall be calculated in accordance with Section 13.4.4.

Mechanical equipment not conforming to the acceptance criteria of Section 13.7.1.3 shall be retrofitted in accordance with Section 13.5.

13.7.1.3 Acceptance Criteria Acceptance criteria shall be applied in accordance with Section 13.3.

13.7.1.3.1 Life Safety Nonstructural Performance Level Anchorage for mechanical equipment and distribution systems as required per Section 13.7.1.1 shall be capable of resisting seismic forces computed in accordance with Section 13.4.3 using a component importance factor, I_p, of 1.0.

Equipment and distribution systems that form part of the Life Safety system, such as fire suppression equipment, or that contain hazardous materials shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using an importance factor, I_p, or 1.5.

Equipment Type 1 or 2 that is 6 ft (1.8 m) or taller, Equipment Type 3, equipment forming part of an emergency power system, and gas-fired equipment in occupied or unoccupied space shall be evaluated and retrofit to the Position Retention Nonstructural Performance Level in areas of moderate or high seismicity.

13.7.1.3.2 Position Retention Nonstructural Performance Level Equipment and distribution systems shall satisfy the requirements of Sections 13.2 and 13.6, of ASCE 7 based on the requirements for a component importance factor, I_p, of 1.0.

Equipment anchorage shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0.

Equipment that forms part of the Life Safety system, such as fire suppression equipment, or that contains hazardous materials shall meet the requirements for Operational Performance Level in Section 13.7.1.3.3.

13.7.1.3.3 Operational Nonstructural Performance Level Equipment shall satisfy the requirements of Sections 13.2 and 13.6, including special certification requirements of Section 13.2.2 of ASCE 7, based on the requirements for a component importance factor, I_p, of 1.5.

Equipment anchorage shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5.

13.7.1.4 Evaluation Requirements Equipment shall be analyzed to establish acceleration-induced forces, and supports, hold-downs, and bracing shall be visually evaluated. Anchors shall be tested in accordance with Section 13.4.6.

In addition, for the Operational Nonstructural Performance Level, equipment shall be analyzed or tested to demonstrate its ability to remain functional after an earthquake commensurate with the Seismic Hazard Level being considered.

13.7.2 Storage Vessels and Water Heaters

13.7.2.1 Definition and Scope Storage vessels and water heaters shall include all vessels that contain fluids used for building operation.

Vessels shall be classified into one of the following two categories:

1. **Category 1**: Vessels with structural support of contents, in which the shell is supported by legs or a skirt; or
2. **Category 2**: Flat-bottom vessels in which the weight of the contents is supported by the floor, roof, or a structural platform.

13.7.2.2 Component Behavior and Retrofit Methods Tanks and vessels shall be considered acceleration sensitive.

Tanks and vessels not conforming to the acceptance criteria of Section 13.7.2.3 shall be retrofitted in accordance with Section 13.5.

Exceptions: (1) Evaluation and retrofit of residential water heaters with capacity less than 100 gal. (378.5 L) by approved prescriptive procedures are permitted; and (2) Retrofit of Category 2 vessels according to approved prescriptive standards are permitted.

13.7.2.3 Acceptance Criteria Acceptance criteria shall be applied in accordance with Section 13.3. Forces for storage vessels and water heaters shall be calculated in accordance with Section 13.4.4.

13.7.2.3.1 Life Safety and Position Retention Nonstructural Performance Level

1. **Category 1 Equipment**: If the analytical procedure is selected based on Table 13-1, Category 1 equipment and supports shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0. Equipment Type 1 or 2, as defined in Section 13.7.1.1, that is 6 ft (1.8 m) or taller, Equipment Type 3, equipment forming part of an emergency power system, and gas-fired equipment in occupied or unoccupied space shall be evaluated and retrofitted to the Position Retention Nonstructural Performance Level in areas of moderate or high seismicity.
2. **Category 2 Equipment**: If the analytical procedure is selected based on Table 13-1, Category 2 equipment and supports shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0. Equipment Type 1 or 2, as defined in Section 13.7.1.1, that is 6 ft (1.8 m) or taller, Equipment Type 3, equipment forming part of an emergency power system, and gas-fired equipment in occupied or unoccupied space shall be evaluated and retrofitted to the Position Retention Nonstructural Performance Level in areas of moderate or high seismicity.

Vessels containing hazardous materials or water for fire suppression shall meet the requirements for the Operational Nonstructural Performance Level of Section 13.7.2.3.3.

13.7.2.3.2 Operational Nonstructural Performance Level

1. **Category 1 Equipment**: If the analytical procedure is selected, Category 1 equipment and supports shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor,

I_p, of 1.5. It must be demonstrated that the vessel will not lose contents in an earthquake commensurate with the Seismic Hazard Level being used for the evaluation or retrofit.

2. **Category 2 Equipment**: If the analytical procedure is Category 2, equipment and supports shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5. It must be demonstrated that the vessel will not lose contents in an earthquake commensurate with the Seismic Hazard Level being used for the evaluation or retrofit.

13.7.2.4 Evaluation Requirements All equipment shall be visually evaluated to determine the existence of hold-downs, supports, and bracing. Water heaters evaluated or retrofitted using prescriptive procedures should have the bracing and anchors verified to conform with the prescriptive procedures. Anchors shall be tested in accordance with Section 13.3.2.

13.7.3 Pressure Piping

13.7.3.1 Definition and Scope The requirements of this section shall apply to all piping (except fire suppression piping) that carries fluids which, in their vapor stage, exhibit a pressure of 15 lb/ft^2 (720 N/m^2) gauge, or higher.

13.7.3.2 Component Behavior and Retrofit Methods Piping shall be considered acceleration sensitive. Piping that runs between floors or across seismic joints shall be considered both acceleration and deformation sensitive.

Piping not conforming to the acceptance criteria of Section 13.7.3.3 shall be retrofitted in accordance with Section 13.5.

Exception: Retrofit of piping systems according to prescriptive standards shall be permitted. Piping shall meet drift provisions of Section 13.4.5 and the force provisions of Section 13.4.4.

13.7.3.3 Acceptance Criteria Acceptance criteria shall be applied in accordance with Section 13.3.

13.7.3.3.1 Life Safety Nonstructural Performance Level If the prescriptive procedure is selected, piping shall meet the prescriptive requirements of ASME B31. If the analytical procedure is selected, piping shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0. Piping that runs between floors or across seismic joints shall be capable of accommodating relative displacements computed in accordance with Section 13.4.5.

13.7.3.3.2 Position Retention Nonstructural Performance Level Piping systems shall meet the requirements of Section 13.6.8 of ASCE 7.

If the prescriptive procedure is selected piping shall meet the prescriptive requirements of ASME B31. If the analytical procedure is selected, piping shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0. Piping that runs between floors or across seismic joints shall be capable of accommodating relative displacements computed in accordance with Section 13.4.5.

13.7.3.3.3 Operational Nonstructural Performance Level Piping systems shall meet the requirements of Section 13.6.8 of ASCE 7.

If the prescriptive procedure is selected, piping shall meet the prescriptive requirements of ASME B31. If the analytical procedure is selected based on Table 13-1, piping shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5. Piping that runs between floors or across seismic joints shall be capable of accommodating relative displacements computed in accordance with Section 13.4.5.

Piping should maintain leak tightness in an earthquake commensurate with the Seismic Hazard Level being used for the evaluation or retrofit.

13.7.3.4 Evaluation Requirements High-pressure piping shall be tested by an approved method. Lines shall be hydrostatically tested to 150% of the maximum anticipated pressure of the system. Anchors shall be tested in accordance with Section 13.3.2.

13.7.4 Fire Suppression Piping

13.7.4.1 Definition and Scope Fire suppression piping shall include fire sprinkler piping consisting of main risers and laterals weighing, loaded, in the range of 30 to 100 lb/ft (0.44 to 1.46 kN/m), with branches of decreasing size to 2 lb/ft (0.03 kN/m).

13.7.4.2 Component Behavior and Retrofit Methods Fire suppression piping shall be considered acceleration sensitive. Fire suppression piping that runs between floors or across seismic joints shall be considered both acceleration and deformation sensitive.

Fire suppression piping not conforming to the acceptance criteria of Section 13.6.4.3 shall be retrofitted in accordance with Section 13.5.

13.7.4.3 Acceptance Criteria Acceptance criteria shall be applied in accordance with Section 13.3.

13.7.4.3.1 Life Safety Nonstructural Performance Level Fire suppression piping shall meet the prescriptive requirements of NFPA 13. If the analytical procedure is selected, fire suppression piping shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0. Fire suppression piping that runs between floors or across seismic joints shall be capable of accommodating relative displacements computed in accordance with Section 13.4.4.

13.7.4.3.2 Position Retention Nonstructural Performance Level Piping systems shall meet the requirements of Section 13.6.8 of ASCE 7.

If the prescriptive procedure is selected, fire suppression piping shall meet the prescriptive requirements of NFPA 13. If the analytical procedure is selected, fire suppression piping shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5. Fire suppression piping that runs between floors or across seismic joints shall be capable of accommodating relative displacements computed in accordance with Section 13.4.4.

13.7.4.3.3 Operational Nonstructural Performance Level Piping systems shall meet the requirements of Section 13.6.8 of ASCE 7.

Fire suppression piping shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5. Fire suppression piping that runs between floors or across seismic joints shall be capable of accommodating relative displacements computed in accordance with Section 13.4.5.

13.7.4.4 Evaluation Requirements The support, flexibility, protection at seismic movement joints, and freedom from impact from adjoining materials at the sprinkler heads shall be evaluated. Anchors shall be tested in accordance with Section 13.3.2.

13.7.5 Fluid Piping Other Than Fire Suppression

13.7.5.1 Definition and Scope Piping, other than pressure piping or fire suppression lines, that transfers fluids under pressure by gravity, or that is open to the atmosphere—including drainage and ventilation piping; hot, cold, and chilled water piping; and piping carrying liquids, as well as fuel gas lines—shall meet the requirements of this section.

Fluid piping other than fire suppression piping shall be classified into one of the following two categories:

1. **Category 1**: Hazardous materials and flammable liquids that would pose an immediate Life Safety danger if exposed because of inherent properties of the contained material; and
2. **Category 2**: Materials that, in case of line rupture, would cause property damage but pose no immediate Life Safety danger.

13.7.5.2 Component Behavior and Retrofit Methods Fluid piping other than fire suppression piping shall be considered acceleration sensitive. Piping that runs between floors or across seismic joints shall be considered both acceleration and deformation sensitive.

Fluid piping not conforming to the acceptance criteria of Section 13.7.5.3 shall be retrofitted in accordance with Section 13.5.

13.7.5.3 Acceptance Criteria Acceptance criteria shall be applied in accordance with Section 13.3. Forces for fluid piping shall be calculated in accordance with Section 13.4.4.

Category 2 pressure pipes conveying nonhazardous materials shall be evaluated for Position Retention Performance Level if the following conditions exist:

1. The structure being evaluated is in an area of moderate or high seismicity,
2. The piping is unbraced, and
3. The pipes are 2.0 in. (51 mm) in diameter or larger and are suspended more than 12 in. (305 mm) from the top of the pipe to the supporting structure at any support point.

13.7.5.3.1 Life Safety Nonstructural Performance Level

1. **Category 1 Piping Systems**: Fluid piping supports and bracing shall meet the prescriptive requirements of Section 13.4.3. Fluid piping shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5. Piping that runs between floors and across seismic joints shall be capable of accommodating relative displacements computed in accordance with Section 13.4.5. Piping systems shall meet the requirements of Section 13.6.8 of ASCE 7. Piping should maintain leak tightness in an earthquake commensurate with the Seismic Hazard Level being used for the evaluation or retrofit.
2. **Category 2 Piping Systems**: Fluid piping supports and bracing shall meet the prescriptive requirements of Section 13.4.3. Fluid piping shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0. Piping that runs between floors and across seismic joints shall be capable of accommodating relative displacements computed in accordance with Section 13.4.5.

13.7.5.3.2 Position Retention Nonstructural Performance Level Piping systems shall meet the requirements of Section 13.6.8 of ASCE 7.

Fluid piping supports and bracing shall meet the prescriptive requirements of Section 13.4.3 for essential facilities. If the analytical procedure is selected, fluid piping shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0. Piping that runs between floors and across seismic joints shall be capable of accommodating relative displacements computed in accordance with Section 13.4.5

13.7.5.3.3 Operational Nonstructural Performance Level Piping systems shall meet the requirements of Section 13.6.8 of ASCE 7.

Fluid piping supports and bracing shall meet the prescriptive requirements of Section 3 equivalent to Risk Category IV. Fluid piping shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5. Piping that runs between floors and across seismic joints shall be capable of accommodating relative displacements computed in accordance with Section 13.4.5.

Piping should maintain leak tightness in an earthquake commensurate with the Seismic Hazard Level being used for the evaluation or retrofit.

13.7.5.4 Evaluation Requirements The support, flexibility, and protection at seismic joints of fluid piping other than fire suppression piping shall be evaluated. Anchors shall be tested in accordance with Section 13.3.2.

Piping shall be insulated from detrimental heat effects.

13.7.6 Ductwork

13.7.6.1 Definition and Scope Ductwork shall include heating, ventilating, and air conditioning (HVAC) and exhaust ductwork systems. Seismic restraints shall not be required for ductwork that is not conveying hazardous materials and that meets either of the following conditions:

1. HVAC ducts are suspended from hangers 12 in. (305 mm) or less from the top of the duct to the supporting structure; hangers shall be installed without eccentricities that induce moments in the hangers; or
2. HVAC ducts have a cross-sectional area of less than 6 ft^2 (0.56 m^2).

13.7.6.2 Component Behavior and Retrofit Methods Ducts shall be considered acceleration sensitive. Ductwork that runs between floors or across seismic joints shall be considered both acceleration and deformation sensitive.

Ductwork not conforming to the acceptance criteria of Section 13.7.6.3 shall be retrofitted in accordance with Section 13.5.

13.7.6.3 Acceptance Criteria Acceptance criteria shall be applied in accordance with Section 13.3. Forces for ductwork shall be calculated in accordance with Section 13.4.4.

13.7.6.3.1 Life Safety Nonstructural Performance Level Ductwork shall meet the requirements of prescriptive standards in accordance with ANSI/SMACNA 001 (2008).

13.7.6.3.2 Position Retention Nonstructural Performance Level Ductwork shall meet the requirements of prescriptive standards in accordance with ANSI/SMACNA 001 and the requirements of ASCE 7, Section 13.6.7 for a component importance factor, I_p, of 1.5.

13.7.6.3.3 Operational Nonstructural Performance Level Ductwork shall meet the requirements of prescriptive standards in accordance with ANSI/SMACNA 001 and the requirements of ASCE 7, Section 13.6.7 for a component importance factor, I_p, of 1.5.

13.7.6.4 Evaluation Requirements Ductwork shall be evaluated visually to determine its length, connection type, and cross-sectional area. Anchors shall be tested in accordance with Section 13.3.2.

13.7.7 Electrical and Communications Equipment

13.7.7.1 Definition and Scope All electrical and communication equipment, including panel boards, battery racks, motor control centers, switch gear, and other fixed components and distribution systems that are in a building, are supported by a building, or are permanently connected to the mechanical or electrical system of a building that meet any of the following criteria, shall be evaluated for seismic anchorage in accordance with this section for the applicable performance level:

1. All equipment weighing more than 400 lb (1.8 kN); and
2. Discrete electrical communications equipment weighing more than 20 lb (0.09 kN) that is attached to floor, ceiling, wall, or other support with a center of mass more than 4 ft (1.2 m) above the floor.

For Hazards Reduced, Life Safety, and Position Retention Performance levels, all other equipment and distribution systems shall be positively fastened to the building or shall have an overturning resistance in accordance with Section 13.4.7.

For the Operational Performance Level, all equipment in regions of high and moderate seismicity shall be evaluated in accordance with this section.

13.7.7.2 Component Behavior and Retrofit Methods Electrical equipment shall be considered acceleration sensitive.

Electrical equipment not conforming to the acceptance criteria of Section 13.7.7.3 shall be retrofitted in accordance with Section 13.5.

13.7.7.3 Acceptance Criteria Acceptance criteria shall be applied in accordance with Section 13.3. Forces for electrical and communications equipment shall be calculated in accordance with Section 13.4.4.

13.7.7.3.1 Life Safety Nonstructural Performance Level Equipment in areas of high seismicity that is 6 ft (1.8 m) or more high, weighs more than 20 lb (0.09 kN), or forms part of an emergency power and/or communication system shall meet the Position Retention Nonstructural Performance Level. Electrical equipment shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0.

Equipment that forms part of the emergency power system shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5.

All other equipment and distribution systems shall be positively fastened to the building or shall have an overturning resistance in accordance with Section 13.4.7.

13.7.7.3.2 Position Retention Nonstructural Performance Level Equipment and distribution systems shall satisfy the requirements of Sections 13.2 and 13.6, of ASCE 7 based on the requirements for a component importance factor, I_p, of 1.0.

Electrical equipment shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0.

Equipment that forms part of the emergency power system shall meet the requirements for Operational Performance Level in Section 13.7.7.3.3.

13.7.7.3.3 Operational Nonstructural Performance Level Equipment shall satisfy the requirements of Sections 13.2 and 13.6, including special certification requirements of Section 13.2.2 of ASCE 7 based on the requirements for a component importance factor, I_p, of 1.5.

Electrical equipment shall meet the prescriptive requirements of Section 13.4.3. Electrical equipment shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5.

13.7.7.4 Evaluation Requirements Equipment shall be visually evaluated to determine its category and the existence of the hold-downs, supports, and braces. Anchors shall be tested in accordance with Section 13.3.2.

In addition, for the Operational Performance Level, equipment shall be analyzed or tested to demonstrate its ability to remain functional after an earthquake commensurate with the Seismic Hazard Level being considered.

13.7.8 Electrical and Communications Distribution Components

13.7.8.1 Definition and Scope All electrical and communications transmission lines, conduit, and cables, and their supports, shall comply with the requirements of this section.

13.7.8.2 Component Behavior and Retrofit Methods Electrical distribution equipment shall be considered acceleration sensitive. Wiring or conduit that runs between floors or across expansion or seismic joints shall be considered both acceleration and deformation sensitive.

Electrical and communications distribution components not conforming to the acceptance criteria of Section 13.7.8.3 shall be retrofitted in accordance with Section 13.5.

13.7.8.3 Acceptance Criteria Acceptance criteria shall be applied in accordance with Section 13.3. Forces for electrical and communications distribution components shall be calculated in accordance with Section 13.4.4.

13.7.8.3.1 Life Safety Nonstructural Performance Level Electrical and communication distribution components shall meet the requirements of prescriptive standards in accordance with Section 13.4.3.

13.7.8.3.2 Position Retention Nonstructural Performance Level Equipment shall satisfy the requirements of Sections 13.2 and 13.6 of ASCE 7 based on the requirements for a component importance factor, I_p, of 1.0.

Electrical and communications distribution components shall meet the requirements of prescriptive standards for essential facilities in accordance with Section 13.4.3 of this standard and Section 13.6.4, of ASCE 7. If the analytical procedure is selected, electrical and communications distribution components shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0. Electrical and communications distribution components that run between floors or across seismic joints shall be capable of accommodating relative displacements computed in accordance with Section 13.4.5.

13.7.8.3.3 Operational Nonstructural Performance Level Equipment shall satisfy the requirements of Sections 13.2 and 13.6, including special certification requirements of Section 13.2.2 of ASCE 7, based on the requirements for a component importance factor, I_p, of 1.5.

Electrical and communications distribution components shall meet the requirements of prescriptive standards for essential facilities in accordance with Section 13.4.3 of this standard and Section 13.6.4 of ASCE 7. If the analytical procedure is selected, electrical and communications distribution components shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5.

Electrical and communications distribution components that run between floors or across seismic joints shall be capable of accommodating relative displacements computed in accordance with Section 13.4.5.

13.7.8.4 Evaluation Requirements Components shall be visually evaluated to determine the existence of supports and bracing. Anchors shall be tested in accordance with Section 13.3.2.

In addition, for the Operational Performance Level, equipment shall be analyzed or tested to demonstrate its ability to remain functional after an earthquake commensurate with the Seismic Hazard Level being considered.

13.7.9 Light Fixtures

13.7.9.1 Definition and Scope Lighting fixtures shall be classified into one of the following categories:

Category 1: Lighting recessed in ceilings,
Category 2: Lighting surface mounted to ceilings or walls,
Category 3: Lighting supported within a suspended ceiling system (integrated ceiling), and
Category 4: Lighting suspended from ceilings or structure by a pendant or chain.

13.7.9.2 Component Behavior and Retrofit Methods Light fixtures not conforming to the acceptance criteria of Section 13.7.9.3 shall be retrofitted in accordance with Section 13.5.

13.7.9.3 Acceptance Criteria Acceptance criteria shall be applied in accordance with Section 13.3. Forces for light fixtures shall be calculated in accordance with Section 13.4.4.

13.7.9.3.1 Life Safety Nonstructural Performance Level

1. **Categories 1 and 2**: The connection to ceiling or wall shall be present with no visible signs of distress.
2. **Category 3**: System bracing and support shall meet prescriptive requirements in accordance with Section 13.4.3.
3. **Category 4**: Fixtures weighing more than 20 lb (0.09 kN) shall be adequately articulated, and the fixture shall be free to swing without impacting adjoining materials. In addition, the connection to the structure shall be capable of accommodating the movement without failure.

13.7.9.3.2 Position Retention Nonstructural Performance Level

1. **Categories 1 and 2**: The connection to ceiling or wall shall be present with no visible signs of distress.
2. **Category 3**: System bracing and support shall meet prescriptive requirements for standard occupancy facilities.
3. **Category 4**: Fixtures weighing more than 20 lb (0.09 kN) shall be articulated, and the fixture shall be free to swing without impacting adjoining materials. In addition, the connection to the structure shall be capable of accommodating the movement without failure.

13.7.9.3.3 Operational Nonstructural Performance Level

1. **Categories 1 and 2**: The connection to ceiling or wall shall be present with no visible signs of distress.
2. **Category 3**: System bracing and support shall meet prescriptive requirements for essential facilities.
3. **Category 4**: Fixtures weighing more than 20 lb (0.09 kN) shall be articulated, and the fixture shall be free to swing without impacting adjoining materials. In addition, the connection to the structure shall be capable of accommodating the movement without failure.

13.7.9.4 Evaluation Requirements Light fixture supports shall be visually evaluated to determine the connection type and adequacy. Anchors shall be tested in accordance with Section 13.3.2.

13.7.10 Rooftop Solar Photovoltaic Arrays

13.7.10.1 Definition and Scope Rooftop solar photovoltaic (PV) arrays shall be categorized as follows:

1. **Category 1**: PV arrays that are not attached to the roof structure, but the weight of the PV array is resisted by the roof framing. Resistance to seismic forces is provided by dead load and friction.
2. **Category 2**: PV arrays that are attached to the roof structure at multiple locations. Gravity loads are resisted by bearing on the roof framing, but the bearing locations may not occur at the attachment points. These systems may include additional weight (ballast).
3. **Category 3**: PV arrays that include structural framing that is positively attached to the roof structure such that vertical and lateral loads from the PV array are only transferred through the points of attachment.

13.7.10.2 Component Behavior and Retrofit Methods Photovoltaic arrays shall be classified as acceleration or displacement sensitive, depending on the category of the PV array. Category 1 PV arrays shall be classified as displacement sensitive. Category 2 and Category 3 PV arrays shall be classified as acceleration sensitive.

Photovoltaic arrays not conforming to the acceptance criteria of Section 13.7.10.3 shall be retrofitted in accordance with Section 13.5.

13.7.10.3 Acceptance Criteria Acceptance criteria shall be applied in accordance with Section 13.3, except as indicated.

13.7.10.3.1 Life Safety and Position Retention Nonstructural Performance Level

1. **Category 1:** The PV array shall accommodate without impact, instability, or loss of support the seismic displacement calculated in accordance with Section 13.6.13 of ASCE 7. Limitations on the roof slope, building height, and distance from the center of mass of the PV array from the roof prescribed by ASCE 7 shall apply.
2. **Categories 2 and 3:** The attachment of the PV array and its supporting framing shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0.

13.7.10.3.2 Operational Nonstructural Performance Level

1. **Category 1:** The PV array shall accommodate without impact, instability, or loss of support for two times the seismic displacement ($2\Delta_{\mathrm{mpv}}$) calculated in accordance with Section 13.6.13 of ASCE 7. Limitations on the roof slope, building height, and distance from the center of mass of the PV array from the roof prescribed by ASCE 7 shall apply.
2. **Categories 2 and 3:** The attachment of the PV array and its supporting framing shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5.

13.7.10.4 Evaluation Requirements Where the requirements of ASCE 7 for Category 1 PV arrays are used, the roof shall be evaluated to verify that the calculated seismic displacement can be accommodated without impact, instability, or loss of support for the PV array or damage to the electrical cables. For all other conditions, the PV array and associated framing shall be analyzed for the acceleration-induced forces, and the presence of adequate supports, anchors, and bracing shall be confirmed.

13.7.11 Elevators

13.7.11.1 Definition and Scope Elevators shall include cabs, shafts, and all equipment and equipment rooms associated with elevator operation, such as hoists, counterweights, cables, and controllers.

13.7.11.2 Component Behavior and Retrofit Methods Components of elevators shall be considered acceleration sensitive. Shafts and hoistway rails, which rise through multiple floors, shall be considered both acceleration and deformation sensitive.

Elevator components not conforming to the acceptance criteria of Section 13.7.11.3 shall be retrofitted in accordance with Section 13.5.

13.7.11.3 Acceptance Criteria Acceptance criteria shall be applied in accordance with Section 13.3.

13.7.11.3.1 Life Safety Nonstructural Performance Level If the prescriptive procedure is selected, elevator components shall meet the prescriptive requirements of Section 13.4.3. If the analytical procedure is selected based on Table 13-1, elevator components shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0.

13.7.11.3.2 Position Retention Nonstructural Performance Level Elevators shall comply with the requirements of Section 13.6.10 of ASCE 7.

If the prescriptive procedure is selected, elevator components shall meet the prescriptive requirements of Section 13.4.3. If the analytical procedure is selected, elevator components shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0.

13.7.11.3.3 Operational Nonstructural Performance Level Elevators shall comply with the requirements of Section 13.6.10 of ASCE 7.

If the prescriptive procedure is selected, elevator components shall meet the prescriptive requirements of Section 13.4.3. If the analytical procedure is selected, elevator components shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5.

13.7.11.4 Evaluation Requirements The construction of elevator shafts shall be considered in the evaluation.

13.7.12 Conveyors

13.7.12.1 Definition and Scope Conveyors shall include material conveyors, including all machinery and controllers necessary to operation.

13.7.12.2 Component Behavior and Retrofit Methods Conveyors shall be considered both acceleration and deformation sensitive.

Conveyors not conforming to the acceptance criteria of Section 13.7.12.3 shall be retrofitted in accordance with Section 13.5.

13.7.12.3 Acceptance Criteria Acceptance criteria shall be applied in accordance with Section 13.3.

13.7.12.3.1 Life Safety Nonstructural Performance Level Conveyors need not be retrofitted for the Life Safety Nonstructural Performance Level.

13.7.12.3.2 Position Retention Nonstructural Performance Level If the analytical procedure is selected, conveyors shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0. If the prescriptive procedure is selected, conveyors shall meet prescriptive standards in accordance with Section 13.4.3.

13.7.12.3.3 Operational Nonstructural Performance Level If the analytical procedure is selected, conveyors shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5. If the prescriptive procedure is selected, conveyors shall meet prescriptive standards in accordance with Section 13.4.3.

13.7.12.4 Evaluation Requirements The stability of machinery shall be considered in the evaluation.

For the Operational Performance Level, conveyors shall be analyzed or tested to demonstrate their ability to resume function after an earthquake commensurate with the Seismic Hazard Level being considered.

13.8 FURNISHINGS AND CONTENTS: DEFINITION, BEHAVIOR, AND ACCEPTANCE CRITERIA

13.8.1 Steel Storage Racks

13.8.1.1 Definition and Scope Steel storage racks shall include systems for holding materials either permanently or temporarily using pallets supported on the rack framing and other multilevel storage systems. Floor-supported steel storage racks constructed with cold-formed or hot-rolled steel structural members with one or more levels shall be evaluated in accordance with this section. Steel storage racks that are less than 8 ft (2.4 m) tall shall be considered as contents and evaluated in accordance with Section 13.8.2.

13.8.1.2 Component Behavior and Retrofit Methods Steel storage racks shall be considered acceleration sensitive. Seismic loads shall be distributed to each level supporting contents using Equation (13-17):

$$F_i = \frac{w_i h_i}{\sum_{i=1}^{n} w_i h_i} F_p \tag{13-17}$$

where

F_p = Seismic horizontal force calculated in accordance with section 13.4.4,
W_i = Weight of the supported contents at level i,
h_i = Height of the supported contents above the supporting floor, and
F_i = Seismic force to be applied at level i of the rack system.

Steel storage racks not conforming to the acceptance criteria of Section 13.8.1.3 shall be retrofitted in accordance with Section 13.5.

Exception: Retrofit shall not be required for steel storage racks in unoccupied spaces being evaluated for Life Safety Performance Level.

13.8.1.3 Acceptance Criteria Acceptance criteria shall be applied in accordance with Section 13.3.

13.8.1.3.1 Life Safety Nonstructural Performance Level Storage racks shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0.

13.8.1.3.2 Position Retention Nonstructural Performance Level Storage racks shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0 and shall conform to the requirements of Section 15.5.3 in ASCE 7.

13.8.1.3.3 Operational Nonstructural Performance Level Storage racks shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5 and shall conform to the requirements of Section 15.5.3 in ASCE 7.

13.8.1.4 Evaluation Requirements Buckling or racking failure of storage rack components, connection to support structures, and type and stability of supporting structure shall be considered in the evaluation. Anchors shall be tested in accordance with Section 13.3.2.

13.8.2 Contents

13.8.2.1 Definition and Scope Contents, such as bookcases constructed of wood or metal, more than 4 ft (1.2 m) high and with a height-to-width ratio greater than 2, shall meet the requirements of this section.

13.8.2.2 Component Behavior and Retrofit Methods Contents shall be considered acceleration sensitive.

Contents not conforming to the acceptance criteria of Section 13.8.2.3 shall be retrofitted in accordance with Section 13.5. Contents 6 ft (1.8 m) or less in height are permitted to be retrofitted using the prescriptive procedure in Section 13.4.3.

13.8.2.3 Acceptance Criteria Acceptance criteria shall be applied in accordance with Section 13.3.

13.8.2.3.1 Life Safety Nonstructural Performance Level Contents shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0.

13.8.2.3.2 Position Retention Nonstructural Performance Level Contents shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.0.

13.8.2.3.3 Operational Nonstructural Performance Level Contents shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5.

13.8.2.4 Evaluation Requirements The loading, type, and condition of bookcases; their connection to support structures; and type and stability of supporting structure shall be considered in the evaluation.

13.8.3 Hazardous Material Storage

13.8.3.1 Definition and Scope Hazardous material storage shall include permanently installed containers—freestanding, on supports, or stored on countertops or shelves—that hold materials defined to be hazardous by the National Institute for Occupational Safety and Health, including the following types:

1. Propane gas tanks,
2. Compressed gas vessels, and
3. Dry or liquid chemical storage containers.

Large non-building structures, such as large tanks found in heavy industry or power plants, floating-roof oil storage tanks, and large [more than 10 ft (3.1 m) long] propane tanks at propane manufacturing or distribution plants need not meet the requirements of this section.

13.8.3.2 Component Behavior and Retrofit Methods Hazardous material storage shall be considered acceleration sensitive.

Hazardous material storage not conforming to the acceptance criteria of Section 13.8.3.3 shall be retrofitted in accordance with Section 13.5.

13.8.3.3 Acceptance Criteria Acceptance criteria shall be applied in accordance with Section 13.3.

13.8.3.3.1 Life Safety and Position Retention Nonstructural Performance Level Hazardous material storage shall meet the requirements for Operational Nonstructural Performance Level of Section 13.8.3.3.2.

13.8.3.3.2 Operational Nonstructural Performance Level Hazardous material storage shall meet prescriptive requirements for essential facilities in accordance with Section 13.4.3. If the analytical procedure is selected, hazardous material storage shall be capable of resisting seismic forces computed in accordance with Section 13.4.4 using a component importance factor, I_p, of 1.5.

13.8.3.4 Evaluation Requirements The location and types of hazardous materials, container materials, manner of bracing, internal seismic force resistance, and the effect of hazardous material spills shall be considered in the evaluation. Anchors shall be tested in accordance with Section 13.4.6.

In addition, for the Operational Performance Level, hazardous material storage shall be analyzed or tested to demonstrate its ability to contain the hazardous material after an earthquake commensurate with the Seismic Hazard Level being considered.

13.8.4 Computer and Communication Racks

13.8.4.1 Definition and Scope Computer and communication racks shall include freestanding rack systems more than 4 ft (1.2 m) high designed to support computer and other electronic equipment. Equipment stored on computer and communication racks need not meet the requirements of this section.

13.8.4.2 Component Behavior and Retrofit Methods Computer and communication racks shall be considered acceleration sensitive.

Computer communication racks not conforming to the acceptance criteria of Section 13.8.4.3 shall be retrofitted in accordance with Section 13.5.

13.8.4.3 Acceptance Criteria Acceptance criteria shall be applied in accordance with Section 13.3.

13.8.4.3.1 Life Safety Nonstructural Performance Level Computer and communication racks need not be retrofitted for the Life Safety Nonstructural Performance Level.

13.8.4.3.2 Position Retention Nonstructural Performance Level If the prescriptive procedure is selected based on Table 13-1, computer and communication racks shall meet the prescriptive requirements of Section 13.4.2. If the analytical procedure is selected based on Table 13-1, computer and communication racks shall be capable of resisting seismic forces computed in accordance with Section 13.4.3 using a component importance factor, I_p, of 1.0.

13.8.4.3.3 Operational Nonstructural Performance Level If the prescriptive procedure is selected based on Table 13-1, computer and communication racks shall meet the prescriptive requirements of Section 13.4.2. If the analytical procedure is selected based on Table 13-1, computer and communication racks shall be capable of resisting seismic forces computed in accordance with Section 13.4.3 using a component importance factor, I_p, of 1.5.

13.8.4.4 Evaluation Requirements Buckling or racking failure of rack components, their connection to support structures, and type and stability of the supporting structure shall be considered in the evaluation. The effect of rack failure on equipment shall also be considered. Anchors shall be tested in accordance with Section 13.4.6.

In addition, for the Operational Performance Level, computer and communication racks shall be analyzed or tested to demonstrate their ability to preserve the functionality of the computer and communication equipment stored in the racks after an earthquake commensurate with the Seismic Hazard Level being considered.

CHAPTER 14

SEISMIC ISOLATION

14.1 SCOPE

This chapter sets forth requirements for the systematic evaluation and retrofit of buildings using seismic isolation systems. In addition to the seismic isolators, the seismic isolation system shall also include any wind-restraint and tie-down systems, displacement-restraint devices, and supplemental energy dissipation devices that cross the isolation interface.

Any of the Performance Objectives of Section 2.4 are permitted for seismic isolation evaluation and retrofit. When the largest hazard level considered is less than the BSE-2E, an evaluation at the BSE-2E hazard level shall also be conducted for select provisions, as indicated in this chapter. The requirements of this chapter shall be satisfied independently using upper-bound and lower-bound properties for each hazard level considered.

Seismic isolation systems and associated structural and nonstructural components shall be evaluated and designed in compliance with the requirements of Section 14.2. Properties of seismic isolation systems shall be based on Section 14.3. Seismic isolation systems shall be modeled and analyzed in accordance with Sections 14.4 and 14.5, respectively. Seismic isolation systems shall be tested in accordance with Section 14.6. Design review shall be conducted in accordance with Section 14.7.

14.2 GENERAL REQUIREMENTS

14.2.1 General For seismically isolated buildings, the coefficients C_0, C_1, C_2, and J defined in Chapter 7 shall be taken as 1.0. Components and elements in buildings with seismic isolation systems shall also comply with the requirements of Chapters 1 through 13 as modified by the requirements of this chapter.

14.2.2 Seismic Hazard The spectral response acceleration parameters at short periods, S_{XS}, and at a 1 s period, S_{X1}, and response spectra shall be determined in accordance with Section 2.3.

14.2.2.1 Ground Motion Acceleration Histories Where the nonlinear dynamic procedure in accordance with Section 14.5.5 is used, the provisions of Section 2.3.4 shall apply except that in lieu of the requirements of Item 3, Section 2.3.4, the period range shall be determined in accordance with the following:

Period Range for Scaling or Matching: A period range shall be determined for each hazard level considered, corresponding to the vibration periods that significantly contribute to the building's lateral dynamic response. The upper bound of this period range shall be greater than or equal to $1.25T_X$, as determined using lower-bound isolation system properties for the hazard level considered and taken as the largest in each principal direction. T_X is the effective period of the seismically isolated structure calculated in accordance with Section 14.5.2.2. The lower bound of this period range shall be established such that the period range includes at least the number of modes necessary to achieve 90% mass participation in each principal horizontal direction using upper-bound isolation system properties for the hazard level considered and shall not exceed T_{fb}. Where vertical response is considered in the analysis, the lower-bound period of the period range used for modification of vertical components of ground motion need not be taken as less than the larger of 0.1 s or the lowest period at which significant vertical mass participation occurs.

14.2.3 Isolation System

14.2.3.1 Environmental Conditions In addition to the requirements for vertical and lateral loads induced by wind and earthquake, the design of the isolation system shall account for other environmental conditions, including aging effects, creep, fatigue, operating temperature, and exposure to moisture or damaging substances.

14.2.3.2 Wind Displacement Displacements across the isolation interface under wind loads determined in accordance with ASCE 7 shall not be greater than 1.5% of the difference in height between floors of the building above the isolation interface.

14.2.3.3 Fire Resistance Fire resistance for the isolation system shall provide at least the same degree of protection as the fire resistance required by the governing regulations, building code, or policies for the columns, walls, or other such gravity-bearing elements in the same region of the building.

14.2.3.4 Lateral Restoring Force The isolation system shall be configured, for both upper-bound and lower-bound isolation system properties, to produce a restoring force such that the lateral force at the displacement D_X is at least 0.025 W greater than the lateral force at 50% of the displacement D_X. D_X shall be computed for the largest hazard level considered.

14.2.3.5 Displacement Restraint The isolation system shall not be configured to include a displacement restraint that limits lateral displacement to less than the total displacement, D_{TX}, computed by Equation (14-10) for the largest hazard level considered except not less than the BSE-2E hazard level unless the seismically isolated building is designed in accordance with all the following criteria:

1. Response shall be calculated in accordance with the nonlinear dynamic procedure of Section 14.5.5, explicitly considering the nonlinear characteristics of the isolation system, displacement restraint, and the structure above the isolation system;
2. The ultimate capacity of the isolation system and structural elements below the isolation system shall exceed the

strength and displacement demands of the response under the largest hazard level considered except not less than the BSE-2E hazard level;
3. The structure above the isolation system shall be checked for stability and ductility demand of the response under the largest hazard level considered; and
4. The displacement restraint shall not become effective at a displacement less than $0.6D_{TX}$. D_{TX} shall be calculated for the largest hazard level considered except not less than the BSE-2E hazard level.

14.2.3.6 Vertical Load Stability Each element of the isolation system shall be designed to remain stable under the design vertical load where subjected to a horizontal displacement equal to D_{TX}. D_{TX} shall be taken at the largest hazard level considered except not less than the BSE-2E hazard level. The design vertical load shall be computed using Load Combination 2 in Section 14.2.6.2 for the maximum vertical load and Load Combination 3 in Section 14.2.6.2 for the minimum vertical load.

14.2.3.7 Overturning The factor of safety against global structural overturning at the isolation interface shall not be less than 1.0 for Load Combination 3 of Section 14.2.6.2. Seismic forces for overturning calculations shall be based on the largest hazard level considered except not less than the BSE-2E hazard level. The vertical restoring force shall be based on the building's weight, W, above the isolation interface.

Local uplift of individual elements shall not be permitted unless the resulting displacements do not cause overstress or instability of the isolation system devices or other structural elements. A tie-down system to limit local uplift of individual components and elements shall be permitted, provided that the seismically isolated building is designed in accordance with all the following criteria:

1. Response shall be calculated in accordance with the nonlinear dynamic procedure of Section 14.5.5, explicitly considering the nonlinear characteristics of the isolation system and the structure above the isolation system;
2. The ultimate capacity of the tie-down system shall not be less than the force and displacement demands of the largest hazard level considered except not less than the BSE-2E hazard level; and
3. The isolation system shall be designed and shown by testing to be stable per Section 14.6.3.4 for demands corresponding to the largest hazard level considered except not less than the BSE-2E hazard level. Demands shall include any additional vertical load resulting from the tie-down system.

14.2.3.8 Inspection and Replacement All the following items shall be addressed as part of the long-term inspection and replacement program:

1. Access for inspection and replacement of all components of the isolation system shall be provided.
2. The design professional responsible for the structure shall complete a final series of observations of structure separation areas and components that cross the isolation interface before the issuance of the certificate of occupancy for the seismically isolated building. Such observations shall verify that conditions allow free and unhindered displacement of the building up to D_{TX}, and that components that cross the isolation interface have been constructed to accommodate D_{TX}. D_{TX} shall be calculated for the largest hazard level considered except not less than the BSE-2E hazard level.
3. Seismically isolated buildings shall have a monitoring, inspection, and maintenance plan for the isolation system established by the design professional responsible for the structure including minimum requirements provided by the isolation system device manufacturer.
4. Remodeling, repair, or retrofitting at the isolation system interface, including that of components that cross the isolation interface, shall be performed under the direction of a design professional.

14.2.4 Structural System

14.2.4.1 Horizontal Distribution of Force A horizontal diaphragm or other structural elements shall provide continuity above the isolation interface and shall have adequate strength and ductility to transmit forces from one part of the structure to another.

14.2.4.2 Minimum Separations Minimum separations between the isolated building and surrounding retaining walls or other fixed obstructions shall be not less than D_{TX} computed at the largest hazard level considered except not less than the BSE-2E hazard level. Minimum separations between the isolated building and adjacent structures shall be not less than that required in Section 7.2.15 for the largest hazard level considered except not less than the BSE-2E hazard level.

14.2.5 Elements of Structures and Nonstructural Components

14.2.5.1 General Parts or portions of an isolated building, permanent nonstructural components and the attachments to them, and the attachments for permanent equipment supported by the building shall be evaluated and designed to resist seismic forces and displacements as prescribed by this section and the applicable requirements of Chapter 13.

14.2.5.2 Components at or above the Isolation Interface Elements of seismically isolated buildings and nonstructural components, or portions thereof, that are at or above the isolation interface shall be evaluated and designed to resist a total lateral seismic force equal to the maximum dynamic response of the element or component under consideration determined using the nonlinear dynamic procedure of Section 14.5.5.

EXCEPTION: Elements of seismically isolated buildings and nonstructural components, or portions thereof, shall be permitted to be evaluated and designed to resist seismic forces and displacements as prescribed in Chapter 13.

14.2.5.3 Components Crossing the Isolation Interface Elements of seismically isolated buildings and nonstructural components, or portions thereof, that cross the isolation interface shall be evaluated and designed to withstand D_{TX} and the vertical displacement of the isolation system at D_{TX} for the largest hazard level considered except not less than the BSE-2E hazard level. These components shall also accommodate, on a long-term basis, any permanent residual displacement.

14.2.5.4 Components below the Isolation Interface Elements of seismically isolated buildings and nonstructural components, or portions thereof, that are below the isolation interface shall be evaluated, designed, and constructed in accordance with the requirements of Chapter 13.

14.2.6 Seismic Load Effects and Load Combinations

14.2.6.1 General All elements of the isolated building shall be evaluated and designed using the load combinations of Section 7.2.3. As specifically referenced elsewhere in this chapter, the additional load combinations of Section 14.2.6.2 shall apply.

14.2.6.2 Isolation System Device Vertical Load Combinations The average, maximum, and minimum vertical load on each isolation system device shall be computed from application of horizontal seismic forces at each hazard level considered and the following vertical load combinations:

1. Average Vertical Load: $Q_D + 0.5Q_L$,
2. Maximum Vertical Load: $1.2Q_D + 0.5Q_L + |Q_E|$, and
3. Minimum Vertical Load: $0.9Q_D - |Q_E|$.

The value of Q_E shall be calculated for the hazard level considered in accordance with the analysis procedure selected in Section 14.5.1.

14.3 SEISMIC ISOLATION SYSTEM DEVICE PROPERTIES

14.3.1 Isolation System Device Types All isolation system devices shall be categorized and grouped in terms of common type and size of isolator and common type and size of supplemental energy dissipation device, if such devices are also components of the isolation system. Elastomeric isolators consist of layers of rubber that are integrally bonded during vulcanization. Sliding isolators consist of one or more interfaces that slide relative to each other on a flat or curved surface. Supplemental energy dissipation devices shall be as defined in Chapter 15.

14.3.2 Nominal Design Properties of Isolation System Devices Nominal design properties of isolation system devices of a common type and size shall be the average properties over the three cycles of prototype testing, specified by Item 3, Section 14.6.3.3.

14.3.3 Bounding Properties of Isolation System Devices

14.3.3.1 Specification Tolerance on Design Properties Specification property modification factors ($\lambda_{\text{spec max}}$ and $\lambda_{\text{spec min}}$) shall be established to account for variation in manufacturing. Specification property modification factors shall be established for individual isolation system devices and for the average across all isolation system devices of a common type and size.

14.3.3.2 Testing Variations on Design Properties Testing property modification factors ($\lambda_{\text{test max}}$ and $\lambda_{\text{test min}}$) shall be established to account for the variation in isolation system device properties caused by required variation in vertical load, rate of loading or velocity effects, effects of heating during cyclic motion, history of loading, scragging (temporary degradation of properties with repeated cycling), and other potential sources of variation as measured by qualification testing in accordance with Section 14.6.2.

Testing property modification factors shall be developed for each isolation system device of a common type and size and shall envelop the prototype test hysteretic response for the range of demands from $\pm 0.67D_X$ up to and including $\pm D_X$ for each hazard level considered in accordance with Section 14.6.6.

EXCEPTION: If the tested values of isolation system device effective stiffness and effective damping for Load Combination 1 of Section 14.2.6.2 differ by less than 15% from those based on the average of tested values for the three vertical load combinations of Section 14.2.6.2, testing property modification factors shall be permitted to be calculated only for Load Combination 1 of Section 14.2.6.2.

14.3.3.3 Aging and Environmental Effects on Design Properties Aging and environmental property modification factors ($\lambda_{\text{ae max}}$ and $\lambda_{\text{ae min}}$) shall be established to account for aging and environmental effects including creep, fatigue, contamination, operating temperature and duration of exposure to that temperature, wear over the life of the building, and exposure to damaging substances.

Aging and environmental property modification factors shall be permitted to be developed from data that do not satisfy the similarity requirements of Section 14.6.3.9.

14.3.4 Property Modification Factors Maximum and minimum property modification factors ($\lambda_{\max}$ and $\lambda_{\min}$) shall be used to account for variation of properties for each isolation system device of a common type and size for the effects of Sections 14.3.3.1 through 14.3.3.3.

For each isolation system device of a common type and size, the maximum property modification factor, $\lambda_{\max}$, and the minimum property modification factor, $\lambda_{\min}$ shall be calculated in accordance with Equations (14-1) and (14-2), respectively:

$$\lambda_{\max} = (1 + 0.75 * (\lambda_{ae\,\max} - 1)) * \lambda_{\text{test max}} * \lambda_{\text{spec max}} \geq \text{Limit of Table 14-1} \quad (14\text{-}1)$$

$$\lambda_{\min} = (1 - 0.75 * (1 - \lambda_{ae\,\min})) * \lambda_{\text{test min}} * \lambda_{\text{spec min}} \leq \text{Limit of Table 14-1} \quad (14\text{-}2)$$

EXCEPTION: The limits of Table 14-1 shall be permitted to be neglected if the property modification factors are developed based on either of the following:

1. Dynamic prototype testing conducted on full-scale specimens in accordance with Section 14.6.3.5, or
2. Manufacturer-specific qualification test data in accordance with Section 14.6.2 as approved by the design professional responsible for the structure.

14.3.5 Upper- and Lower-Bound Properties Upper-bound and lower-bound properties for each isolation system device of a common type and size shall be calculated for each property of interest in accordance with Equations (14-3) and (14-4), respectively:

$$\text{Upper-bound design property} = \text{Nominal design property} \times \lambda_{\max} \quad (14\text{-}3)$$

$$\text{Lower-bound design property} = \text{Nominal design property} \times \lambda_{\min} \quad (14\text{-}4)$$

Upper-bound strength, stiffness, and energy dissipation shall be considered together as the upper-bound case, and lower-bound strength, stiffness, and energy dissipation shall be considered together as the lower-bound case. Upper-bound and lower-bound properties shall be used to establish loads and displacements corresponding to each hazard level considered.

Table 14-1. Limits of Property Modification Factors.

Variable	Sliding Isolators	Elastomeric Isolators
Equation (14-1): $\lambda_{\max}$	2.1	1.8
Equation (14-2): $\lambda_{\min}$	0.6	0.8

14.4 MODELING

14.4.1 Isolation System Device Modeling

14.4.1.1 Upper-Bound and Lower-Bound Force–Deflection Behavior of Isolation System Devices A mathematical model of upper-bound force–deflection behavior of each type and size of isolation system device shall be developed. Upper-bound force–deflection behavior of isolation system devices that are essentially hysteretic devices shall be modeled using the maximum values of isolation system device properties calculated in accordance with Section 14.3.5. Upper-bound force–deflection behavior of isolation system devices that are essentially viscous devices shall be modeled in accordance with the requirements of Chapter 15.

A mathematical model of lower-bound force–deflection behavior of each type and size of isolation system device shall be developed. Lower-bound force–deflection behavior of isolation system devices that are essentially hysteretic devices shall be modeled using the minimum values of isolation system device properties calculated in accordance with Section 14.3.5. Lower-bound force–deflection behavior of isolation system devices that are essentially viscous devices shall be modeled in accordance with the requirements of Chapter 15.

14.4.1.2 Isolation System Properties The effective stiffness, k_X, of the isolation system at displacement D_X shall be computed using both upper-bound and lower-bound force–deflection behavior of individual isolation system devices in accordance with Equation (14-5):

$$k_X = \frac{|\Sigma F_X^+| + |\Sigma F_X^-|}{2D_X} \tag{14-5}$$

The effective damping, β_X, of the isolation system at displacement D_X shall be computed using both upper-bound and lower-bound force–deflection behavior of individual isolation system devices in accordance with Equation (14-6):

$$\beta_X = \frac{\sum E_X}{2\pi k_X D_X^2} \tag{14-6}$$

where

ΣE_X = Total energy dissipated, in kips-in. (kN-mm), in the isolation system during a full cycle of response at the displacement D_X;

$|\Sigma F_X^+|$ = Absolute value of the sum, over all isolation system devices, of the force, in kips (kN), at a positive displacement equal to D_X; and

$|\Sigma F_X^-|$ = Absolute value of the sum, over all isolation system devices, of the force, in kips (kN), at a negative displacement equal to D_X.

The analysis of the isolation system and structure shall be performed separately for upper-bound and lower-bound properties, and the governing case for each response parameter of interest shall be used for evaluation and design.

14.4.1.3 Isolation System Models for Linear Procedures For the linear static procedure of Section 14.5.2 and the linear dynamic procedure of Section 14.5.3, the maximum force, F, of the isolation system shall be calculated as the product of effective stiffness, k_X, and displacement, D_X, in accordance with Equation (14-7):

$$F = k_X D_X \tag{14-7}$$

The effective stiffness, k_X, of the isolation system shall be calculated in accordance with Equation (14-5). The effective damping, β_X, of the isolation system shall be calculated in accordance with Equation (14-6). Effective stiffness and effective damping shall be calculated for each hazard level considered.

14.4.1.4 Isolation System Device Models for Nonlinear Procedures For the nonlinear static procedure of Section 14.5.4 and the nonlinear dynamic procedure of Section 14.5.5, the nonlinear force–deflection properties of isolation system devices shall be explicitly modeled using the mathematical models developed in accordance with Section 14.4.1.1. The structural response shall be determined independently using both upper-bound and lower-bound models for each hazard level considered.

Additional viscous damping shall not be included in the isolation system device models unless it is supported by rate-dependent tests of isolation system devices. Inherent damping in the structural modes shall be separately considered.

14.4.2 Isolation System and Superstructure Modeling

14.4.2.1 General Mathematical models of the isolated building, including the isolation system, the superstructure, other structural components and elements, and connections between the isolation system and the structure, shall conform to the requirements of Sections 14.4.1.3 and 14.4.1.4 and this section. Three-dimensional models shall be used for the nonlinear static procedure of Section 14.5.4 and the nonlinear dynamic procedure of Section 14.5.5.

14.4.2.2 Isolation System Model The isolation system shall be modeled with sufficient detail to capture all the following:

1. Spatial distribution of isolation system devices;
2. Translation, in both horizontal directions, and torsion of the structure above the isolation interface considering the most disadvantageous location of eccentric mass;
3. Overturning and uplift forces on individual isolation system devices;
4. Effects of vertical load, bilateral load, and/or the rate of loading if the force–deflection properties of the isolation system are dependent on one or more of these attributes; and
5. Effects of the wind-restraint, displacement-restraint, and tie-down systems, if such systems are used.

The deformation characteristics of the isolation system shall be based on properly substantiated prototype tests performed in accordance with Section 14.6.3 and shall incorporate property modification factors in accordance with Section 14.3.5.

14.4.2.3 Superstructure Model Modeling of the structure above the isolation system shall include all primary components. Force-controlled actions shall be modeled using linear elements. Deformation-controlled actions shall be modeled using nonlinear elements.

EXCEPTION: The structure above the base level shall be permitted to be modeled, evaluated, and designed as linear where Equation (7-39) is satisfied for all deformation-controlled actions above the base level for the hazard level considered using an *m*-factor equal to the lesser of the following values:

1. Those specified by Chapters 8 through 12 at the selected structural performance level,
2. 1.0 for the Immediate Occupancy Performance Level,
3. 1.5 for the Life Safety Performance Level, and
4. 2.0 for the Collapse Prevention Performance Level.

14.5 ANALYSIS PROCEDURES

14.5.1 Selection of Analysis Procedure An analysis procedure shall be selected subject to the limitations set forth in Sections 14.5.1.1 through 14.5.1.4.

14.5.1.1 Linear Static Procedure The linear static procedure shall be permitted for the evaluation and design of seismically isolated buildings provided that all the following criteria are satisfied using nominal isolation properties for the hazard level considered:

1. The limitations of Section 7.3.1.1 for the portion of the structure above the isolation interface.
2. The limitations of Section 7.3.1.2 requirements 2 through 5 for the portion of the structure above the isolation interface.
3. The structure is located on Site Class A, B, BC, C, CD, or D.
4. The effective period of the isolated building, T_X, at D_X for the hazard level considered is less than or equal to 5.0 s.
5. The structure above the isolation interface is less than or equal to four stories or 65 ft (19.8 m) in height measured from the base level.

   EXCEPTION: These story and height limits shall be permitted to be exceeded if there is no tension or uplift on the isolation system devices.
6. The effective damping of the isolation system, β_X, at D_X for the hazard level considered is less than or equal to 30%.
7. The effective period of the isolated building, T_X, for the hazard level considered is greater than three times the elastic, fixed-base period of the structure above the isolation system, T_{fb}, determined using modal analysis.
8. The isolation system satisfies all the following criteria:
   - 8.1 The effective stiffness of the isolation system, k_X, at D_X for the hazard level considered is greater than one-third of the effective stiffness at 20% of D_X.
   - 8.2 The isolation system is capable of producing a restoring force in accordance with Section 14.2.3.4.
   - 8.3 The isolation system does not limit the isolation system displacement to less than the total displacement, D_{TX}, for the hazard level considered.
9. The structure above the isolation system is permitted to be modeled, evaluated, and designed as linear in accordance with the exception of Section 14.4.2.3.

14.5.1.2 Linear Dynamic Procedure The linear dynamic procedure shall be permitted for the evaluation and design of seismically isolated buildings provided that the criteria of Items 1, 3, 4, 5, 6, 8, and 9 of Section 14.5.1.1 are satisfied using nominal isolation properties for the hazard level considered.

14.5.1.3 Nonlinear Static Procedure The nonlinear static procedure shall be permitted for the evaluation and design of seismically isolated buildings provided that the criteria of Items 3 through 8 of Section 14.5.1.1 are satisfied using nominal isolation properties for the hazard level considered.

14.5.1.4 Nonlinear Dynamic Procedure The nonlinear dynamic procedure shall be required when the building does not conform to the requirements in Sections 14.5.1.1, 14.5.1.2, or 14.5.1.3.

14.5.1.5 Design Forces and Deformations Components and elements shall be separately checked for the demands from analyses performed with upper-bound and lower-bound isolation system device properties, and the governing case for each response parameter of interest shall be used for evaluation and design for each hazard level considered.

When the linear static procedure of Section 14.5.2 or the linear dynamic procedure of Section 14.5.3 are used, components and elements of the building shall be evaluated and designed for the forces and displacements in accordance with Section 7.5.2.2 and the requirements of the exception of Section 14.4.2.3.

When the nonlinear static procedure of Section 14.5.4 or the nonlinear dynamic procedure of Section 14.5.5 are used, components and elements of the building shall be evaluated and designed for the forces and deformations in accordance with Section 7.5.3.2, except that Section 7.5.3.2.1 shall not apply. When the nonlinear dynamic procedure of Section 14.5.5 is used, component actions shall be determined in accordance with Section 7.4.4.3, and the displacement of the isolation system shall be calculated as the vector sum of the two orthogonal displacements at each time step.

EXCEPTION: When the requirements of the exception of Section 14.4.2.3 are satisfied for the nonlinear static procedure of Section 14.5.4 or the nonlinear dynamic procedure of Section 14.5.5, the structure above the isolation system shall be permitted to be modeled, evaluated, and designed as linear.

14.5.2 Linear Static Procedure

14.5.2.1 General Where the linear static procedure is used to evaluate and design seismically isolated buildings, subject to the limitations of Section 14.5.1.1, the requirements of this section shall apply. The analysis of the isolation system and structure shall be performed separately for upper-bound and lower-bound properties, and the governing case for each response parameter of interest shall be used for evaluation and design for each hazard level considered.

14.5.2.2 Minimum Lateral Displacements

14.5.2.2.1 Isolation System Displacement. The displacement D_X at the center of rigidity corresponding to a specified hazard level shall be calculated in accordance with Equation (14-8):

$$D_X = \frac{gS_a(T_X)T_X^2}{4\pi^2 B_X} \tag{14-8}$$

where

g = Acceleration caused by gravity, in inches per seconds squared (in./s^2) or millimeters per seconds squared (mm/s^2) if the units of the displacement D_X are in inches (millimeters);

$S_a(T_X)$ = 5% damped spectral acceleration parameter, in g, determined in accordance with Section 14.2.2 at the effective period, T_X;

T_X = Effective period of the seismically isolated building, in seconds, at the displacement D_X in the direction under consideration calculated in accordance with Equation (14-9); and

B_X = Numerical coefficient equal to the value of B_1 per Section 2.3.2 for the effective damping of the isolation system β_X, at the displacement D_X determined in accordance with Equation (14-6).

14.5.2.2.2 Effective Period at the Displacement D_X. The effective period of the isolated building, T_X, at the displacement D_X shall be determined in accordance with Equation (14-9):

$$T_X = 2\pi\sqrt{\frac{W}{k_X g}} \tag{14-9}$$

where

W = Effective seismic weight, in kips (kN), of the building above the isolation interface;

k_X = Effective stiffness, in kips/in. (kN/mm), of the isolation system at the displacement D_X calculated in accordance with Equation (14-5); and

g = Acceleration of gravity, in inches per seconds squared (in./s^2) or millimeters per seconds squared (mm/s^2), if the units of k_X are in kips per inch (kips/in.) or kilonewton per millimeter (kN/mm).

14.5.2.2.3 Total Isolation System Displacement. The total displacement, D_{TX}, of elements of the isolation system shall include additional displacement caused by actual and accidental torsion calculated from the spatial distribution of the lateral stiffness of the isolation system and the most disadvantageous location of eccentric mass. The total displacement, D_{TX}, of elements of an isolation system shall not be taken as less than that calculated in accordance with Equation (14-10):

$$D_{TX} = D_X\left[1 + \left(\frac{y}{P_T^2}\right)\frac{12e}{b^2 + d^2}\right] \qquad (14\text{-}10)$$

where

D_X = Displacement, in inches (millimeters), at the center of rigidity of the isolation system in the direction under consideration calculated in accordance with Equation (14-8);
y = Distance, in inches (millimeters), between the centers of rigidity of the isolation system and the element of interest measured perpendicular to the direction of seismic loading under consideration;
e = Actual eccentricity, in feet (millimeters), measured in plan between the center of mass of the structure above the isolation interface and the center of rigidity of the isolation system, plus accidental eccentricity taken as 5% of the longest plan dimension of the structure perpendicular to the direction of force under consideration;
b = Shortest plan dimension of the structure, in feet (millimeters), measured perpendicular to d;
d = Longest plan dimension of the structure, in feet (millimeters); and
P_T = Ratio of the effective translational period of the isolation system to the effective torsional period of the isolation system, as calculated by dynamic analysis or in accordance with Equation (14-11), except P_T need not be taken as less than unity.

$$P_T = \frac{1}{r_I}\sqrt{\frac{\sum_{i=1}^{N}(x_i^2 + y_i^2)}{N}} \qquad (14\text{-}11)$$

where

x_i, y_i = Horizontal distances, in feet (millimeters), from the center of mass to the ith isolation system device in the two horizontal axes of the isolation system;
N = Number of isolation system devices; and
r_I = Radius of gyration of the isolation system, in feet (millimeters), which shall be calculated as $((b^2 + d^2)/12)^{1/2}$ for isolation systems of rectangular plan dimension, $b \times d$.

The total displacement, D_{TX}, shall not be taken as less than 1.15 times D_X.

14.5.2.3 Minimum Lateral Forces

14.5.2.3.1 Isolation System and Structural Elements at or below the Base Level The level immediately above the isolation interface shall be taken as the isolation base level. The isolation system, the foundation, and all structural elements at or below the base level shall be evaluated and designed to withstand a minimum lateral seismic force, V_b, for the largest hazard level considered calculated in accordance with Equation (14-12):

$$V_b = k_X D_X \qquad (14\text{-}12)$$

where

k_X = Effective stiffness, in kips per inch (kips/in.) or kilonewtons per millimeters (kN/mm), of the isolation system at the displacement D_X calculated in accordance with Equation (14-5); and
D_X = Displacement, in inches (millimeters), at the center of rigidity of the isolation system in the direction under consideration, calculated in accordance with Equation (14-8).

V_b shall not be taken less than the maximum force in the isolation system at any displacement up to and including the displacement D_X, as defined in this section.

Overturning loads on elements of the isolation system, the foundation, and structural elements at or below the base level caused by lateral seismic force, V_b, shall be based on the vertical distribution of force calculated in accordance with Section 14.5.2.4.

All elements and components at or below the base level, excluding the isolation system devices, shall be evaluated and designed as force controlled in accordance with Section 7.5.2.2.2.

14.5.2.3.2 Structural Elements above the Base Level. The structure above the base level shall be evaluated and designed for a minimum shear force, V_{st}, determined in accordance with Equation (14-13):

$$V_{st} = V_b\left(\frac{W_s}{W}\right)^{(1-2.5\beta_X)} \qquad (14\text{-}13)$$

where

V_b = Lateral seismic force, in kips (kilonewtons), on the isolation system and structural elements at or below the base level;
W = Effective seismic weight, in kips (kilonewtons), of the building above the isolation interface;
W_s = Effective seismic weight, in kips (kilonewtons), of the building above the isolation interface, excluding the effective seismic weight, in kips (kilonewtons), of the base level; and
β_X = Effective damping of the isolation system at displacement D_X in the direction under consideration.

The effective seismic weight W_s in Equation (14-13) shall be taken as equal to W when the average distance from the top of isolation system devices to the underside of the base level floor framing above the isolation system devices exceeds 3 ft (0.9 m).

EXCEPTION: For isolation systems whose hysteretic behavior is characterized by an abrupt transition from pre-yield to post-yield or pre-slip to post-slip behavior, the exponent term $(1–2.5\beta_X)$ in Equation (14-13) shall be replaced by $(1–3.5\beta_X)$.

14.5.2.3.3 Limits on V_{st} The value of V_{st} for the BSE-1X hazard level shall not be taken as less than the greater of all the following:

1. The base shear corresponding to the factored design wind load determined in accordance with ASCE 7; and
2. The lateral seismic force, V_{st}, calculated in accordance with Equation (14-13), with V_b set equal to the force required to fully activate the isolation system using the greater of the following:
   (a) Upper-bound properties,
   (b) 1.5 times the nominal design properties for the yield level of a softening system,
   (c) Ultimate capacity of a sacrificial wind-restraint system,
   (d) Breakaway friction force of a sliding system, and
   (e) Force at zero displacement of a sliding system following a complete dynamic cycle of motion at D_X.

14.5.2.4 Vertical Distribution of Force The lateral seismic force V_{st} shall be distributed over the height of the structure above the base level in accordance with Equations (14-14) through (14-17):

$$F_1 = (V_b - V_{st}) \tag{14-14}$$

$$F_x = C_{vx} V_{st} \tag{14-15}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=2}^{n} w_i h_i^k} \tag{14-16}$$

$$k = 14\beta_X T_{fb} \tag{14-17}$$

where

F_1 = Pseudo lateral seismic force, in kips (kilonewtons), applied at Level 1, the base level;
F_x = Pseudo lateral seismic force, in kips (kilonewtons), applied at Level x, $x > 1$;
C_{vx} = Vertical distribution factor;
V_b = Lateral seismic force, in kips (kilonewtons), on the isolation system and structural elements at or below the base level;
V_{st} = Total lateral seismic force, in kips (kilonewtons), on structural elements above the base level calculated in accordance with Equation (14-13) and the limits of Section 14.5.2.3.3.
w_i,w_x = Portion of W_s, in kips (kilonewtons), that is located at Level i or x;
h_i,h_x = Height, in feet (millimeters), from the base level to Level i or x;
T_{fb} = Fundamental period, in seconds, of the structure above the isolation interface determined in accordance with Section 7.4.1.2 assuming fixed-base conditions; and
β_X = Effective damping of the isolation system at displacement D_X in the direction under consideration.

EXCEPTION: In lieu of Equations (14-13) through (14-15), the pseudo lateral seismic forces F_1 and F_x are permitted to be calculated as the average value of the force at Level 1 and Level x, respectively, in the direction of interest using the results of a simplified stick model of the building and a lumped representation of the isolation system using response history analysis scaled to V_b across the isolation interface.

14.5.2.5 Design Forces and Deformations Design forces and deformations shall be determined in accordance with Section 14.5.1.5.

14.5.3 Linear Dynamic Procedure

14.5.3.1 General Where the linear dynamic procedure is used to evaluate and design seismically isolated buildings, subject to the limitations of Section 14.5.1.2, the requirements of Section 7.4.2 and this section shall apply. The analysis of the isolation system and structure shall be performed separately for upper-bound and lower-bound properties, and the governing case for each response parameter of interest shall be used for evaluation and design for each hazard level considered.

14.5.3.2 Response Spectrum Method Dynamic analysis using the response spectrum method shall be performed using a modal damping value for the fundamental mode in the direction of interest not greater than the effective damping of the isolation system. Modal damping values for higher modes shall be selected consistent with those appropriate for response spectrum analysis of the structure above the isolation system assuming a fixed base.

Use of the response spectrum method to determine D_X and D_{TX} shall include simultaneous excitation of the model by 100% of the ground motion in the critical direction and 30% of the ground motion in the perpendicular, horizontal direction. D_X and D_{TX} of the isolation system shall be calculated as the vector sum of the two orthogonal displacements.

14.5.3.3 Isolation System and Structural Elements at or below the Base Level The isolation system, the foundation, and all structural elements at or below the base level shall be evaluated and designed to withstand the forces obtained from the dynamic analysis, except the design lateral force shall not be taken as less than 90% of V_b calculated in accordance with Equation (14-12) for each hazard level considered. All structural elements at or below the base level, excluding the isolation system devices, shall be designed as force controlled in accordance with Chapter 7, Section 7.5.2.2.2.

The total displacement of the isolation system shall not be taken as less than 80% of D'_{TX} calculated in accordance with Equation (14-18):

$$D'_{TX} = \frac{D_{TX}}{\sqrt{1 + (T_{fb}/T_X)^2}} \tag{14-18}$$

where

D_{TX} = Total displacement, in inches (millimeters), of the isolation system in the direction under consideration determined in accordance with Section 14.5.2.2.3;
T_{fb} = Fundamental fixed-base period, in s, of the structure above the isolation interface determined in accordance with Section 7.4.1.2; and
T_X = Effective period of the seismically isolated building, in s, at the displacement D_X in the direction under consideration, calculated in accordance with Equation (14-9).

14.5.3.4 Structural Elements above the Base Level Structural elements above the base level shall be evaluated and designed for the forces obtained from the dynamic analysis. The design shear at any story shall not be less than the story shear resulting from application of the forces calculated in accordance with Equation (14-15) with V_b equal to the base shear obtained from the dynamic analysis in the direction of interest.

14.5.3.5 Scaling of Results Where the demand on structural elements from the linear dynamic procedure is less than the minimum values prescribed by Sections 14.5.3.3 and 14.5.3.4, all evaluation and design parameters shall be adjusted upward proportionally.

14.5.3.6 Design Forces and Deformations Design forces and deformations shall be determined in accordance with Section 14.5.1.5.

14.5.4 Nonlinear Static Procedure

14.5.4.1 General Where the nonlinear static procedure is used to evaluate and design seismically isolated buildings, subject to the limitations of Section 14.5.1.3, the requirements of Section 7.4.3 as modified by this section shall apply. The analysis of the isolation system and structure shall be performed separately for upper-bound and lower-bound properties, and the governing case for each response parameter of interest shall be used for evaluation and design for each hazard level considered.

14.5.4.2 Target Displacement In each principal direction, the building model shall be pushed to the target displacement, D'_X, calculated in accordance with Equation (14-19). The target displacement, D'_X, shall be determined at a control node that is located at the center of rigidity of the isolation base level:

$$D'_X \frac{D_X}{\sqrt{1+(T_{fb}/T_X)^2}} \qquad (14\text{-}19)$$

where

D_X = Displacement, in inches (millimeters), at the center of rigidity of the isolation system in the direction under consideration calculated in accordance with Equation (14-8);

T_{fb} = Fundamental period, in seconds, of the structure above the isolation interface determined in accordance with by Section 7.4.1.2; and

T_X = Effective period of the seismically isolated building, in s, at the displacement D_X in the direction under consideration calculated in accordance with Equation (14-9).

14.5.4.3 Seismic Force Pattern The pattern of applied seismic forces shall be determined in accordance with Section 14.5.2.4.

14.5.4.4 Design Forces and Deformations Design forces and deformations shall be determined in accordance with Section 14.5.1.5.

14.5.5 Nonlinear Dynamic Procedure

14.5.5.1 General Where the nonlinear dynamic procedure is used to evaluate and design seismically isolated buildings, the requirements of this section shall apply. Dynamic analysis shall be performed for a suite of ground motions selected and modified in accordance with Section 14.2.2.1 for each hazard level considered. Inherent damping shall meet the requirements of Section 7.4.4.4.

The analysis of the isolation system and structure shall be performed separately for upper-bound and lower-bound properties, and the governing case for each response parameter of interest shall be used for evaluation and design for each hazard level considered.

14.5.5.2 Accidental Mass Eccentricity Accidental mass eccentricity shall consist of shifting the computed center of mass by an amount equal to not less than 2% of the diaphragm dimension at each level, separately in each of two orthogonal directions.

EXCEPTION: The effects of accidental mass eccentricity shall be permitted to be accounted for by amplifying forces and deformations determined from analysis using only the computed center of mass, provided that factors used to amplify forces and deformations of the center of mass case produce results which envelope all accidental mass eccentricity cases.

14.5.5.3 Isolation System and Structural Elements at or below the Base Level The isolation system, foundation, and all structural elements at or below the base level shall be evaluated and designed for a lateral force not less than 90% of V_b calculated in accordance with Equation (14-12). The total displacement of the isolation system shall not be taken as less than 80% of D'_{TX} calculated in accordance with Equation (14-18) with D_{TX} determined in accordance with the requirements of the linear static procedure.

14.5.5.4 Structural Elements above the Base Level Structural elements above the base level shall be evaluated and designed for a base shear, V_b, not less than 80% of that calculated in accordance with Equation (14-12), and for story shears not less than 100% of those determined in accordance with Section 14.5.2.4. If the structure above the base level exhibits any of the irregularities in Section 7.3.1.1 assuming that it is fixed base, V_b shall not be taken as less than 100% of that calculated in accordance with Equation (14-12).

14.5.5.5 Scaling of Results Where the demand on structural elements from the nonlinear dynamic procedure is less than the minimum values prescribed by Sections 14.5.5.3 and 14.5.5.4, all evaluation and design parameters shall be adjusted upward proportionally.

14.5.5.6 Design Forces and Deformations Design forces and deformations shall be determined in accordance with Section 14.5.1.5.

14.6 ISOLATION SYSTEM TESTING AND DESIGN PROPERTIES

14.6.1 General The deformation characteristics and damping values of the isolation system devices for use in Section 14.3 shall be determined in accordance with the requirements of this section.

The isolation system devices to be tested shall include isolators, components of the wind-restraint system, and supplemental energy dissipation devices, if such components and devices are used in the design.

14.6.2 Qualification Tests Isolation system device manufacturers shall submit for approval by the design professional responsible for the structure the results of qualification tests, analysis of test data, and supporting scientific studies used to quantify the effects of heating caused by cyclic dynamic motion, loading rate, scragging, variability and uncertainty in isolation system device properties, temperature, aging, environmental exposure, and contamination. The qualification testing shall be applicable to the component types, models, materials, and sizes to be used in the construction. The qualification testing shall have been performed on components manufactured by the same manufacturer supplying the components to be used in the construction. When scaled specimens are used in the qualification testing, principles of scaling and similarity shall be used in the interpretation of the data.

14.6.3 Prototype Tests

14.6.3.1 General Prototype tests shall be performed separately on two full-sized specimens of each predominant type and size of isolation system device. If prototype testing is performed on a pair of isolation system devices simultaneously, two pairs of each predominant type and size of isolation system device shall be tested. The test specimens shall include components of the wind-restraint system if such components are used in the design. Supplemental energy dissipation devices shall be tested in accordance with Section 15.8. Specimens tested shall not be used for construction unless approved by the design professional responsible for the structure.

14.6.3.2 Record For each cycle of each test, the force–deflection behavior of the test specimen shall be recorded.

14.6.3.3 Sequence and Cycles The following sequence of tests shall be performed for the prescribed number of cycles for the average vertical load corresponding to Load Combination 1 of Section 14.2.6.2 on all isolation system devices of a common type and size. Prior to performing these tests, the production set of tests of Section 14.6.4 shall be performed on each isolation system device.

1. Twenty fully reversed cycles of loading at a lateral force corresponding to the factored wind design force. The factored wind design force shall be calculated in accordance with ASCE 7.
2. The sequence of either Item (a) or Item (b) shall be performed:
   (a) Three fully reversed cycles of quasi-static loading at each of the following increments of displacement:

$0.25D_X$, $0.50D_X$, $0.67D_X$, and $1.0D_X$. D_X shall be calculated for the largest hazard level considered except not less than the BSE-2E hazard level.

   (b) The following sequences, performed dynamically at a frequency equal to the inverse of the effective period, T_X: continuous loading of one fully reversed cycle at each of the following increments of displacement: $1.0D_X$, $0.67D_X$, $0.50D_X$, and $0.25D_X$ followed by continuous loading of one fully reversed cycle at each of $0.25D_X$, $0.50D_X$, $0.67D_X$, and $1.0D_X$. T_X and D_X shall be calculated for the largest hazard level considered except not less than the BSE-2E hazard level. A rest interval shall be permitted between these two sequences.

3. Three fully reversed cycles of quasi-static loading, or dynamic loading at a frequency equal to the inverse of the effective period, T_X, and a displacement of $1.0D_X$. T_X and D_X shall be calculated for the largest hazard level considered except not less than the BSE-2E hazard level.
4. The sequence of either Item (a) or Item (b) shall be performed:
   (a) $30S_{X1}/(S_{XS}B_X)$, except not fewer than 10, continuous fully reversed cycles of quasi-static loading at the displacement, $0.75D_X$. S_{X1}, S_{XS}, B_X, and D_X shall be calculated for the largest hazard level considered except not less than the BSE-2E hazard level.
   (b) The test of Section 14.6.3.3, Item 4(a), performed dynamically at a frequency equal to the inverse of the effective period, T_X. T_X shall be calculated for the largest hazard level considered except not less than the BSE-2E hazard level. This test shall be permitted to consist of separate sets of multiple cycles of loading, with each set consisting of not fewer than five continuous cycles.

14.6.3.4 Vertical-Load-Carrying Isolation System Devices If an isolation system device is also a vertical-load-carrying component, then Section 14.6.3.3, Item 2, shall also be performed for the average vertical load corresponding to Load Combinations 2 and 3 of Section 14.2.6.2 on all isolation system devices of a common type and size. The load increment caused by earthquake overturning, Q_E, for each isolation system device shall be equal to or greater than the peak earthquake vertical force response corresponding to the test displacement being evaluated.

Vertical loads and horizontal displacements shall be the envelope of those determined from separate analyses using upper-bound and lower-bound isolation system device properties.

EXCEPTION: In lieu of envelope values, it shall be permitted to perform multiple, separate tests for the combinations of vertical load and horizontal displacement obtained from the upper-bound and lower-bound isolation system device property analyses.

14.6.3.5 Dynamic Testing Tests specified in Section 14.6.3.3 shall be performed dynamically at the lesser of the effective periods, T_X, determined using upper-bound and lower-bound properties for the largest hazard level considered except not less than the BSE-2E hazard level.

Dynamic testing shall not be required if the prototype testing has been performed dynamically on similar-sized isolation system devices meeting the requirements of Items 2, 3, 4, 7, and 8 of Section 14.6.3.9, and the testing was conducted at similar loads and accounted for the effects of velocity, amplitude of displacement, and heating effects. The prior dynamic prototype test data shall be used to establish factors that adjust nominal values of k_d and E_{loop} to account for the difference in test velocity and heating effects and to establish $\lambda_{\text{test min}}$ and $\lambda_{\text{test max}}$ in accordance with Section 14.3.3.2.

Reduced-scale prototype specimens shall be permitted to quantify the rate-dependent properties of isolation system devices. Reduced-scale prototype specimens shall be of the same type and material and shall be manufactured with the same processes and quality as full-scale prototypes and shall be tested at a frequency that represents full-scale prototype loading rates.

14.6.3.6 Isolation System Devices Dependent on Bilateral Load If the force–deflection properties of the isolation system devices are dependent on bilateral load, the tests specified in Sections 14.6.3.3 and 14.6.3.4 shall be augmented to include bilateral load at the following increments of the displacement D_X: 0.25 and 1.0, 0.50 and 1.0, 0.75 and 1.0, and 1.0 and 1.0. D_X shall be calculated for the largest hazard level considered except not less than the BSE-2E hazard level.

EXCEPTION: In lieu of augmenting the Sections 14.6.3.3 and 14.6.3.4 tests to include bilateral displacement, property modification factors for bilateral effects shall be permitted to be established using data from testing of similar isolators in accordance with Section 14.6.3.9. If reduced-scale prototype specimens are used to quantify bilateral-load-dependent properties, the reduced-scale specimens shall be of the same type and material and shall be manufactured with the same processes and quality as full-scale prototypes.

The force–deflection properties of an isolation system device shall be considered to be dependent on bilateral load if the effective stiffness when subjected to bilateral loading is different by more than 15% from the effective stiffness subjected to unilateral loading.

14.6.3.7 Maximum and Minimum Vertical Load Isolation system devices that carry vertical load shall be subjected to one fully reversed cycle of loading at the total displacement, D_{TX}, at the maximum and minimum vertical loads corresponding to Load Combinations 2 and 3 of Section 14.2.6.2 on any one isolation system device of a common type and size. D_{TX} shall be calculated for the largest hazard level considered except not less than the BSE-2E hazard level. Vertical loads and horizontal displacements for each test shall be the envelope of those determined from separate analyses using upper-bound and lower-bound isolation system device properties.

EXCEPTION: In lieu of envelope values, it shall be permitted to perform two tests, one each for the combination of vertical load and horizontal displacement obtained from analysis using the upper-bound and lower-bound isolation system device properties.

14.6.3.8 Sacrificial Wind-Restraint Systems If a sacrificial wind-restraint system is part of the isolation system, its ultimate capacity shall be established by testing.

14.6.3.9 Testing Similar Isolation System Devices Prototype tests shall be permitted to be satisfied by previous testing of similar isolation system devices complying with Items 1 through 6 of the following:

1. The isolation system device design shall not be more than 15% larger nor more than 30% smaller than the previously tested prototype, in terms of governing dimensions;
2. The isolation system device shall be of the same type and materials;
3. The isolation system device shall have an energy dissipated per cycle, E_{loop}, that is not less than 85% of the previously tested prototype;

4. The isolation system device shall be fabricated by the same manufacturer using the same or more stringent documented manufacturing and quality control procedures;
5. For elastomeric isolators, the isolation system device design shall not be subject to a greater shear strain nor greater vertical stress than that of the previously tested prototype; and
6. For sliding isolators, the isolation system device design shall not be subject to a greater vertical stress or sliding velocity than that of the previously tested prototype using the same sliding material.

This prototype testing exemption shall be approved by the independent design reviewer specified in Section 14.7.

When the results of tests of similar isolation system devices are used to establish dynamic properties in accordance with Section 14.6.3.5, in addition to Items 2 through 4, the following criteria of Items 7 and 8 shall be satisfied:

7. The similar isolation system device shall be tested at a frequency that represents design full-scale loading rates in accordance with principles of scaling and similarity, and
8. The length scale of reduced-scale specimens shall not be greater than two.

14.6.4 Production Testing A test program for the isolation system devices used in the construction shall be established by the design professional responsible for the structure. The test program shall evaluate the consistency of measured values of nominal isolation system device properties by testing 100% of the isolation system devices in combined compression and shear at not less than $0.67D_X$ determined using lower-bound properties. D_X shall be calculated for the largest hazard level considered except not less than the BSE-2E hazard level.

The mean results of all tests shall fall within the range of values defined by $\lambda_{\text{spec max}}$ and $\lambda_{\text{spec min}}$ established in Section 14.3.3.1. Different ranges shall be permitted for individual isolation system devices and for the average value across all isolation system devices of a common type and size provided that differences in the ranges of values are accounted for in the design of each component of the isolation system.

14.6.5 Determination of Force–Deflection Characteristics The force–deflection characteristics of an isolation system device shall be based on the cyclic load testing of prototype isolation system devices in Section 14.6.3.

The effective stiffness of an isolation system device, k_X, shall be calculated for each cycle of deformation in accordance with Equation (14-20):

$$k_X = \frac{|F^+| + |F^-|}{|\Delta^+| + |\Delta^-|} \tag{14-20}$$

where F^+ and F^- are the positive and negative forces at positive and negative test displacements, Δ^+ and Δ^-, respectively.

The effective damping of an isolation system device, β_X, shall be calculated for each cycle of deformation in accordance with Equation (14-21):

$$\beta_x = \frac{2}{\pi}\left(\frac{E_{\text{loop}}}{k_x(|\Delta^+| + |\Delta^-|)^2}\right) \tag{14-21}$$

where the energy dissipated per cycle of loading, E_{loop}, and the effective stiffness, k_X, are based on test displacements, Δ^+ and Δ^-.

The post-yield stiffness of an isolation system device shall be calculated for each cycle of loading in accordance with all the following:

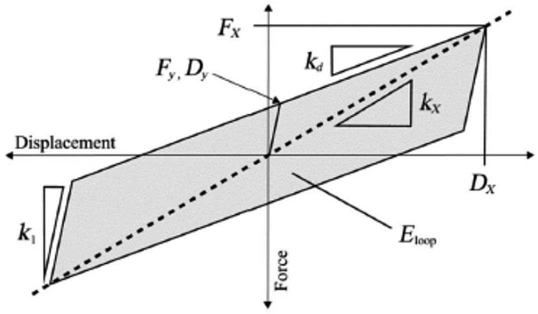

Figure 14-1. Properties of the isolation system device bilinear force–deflection model.

1. A test loop shall be assumed to have bilinear hysteretic characteristics with values of k_1, k_d, F_y, k_x, and E_{loop} as shown in Figure 14-1;
2. The computed loop shall have the same values of effective stiffness, k_x, and energy dissipated per cycle of loading, E_{loop}, as the test loop; and
3. The value of k_1 shall be a visual fit to the elastic stiffness of the isolation system device during unloading immediately after D_x.

EXCEPTION: Alternate methods for determining k_d shall be permitted, subject to design review in accordance with Section 14.7.

14.6.6 Test Specimen Adequacy The performance of the prototype test specimens shall be deemed adequate if all the following conditions are satisfied:

1. The force–deflection plots of all tests specified in Section 14.6.3 shall have a non-negative incremental force-resisting capacity.
2. For each test specimen, the average post-yield stiffness, k_d, and energy dissipated per cycle, E_{loop}, for the three cycles of test specified in Section 14.6.3.3, Item 3 for Load Combination 1 of Section 14.2.6.2, shall fall within the range defined by $\lambda_{\text{spec min}}$ and $\lambda_{\text{spec max}}$ for individual isolation system devices established in Section 14.3.3.1 multiplied by the nominal design properties established in Section 14.3.2.
3. For each test specimen, for each increment of test displacements $0.67D_X$ and $1.0D_X$ specified in Section 14.6.3.3, Items 2 and 3, and for each vertical load combination of Section 14.2.6.2 when required by Section 14.6.3.4, the value of the post-yield stiffness, k_d for each cycle shall fall within the range defined by $\lambda_{\text{test min}}$ and $\lambda_{\text{test max}}$ established in Section 14.3.3.2 multiplied by the average tested value.
4. For each test specimen, there shall be no greater than a 20% change in the effective stiffness over the cycles of test specified in Section 14.6.3.3, Item 4.
5. For each test specimen, the value of the post-yield stiffness, k_d, and energy dissipated per cycle, E_{loop}, for any cycle of test in Section 14.6.3.3, Item 4, shall fall within the range defined by $\lambda_{\text{test min}}$ and $\lambda_{\text{test max}}$ established in Section 14.3.3.2 multiplied by the average tested value.

   EXCEPTION: In lieu of satisfying this requirement for all cycles of the test in Section 14.6.3.3, Item 4, it shall be permitted to consider only the equivalent number of cycles at $0.75D_X$ representative of the isolation system for the local seismic hazard conditions. The equivalent number of cycles shall not be taken less than four. The design professional responsible for the structure shall specify the number of cycles. Design review of the number of cycles shall be performed in accordance with Section 14.7.

6. For each test specimen, there shall be no greater than a 20% decrease in the effective damping over the cycles of test specified in Section 14.6.3.3, Item 4.
7. All specimens of vertical-load-carrying elements of the isolation system shall remain stable where tested in accordance with Section 14.6.3.7.

EXCEPTION: The design professional responsible for the structure shall be permitted to adjust the limits of Items 4 and 6 to account for the testing property modification factors of Section 14.3.3.2 used for design of the isolation system.

14.7 DESIGN REVIEW

An independent design review of the isolation system and related test programs shall be performed by one or more individuals experienced in the design and analysis of buildings incorporating seismic isolation with a minimum of one reviewer being a registered design professional. Where ground motion histories are used, a minimum of one individual shall be experienced in the selection and scaling of ground motions. Design review shall include, at a minimum, all the following:

1. Project evaluation and design criteria, including site-specific spectra and ground motion histories, where applicable;
2. Preliminary evaluation and design, including the selection of the isolation system devices and determination of structure and isolation system displacement and force demands;
3. Property modification factors for the manufacturer and isolation system device selected in accordance with Section 14.3.3;
4. Qualification data and prototype testing program in accordance with Sections 14.6.2 and 14.6.3, respectively, and, in particular, Section 14.6.3.9, where applicable;
5. Final evaluaton and design of the structure and isolation system and all supporting analyses, including modeling of isolation system devices when the nonlinear static or nonlinear dynamic procedures are performed; and
6. Production testing program in accordance with Section 14.6.4.

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CHAPTER 15

DESIGN REQUIREMENTS FOR STRUCTURES WITH SUPPLEMENTAL ENERGY DISSIPATION

15.1 SCOPE

This chapter sets forth requirements for the systematic evaluation and retrofit of buildings using supplemental energy dissipation systems. Any of the Performance Objectives specified in this standard are permitted for supplemental energy dissipation retrofits.

Whenever the Limited Performance Objective of Section 2.4.3 or a Partial Retrofit of Section 2.4.5 is selected, the devices used shall be able to achieve performance responses larger than those used for the Limited Performance Objectives. Components and elements in buildings with energy dissipation systems shall comply with the requirements of this standard, unless they are modified by the requirements of this chapter.

Independent design review is required for all retrofit schemes that use energy dissipation systems. This design review shall be conducted in accordance with Section 15.7.

15.2 GENERAL DESIGN REQUIREMENTS

15.2.1 General Requirements Damping devices shall comply with the requirements of Section 15.1. Linear and nonlinear analyses shall be performed, as required, in accordance with Sections 15.4 and 15.5, respectively. Additional requirements for energy dissipation systems, as defined in Section 15.6, shall be met. Energy dissipation systems shall be subject to design review and tested in accordance with Sections 15.7 and 15.8.

15.2.2 Seismic Hazard The spectral response acceleration parameters at short periods, S_{XS}, and at a 1 s period, S_{X1}, and response spectra shall be determined in accordance with Section 2.3.

15.2.2.1 Ground Motion Acceleration Histories Where the nonlinear dynamic procedure in accordance with Section 15.5 is used, the provisions of Section 2.3.4 shall apply. For establishing the period range in Item 3 of Section 2.3.4, nominal properties of the energy dissipation devices for the hazard level considered shall be used.

15.2.3 Damping Device Requirements The energy dissipation devices shall be designed with consideration given to environmental conditions, including wind, aging effects, creep, fatigue, ambient temperature, operating temperature, and exposure to moisture or damaging substances.

The design of damping devices shall consider all of the following:

1. Potential low-cycle, large-displacement degradation caused by seismic loads;
2. Potential high-cycle, small-displacement degradation caused by wind, thermal, or other cyclic loads;
3. Forces or displacements caused by gravity loads;
4. Potential adhesion of device parts caused by corrosion, abrasion, biodegradation, moisture, or chemical exposure; and
5. Exposure to environmental conditions, including, but not limited to, temperature, humidity, moisture, radiation (e.g., ultraviolet light), and reactive or corrosive substances (e.g., saltwater).

Damping devices subject to failure by low-cycle fatigue shall be designed to resist wind forces without slip, movement, or inelastic cycling.

The design of damping devices shall consider and accommodate the range of thermal conditions, device wear, manufacturing tolerances, and other effects that cause device properties to vary during the design life of the device. Ambient temperature shall be the normal in-service temperature of the damping device. The design temperature range shall cover the annual minimum and maximum in-service temperatures of the damping device.

15.2.3.1 Device Classification Energy dissipation systems are classified as displacement dependent, velocity dependent, or other, as defined in Section 1.2. Displacement-dependent devices shall include devices that exhibit either rigid plastic (friction devices), bilinear (metallic yielding devices), or trilinear hysteresis. The response of displacement-dependent devices shall be independent of velocity and frequency of excitation. Velocity-dependent devices shall include solid and fluid viscoelastic devices and fluid viscous devices. Devices utilizing bimetallic interfaces subject to cold welding of the sliding interface shall not be permitted.

15.2.3.2 Multiaxis Movement Connection points of damping devices shall provide articulation to accommodate simultaneous longitudinal, lateral, and vertical displacements of the damping system.

15.2.3.3 Inspection and Periodic Testing Means of access for inspection and removal of all damping devices shall be provided. The registered design professional (RDP) responsible for design of the structure shall establish an inspection, maintenance, and testing schedule for each type of damping device to ensure that the devices respond in a dependable manner throughout their design life. The degree of inspection and testing shall reflect the established in-service history of the damping devices and the likelihood of change in properties over the design life of the devices.

15.2.3.4 Performance Objectives and System Redundancy If a Limited Performance Objective (LPO) is adopted, each story shall have at least four energy dissipation devices in each principal direction of the building, with at least two devices located on each side of the center of stiffness of the story in the direction under consideration.

The mathematical model of the building shall include the plan and vertical distribution of the energy dissipation devices. Analyses shall account for the dependence of the devices on excitation frequency, ambient and operating temperature, velocity, sustained loads, and bilateral loads. Multiple analyses of the building shall be conducted to bound the effects of the varying mechanical characteristics of the devices.

Displacement-dependent energy dissipation devices shall be capable of sustaining larger displacements and forces, and velocity-dependent devices shall be capable of sustaining larger displacements, velocities, and forces than the maximum calculated in accordance with the following criteria:

1. If four or more energy dissipation devices are provided in a given story of a building in one principal direction of the building, with a minimum of two devices located on each side of the center of stiffness of the story in the direction under consideration, all energy dissipation devices shall be capable of sustaining displacements equal to 130% of the maximum calculated displacement in the device in the BSE-2X or 200% of the maximum calculated displacement in the device at BSE-1X for an LPO. A velocity-dependent device, as described in Section 15.2.3.1, shall be capable of sustaining the force and displacement associated with a velocity equal to 130% of the maximum calculated velocity for that device in the BSE-2X or the force and displacement associated with 200% of the maximum calculated velocity for that device at BSE-1X for an LPO.
2. If fewer than four energy dissipation devices are provided in a given story of a building in one principal direction of the building, or fewer than two devices are located on each side of the center of stiffness of the story in the direction under consideration, all energy dissipation devices shall be capable of sustaining displacements equal to 200% of the maximum calculated displacement in the device in the BSE-2X. A velocity-dependent device shall be capable of sustaining the force and displacement associated with a velocity equal to 200% of the maximum calculated velocity for that device in the BSE-2X.

The components and connections of the damping device shall be designed to remain linearly elastic for the forces described in Items 1 or 2. Other elements of the damping system are permitted to have inelastic response if it is shown by analysis or test that inelastic response of these elements would not adversely affect the damping system or the performance of the structure.

15.3 PROPERTIES OF ENERGY DISSIPATION DEVICES

15.3.1 Nominal Design Properties Nominal design properties of energy dissipation devices shall be established from either project-specific prototype test data or prior prototype tests on a device of similar size and construction based on requirements of Section 15.8.1 and Section 15.8.1.3, respectively. These nominal design properties shall be modified by property variation or lambda (λ) factors to account for (1) manufacturing tolerances, (2) device characteristics not explicitly accounted for during testing, and (3) environmental effects and aging, to develop upper- and lower-bound properties for the design and analysis of the energy dissipated structure, as specified in Section 15.3.2.

15.3.2 Maximum and Minimum Damper Properties Maximum and minimum property modification (λ) factors shall be established in accordance with Equations (15-1) and (15-2) for each device by the RDP and shall be used in analysis and design to account for the variation from nominal properties:

$$\lambda_{max} = \lambda_{test\,max} * \lambda_{spec\,max} * (1 + \mathrm{SPAF}(\lambda_{ae\,max} - 1)) \geq 1.2 \quad (15\text{-}1)$$

$$\lambda_{min} = \lambda_{test\,min} * \lambda_{spec\,min} * (1 - \mathrm{SPAF}(1 - \lambda_{ae\,min})) \leq 0.85 \quad (15\text{-}2)$$

where

λ_{test} = Lambda factors obtained from testing,
λ_{spec} = Variation on the average of the manufacturing production test values from the nominal design value,
λ_{ae} = Property variation factor caused by the individual aging and environmental effects, and
SPAF = System property adjustment factor equal to 0.67 for all Performance Objectives.

EXCEPTION: When test data are reviewed by the RDP and accepted by a professional conducting design review, it is permitted to use λ_{max} less than 1.2 and λ_{min} greater than 0.85.

Maximum and minimum analysis and design properties for each device shall be determined in accordance with Equations (15-3) and (15-4):

$$\text{Maximum Design Property} = \text{Nominal Design Property} \times \lambda_{max} \quad (15\text{-}3)$$

$$\text{Minimum Design Property} = \text{Nominal Design Property} \times \lambda_{min} \quad (15\text{-}4)$$

A maximum and minimum analysis and design property shall be established for each modeling parameter as necessary for the selected method of analysis. Maximum velocity coefficients, stiffness, strength, and energy dissipation shall be considered together as the maximum analysis and design case, and minimum velocity coefficients, stiffness, strength, and energy dissipation shall be considered together as the minimum analysis and design case.

Separate maximum and minimum properties shall be established for loads and displacements corresponding to each Seismic Hazard Level under consideration.

15.4 ANALYSIS PROCEDURE SELECTION

Structures with a damping system provided for seismic resistance shall be analyzed and designed using the nonlinear response history procedure of Section 15.5.

EXCEPTION: It shall be permitted to analyze and design the structure using the linear static procedure of Section 15.9.2 subject to the limits of Sections 15.4.1 and 15.9.2 or the response-spectrum procedure of 15.9.2.4 subject to the limits of Sections 15.4.1 and 15.9.2.4.

15.4.1 General Limitations for the Linear Analysis Procedures The use of analysis methods listed in this section is restricted to cases where the energy dissipation devices are present in all stories of the upgraded building. All analyses shall be performed for the upper- and lower-bound properties specified in Section 15.3.

Linear procedures shall be permitted only if all the following criteria are met:

1. The building, including retrofit measures, shall comply with the requirements of Sections 7.3 through 7.5 for linear procedures, except as modified in this section.

2. For all deformation-controlled actions exclusive of the energy dissipation devices, Equation (7-39) shall be satisfied whereby the *m*-factor shall be taken as the least of the following values:
   (a) One-half of those specified by Chapters 8 through 12 at the selected structural performance level but not less than unity,
   (b) 1.0 for the Immediate Occupancy Performance Level,
   (c) 1.5 for the Life Safety Performance Level, and
   (d) 2.0 for the Collapse Prevention Performance Level.
3. The effective damping afforded by the energy dissipation system does not exceed 30% of critical in the fundamental mode.
4. The secant stiffness of each energy dissipation device, calculated at the maximum displacement in the device, is included in the mathematical model of the rehabilitated building.
5. Where evaluating the regularity of a building, the energy dissipation devices are included in the mathematical model.

15.5 NONLINEAR DYNAMIC PROCEDURES

15.5.1 General Requirements If the nonlinear dynamic procedure (NDP) is selected based on the requirements of Section 15.4 and Section 7.3 a nonlinear dynamic analysis shall be performed as required by Section 7.4.4, except as modified by this section. A nonlinear response history analysis shall use a mathematical model of the seismic-force-resisting system and the damping system as provided in this section.

The mathematical model shall account for both the plan and vertical spatial distribution of the energy dissipation devices in the building. The stiffness and damping properties of the damping devices used in the models shall be based on or verified by testing of the damping devices as specified in Section 15.8. The nonlinear force–velocity–displacement characteristics of damping devices shall be modeled, as required, to explicitly account for device dependence on frequency, amplitude, and duration of seismic loading.

If the energy dissipation devices are dependent on excitation frequency, operating temperature (including temperature rise caused by excitation), deformation (or strain), velocity, sustained loads, and bilateral loads, such dependence shall be accounted for in the analysis by assuming upper- and lower-bound properties to bound the solution as specified in Section 15.3.

In the NDP, the energy dissipation devices shall be modeled as nonlinear elements. All other components of the structure shall be modeled and evaluated in accordance with Sections 7.4.4 and 7.5. The viscous forces in velocity-dependent energy dissipation devices shall be included in the calculation of design actions and deformations. Substitution of viscous effects in energy dissipation devices by global structural damping for nonlinear time-history analysis shall not be permitted.

15.5.2 Modeling of Energy Dissipation Devices Mathematical models of displacement-dependent damping devices shall include the hysteretic behavior of the devices consistent with test data and accounting for all significant changes in strength, stiffness, and hysteretic loop shape. Mathematical models of velocity-dependent damping devices shall include the velocity coefficient consistent with test data. If damping device properties change with time and/or temperature, such behavior shall be modeled explicitly or through bounding per the requirements of Section 15.3. Models of the energy dissipation system shall include the flexibility of structural components of the damping system. These structural components, whose flexibility affects the performance of the energy dissipation system, include components of the foundation, braces that work in series with the energy dissipation devices, and connections between braces and the energy dissipation devices.

Energy dissipation devices shall be modeled as described in the following subsections, unless other methods approved by the Authority Having Jurisdiction are used.

15.5.2.1 Displacement-Dependent Devices A displacement-dependent device shall have a force–displacement relationship that is a function of the relative displacement between each end of the device. The response of a displacement-dependent device shall be independent of the relative velocity between each end of the device and frequency of excitation.

Displacement-dependent devices shall be modeled to capture their force–displacement response and their dependence, if any, on axial–shear–flexure interaction or bilateral deformation response.

For evaluating the response of a displacement-dependent device from testing data, the force, F, in a displacement-dependent device shall be calculated in accordance with Equation (15-5):

$$F = k(D)D \tag{15-5}$$

where

$k(D)$ = Displacement-dependent stiffness; and
D = Relative displacement between each end of the device.

15.5.2.2 Velocity-Dependent Devices

15.5.2.2.1 Solid Viscoelastic Devices Solid viscoelastic devices shall be modeled using a spring and dashpot in parallel (Kelvin model). The spring and dashpot constants selected shall capture the frequency and temperature dependence of the device consistent with fundamental frequency of the building (f_1) and the operating temperature range. If the cyclic response of a solid viscoelastic device cannot be captured by single estimates of the spring and dashpot constants, the response of the building shall be estimated by multiple analyses of the building frame, using limiting upper- and lower-bound values for the spring and dashpot constants.

The force in a solid viscoelastic device shall be determined in accordance with Equation (15-6):

$$F = k(D)D + C\dot{D} \tag{15-6}$$

where

C = Damping coefficient for the solid viscoelastic device;
D = Relative displacement between each end of the device;
$\dot{D}$ = Relative velocity between each end of the device; and
$k(D)$ = Displacement-dependent stiffness.

15.5.2.2.2 Fluid Viscoelastic Devices Fluid viscoelastic devices shall be modeled using a combination of springs and dashpots in series and parallel to represent the constitutive relation of the device. The spring and dashpot constants selected shall capture the frequency and temperature dependence of the device consistent with fundamental frequency of the building (f_1) and the operating temperature range. If the cyclic response of a fluid viscoelastic device cannot be captured by single estimates of the spring and dashpot constants, the response of the building shall be estimated by multiple analyses of the building frame, using limiting upper- and lower-bound values for the spring and dashpot constants.

15.5.2.2.3 Fluid Viscous Devices Linear fluid viscous dampers exhibiting stiffness in the frequency range $0.5f_1$ to $2.0f_1$ shall be modeled as fluid viscoelastic devices.

In the absence of stiffness in the frequency range $0.5f_1$ to $2.0f_1$, the force in the fluid viscous device shall be computed in accordance with Equation (15-7):

$$F = C_0|\dot{D}|^{\alpha} \times \text{sgn}(\dot{D}) \tag{15-7}$$

where

C_0 = Damping coefficient for the device;

$\dot{D}$ = Relative velocity between each end of the device; sgn is the signum function, which in this case, defines the sign of the relative velocity term; and

α = Velocity exponent of the device.

15.5.2.3 Other Types of Devices Energy dissipation devices not classified as either displacement dependent or velocity dependent shall be modeled using methods approved by the Authority Having Jurisdiction. Such models shall accurately describe the force–velocity–displacement response of the device under all sources of loading, including gravity, seismic, environmental, and thermal.

15.5.3 Accidental Eccentricity Inherent eccentricity resulting from lack of symmetry in mass and stiffness shall be accounted for in the analysis. In addition, accidental eccentricity consisting of displacement of the center of mass from the computed location by an amount equal to 5% of the diaphragm dimension separately in each of two orthogonal directions at each diaphragm level shall be accounted for in the analysis. Alternatively, amplification factors on forces, drifts, and deformations are permitted to be rationally established to account for the effects of mass eccentricity. These factors shall be applied to the center-of-mass analysis results to incorporate accidental eccentricity effects.

15.6 DETAILED SYSTEM REQUIREMENTS

15.6.1 General The energy dissipation system and the remainder of the seismic-force-resisting system shall comply with the detailed system requirements specified in this section. Upper- and lower-bounding analyses shall be performed to account for the variation in device properties, as specified in Section 15.3.

15.6.2 Wind Forces The fatigue life of energy dissipation devices, or components thereof, including seals in a fluid viscous device, shall be investigated and shown to be adequate for the design life of the devices. Devices subject to failure by low-cycle fatigue shall resist wind forces in the linearly elastic range.

15.6.3 Inspection and Replacement Access for inspection and replacement of the energy dissipation devices shall be provided.

15.6.4 Maintenance The RDP shall establish maintenance and testing schedules for energy dissipation devices to obtain reliable responses of the devices over the design life of the structure. The degree of maintenance and testing shall reflect the established in-service history of the devices.

15.7 DESIGN REVIEW

A review of the design of a structure with energy dissipation devices and related test programs shall be performed by an independent engineer (or engineers) experienced in design and analysis of structures incorporating energy dissipation devices, with a minimum of one reviewer being a RDP. Damping system design review shall include, but need not be limited to, all of the following:

1. Project design criteria, including site-specific spectra and ground motion histories;
2. Selection of the devices and their design parameters;
3. Preliminary design, including the determination of the structure lateral displacements and the device displacement, velocity, and force demands;
4. Review of a prototype testing program to be conducted in accordance with Section 15.8.1, or on the basis of use of data from similar devices;
5. Review of manufacturer test data and property modification factors for the manufacturer and device selected;
6. Final design of the entire structural system and supporting analyses including modeling of the damping devices for response history analysis if performed; and
7. Damping device production testing program (Section 15.8.2).

15.8 REQUIRED TESTS OF ENERGY DISSIPATION DEVICES

The force–velocity–displacement relations and damping properties assumed as the damping device nominal design properties in Section 15.3.1 shall be confirmed by the tests conducted in accordance with Section 15.8.1, or they shall be based on prior tests of devices meeting the similarity requirements of Section 15.8.1.3.

The tests specified in this section shall be conducted to confirm the force–velocity–displacement properties of the energy dissipation devices assumed for analysis and design and to demonstrate the robustness of individual devices under seismic excitation. These tests shall be conducted before production of devices for construction. The production testing requirements are specified in Section 15.8.2.

Device nominal properties determined from the prototype testing shall meet the acceptance criteria established using $\lambda_{\text{spec max}}$ and $\lambda_{\text{spec min}}$ from Section 15.3.2. These criteria shall account for likely variations in material properties.

Device nominal properties determined from the production testing of Section 15.8.2 shall meet the acceptance criteria established using $\lambda_{\text{spec max}}$ and $\lambda_{\text{spec min}}$ from Section 15.3.2.

The fabrication and quality control procedures used for all prototype and production devices shall be identical. These procedures shall be approved by the RDP before the fabrication of prototype devices.

The force–velocity–displacement relationship for each cycle of each test shall be recorded electronically for all prototype tests of Section 15.8.1 and production tests of Section 15.8.2.

15.8.1 Prototype Tests

15.8.1.1 General The tests specified in this section shall be performed separately on two full-size damping devices of each type and size used in the design, in the order listed as follows.

Representative sizes of each type of device are permitted to be used for prototype testing, provided that both of the following conditions are met:

1. Fabrication and quality control procedures are identical for each type and size of device used in the structure, and
2. Prototype testing of representative sizes is approved by the RDP responsible for design of the structure.

Test specimens shall not be used for construction, unless they are approved by the RDP responsible for design of the structure and meet the requirements for prototype and production tests.

15.8.1.2 Sequence and Cycles of Testing For all of the following test sequences, each damping device shall be subjected to gravity load effects and thermal environments representative of the installed condition.

Before the sequence of prototype tests defined in this section, a production test in accordance with Section 15.8.2 shall be performed and data from this test shall be used as a baseline for comparison with subsequent tests on production dampers.

1. Each device shall be loaded with the number of cycles expected in the design windstorm, but not less than 2,000 fully reversed cycles of load (displacement-dependent and viscoelastic devices) or displacement (viscous devices) at amplitudes expected in the design windstorm, at a frequency equal to the inverse of the fundamental period of the building.

   EXCEPTION: Devices not subject to wind-induced forces or displacements need not be subjected to these tests. Alternate loading protocols that apportion the total wind displacement into its expected static, pseudo static, and dynamic components shall be acceptable.
2. Each device shall be subjected to the following sequence of tests, all at a frequency equal to the inverse of the fundamental period of the upgraded building:
   (a) Ten fully reversed cycles at the displacement in the energy dissipation device corresponding to 0.33 times the BSE-2X device displacement or 0.67 times the BSE-1X displacement for an LPO,
   (b) Five fully reversed cycles at the displacement in the energy dissipation device corresponding to 0.67 times the BSE-2X device displacement or 1.33 times the BSE-1X displacement for an LPO, and
   (c) Three fully reversed cycles at the displacement in the energy dissipation device corresponding to 1.0 times the BSE-2X device displacement or 2.0 times the BSE-1X displacement for an LPO.
3. Where the damping device characteristics vary with operating temperature, the tests of Items 2a through c in this list shall be conducted on at least one device, at a minimum of two additional temperatures (minimum and maximum) that bracket the design temperature range.

   EXCEPTION: Testing methods for energy dissipation devices other than those previously noted shall be permitted, provided that all of the following conditions are met:
   (a) Equivalency between the proposed method and cyclic testing can be demonstrated;
   (b) The proposed method captures the dependence of the energy dissipation device response to ambient temperature, frequency of loading, and temperature rise during testing; and
   (c) The proposed method is approved by the RDP.
4. If the force–deformation properties of the damping device at any displacement less than or equal to the BSE-2X device displacement (or twice BSE-1X displacement for an LPO) change by more than 15% for changes in testing frequency from $1/(1.5T_1)$ to $2.5/T_1$, then the preceding tests [Items 2(a) through 2(c)] shall also be performed at frequencies equal to $1/T_1$ and $2.5/T_1$.

   EXCEPTION: When full-scale dynamic testing is not possible because of test machine limitations, it is permitted to use reduced-scale prototypes to qualify the rate-dependent properties of damping devices provided that scaling principles and similitude are used in the design of the reduced-scale devices and the test protocol.

15.8.1.3 Testing Similar Devices Prototype tests need not be performed on a particular damping device if there exists a previously prototype-tested unit that meets all of the following conditions:

1. It is of similar dimensional characteristics, internal construction, and static and dynamic internal pressures (if any) to the subject damping device;
2. It is of the same type and materials as the subject damping device;
3. It was fabricated using identical documented manufacturing and quality control procedures that govern the subject damping device; and
4. It was tested under similar maximum strokes and forces to those required of the subject damping device.

Provided that the following conditions are also true:

1. All pertinent testing data are made available to, and are approved by, the RDP;
2. The manufacturer can substantiate the similarity of the previously tested devices to the satisfaction of the RDP; and
3. The submission of data from a previous testing program is approved in writing by the RDP.

15.8.1.4 Determination of Force–Velocity–Displacement Characteristics The force–velocity–displacement characteristics of an energy dissipation device shall be based on the cyclic load and displacement tests of prototype devices specified in Section 15.8.1.2 and all of the following:

1. The maximum force and minimum force at zero displacement, the maximum force and minimum force at maximum device displacement, and the area of hysteresis loop (E_{loop}) shall be calculated for each cycle of deformation;
   (a) As required, the effective stiffness (K_{eff}) of an energy dissipation device exhibiting stiffness shall be calculated for each cycle of deformation in accordance with Equation (15-8):

   $$K_{\text{eff}} = \frac{|F^+| + |F^-|}{|\Delta^+| + |\Delta^-|} \tag{15-8}$$

   where forces F^+ and F^- shall be calculated at displacements Δ^+ and Δ^-, respectively. The effective stiffness of an energy dissipation device shall be established at the test displacements given in Section 15.8.1.2.
   (b) The equivalent viscous damping of an energy dissipation device (β_{eff}) exhibiting stiffness shall be calculated for each cycle of deformation based on Equation (15-9):

   $$\beta_{\text{eff}} = \frac{1}{2\pi}\frac{E_{\text{loop}}}{K_{\text{eff}}\Delta_{\text{ave}}^2} \tag{15-9}$$

   where K_{eff} shall be calculated in accordance with Equation (15-8), and E_{loop} shall be taken as the area enclosed by one complete cycle of the force–displacement response for a single energy dissipation device at a prototype test displacement, Δ_{ave}, equal to the average of the absolute values of displacements Δ^+ and Δ^-, $(|\Delta^+| + |\Delta^-|)/2$.
2. Damping device nominal test properties for analysis and design shall be based on the average value for the first three cycles of test at a given displacement. For each cycle of each test, corresponding lambda factors (λ_{test}) for cyclic effects shall be established by comparison of nominal and per-cycle properties; and
3. Lambda (λ) factors for velocity and temperature shall be determined simultaneously with those for cyclic effects where full-scale prototype test data are available. Where these or similar effects are determined from separate tests, lambda factors shall be established by comparison of

properties determined under prototype test conditions with corresponding properties determined under the range of test conditions applicable to the property variation parameter.

15.8.1.5 Device Adequacy The performance of a prototype device shall be considered adequate if all of the conditions are satisfied. The 15% limits specified in the following text are permitted to be increased by the RDP responsible for the design of the structure, provided that the increased limit has been demonstrated by analysis not to have a deleterious effect on the response of the structure.

15.8.1.5.1 General Requirements The performance of the prototype damping devices shall be deemed acceptable if all the following requirements are met and, in addition, all the requirements of Section 15.8.1.5.2 are met for displacement-dependent devices or all the requirements of Section 15.8.1.5.3 are met for velocity-dependent damping devices.

1. For Test 1, no signs of damage, including leakage, yielding, or breakage;
2. For Tests 2, 3, and 4, the maximum force and minimum force at zero displacement for a damping device for any one cycle do not differ by more than 15% from the average maximum and minimum forces at zero displacement as calculated from all cycles in that test at a specific frequency and temperature;
3. For Tests 2, 3, and 4, the area of hysteresis loop (E_{loop}) of a damping device for any one cycle does not differ by more than 15% from the average area of the hysteresis loop as calculated from all cycles in that test at a specific frequency and temperature; and
4. The test values for damping units, determined in accordance with Section 15.8.1.2, shall not exceed the values specified by the RDP in accordance with Section 15.3.2.

15.8.1.5.2 Displacement-Dependent Devices The performance of the prototype displacement-dependent damping devices shall be deemed adequate if, in addition to the general requirements of Section 15.8.1.5.1, all of the following conditions, based on tests specified in Section 15.8.1.2, are satisfied:

1. For Tests 2, 3, and 4, the maximum force and minimum force at maximum device displacement for a damping device for any one cycle does not differ by more than 15% from the average maximum and minimum forces at the maximum device displacement as calculated from all cycles in that test at a specific frequency and temperature;
2. The average maximum and minimum forces at zero displacement and maximum displacement, and the average area of the hysteresis loop (E_{loop}), calculated for each test in the sequence of Tests 2, 3, and 4, shall not differ by more than 15% from the target values specified by the RDP responsible for the design of the structure; and
3. The average maximum and minimum forces at zero displacement and the maximum displacement, and the average area of the hysteresis loop (E_{loop}), calculated for Section 15.8.1.2, Item 2c, shall fall within the limits specified by the RDP, as described by the nominal properties and the lambda factor for specification tolerance ($\lambda_{\text{spec,max}}$ and $\lambda_{\text{spec,min}}$) from Section 15.3.2.

15.8.1.5.3 Velocity-Dependent Damping Devices The performance of the prototype velocity-dependent damping devices shall be deemed adequate if in addition to the general requirements of Section 15.8.1.5.1, all of the following conditions, based on tests specified in Section 15.8.1.2, are satisfied:

1. For velocity-dependent damping devices exhibiting stiffness, the effective stiffness of a damping device in any one cycle of Tests 2, 3, and 4 does not differ by more than 15% from the average effective stiffness as calculated from all cycles in that test at a specific frequency and temperature; and
2. The average maximum and minimum forces at zero displacement, effective stiffness (for damping devices exhibiting stiffness only), and average area of the hysteresis loop (E_{loop}), calculated for Section 15.8.1.2, Item 2c, shall fall within the limits specified by the RDP, as described by the nominal properties and the lambda factor for specification tolerance ($\lambda_{\text{spec,max}}$ and $\lambda_{\text{spec,min}}$) from Section 15.3.2.

15.8.2 Production Tests Before installation in a building, damping devices shall be tested in accordance with the requirements of this section. A test program for the production damping devices shall be established by the RDP. The test program shall validate the nominal properties by testing 100% of the devices for three cycles at 0.67 times the BSE-2X stroke or 1.33 times BSE-1X for an LPO at a frequency equal to $1/(1.5T_1)$. The measured values of the nominal properties shall fall within the limits provided in the project specifications. These limits shall agree with the specification tolerances on nominal design properties established in Section 15.3.

EXCEPTION: Production damping devices need not be subjected to this test program if it can be shown by other means that their properties meet the requirements of the project specifications. In such cases, the RDP shall establish an alternative program to ensure the quality of the installed damping devices. This alternative program shall include production testing of at least one device of each type and size, unless project-specific prototype tests have been conducted on that identical device type and size. Devices that undergo inelastic action or are otherwise damaged during this test shall not be used in construction.

15.9 LINEAR ANALYSIS PROCEDURES

15.9.1 Modeling of Energy Dissipation Devices

15.9.1.1 Displacement-Dependent Devices For evaluating the response of a displacement-dependent device from testing data, the force, F, in a displacement-dependent device shall be calculated in accordance with Equation (15-10):

$$F = K_{\text{eff}} D \tag{15-10}$$

where

K_{eff} = Effective stiffness of the device calculated in accordance with Equation (15-11), and

D = Relative displacement between two ends of the energy dissipation device.

$$K_{\text{eff}} = \frac{|F^+| + |F^-|}{|D^+| + |D^-|} \tag{15-11}$$

The forces in the device, F^+ and F^-, shall be evaluated at displacements D^+ and D^-, respectively.

15.9.1.2 Velocity-Dependent Devices

15.9.1.2.1 Solid Viscoelastic Devices The force in a solid viscoelastic device shall be determined in accordance with Equation (15-12):

$$F = K_{\text{eff}} D + C\dot{D} \tag{15-12}$$

where

C = Damping coefficient for the solid viscoelastic device;
D = Relative displacement between each end of the device;
$\dot{D}$ = Relative velocity between each end of the device; and
K_{eff} = Effective stiffness of the device calculated in accordance with Equation (15-13).

$$K_{\text{eff}} = \frac{|F^+| + |F^-|}{|D^+| + |D^-|} = K' \tag{15-13}$$

where K' is the storage stiffness.

The damping coefficient, C, for the device shall be calculated in accordance with Equation (15-14):

$$C = \frac{W_D}{\pi D_{\text{ave}}^2 \omega_1} = \frac{K''}{\omega_1} \tag{15-14}$$

where

K'' = Loss stiffness,
ω_1 = Angular frequency equal to $2\pi f_1$,
$f_1 = 1/T_1$,
D_{ave} = Average of the absolute values of displacements D^+ and D^- equal to $(|D^+| + |D^-|)/2$, and
W_D = Area enclosed by one complete cycle of the force–displacement response of the device.

15.9.2 Linear Static Procedure

15.9.2.1 Displacement-Dependent Devices Use of the linear static procedure (LSP) shall be permitted to analyze displacement-dependent energy dissipation devices provided that, in addition to the requirements of Section 15.4.1, the following requirements are satisfied:

1. The ratio of the maximum resistance in each story, in the direction under consideration, to the story shear demand calculated using Equations (7-25) and (7-26) shall range between 80% and 120% of the average value of the ratio for all stories. The maximum story resistance shall include the contributions from all components, elements, and energy dissipation devices; and
2. The maximum resistance of all energy dissipation devices in a story, in the direction under consideration, shall not exceed 50% of the resistance of the remainder of the framing where said resistance is calculated at the displacements anticipated in the BSE-2X or BSE-1X for an LPO. Aging and environmental effects shall be considered in calculating the maximum resistance of the energy dissipation devices.

The pseudo seismic force of Equation (7-21) shall be calculated with an S_a reduced by the damping modification factor, B_1, in Section 2.3.2 to account for the energy dissipation (damping) afforded by the energy dissipation devices. The damping modification factor, B_1, shall be calculated based on an effective damping ratio, β_{eff}, calculated in accordance with Equation (15-15):

$$\beta_{\text{eff}} = \beta + \frac{\sum_j W_j}{4\pi W_k} \tag{15-15}$$

where

β = Damping in the framing system and shall be set equal to 0.02 unless modified in Section 7.2.4.6;
W_j shall be taken as the work done by device j in one complete cycle corresponding to floor displacements δ_i, where the summation extends over all devices j; and
W_k = Maximum strain energy in the frame, determined using Equation (15-16):

$$W_k = \frac{1}{2}\sum_i F_i \delta_i \tag{15-16}$$

where F_i shall be taken as the inertia force at floor level i and the summation extends over all floor levels.

15.9.2.2 Velocity-Dependent Devices Use of the LSP shall be permitted to analyze velocity-dependent energy dissipation devices, provided that in addition to the requirements of Section 15.4.1, the following requirements are satisfied:

1. The maximum resistance of all energy dissipation devices in a story in the direction under consideration shall not exceed 50% of the resistance of the remainder of the framing where said resistance is calculated at the displacements anticipated in the BSE-2X or BSE-1X for an LPO. Aging and environmental effects shall be considered in calculating the maximum resistance of the energy dissipation devices; and
2. The pseudo seismic force of Equation (7-21) shall be calculated with an S_a reduced by the damping modification factors, B_1, in Section 2.3.2 to account for the energy dissipation (damping) provided by the energy dissipation devices. The damping modification factor, B_1, shall be calculated based on an effective damping ratio, β_{eff}, calculated in accordance with Equation (15-17):

$$\beta_{\text{eff}} = \beta + \frac{\sum_j W_j}{4\pi W_k} \tag{15-17}$$

where

β = Damping in the structural frame and shall be set equal to 0.02 unless modified in Section 7.2.4.6;
W_j = Work done by device j in one complete cycle corresponding to floor displacements δ_i, the summation extends over all devices j; and
W_k = Maximum strain energy in the frame, determined using Equation (15-16).

The work done by linear viscous device j in one complete cycle of loading shall be calculated in accordance with Equation (15-18):

$$W_j = \frac{2\pi^2}{T} C_j \delta_{rj}^2 \tag{15-18}$$

where

T = Fundamental period of the building including the stiffness of the velocity-dependent devices,
C_j = Damping constant for device j, and
δr_j = Relative displacement between the ends of device j along the axis of device j.

Calculation of effective damping in accordance with Equation (15-19) rather than Equation (15-17) shall be permitted for linear viscous devices:

$$\beta_{\text{eff}} = \beta + \frac{T\sum_j C_j \cos^2\theta_j \phi_{rj}^2}{4\pi \sum_i \left(\frac{w_i}{g}\right)\phi_i^2} \tag{15-19}$$

where

θ_j = Angle of inclination of device j to the horizontal,
ϕ_{rj} = First mode relative displacement between the ends of device j in the horizontal direction,
w_i = Reactive weight of floor level i, and
ϕ_i = First mode displacement at floor level i.

15.9.2.3 Design Actions The design actions for components of the building shall be calculated for both the upper- and lower-bound properties of Section 15.3 in three distinct stages of deformation as follows. The maximum action shall be used for design.

1. *At the Stage of Maximum Drift.* The seismic forces at each level, F_x, of the building shall be calculated using Equation (7-25), where V is replaced with the modified equivalent base shear, V^*.
2. *At the Stage of Maximum Velocity and Zero Drift.* The viscous component of force in each energy dissipation device shall be calculated by Equation (15-7) or (15-12) where the relative velocity, $\dot{D}$, is given by $2\pi f_1 D$, where D is the relative displacement between the ends of the device calculated at the stage of maximum drift. The calculated viscous forces shall be applied to the mathematical model of the building at the points of attachment of the devices and in directions consistent with the deformed shape of the building at maximum drift. The horizontal inertia forces at each floor level of the building shall be applied concurrently with the viscous forces so that the horizontal displacement of each floor level is zero.
3. *At the Stage of Maximum Floor Acceleration.* Design actions in components of the building shall be determined as the sum of actions determined at the stage of maximum drift times CF_1 and actions determined at the stage of maximum velocity times CF_2, where

$$CF_1 = \cos[\tan^{-1}(2\beta_{\text{eff}})] \tag{15-20}$$

$$CF_2 = \sin[\tan^{-1}(2\beta_{\text{eff}})] \tag{15-21}$$

in which β_{eff} is defined by either Equation (15-17) or Equation (15-19).

15.9.2.4 Linear Dynamic Procedure If the linear dynamic procedure (LDP) is selected based on the requirements of Section 15.4.1 and Section 7.3, the LDP of Section 7.4.2 shall be followed unless explicitly modified by this section.

Use of the response-spectrum method of the LDP shall be permitted where the effective damping in the fundamental mode of the building, in each principal direction, does not exceed 30% of critical.

15.9.2.5 Displacement-Dependent Devices Application of the LDP for the analysis of buildings incorporating displacement-dependent devices shall comply with the restrictions set forth in Section 15.9.1.1.

For analysis by the response-spectrum analysis method, modification of the 5% damped response spectrum shall be permitted to account for the damping afforded by the displacement-dependent energy dissipation devices. The 5% damped acceleration spectrum shall be reduced by the modal-dependent damping modification factor, B_1, for periods in the vicinity of the mode under consideration; the value of B_1 is different for each mode of vibration. The damping modification factor in each significant mode shall be determined in accordance with Section 2.3.2, and the calculated effective damping in that mode. The effective damping shall be determined using a procedure similar to that described in Section 15.9.2.1.

If the maximum base shear force calculated by dynamic analysis is less than 80% of the modified equivalent base shear of Section 15.9.2.1, component and element actions and deformations shall be proportionally increased to correspond to 80% of the modified equivalent base shear of Section 15.9.2.1.

15.9.2.6 Velocity-Dependent Devices For analysis by the response-spectrum analysis method, modification of the 5% damped response spectrum shall be permitted to account for the damping afforded by the velocity-dependent energy dissipation devices. The 5% damped acceleration spectrum shall be reduced by the modal-dependent damping modification factor, B_1, for periods in the vicinity of the mode under consideration; note that the value of B_1 is different for each mode of vibration. The damping modification factor in each significant mode shall be determined in accordance with Section 2.3.2, and the calculated effective damping in that mode.

The effective damping in the mth mode of vibration ($\beta_{\text{eff},m}$) shall be calculated in accordance with Equation (15-22):

$$\beta_{\text{eff},m} = \beta + \frac{\sum_j W_{mj}}{4\pi W_{mk}} \tag{15-22}$$

where

β_m = mth mode damping in the building frame;
W_{mj} = Work done by device j in one complete cycle corresponding to modal floor displacements, δ_{mi}; and
W_{mk} = Maximum strain energy in the frame in the mth mode, determined using Equation (15-23):

$$W_{mk} = \frac{1}{2}\sum F_{mi}\delta_{mi} \tag{15-23}$$

where

F_{mi} = mth mode horizontal inertia force at floor level i, and
δ_{mi} = mth mode horizontal displacement at floor level i.

The work done by linear viscous device j in one complete cycle of loading in the mth mode may be calculated in accordance with Equation (15-24):

$$W_{mj} = \frac{2\pi^2}{T_m} C_j \delta_{mrj}^2 \tag{15-24}$$

where

T_m = mth mode period of the building, including the stiffness of the velocity-dependent devices;
C_j = Damping constant for device j; and
δ_{mrj} = mth mode relative displacement between the ends of device j along the axis of device j.

In addition to direct application of the response-spectrum analysis method in accordance with this section to obtain member actions at maximum drift, member actions at maximum velocity, and maximum acceleration in each significant mode shall be determined using the procedure described in Section 15.9.2.3. The combination factors CF_1 and CF_2 shall be determined based on Equations (15-20) and (15-21) using $\beta_{\text{eff},m}$ for the mth mode.

If the maximum base shear force calculated by dynamic analysis is less than 80% of the modified equivalent base shear of Section 15.9.2.2, component and element actions and deformations shall be proportionally increased to correspond to 80% of the modified equivalent base shear of Section 15.9.2.2.

15.10 NONLINEAR STATIC PROCEDURE

If the nonlinear static procedure (NSP) is selected based on the requirements of Section 15.4 and Section 7.3, the NSP of Section 7.4.3 shall be followed unless explicitly modified by this section.

The nonlinear mathematical model of the building shall include the nonlinear force–velocity–displacement characteristics of the energy dissipation devices explicitly and the mechanical characteristics of the components supporting the devices. Stiffness characteristics shall be consistent with the deformations corresponding to the target displacement and a frequency equal to the inverse of period T_e, as defined in Section 7.4.3.2.

Energy dissipation devices with stiffness and damping characteristics that are dependent on excitation frequency and/or temperature shall be modeled with characteristics consistent with (1) the deformations expected at the target displacement and (2) a frequency equal to the inverse of the effective period.

Equation (7-29) shall be used to calculate the target displacement.

15.10.1 Displacement-Dependent Devices The stiffness characteristics of the energy dissipation devices shall be included in the mathematical model.

15.10.2 Velocity-Dependent Devices The target displacement and the spectral acceleration, S_a in Chapter 7, shall be reduced to account for the damping added by the velocity-dependent energy dissipation devices. The damping effect shall be calculated in accordance with Equation (15-25):

$$\beta_{\text{eff}} = \beta + \frac{\sum_j W_j}{4\pi W_k} \qquad (15\text{-}25)$$

where

β = Damping in the structural frame and shall be set equal to 0.02 unless modified in Section 7.2.4.6;

W_j = Work done by device j in one complete cycle corresponding to floor displacements δ_j, the summation extends over all devices j; and

W_k = Maximum strain energy in the frame, determined using Equation (15-16).

The work done by device j in one complete cycle of loading shall be calculated based on Equation (15-26):

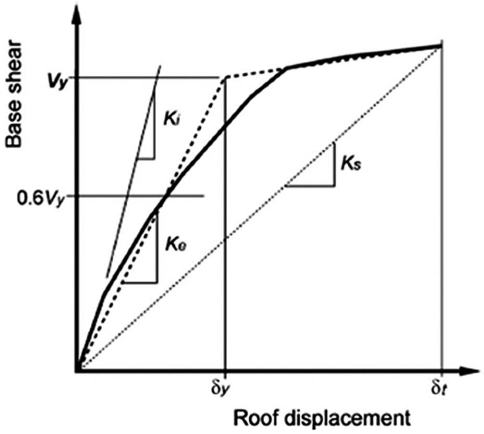

Figure 15-1. Calculation of Secant Stiffness, K_s.

$$W_j = \frac{2\pi^2}{T_{SS}} C_j \delta_{rj}^2 \qquad (15\text{-}26)$$

where

T_{ss} = Secant fundamental period of the building, including the stiffness of the velocity-dependent devices (if any), calculated using Equation (7-28) but replacing the effective stiffness, K_e, with the secant stiffness, K_s, at the target displacement, as shown in Figure 15-1;

C_j = Damping constant for device j; and

δ_{rj} = Relative displacement between the ends of device j along the axis of device j at a roof displacement corresponding to the target displacement.

The acceptance criteria of Section 7.5.3 shall apply to buildings incorporating energy dissipation devices. Checking for displacement-controlled actions shall use deformations corresponding to the target displacement. Checking for force-controlled actions shall use component actions calculated for three limit states: maximum drift, maximum velocity, and maximum acceleration. Maximum actions shall be used for design. Higher mode effects shall be explicitly evaluated.

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CHAPTER 16

SYSTEM-SPECIFIC PERFORMANCE PROCEDURES

16.1 SCOPE

This chapter provides alternate procedures for the seismic evaluation and retrofit of certain building types as indicated in this section. The application of an alternative procedure is limited to those conditions specified for the alternative procedure. When applied consistently with the limitations of the alternative procedure, the resulting modified structure is deemed to comply with the requirements of this standard as stated in the alternative procedure.

The individual procedures are provided as stand-alone procedures to be used on their own basis and not as a part of other procedures in this document. The basis for the individual procedures is that the information available warrants its use for the particular system application and limitations to achieve the stated seismic Performance Objective. In each case, the standard contains the following items:

1. A specification of the structural systems and circumstances in which the procedure may be used. This specification includes the limits on application of the procedure in lieu of other procedures of this standard.
2. The seismic Performance Objective of the building using the alternate procedure; the same terminology is used for specification of the seismic Performance Objective and hazard levels as used in this standard for the specified Seismic Hazard Level.
3. The specific technical procedures.

In each case, application of the system-specific performance procedure is whole unto itself and is considered to be an acceptable alternative to achieve the stated seismic Performance Objective compared with other approaches contained within this standard.

16.2 SPECIAL PROCEDURE FOR UNREINFORCED MASONRY

16.2.1 Scope This procedure shall be permitted to meet a Limited Performance Objective Section 2.2.3 for unreinforced masonry bearing wall buildings meeting the limitations of this section. Specifically, the procedure shall be permitted for Limited Performance Objective, Item 3 of Section 2.2.3, the Collapse Prevention Performance Level (S-5) for BSE-1E Seismic Hazard Level demands. This special procedure is consistent with the Tier 2 deficiency-based procedures of Chapter 5 for the Performance Objective indicated but is not permitted for Tier 3 systematic evaluation and retrofit in accordance with Chapter 6.

This special procedure shall be permitted to apply to unreinforced masonry bearing wall buildings with the following characteristics:

- Flexible diaphragms at all levels above the base of the structure;
- Vertical elements of the seismic-force-resisting system consisting of unreinforced masonry shear walls or a combination of predominantly unreinforced masonry and incidental concrete shear walls;
- A minimum of two lines of walls in each principal direction, except for single-story buildings with an open front on one side; and
- A maximum of six stories above the base of the structure.

16.2.2 Condition of Existing Materials Existing materials used as part of the required vertical-load-carrying or seismic-force-resisting system shall be evaluated using the on-site investigation provisions in Section 4.2.1 and Table 4-1. If existing masonry is determined to be in poor condition including degraded mortar, degraded masonry units, or significant cracking, the masonry shall be repaired, enhanced, removed, or replaced with new materials. Deteriorated mortar joints shall be repaired by pointing in accordance with Section 11.2.2.5 and shall be retested. Existing significant cracks in solid-unit unreinforced and in solid grouted hollow-unit masonry shall be repaired.

16.2.2.1 Layup of Walls Unreinforced masonry walls shall be laid in a running bond pattern.

16.2.2.1.1 Headers in Multiwythe Solid Brick The facing and backing wythes of multiwythe walls shall be bonded so that not less than 10% of the exposed face area is composed of solid headers extending not less than 4 in. (101.6 mm) into the backing. The clear distance between adjacent full-length headers shall not exceed 24 in. (609.6 mm) vertically or horizontally. Where backing consists of two or more wythes, the headers shall extend not less than 4 in. (101.6 mm) into the most distant wythe, or the backing wythes shall be bonded together with separate headers for which the area and spacing conform to the foregoing. Wythes of walls not meeting these requirements shall be considered veneer and shall not be included in the effective thickness used in calculation of the height-to-thickness ratio and shear strength of the wall.

EXCEPTION: Where S_{X1} is 0.3 or less, veneer wythes anchored and made composite with backup masonry are permitted to be used for calculation of the effective thickness.

16.2.2.1.2 Concrete Masonry Units and Structural Clay Load-Bearing Wall Tile Grouted or ungrouted hollow concrete masonry units shall be in accordance with ASTM C140. Grouted or ungrouted structural clay load-bearing wall tile shall be in accordance with ASTM C34.

16.2.2.1.3 Walls with Other Layups Layup patterns other than those previously specified are allowed if their performance can be justified.

16.2.2.2 Testing All unreinforced masonry (URM) walls used to carry vertical loads or resist seismic forces parallel and perpendicular to the wall plane shall be tested. The shear tests shall be taken at locations representative of the mortar conditions throughout the building. Test locations shall be determined by the design professional in charge. Results of all tests and their locations shall be recorded.

The minimum number of tests per masonry class shall be determined as follows:

- At each of both the first and top stories, not less than two tests per wall or line of wall elements providing a common line of resistance to seismic forces;
- At each of all other stories, not less than one test per wall or line of wall elements providing a common line of resistance to seismic forces; and
- Not less than one test per 1,500 ft^2 (457.2 m^2) of wall surface or less than a total of eight tests.

For masonry walls that use high shear strength mortar, masonry testing shall be performed in accordance with Section 16.2.2.2.2. The quality of mortar in all other coursed masonry walls shall be determined by performing tests in accordance with Section 16.2.2.2.1.

Collar joints of multiwythe masonry shall be inspected at the test locations during each in-place shear test, and estimates of the percentage of the surfaces of adjacent wythes that are covered with mortar shall be reported with the results of the in-place shear tests.

Existing unreinforced masonry shall be categorized into one or more classes based on shear strength, quality of construction, state of repair, deterioration, and weathering. Classes shall be defined for whole walls, not for small areas of masonry within a wall. Discretion in the definition of classes of masonry is permitted to avoid unnecessary testing.

Deteriorated mortar joints in unreinforced masonry walls shall be pointed in accordance with Section 11.2.2.5. Nothing shall prevent pointing of any masonry wall joints before tests are made.

16.2.2.2.1 In-Place Mortar Tests Mortar shear test values, v_{to}, shall be calculated for each in-place shear test in accordance with Equation (16-1) when testing is performed in accordance with ASTM C1531. Individual unreinforced masonry walls with more than 50% of mortar test values, v_{to}, less than 30 lb/in.^{2} (206.8 kN/m^2) shall be pointed and retested:

$$v_{to} = \frac{V_{\text{test}}}{A_b} - P_{D+L} \tag{16-1}$$

where

V_{test} = Load at first observed movement,
A_b = Total area of the bed joints above and below the test specimen, and
P_{D+L} = Stress resulting from actual dead plus live loads in place at the time of testing.

The lower-bound mortar shear strength, v_{tL}, is defined as the mean minus one standard deviation of the mortar shear test values, v_{to}. Unreinforced masonry with mortar shear strength, v_{te}, less than 30 lb/in.^{2} (206.8 kN/m^2) shall be pointed and retested, or shall have its structural function replaced, and shall be anchored to supporting elements in accordance with Section 16.2.4.

When existing mortar in any wythe is pointed to increase its shear strength and is retested, the condition of the mortar in the adjacent bed joints of the inner wythe or wythes and the opposite outer wythe shall be examined for the extent of deterioration. The shear strength of any wall class shall be no greater than that of the weakest wythe of the class.

16.2.2.2.2 Masonry The tensile-splitting strength, f_{sp}, of existing masonry using high-strength mortar shall be determined in accordance with ASTM C496 and calculated in accordance with Equation (16-2):

$$f_{sp} = \frac{2P_{\text{test}}}{\pi a_n} \tag{16-2}$$

where

P_{test} = Splitting test load, and
a_n = Diameter of core multiplied by its length or area of the side of a square prism.

The minimum average value of tensile-splitting strength, f_{sp}, as calculated by Equation (16-2), shall be 50 lb/in.^{2} (344.7 kN/m^2). Individual unreinforced masonry with tensile-splitting strength, f_{sp}, less than 50 lb/in.^{2} (344.7 kN/m^2) shall be pointed and retested or shall have its structural function replaced, and shall be anchored to supporting elements in accordance with Section 16.2.4.

The lower-bound mortar shear strength, f_{spL}, is defined as the mean minus one standard deviation of the tensile-splitting test values, f_{sp}.

16.2.2.2.3 Wall Anchors Wall anchors used as part of the required tension anchors shall be tested in pullout.

EXCEPTION: New anchors that extend through the wall with a 30 in.^{2} (0.76 m^2) minimum steel plate on the far side of the wall need not be tested.

Results of all tests shall be reported to the Authority Having Jurisdiction. The report shall include the test results as related to anchor size and type, orientation of loading, details of the anchor installation and embedment, wall thickness, and joint orientation.

A minimum of four anchors per floor shall be tested but not less than 10% of the total number of tension anchors at each level. A minimum of two tests per floor shall occur at walls with joists framing into the wall, and two tests per floor shall occur at walls with joists parallel to the wall. A minimum of 5% of all bolts that do not extend through the wall shall be subject to a direct-tension test, and an additional 20% shall be tested using a calibrated torque wrench in accordance with Section 16.2.2.2.3.2. The strength of the wall anchors shall be the average of the tension test values for anchors having the same wall thickness and framing orientation.

16.2.2.2.3.1 Direct-Tension Testing of Existing and New Anchors The test apparatus for testing wall anchors shall be supported by the masonry wall. The test procedure for prequalification of tension and shear anchors shall comply with ASTM E488, except, where obstructions occur, the distance between the anchor and the test apparatus support is permitted to be less than one-half the wall thickness and 75% of the embedment for new embedded anchors. Existing wall anchors shall be given a preload of 300 lb (1.3 kN) before establishing a datum for recording elongation. The tension test load reported shall be recorded at 1/8 in. (3.2 mm) relative movement of the anchor and the adjacent masonry surface. New embedded tension anchors shall be subject to a direct-tension load of not less than 2.5 times the design load but not less than 1,500 lb (6.7 kN) for 5 min.

16.2.2.2.3.2 Torque Testing of New Anchors Anchors embedded in unreinforced masonry walls shall be tested using a torque-calibrated wrench to the following minimum torques:

1/2 in. diameter bolts: 40 ft lb (13 mm diameter bolts: 54 Nm)
5/8 in. diameter bolts: 50 ft lb (16 mm diameter bolts: 68 Nm)
3/4 in. diameter bolts: 60 ft lb (19 mm diameter bolts: 81 Nm)

Table 16-1. Strengths of Anchors in Unreinforced Masonry Walls.

	New Materials or Configuration of Materials	Strength Values
Tension anchors[a,b,c]	Anchors extending entirely through unreinforced masonry wall secured with bearing plates on far side of a wall with at least 30 in.^{2} (762 mm^2) of area.	5,400 lb (24 kN) per anchor for three-wythe minimum walls; 2,700 lb (12 kN) for two-wythe walls.
Shear anchors[b,c]	Anchors embedded a minimum of 8 in. (203.2 mm) into unreinforced masonry walls; anchors should be centered in 2-1/2 in. (63.5 mm) diameter holes with dry-pack or nonshrink grout around the circumference of the anchor.	The value for plain masonry specified for solid masonry in TMS 402; no value larger than those given for 3/4-in. (19 mm) anchors should be used.
Combined tension and shear anchors[b,c]	Through-bolts and anchors: anchors meeting the requirements for shear and for tension anchors.	Tension: same as for tension anchors. Shear: same as for shear bolts.
	Embedded anchors: anchors extending to the exterior face of the wall with a 2-1/2 in. (63.5 mm) round plate under the head, drilled at an angle of 22-1/2 degrees to the horizontal; installed as specified for shear anchors.[d]	Tension: 3,600 lb (16 kN) per bolt. Shear: same as for shear bolts.

[a] Anchors to be 1/2 in. (12.7 mm) minimum in diameter.

[b] Drilling for anchors shall be done with an electric rotary drill; impact tools should not be used for drilling holes or tightening anchors and shear bolt nuts.

[c] An alternative adhesive anchor system is permitted to be used providing (a) its properties and installation conform to an ICC Evaluation Service Report or equivalent evaluation report; and (b) the report states that the system's use is in unreinforced masonry as an acceptable alternative to the *International Existing Building Code*'s Sections A107.4 and A113.1, or TMS 402, Section 2.1.4. The report's allowable values shall be multiplied by a factor of 3 to obtain lower-bound strength values, and the strength reduction factor ϕ shall be taken equal to 1.0.

[d] Embedded anchors shall be tested as specified in Section 16.2.2.2.3.

16.2.2.2.3.3 Prequalification Tests for Nonconforming Anchors ASTM E488 or the test procedure in Section 16.2.2.2.3.1 are permitted to be used to determine tension or shear strength values greater than those permitted by Table 16-1. Anchors shall be installed in the same manner and using the same materials as will be used in the actual construction. A minimum of five tests for each bolt size and type shall be performed for each class of masonry in which they are proposed to be used. The tension and shear strength values for such anchors shall be the lesser of the average ultimate load divided by 5.0 or the average load at which 1/8 in. (3.2 mm) elongation occurs for each size and type of anchor and class of masonry.

16.2.2.3 Masonry Strength

16.2.2.3.1 Shear Strength The lower-bound unreinforced masonry strength, v_{mL}, shall be determined for each masonry class from one of the following:

- When testing in accordance with Section 16.2.2.2.1 is performed, v_{mL} shall be determined by Equation (16-3):

$$v_{mL} = \frac{0.75\left(0.75v_{tL} + \frac{P_D}{A_n}\right)}{1.5} \qquad (16\text{-}3)$$

- When testing in accordance with Section 16.2.2.2.2 is performed, v_{mL} shall be determined by Equation (16-4):

$$v_{mL} = \frac{0.75\left(f_{spL} + \frac{P_D}{A_n}\right)}{1.5} \qquad (16\text{-}4)$$

where

v_{tL} = Mortar shear strength calculated in Section 16.2.2.2.1,
f_{spL} = Tensile-splitting strength calculated in Section 16.2.2.2.2,
P_D = Superimposed dead load at the top of the pier under consideration (lb), and
A_n = Area of net mortared and/or grouted section of a wall or wall pier [in.^{2} (mm^2)].

16.2.2.3.2 Masonry Compression Where any increase in wall dead plus live load compression stress occurs, the maximum compression stress in unreinforced masonry, Q_G/A_n, shall not exceed 300 lb/in.^{2} (2,068 kN/m^2).

16.2.2.3.3 Masonry Tension Unreinforced masonry shall be assumed to have no tensile capacity.

16.2.2.3.4 Foundations For existing foundations, new total dead loads are permitted to be increased over the existing dead load by 25%. New total dead load plus live load plus seismic forces may be increased over the existing dead load plus live load by 50%. Higher values may be justified only in conjunction with a geotechnical investigation and Chapter 8.

16.2.3 Analysis The URM special procedures for shear wall and diaphragm analysis requirements shall be in accordance with this section. The analysis requirements for other components and systems of URM buildings shall be in accordance with Section 16.2.4.

Material strengths for new elements shall be as prescribed in the applicable material standards referenced in Chapters 9 through 12, unless otherwise required by this section. The strength reduction factor, ϕ, shall be taken equal to 1.0. Specified values rather than expected values for nominal strength, as defined in Chapters 9 through 12 and the applicable referenced material standards, shall be used in the strength determination.

16.2.3.1 Cross Walls

16.2.3.1.1 General Only wood-framed walls sheathed with materials listed in Table 16-2 may be considered as cross walls. Cross walls shall not be spaced more than 40 ft (12.2 m) on center, measured perpendicular to the direction under consideration, and should be present in each story of the building. Cross walls shall extend the full story height between diaphragms. Cross walls shall have a length-to-height ratio between openings equal to or greater than 1.5. Addition of new wood-framed cross

Table 16-2. Cross Wall Shear Strengths.*

Material[a] and Configuration	Seismic Shear Strength[b,c] (lb/ft)
Plaster on wood or metal lath	600
Plaster on gypsum lath	550
Gypsum wallboard, unblocked edges	200
Gypsum wallboard, blocked edges	400
Existing wood structural use panels applied directly over wood studs	600
New wood structural use panels applied over wood sheathing	600
Plywood sheathing applied over existing plaster	0
Existing drywall or plaster applied directly over wood studs	230
Drywall or plaster applied to sheathing over existing wood studs	0
New structural wood use panels applied directly over wood studs	Expected strength
New drywall or plaster applied directly over wood studs	Expected strength

*Table is given in customary units only.

[a] Materials shall conform to the existing condition criteria in accordance with Chapter 4.

[b] Shear values are per side of wall and are permitted to be combined. However, total combined value shall not exceed 900 lb/ft (408.2 kg/m).

[c] No increase in stress is allowed.

walls is permitted to satisfy Section 16.2.3.2.2 provided that they are sheathed with materials listed in Table 16-2. New cross wall connections to the diaphragm shall develop the cross wall shear capacity.

EXCEPTIONS:

1. Cross walls need not be present at all levels in accordance with Section 16.2.3.2.2;
2. Cross walls that meet the following requirements need not be continuous:
   - Shear connections and anchorage at all edges of the diaphragm shall meet the requirements of Section 16.2.3.2.6;
   - Cross walls shall have a shear strength of 0.5 $S_{X1}\Sigma W_d$ and shall interconnect the diaphragm to the foundation; and
   - Diaphragms spanning between cross walls that are continuous shall comply with Equation (16-5):

$$\frac{2.1 S_{X1} W_d + V_{ca}}{2 v_u D} \leq 2.5 \qquad (16\text{-}5)$$

where

S_{X1} = Spectral response acceleration parameter at a 1 s period;
W_d = Total dead load tributary to the diaphragm, pound;
V_{ca} = Total shear strength of cross walls in the direction of analysis immediately above the diaphragm level being evaluated, pound;
v_u = Unit shear strength of diaphragm, (lb/ft); and
D = Depth of diaphragm, feet.

16.2.3.1.2 Shear Strength Within any 40 ft (12.2 m) measured along the span of the diaphragm, the sum of the cross wall shear strengths shall be greater than or equal to 30% of the diaphragm shear strength of the strongest diaphragm at or above the level under consideration. The values in Table 16-2 may be assumed for cross wall strengths for the purposes of this procedure.

16.2.3.2 Diaphragms

16.2.3.2.1 Shear Strength The values in Table 16-3 may be assumed for diaphragm strengths for the purposes of this procedure.

16.2.3.2.2 Demand–Capacity Ratios Demand–capacity ratios (DCRs) shall be evaluated when S_{X1} exceeds 0.20. Demand–capacity ratios shall be calculated for a diaphragm at any level in accordance with Equations (16-6) through (16-9):

- Diaphragms without cross walls at levels immediately above or below:

$$DCR = \frac{2.1 S_{X1} W_d}{\Sigma v_u D} \qquad (16\text{-}6)$$

- Diaphragms in a one-story building with cross walls:

$$DCR = \frac{2.1 S_{X1} W_d}{\Sigma v_u D + V_{cb}} \qquad (16\text{-}7)$$

- Diaphragms in a multistory building with cross walls at all levels:

$$DCR = \frac{2.1 S_{X1} \Sigma W_d}{\Sigma(\Sigma v_u D) + V_{cb}} \qquad (16\text{-}8)$$

  DCR shall be calculated at each level for the set of diaphragms at and above the level under consideration. In addition, the roof diaphragm shall also meet the requirements of Equation (16-7).

- Roof diaphragms and the diaphragms directly below if coupled by cross walls where walls do not exist at all levels:

$$DCR = \frac{2.1 S_{X1} \Sigma W_d}{\Sigma(\Sigma v_u D)} \qquad (16\text{-}9)$$

where

S_{X1} = Spectral response acceleration parameter at a 1 s period;
W_d = Total dead load tributary to the diaphragm, pounds;
V_{cb} = Total shear strength of cross walls in the direction of analysis immediately below the lowest diaphragm level being evaluated, pounds;
v_u = Unit shear strength of diaphragm, (lb/ft); and
D = Depth of diaphragm, feet.

16.2.3.2.3 Acceptability Criteria The intersection of diaphragm span between vertical lateral-force-resisting elements that meet

Table 16-3. Diaphragm Shear Strengths.

Material[a] and Configuration	Seismic Shear Strength (lb/ft)
Roofs with straight sheathing and roofing applied directly to sheathing	300
Roofs with diagonal sheathing and roofing applied directly to sheathing	750
Floors with straight tongue-and-groove sheathing	300
Floors with straight sheathing and finished wood flooring with board edges offset or perpendicular	1,500
Floors with diagonal sheathing and finished wood flooring	1,800
Metal deck[b]	1,800
Metal deck welded for seismic resistance[c]	3,000
Plywood sheathing applied directly over existing straight sheathing with ends of plywood sheets bearing on joists or rafters and edges of plywood located on center of individual sheathing boards	675

[a] Materials shall conform to the existing condition criteria in accordance with Chapter 4.
[b] Minimum 22-gauge steel deck with welds to support at a maximum average spacing of 12 in. (304.8 mm).
[c] Minimum 22-gauge steel deck with 3/4 in. (19 mm) diameter plug welds at a maximum average spacing of 8 in. (203.2 mm) and with sidelap welds, screws, or button punches at a spacing of 24 in. (609.6 mm) or less.
Note: Values are taken from ABK (1981).

the drift requirements of Section 16.2.3.5.6, *L*, and the DCR shall be located within Region 1, 2, or 3 on Figure 16-1.

16.2.3.2.4 Chords and Collectors An analysis for diaphragm flexure need not be made, and chords need not be present.

Where walls do not extend the length of the diaphragm, collectors shall be present. The collectors shall be able to transfer diaphragm shears calculated in accordance with Section 16.2.3.2.6 into the shear walls.

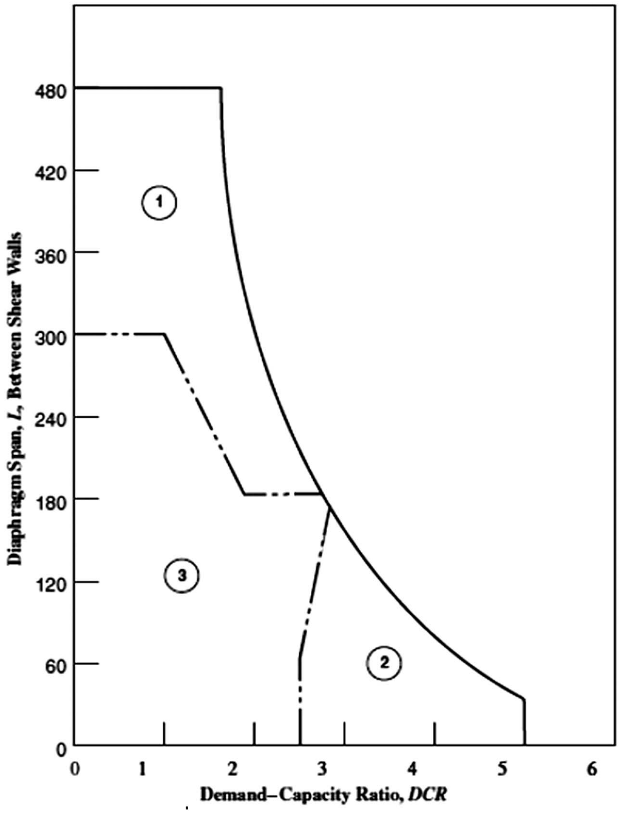

Figure 16-1. Demand–capacity ratio for diaphragms between shear walls.

16.2.3.2.5 Diaphragm Openings Diaphragm forces at corners of openings shall be investigated.

There shall be sufficient capacity to develop the strength of the diaphragm at opening corners.

The demand–capacity ratio shall be calculated and evaluated in accordance with Sections 16.2.3.2.2 using Equation (16-6) and Section 16.2.3.2.3 for the portion of the diaphragm adjacent to an opening using the opening dimension as the diaphragm span.

The demand–capacity ratio shall be calculated and evaluated in accordance with Sections 16.2.3.2.2 using Equation (16-6) and 16.2.3.2.3 for openings occurring in the end quarter of the diaphragm span. The diaphragm strength, $v_u D$, shall be based on the net depth of the diaphragm.

16.2.3.2.6 Diaphragm Shear Transfer Diaphragm shear transfer shall be evaluated when S_{X1} exceeds 0.133. Diaphragms shall be connected to shear walls at each end and shall be able to develop the minimum of the forces calculated in accordance with Equations (16-10) and (16-11):

$$V_d = 1.25 S_{X1} C_p W_d \tag{16-10}$$

$$V_d = v_u D \tag{16-11}$$

where

S_{X1} = Response acceleration parameter at a 1 s period;
W_d = Total dead load tributary to diaphragm, pounds;
v_u = Unit shear strength of diaphragm, (lb/ft);
D = Depth of diaphragm, feet; and
C_p = Horizontal force factor (Table 16-4).

Table 16-4. Horizontal Force Factor, C_p.

Configuration of Materials	C_p
Roofs with straight or diagonal sheathing and roofing applied directly to the sheathing, or floors with straight tongue-and-groove sheathing	0.50
Diaphragm with double or multiple layers of boards with edges offset, and blocked structural panel systems	0.75

16.2.3.3 Shear Walls

16.2.3.3.1 Shear Wall Actions In-plane shear shall be evaluated when S_{X1} exceeds 0.133. The story force distributed to a shear wall at any diaphragm level shall be determined in accordance with Equations (16-12) and (16-13):

$$F_{wx} = 0.8S_{X1}(W_{wx} + 0.5W_d) \tag{16-12}$$

but not exceeding

$$F_{wx} = 0.8S_{X1}W_{wx} + v_uD \tag{16-13}$$

The wall story shear shall be calculated in accordance with Equation (16-14):

$$V_{wx} = \Sigma F_{wx} \tag{16-14}$$

where

S_{X1} = Spectral response acceleration parameter at a 1 s period;
W_{wx} = Dead load of an unreinforced masonry wall assigned to level x, taken from midstory below level x to midstory above level x, pounds;
W_d = Total dead load tributary to the diaphragm, inches pounds;
v_u = Unit shear strength of diaphragm, lb/ft; and
D = Depth of diaphragm, feet.

16.2.3.3.2 Shear Wall Strengths The shear wall strength shall be calculated in accordance with Equation (16-15):

$$V_a = v_{mL}A_n \tag{16-15}$$

where

v_{Ml} = Lower-bound masonry shear strength, lb/in.^{2}, calculated in accordance with Section 16.2.2.3.1; and
A_n = Area of net mortared or grouted section of wall or pier.

The wall or pier rocking shear strength shall be calculated in accordance with Equations (16-16) and (16-17):

For walls without openings:

$$V_r = 0.9(P_D + 0.5P_w)\frac{D}{H} \tag{16-16}$$

For walls with openings:

$$V_r = 0.9P_D\frac{D}{H} \tag{16-17}$$

where

P_D = Superimposed dead load at the top of the pier under consideration, pounds;
P_W = Weight of wall, pounds;
D = In-plane width dimension of masonry, inches; and
H = Least clear height of opening on either side of pier, inches

16.2.3.3.3 Shear Wall Acceptance Criteria The acceptability of unreinforced masonry shear walls shall be determined in accordance with Equations (16-18), (16-19), and (16-20):

When $V_r < V_a$

$$0.7V_{wx} < \Sigma V_r \tag{16-18}$$

When $V_a < V_r$, V_{wx} shall be distributed to the individual wall piers, V_p, in proportion to D/H, and Equations (16-19) and (16-20) shall be met:

$$V_p < V_a \tag{16-19}$$

$$V_p < V_r \tag{16-20}$$

If $V_p < V_a$ and $V_p > V_r$ for any pier, the pier shall be omitted from the analysis and the procedure shall be repeated.

16.2.3.4 Buildings with Open Fronts Single-story buildings with an open front on one side shall have cross walls parallel to the open front. The effective diaphragm span, L_i, for use in Figure 16-1, shall be calculated in accordance with Equation (16-21):

$$L_i = 2L\left(\frac{W_w}{W_d} + 1\right) \tag{16-21}$$

where

L = Span of diaphragm between shear wall and open front, feet;
W_w = Total weight of wall above open front; and
W_d = Total dead load tributary to the diaphragm, pounds.

The DCR shall be calculated in accordance with Equation (16-22):

$$DCR = \frac{2.1S_{X1}(W_d + W_w)}{(v_uD + V_{cb})} \tag{16-22}$$

where

S_{X1} = Spectral response acceleration parameter at a 1 s period;
v_u = Unit shear strength of diaphragm, lb/ft;
D = Depth of diaphragm, feet;
V_{cb} = Total shear strength of cross walls in the direction of analysis immediately below the diaphragm level being evaluated, pounds;
W_w = Total weight of wall above open front; and
W_d = Total dead load tributary to the diaphragm, pounds.

16.2.3.5 New Vertical Elements

16.2.3.5.1 General New vertical elements may be added to resist lateral forces.

16.2.3.5.2 Combinations of Vertical Elements

16.2.3.5.2.1 Lateral Force Distribution For vertical elements in the same line of resistance, lateral forces shall be distributed among the vertical elements in proportion to their relative rigidities. The masonry assemblage of units, mortar, and grout shall be considered to be a homogeneous medium for stiffness computations with an elastic modulus in compression, E_m, as specified in Section 11.2.3.4. The shear modulus, G_m, shall be permitted to be equal to $0.4E_m$. The stiffness of a URM wall or wall pier resisting seismic forces parallel to its plane shall be considered to be linear and proportional with the geometrical properties of the uncracked section, excluding veneer wythes. For vertical elements not in the same line, lateral forces shall be permitted to be distributed in accordance with the tributary area method. The existing masonry shall be evaluated and shall be adequate to resist the forces determined in accordance with Section 16.2.3.3 and distributed in proportion to relative rigidity, regardless of the design force used for new vertical elements.

EXCEPTION: The existing masonry is not required to have adequate capacity to resist the distributed forces if all the following conditions are met:

1. The new vertical elements are designed for 100% of the required forces on the wall line;

2. Truss, post, or beam supports per Section 16.2.4.4 are added at rafters, girders, and joists at that wall line; and
3. Vertical bracing per Section 16.2.4.2.2 is added at that wall line.

In addition, moment-resisting frames shall comply with Section 16.2.3.5.2.2.

16.2.3.5.2.2 Moment-Resisting Frames Moment-resisting frames shall not be used in combination with an unreinforced masonry wall in a single line of resistance unless the wall has piers that have adequate shear capacity to sustain rocking in accordance with Section 16.2.3.3.2. The frames shall be designed to carry 100% of the forces tributary to that line of resistance.

16.2.3.5.3 Wood Structural Panels Wood structural panels shall not be used to share lateral forces with other materials along the same line of resistance.

16.2.3.5.4 Forces on New Vertical Elements Story shear per Section 16.2.3.3 shall be used to determine forces on new and existing vertical lateral-force-resisting elements. The additional weight of new elements shall be included in the force determination.

16.2.3.5.5 Acceptance Criteria for New Vertical Elements New vertical elements shall satisfy the requirements of Section 16.2.3. Footings shall be provided for new vertical elements to transfer loads into the supporting soil. Existing footings supporting new vertical elements shall be evaluated per Section 16.2.2.3.4. Bearing pressure capacities used for new footings similar to existing footings shall be permitted to use the provisions of Section 16.2.2.3.4. For new footings that are not similar to existing footings, bearing pressure capacities shall be determined by a geotechnical investigation.

Table 16-5. Other Components and Systems Requirements for Unreinforced Masonry Buildings.

Other Components and Systems	Chapter, Section(s)
Continuity	7.2.12
Structures sharing common elements	7.2.14
Building separation	7.2.15
Anchored veneer	13.5, 13.6.1.2
URM partitions	13.5, 13.6.1.3, 13.6.2
Parapets and cornices	13.6.5, 13.4.4.1, 13.5
URM bearing wall out-of-plane demands	16.2.4.2
Wall anchorage	16.2.4.3
Truss and beam supports	7.2.12; 16.2.4.4
Diaphragm chords	16.2.3.2.4
Collectors	16.2.3.2.4

16.2.3.5.6 Drift Limits The story drift ratio for all new vertical elements shall be limited to 0.0075.

16.2.4 Other Components and Systems of Unreinforced Masonry Buildings

16.2.4.1 References to Applicable Sections Requirements for other components and systems of URM buildings are listed in Table 16-5.

16.2.4.2 Out-of-Plane Demands Where S_{X1} exceeds 0.133, the height-to-thickness ratios of all unreinforced masonry walls shall be less than or equal to the values in Table 16-6. For the purpose of Table 16-6 and this section, S_{X1} shall be permitted to be taken as the value of S_a at 1 s for the specified Seismic Hazard Level. Alternatively, the height-to-thickness ratios shall be permitted to be calculated per Section 11.3.3.3 with $C_{pl} = 1.1$. Walls not in compliance shall be strengthened in accordance with Section 16.2.4.2.1.

The following limitations shall apply to Table 16-6 when S_{X1} exceeds 0.4:

1. For a wall with both adjacent diaphragms in Region 1 of Figure 16-1 as defined in Section 16.2.3.2.3, height-to-thickness ratios in Column A of Table 16-6 are permitted to be used if cross walls comply with the requirements of Section 16.2.3.1 and are present in all stories;
2. For a wall with both adjacent diaphragms in Region 2 of Figure 16-1, as defined in Section 16.2.3.2.3, height-to-thickness ratios in Column A are permitted to be used; and
3. For a wall with both adjacent diaphragms in Region 3 of Figure 16-1, as defined in Section 16.2.3.2.3, height-to-thickness ratios in Column B are permitted to be used.

When diaphragms above and below the wall under consideration have demand–capacity ratios in different regions of Figure 16-1, the lesser of the height-to-thickness ratios shall be used.

16.2.4.2.1 Wall Bracing General Where a wall height-to-thickness ratio exceeds the specified limits of Table 16-6, the wall shall be required to be laterally supported by vertical bracing

Table 16-6. Allowable Height-to-Thickness Ratios of Unreinforced Masonry Walls.*

Wall Type	$0.133 \leq S_{X1} < 0.25$	$0.25 \leq S_{X1} < 0.4$	$0.4 \leq S_{X1} < 0.50$		$0.50 \leq S_{X1} < 0.60$		$0.60 \leq S_{X1}$	
			A	B	A	B	A	B
Walls of 1-story buildings	20	16	16[a,b]	13	13[a,b]	8	8	8
Top story of a multistory building	14	14	14[a,b]	9	9[a,b]	8	8	8
First story of a multistory building	20	18	16	15	15	11	11	8
All other conditions	20	16	16	13	13	10	10	8

*Table is given in customary units only.

[a] Value is permitted to be used when in-plane shear tests in accordance with Section 16.2.2.2.1 have a minimum ν_{tL} of 100 lb/in.^{2} or a minimum ν_{tL} of 60 lb/in.^{2} and a minimum of 50% mortar coverage of the collar joint.

[b] Values are permitted to be interpolated between Columns A and B where in-plane shear tests in accordance with Section 16.2.2.2.1 have a ν_{tL} between 30 and 60 lb/in.^{2} and a minimum of 50% mortar coverage of the collar joint.

members per Section 16.2.4.2.2 or by reducing the effective wall span by intermediate wall bracing in accordance with Section 16.2.4.2.3. Bracing shall be designed for the minimum of $0.4S_{xs}W$ or $0.1W$, where W is the weight of the wall per unit area.

16.2.4.2.2 Vertical Bracing Members Vertical bracing members shall be attached to the floor or foundation below and the roof or floor above to resist their force demands in accordance with Section 16.2.4.2.1, independently of required wall anchors. Horizontal spacing of vertical bracing members shall not exceed one-half of the unsupported height of the wall or 10 ft (3 m). Deflection of such bracing members at force demands in accordance with Section 16.2.4.2.1 shall not exceed one-tenth of the wall thickness.

16.2.4.2.3 Intermediate Wall Bracing The wall height shall be reduced by bracing elements connected to the floor or roof. Horizontal spacing of the bracing elements and wall anchors shall be as required by force demands in accordance with Section 16.2.4.2.1, but shall not exceed 6 ft (1.8 m) on center. Bracing elements shall be detailed to limit the horizontal displacement to one-tenth of the wall thickness.

16.2.4.3 Wall Anchorage Wall anchorage shall be evaluated when S_{X1} exceeds 0.067. Anchors shall be capable of developing the maximum of the following: $0.9S_{XS}$ times the weight of the wall, or 200 lb/ft (2,919 N/m), acting normal to the wall at the level of the floor or roof.

Walls shall be anchored at the roof and all floor levels at a spacing equal to or less than 6 ft (1.8 m) on center.

At the roof and all floor levels, anchors shall be provided within 2 ft (610 mm) horizontally from the inside corners of the wall.

The connection between the walls and the diaphragm shall not induce cross-grain bending or tension in the wood ledgers.

Anchors shall be located a minimum distance of 12 in. (305 mm) from wall openings or from the top of parapets.

EXCEPTION: If a reinforced beam or column is provided at the top of the wall or adjacent to the wall opening, the minimum distance is permitted to be 6 in. (152 mm).

16.2.4.3.1 Transfer of Anchorage Forces into Diaphragm The wall anchorage force in this section shall be fully developed into the diaphragm when S_{X1} exceeds 0.2. If subdiaphragms are used, each subdiaphragm shall be capable of transmitting the shear forces caused by wall anchorage to a continuous diaphragm crosstie. Subdiaphragms shall have length-to-depth ratios not exceeding 3:1.

Alternatively, the wood diaphragm systems listed in Section 16.2.4.3.1.1 shall be permitted to develop the wall anchorage as follows, but subdiaphragm analysis, crossties, and chords are not required:

1. For joists parallel to the masonry walls, the anchorage shall be developed a minimum of 8 ft (2.4 m) into the diaphragm.
2. For joists perpendicular to the masonry walls, anchors attached to joists 8 ft (2.4 m) or longer shall be deemed sufficient development. If joists are shorter than 8 ft (2.4 m) or if attachment is between joists, the wall anchorage shall be developed into the diaphragm similar to conditions where joists are parallel to the masonry walls as outlined in Item 1.

16.2.4.3.1.1 Wood Diaphragms Allowed in Alternate Method Wood diaphragms consisting of the following shall be permitted to use the alternate anchorage transfer without subdiaphragm analysis, crossties, and chords:

1. Diagonal sheathing overlaid with straight sheathing, finished wood flooring, or wood structural panel sheathing;
2. Double straight sheathing (with board edges offset or perpendicular);
3. Straight sheathing overlaid with wood structural panel sheathing (with panel edges offset);
4. Wood structural panel sheathing; or
5. Nail-laminated timber.

16.2.4.4 Truss and Beam Supports Where S_{X1} is greater than $0.3g$ and where trusses and beams other than rafters or joists are supported on masonry, independent secondary columns or equivalent components shall be installed to support vertical loads of the roof or floor members.

16.2.5 Detailing for New Elements New elements and systems shall conform with, at a minimum, the detailing requirements and strength design procedures as prescribed in the applicable material standards referenced in Chapters 9 through 12. Where prescribed by the applicable material standards, detailing shall also, at a minimum, conform with the requirements for “ordinary” systems. For reinforced concrete walls, 90-degree, 135-degree, or 180-degree hooks shall be provided on horizontal reinforcing at wall ends.

In addition, seismic-force-resisting systems that are Not Permitted (NP) in accordance with ASCE 7 for buildings assigned to Seismic Design Category C shall not be used for new vertical elements in regions of High Seismicity as defined in Section 2.5.

CHAPTER 17
TIER 1 CHECKLISTS

17.1 BASIC CHECKLISTS

17.1.1 Very Low Seismicity Checklist The Very Low Seismicity Checklist in Table 17-1 shall be completed for all building types in Very Low Seismicity being evaluated to the Collapse Prevention Performance Level only. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure (given as the section number in the provisions) listed next to each evaluation statement.

17.1.2 Basic Configuration Checklist The Collapse Prevention Basic Configuration Checklist in Table 17-2 shall be completed for all building types, except buildings in Very Low Seismicity, being evaluated to the Collapse Prevention Performance Level. The Immediate Occupancy Basic Configuration Checklist in Table 17-3 shall be completed for all building types being evaluated to the Immediate Occupancy Structural Performance Level. Once the appropriate Basic Configuration Checklist has been completed, complete the appropriate building type checklist in Sections 17.2 through Section 17.18 for the relevant building type and the desired performance level in accordance with Table 4-6. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

17.2 STRUCTURAL CHECKLISTS FOR BUILDING TYPES W1: WOOD LIGHT FRAMES, SMALL RESIDENTIAL

For building systems and configurations that comply with the W1 building type description in Table 3-1 the Collapse Prevention Structural Checklist in Table 17-4 shall be completed where required by Table 4-6 for Collapse Prevention Structural Performance, and the Immediate Occupancy Structural Checklist in Table 17-5 shall be completed where required by Table 4-6 for Immediate Occupancy Structural Performance. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

17.3 STRUCTURAL CHECKLISTS FOR BUILDING TYPE W2: WOOD FRAMES, LARGE RESIDENTIAL, COMMERCIAL, INDUSTRIAL, AND INSTITUTIONAL

For building systems and configurations that comply with the W2 building type description in Table 3-1, the Collapse Prevention Structural Checklist in Table 17-6 shall be completed where required by Table 4-6 for Collapse Prevention Structural Performance, and the Immediate Occupancy Structural Checklist in Table 17-7 shall be completed where required by Table 4-6 for Immediate Occupancy Structural Performance. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements

Table 17-1. Very Low Seismicity Checklist.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Structural Components			
C NC N/A U	LOAD PATH: The structure contains a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation.	5.4.1.1	A.2.1.1
C NC N/A U	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-2. Collapse Prevention Basic Configuration Checklist.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low Seismicity			
Building System—General			
C NC N/A U	LOAD PATH: The structure contains a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation.	5.4.1.1	A.2.1.1
C NC N/A U	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.25% of the height of the shorter building in low seismicity, 0.5% in moderate seismicity, and 1.5% in high seismicity.	5.4.1.2	A.2.1.2
C NC N/A U	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure.	5.4.1.3	A.2.1.3
Building System—Building Configuration			
C NC N/A U	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above.	5.4.2.1	A.2.2.2
C NC N/A U	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above.	5.4.2.2	A.2.2.3
C NC N/A U	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation.	5.4.2.3	A.2.2.4
C NC N/A U	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding 1-story penthouses and mezzanines.	5.4.2.4	A.2.2.5
C NC N/A U	MASS: There is no change in effective mass of more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered.	5.4.2.5	A.2.2.6
C NC N/A U	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. This statement does not apply to buildings with flexible diaphragms.	5.4.2.6	A.2.2.7
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Geologic Site Hazards			
C NC N/A U	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 ft (15.2 m) under the building.	5.4.3.1	A.6.1.1

continues

Table 17-2 *(Continued)*. Collapse Prevention Basic Configuration Checklist.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure.	5.4.3.1	A.6.1.2
C NC N/A U	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated.	5.4.3.1	A.6.1.3
High Seismicity (Complete the Following Items in Addition to the Items for Moderate Seismicity)			
Foundation Configuration			
C NC N/A U	TIES BETWEEN FOUNDATION ELEMENTS: For buildings supported on soils classified as Site Class D, DE, E, or F, the individual pile caps, piles, and piers are restrained by concrete beams or slabs adequate to resist seismic forces. For buildings supported on soils classified as Site Class E or F, individual spread footings are restrained by concrete beams or slabs adequate to resist seismic forces.	5.4.3.4	A.6.2.2

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-3. Immediate Occupancy Basic Configuration Checklist.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Building System—General			
C NC N/A U	LOAD PATH: The structure contains a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation.	5.4.1.1	A.2.1.1
C NC N/A U	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.5% of the height of the shorter building in low seismicity, 1.0% in moderate seismicity, and 3.0% in high seismicity.	5.4.1.2	A.2.1.2
C NC N/A U	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure.	5.4.1.3	A.2.1.3
Building System—Building Configuration			
C NC N/A U	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above.	5.4.2.1	A.2.2.2
C NC N/A U	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above.	5.4.2.2	A.2.2.3
C NC N/A U	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation.	5.4.2.3	A.2.2.4
C NC N/A U	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding 1-story penthouses and mezzanines.	5.4.2.4	A.2.2.5
C NC N/A U	MASS: There is no change in effective mass of more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered.	5.4.2.5	A.2.2.6
C NC N/A U	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. This statement does not apply to buildings with flexible diaphragms.	5.4.2.6	A.2.2.7

continues

Table 17-3 *(Continued)*. Immediate Occupancy Basic Configuration Checklist.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Geologic Site Hazards			
C NC N/A U	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 ft (15.2 m) under the building.	5.4.3.1	A.6.1.1
C NC N/A U	SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure.	5.4.3.1	A.6.1.2
C NC N/A U	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated.	5.4.3.1	A.6.1.3
Tsunami Hazards			
C NC N/A U	TSUNAMI: The building is not located within a Tsunami Design Zone as defined by ASCE 7 Chapter 6 or is located in a Tsunami Design Zone where the inundation depth per ASCE 7 Chapter 6 is less than 3 ft (0.9 m).	5.4.3.1	A.6.1.4
Moderate and High Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Foundation Configuration			
C NC N/A U	TIES BETWEEN FOUNDATION ELEMENTS: For buildings supported on soils classified as Site Class D, DE, E, or F, the individual pile caps, piles, and piers are restrained by concrete beams or slabs adequate to resist seismic forces. For buildings supported on soils classified as Site Class E or F, individual spread footings are restrained by concrete beams or slabs adequate to resist seismic forces.	5.4.3.4	A.6.2.2
C NC N/A U	DEEP FOUNDATIONS: Piles that are required to transfer lateral and/or overturning forces between the structure and the soil shall have a positive connection between the piles and the pile cap, foundation mat, grade beam, or other element of the building foundation system. Cast-in-place and precast non-prestressed piles shall have a minimum longitudinal reinforcement ratio of 0.0025 and transverse reinforcing spaced at no more than 6 in (152.4 mm) within a distance of three times the pile diameter from the bottom of the pile cap. Precast prestressesd piles shall have a minimum effective prestress of 400 psi and transverse reinforcing spaced at no more than 6 in. (152.4 mm) within a distance of 20 ft (6 m) from the top of the pile.		A.6.2.3
C NC N/A U	SLOPING SITES: The exterior grade difference from one side of the building to another does not exceed one story in height.		A.6.2.4

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-4. Collapse Prevention Structural Checklist for Building Type W1.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the following values: Structural panel sheathing 1,000 lb/ft (14.6 kN/m), Diagonal sheathing 700 lb/ft (10.2 kN/m), Straight sheathing 100 lb/ft (1.5 kN/m), and All other conditions 100 lb/ft (1.5 kN/m).	5.5.3.1.1	A.3.2.7.1
C NC N/A U	STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multistory buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system.	5.5.3.6.1	A.3.2.7.2

continues

Table 17-4 *(Continued)*. Collapse Prevention Structural Checklist for Building Type W1.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard is not used for shear walls on buildings more than one story high with the exception of the uppermost level of a multistory building.	5.5.3.6.1	A.3.2.7.3
C NC N/A U	NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces.	5.5.3.6.1	A.3.2.7.4
C NC N/A U	WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor.	5.5.3.6.2	A.3.2.7.5
C NC N/A U	HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-1.	5.5.3.6.3	A.3.2.7.6
C NC N/A U	CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels.	5.5.3.6.4	A.3.2.7.7
C NC N/A U	OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces.	5.5.3.6.5	A.3.2.7.8
Connections			
C NC N/A U	WOOD POSTS: There is a positive connection of wood posts to the foundation.	5.7.3.3	A.5.3.3
C NC N/A U	WOOD SILLS: All wood sills are bolted to the foundation.	5.7.3.3	A.5.3.4
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Connections			
C NC N/A U	WOOD SILL BOLTS: Sill bolts are spaced at 6 ft (1.8 m) or less with acceptable edge and end distance provided for wood and concrete.	5.7.3.3	A.5.3.7
Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 24 ft (7.3 m) and aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.2
C NC N/A U	OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-5. Immediate Occupancy Checklist for Building Type W1.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of nonlines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1

continues

Table 17-5 *(Continued)*. Immediate Occupancy Checklist for Building Type W1.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the following values: Structural panel sheathing 1,000 lb/ft (14.5 kN/m), Diagonal sheathing 700 lb/ft (10.2 kN/m), Straight sheathing 100 lb/ft (1.45 kN/m), and All other conditions 100 lb/ft (1.45 kN/m).	5.5.3.1.1	A.3.2.7.1
C NC N/A U	STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multistory buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system.	5.5.3.6.1	A.3.2.7.2
C NC N/A U	GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard is not used for shear walls on buildings more than one story high with the exception of the uppermost level of a multistory building.	5.5.3.6.1	A.3.2.7.3
C NC N/A U	NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces.	5.5.3.6.1	A.3.2.7.4
C NC N/A U	WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor.	5.5.3.6.2	A.3.2.7.5
C NC N/A U	HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-2.	5.5.3.6.3	A.3.2.7.6
C NC N/A U	CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels.	5.5.3.6.4	A.3.2.7.7
C NC N/A U	OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces.	5.5.3.6.5	A.3.2.7.8
Connections			
C NC N/A U	WOOD POSTS: There is a positive connection of wood posts to the foundation.	5.7.3.3	A.5.3.3
C NC N/A U	WOOD SILLS: All wood sills are bolted to the foundation.	5.7.3.3	A.5.3.4
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
Low, Moderate, and High Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	HOLD-DOWN ANCHORS: All shear walls have hold-down anchors attached to the end studs constructed in accordance with acceptable construction practices.	5.5.3.6.6	A.3.2.7.9
C NC N/A U	NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 1.5-to-1 are not used to resist seismic forces.	5.5.3.6.1	A.3.2.7.4
Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 12 ft (3.65 m) and aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.1 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.2

continues

Table 17-5 *(Continued)*. Immediate Occupancy Checklist for Building Type W1.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	WOOD SILL BOLTS: Sill bolts are spaced at 4 ft (1.2 m) or less, with acceptable edge and end distance provided for wood and concrete.	5.7.3.3	A.5.3.7

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-6. Collapse Prevention Structural Checklist for Building Type W2.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the following values: Structural panel sheathing 1,000 lb/ft (14.6 kN/m), Diagonal sheathing 700 lb/ft (10.2 kN/m), Straight sheathing 100 lb/ft (1.45 kN/m), and All other conditions 100 lb/ft (1.45 kN/m).	5.5.3.1.1	A.3.2.7.1
C NC N/A U	STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multistory buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system.	5.5.3.6.1	A.3.2.7.2
C NC N/A U	GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard is not used for shear walls on buildings more than one story high with the exception of the uppermost level of a multistory building.	5.5.3.6.1	A.3.2.7.3
C NC N/A U	NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces.	5.5.3.6.1	A.3.2.7.4
C NC N/A U	WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor.	5.5.3.6.2	A.3.2.7.5
C NC N/A U	HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-1.	5.5.3.6.3	A.3.2.7.6
C NC N/A U	CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels.	5.5.3.6.4	A.3.2.7.7
C NC N/A U	OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces.	5.5.3.6.5	A.3.2.7.8
Connections			
C NC N/A U	WOOD POSTS: There is a positive connection of wood posts to the foundation.	5.7.3.3	A.5.3.3
C NC N/A U	WOOD SILLS: All wood sills are bolted to the foundation.	5.7.3.3	A.5.3.4
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Connections			
C NC N/A U	WOOD SILL BOLTS: Sill bolts are spaced at 6 ft (1.8 m) or less with acceptable edge and end distance provided for wood and concrete.	5.7.3.3	A.5.3.7

continues

Table 17-6 *(Continued)*. Collapse Prevention Structural Checklist for Building Type W2.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 24 ft (7.3 m) and aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.2
C NC N/A U	OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-7. Immediate Occupancy Checklist for Building Type W2.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the following values: Structural panel sheathing 1,000 lb/ft (14.6 kN/m), Diagonal sheathing 700 lb/ft (10.2 kN/m), Straight sheathing 100 lb/ft (1.45 kN/m), and All other conditions 100 lb/ft (1.45 kN/m).	5.5.3.1.1	A.3.2.7.1
C NC N/A U	STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multistory buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system.	5.5.3.6.1	A.3.2.7.2
C NC N/A U	GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard is not used for shear walls on buildings more than one story high with the exception of the uppermost level of a multistory building.	5.5.3.6.1	A.3.2.7.3
C NC N/A U	NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces.	5.5.3.6.1	A.3.2.7.4
C NC N/A U	WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor.	5.5.3.6.2	A.3.2.7.5
C NC N/A U	HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-2.	5.5.3.6.3	A.3.2.7.6
C NC N/A U	CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels.	5.5.3.6.4	A.3.2.7.7

continues

Table 17-7 *(Continued)*. Immediate Occupancy Checklist for Building Type W2.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces.	5.5.3.6.5	A.3.2.7.8
C NC N/A U	HOLD-DOWN ANCHORS: All shear walls have hold-down anchors attached to the end studs constructed in accordance with acceptable construction practices.	5.5.3.6.6	A.3.2.7.9
Connections			
C NC N/A U	WOOD POSTS: There is a positive connection of wood posts to the foundation.	5.7.3.3	A.5.3.3
C NC N/A U	WOOD SILLS: All wood sills are bolted to the foundation.	5.7.3.3	A.5.3.4
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
Low, Moderate, and High Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 1.5-to-1 are not used to resist seismic forces.	5.5.3.6.1	A.3.2.7.4
Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 12 ft (3.6 m) and aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.2 m) and have aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.2
C NC N/A U	OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	WOOD SILL BOLTS: Sill bolts are spaced at 4 ft (1.2 m) or less with acceptable edge and end distance provided for wood and concrete.	5.7.3.3	A.5.3.7

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

17.4 STRUCTURAL CHECKLISTS FOR BUILDING TYPES S1: STEEL MOMENT FRAMES WITH STIFF DIAPHRAGMS, AND S1A: STEEL MOMENT FRAMES WITH FLEXIBLE DIAPHRAGMS

For building systems and configurations that comply with the S1 or S1a building type description in Table 3-1 the Collapse Prevention Structural Checklist in Table 17-8 shall be completed where required by Table 4-6 for Collapse Prevention Structural Performance, and the Immediate Occupancy Structural Checklist in Table 17-9 shall be completed where required by Table 4-6 for Immediate Occupancy Structural Performance. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

Table 17-8. Collapse Prevention Structural Checklist for Building Types S1 and S1a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.1.1.1
C NC N/A U	DRIFT CHECK: The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 4.4.3.1 is less than 0.030.	5.5.2.1.2	A.3.1.3.1
C NC N/A U	COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6 is less than $0.30F_y$.	5.5.2.1.3	A.3.1.3.2
C NC N/A U	FLEXURAL STRESS CHECK: The average flexural stress in the moment-frame columns and beams, calculated using the Quick Check procedure of Section 4.4.3.9, is less than F_y. Columns need not be checked if the strong column–weak beam checklist item is compliant.	5.5.2.2.2	A.3.1.3.3
Connections			
C NC N/A U	TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames.	5.7.2	A.5.2.2
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation with a minimum of two anchor rods and with the base plates bearing on concrete or a grout pad.	5.7.3.1	A.5.3.1
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of bays of moment frames in each line is greater than or equal to 2.	5.5.1.1	A.3.1.1.1
C NC N/A U	INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements.	5.5.2.1.1	A.3.1.2.1
C NC N/A U	MOMENT-RESISTING CONNECTIONS: All moment connections can develop the strength of the adjoining members based on the specified minimum yield stress of steel.	5.5.2.2.1	A.3.1.3.4
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members or panel zones based on 110% of the expected yield stress of the steel in accordance with AISC 341, Section A3.2.	5.5.2.2.1	A.3.1.3.4
C NC N/A U	PANEL ZONES: All panel zones have the shear capacity to resist the shear demand required to develop 0.8 times the sum of the flexural strengths of the girders framing in at the face of the column.	5.5.2.2.3	A.3.1.3.5
C NC N/A U	COLUMN SPLICES: At all column splice details located in moment-resisting frames, the web and flanges of I-shaped members, or all walls of tube/box members are connected to each other with the partial penetration welds with effective throat of at least 85% of the smaller member thickness or with plates that have been bolted or welded to the columns capable of developing A_gF_{ye} of the thinner flange, web, or tube/box wall.	5.5.2.2.4	A.3.1.3.6
C NC N/A U	STRONG COLUMN—WEAK BEAM: The percentage of strong column–weak beam joints in each story of each line of moment frames is greater than 50%.	5.5.2.1.5	A.3.1.3.7
C NC N/A U	COMPACT MEMBERS: All frame elements meet section requirements in accordance with AISC 341, Table D1.1a, for moderately ductile members.	5.5.2.2.5	A.3.1.3.8
Diaphragms (Stiff or Flexible)			
C NC N/A U	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the moment frames extend less than 25% of the total frame length.	5.6.1.3	A.4.1.5
Flexible Diaphragms			
C NC N/A U	CROSSTIES: There are continuous crossties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 24 ft (7.3 m) and aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1

continues

Table 17-8 *(Continued)*. Collapse Prevention Structural Checklist for Building Types S1 and S1a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.2
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-9. Immediate Occupancy Checklist for Building Types S1 and S1a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	DRIFT CHECK: The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 4.4.3.1 is less than 0.015.	5.5.2.1.2	A.3.1.3.1
C NC N/A U	COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6 is less than $0.30F_y$.	5.5.2.1.3	A.3.1.3.2
C NC N/A U	FLEXURAL STRESS CHECK: The average flexural stress in the moment-frame columns and beams, calculated using the Quick Check procedure of Section 4.4.3.9, is less than F_y. Columns need not be checked if the strong column–weak beam checklist item is compliant.	5.5.2.2.2	A.3.1.3.3
Connections			
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation with a minimum of two anchor rods and with the base plates bearing on concrete or a grout pad. The anchor rods are capable of resisting the overturning force using the Quick Check procedure of Section 4.4.3.6.	5.7.3.1	A.5.3.1
Low Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2. The number of bays of moment frames in each line is greater than or equal to 3.	5.5.1.1	A.3.1.1.1
C NC N/A U	INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements.	5.5.2.1.1	A.3.1.2.1
Connections			
C NC N/A U	TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames, and the connections are able to develop the lesser of the strength of the frames or the diaphragms.	5.7.2	A.5.2.2
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation, and the anchorage is able to develop the least of the following: the tensile capacity of the column, the tensile capacity of the lowest-level column splice (if any), or the uplift capacity of the foundation.	5.7.3.1	A.5.3.1
Moderate Seismicity (Complete the Following Items in Addition to the Items for Very Low and Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the expected strength of the adjoining members based on the specified minimum yield stress of the steel.	5.5.2.2.1	A.3.1.3.4
C NC N/A U	PANEL ZONES: All panel zones have the shear capacity to resist the shear demand required to develop 0.8 times the sum of the flexural strengths of the girders framing in at the face of the column.	5.5.2.2.3	A.3.1.3.5

continues

Table 17-9 *(Continued)*. Immediate Occupancy Checklist for Building Types S1 and S1a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	COLUMN SPLICES: All column splice details located in moment frames include connection of both flanges and the web, and the splice develops the strength of the column.	5.5.2.2.4	A.3.1.3.6
C NC N/A U	STRONG COLUMN—WEAK BEAM: The percentage of strong column–weak beam joints in each story of each line of moment-resisting frames is greater than 50%.	5.5.2.1.5	A.3.1.3.7
C NC N/A U	COMPACT MEMBERS: All frame elements meet section requirements in accordance with AISC 341, Table D1.1a, for highly ductile members.	5.5.2.2.5	A.3.1.3.8
C NC N/A U	BEAM PENETRATIONS: All openings in frame-beam webs are less than one-quarter of the beam depth and are located in the center half of the beams.	5.5.2.2.6	A.3.1.3.9
C NC N/A U	GIRDER FLANGE CONTINUITY PLATES: There are girder flange continuity plates at all moment-frame joints.	5.5.2.2.7	A.3.1.3.10
C NC N/A U	OUT-OF-PLANE BRACING: Beam–column joints are braced out of plane.	5.5.2.2.8	A.3.1.3.11
C NC N/A U	BOTTOM FLANGE BRACING: The bottom flanges of beams are braced out of plane.	5.5.2.2.9	A.3.1.3.12
Diaphragms (Stiff or Flexible)			
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
C NC N/A U	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the moment frames extend less than 15% of the total frame length.	5.6.1.3	A.4.1.5
Flexible Diaphragms			
C NC N/A U	CROSSTIES: There are continuous crossties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 12 ft (3.6 m) and aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.2 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.2
C NC N/A U	NON-CONCRETE-FILLED DIAPHRAGMS: Bare steel deck diaphragms or steel deck diaphragms with fill other than reinforced concrete consist of horizontal spans of less than 40 ft (12.2 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Foundation System			
C NC N/A U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$.	5.4.3.3	A.6.2.1
High Seismicity (Complete the Following Items in Addition to the Items for Very Low, Low, and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members or panel zones based on 110% of the expected yield stress of the steel in accordance with AISC 341, Section A3.2.	5.5.2.2.1	A.3.1.3.4

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

17.5 STRUCTURAL CHECKLIST FOR BUILDING TYPES S2: STEEL BRACED FRAMES WITH STIFF DIAPHRAGMS, AND S2A: STEEL BRACED FRAMES WITH FLEXIBLE DIAPHRAGMS

For building systems and configurations that comply with the S2 or S2a building type description in Table 3-1, the Collapse Prevention Structural Checklist in Table 17-10 shall be completed where required by Table 4-6 for Collapse Prevention Structural Performance, and the Immediate Occupancy Structural Checklist in Table 17-11 shall be completed where required by Table 4-6 for Immediate Occupancy Structural Performance. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Table 17-10. Collapse Prevention Structural Checklist for Building Types S2 and S2a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of braced frames in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.3.1.1
C NC N/A U	COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6 is less than $0.30F_y$.	5.5.2.1.3	A.3.1.3.2
C NC N/A U	BRACE AXIAL STRESS CHECK: The axial stress in the diagonals, calculated using the Quick Check procedure of Section 4.4.3.4 is less than $0.50F_y$.	5.5.4.1	A.3.3.1.2
Connections			
C NC N/A U	TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames.	5.7.2	A.5.2.2
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation with a minimum of two anchor rods and with the base plates bearing on concrete or a grout pad.	5.7.3.1	A.5.3.1
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of braced bays in each line is greater than 2.	5.5.1.1	A.3.3.1.1
C NC N/A U	CONNECTION STRENGTH: All the brace connections develop the buckling capacity of the diagonals.	5.5.4.4	A.3.3.1.5
C NC N/A U	COMPACT MEMBERS: All brace elements meet compact section requirements in accordance with AISC 360, Table B4.1a.	5.5.4.3	A.3.3.1.7
C NC N/A U	K-BRACING: The bracing system does not include K-braced bays.	5.5.4.6	A.3.3.2.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	COLUMN SPLICES: All column splice details located in braced frames develop 50% of the tensile strength of the column.	5.5.4.2	A.3.3.1.3
C NC N/A U	SLENDERNESS OF DIAGONALS: All diagonal elements required to carry compression have Kl/r ratios less than 200.	5.5.4.3	A.3.3.1.4
C NC N/A U	CONNECTION STRENGTH: All the brace connections develop the yield capacity of the diagonals.	5.5.4.4	A.3.3.1.5
C NC N/A U	COMPACT MEMBERS: All brace elements meet section requirements in accordance with AISC 341, Table D1.1a, for moderately ductile members.	5.5.4.3	A.3.3.1.7
C NC N/A U	CHEVRON BRACING: Beams in chevron, or V-braced, bays are capable of resisting the vertical load resulting from the simultaneous yielding and buckling of the brace pairs.	5.5.4.6	A.3.3.2.3
C NC N/A U	CONCENTRICALLY BRACED FRAME JOINTS: All the diagonal braces frame into the beam–column joints concentrically.	5.5.4.8	A.3.3.2.4
Diaphragms (Stiff or Flexible)			
C NC N/A U	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the braced frames extend less than 25% of the frame length.	5.6.1.3	A.4.1.5
Flexible Diaphragms			
C NC N/A U	CROSSTIES: There are continuous crossties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 24 ft (7.3 m) and aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.2
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-11. Immediate Occupancy Structural Checklists for Building Types S2 and S2a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6 is less than $0.30F_y$.	5.5.2.1.3	A.3.1.3.2
C NC N/A U	BRACE AXIAL STRESS CHECK: The axial stress in the diagonals, calculated using the Quick Check procedure of Section 4.4.3.4 is less than $0.50F_y$.	5.5.4.1	A.3.3.1.2
Connections			
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation with a minimum of two anchor rods and with the base plates bearing on concrete or a grout pad. The anchor rods are capable of resisting the overturning force using the Quick Check procedure of Section 4.4.3.6.	5.7.3.1	A.5.3.1
Low Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Connections			
C NC N/A U	TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames, and the connections are able to develop the lesser of the strength of the frames or the diaphragms.	5.7.2	A.5.2.2
Moderate Seismicity (Complete the Following Items in Addition to the Items for Very Low and Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of braced frames in each principal direction is greater than or equal to 2. The number of braced bays in each line is greater than 3.	5.5.1.1	A.3.1.1.1
C NC N/A U	COLUMN SPLICES: All column splice details located in braced frames develop 100% of the tensile strength of the column.	5.5.4.2	A.3.3.1.3
C NC N/A U	SLENDERNESS OF DIAGONALS: All diagonal elements required to carry compression have *Kl*/*r* ratios less than 200.	5.5.4.3	A.3.3.1.4
C NC N/A U	CONNECTION STRENGTH: All the brace connections develop the buckling capacity of the diagonals.	5.5.4.4	A.3.3.1.5
C NC N/A U	OUT-OF-PLANE BRACING: Braced frame connections that are attached to beam bottom flanges located away from beam–column joints are braced out of plane at the bottom flange of the beams.	5.5.4.5	A.3.3.1.6
C NC N/A U	COMPACT MEMBERS: All brace elements meet compact section requirements in accordance with AISC 341, Table B4.1a.	5.5.4.3	A.3.3.1.7
C NC N/A U	K-BRACING: The bracing system does not include K-braced bays.	5.5.4.6	A.3.3.2.1
C NC N/A U	TENSION-ONLY BRACES: Tension-only braces do not comprise more than 70% of the total seismic-force-resisting capacity in structures more than two stories high.	5.5.4.7	A.3.3.2.2
C NC N/A U	CHEVRON BRACING: Beams in chevron, or V-braced, bays are capable of resisting the vertical load resulting from the simultaneous yielding and buckling of the brace pairs.	5.5.4.6	A.3.3.2.3
C NC N/A U	CONCENTRICALLY BRACED FRAME JOINTS: All the diagonal braces frame into the beam–column joints concentrically.	5.5.4.8	A.3.3.2.4
Diaphragms (Stiff or Flexible)			
C NC N/A U	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the braced frames extend less than 15% of the frame length.	5.6.1.3	A.4.1.5
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8

continues

Table 17-11 *(Continued)*. Immediate Occupancy Structural Checklists for Building Types S2 and S2a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Flexible Diaphragms			
C NC N/A U	CROSSTIES: There are continuous crossties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 12 ft (3.6 m) and aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.2 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.2
C NC N/A U	NON-CONCRETE-FILLED DIAPHRAGMS: Bare steel deck diaphragms or steel deck diaphragms with fill other than reinforced concrete consist of horizontal spans of less than 40 ft (12.2 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Foundation System			
C NC N/A U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$.	5.4.3.3	A.6.2.1
High Seismicity (Complete the Following Items in Addition to the Items for Very Low, Low, and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	CONNECTION STRENGTH: All the brace connections develop the yield capacity of the diagonals.	5.5.4.4	A.3.3.1.5
C NC N/A U	COMPACT MEMBERS: All column and brace elements meet section requirements in accordance with AISC 341, Table D1.1a, for highly ductile members. Braced frame beams meet the requirements for moderately ductile members.	5.5.4.3	A.3.3.1.7
C NC N/A U	NET AREA: The brace effective net area is not less than the brace gross area for hollow structural section (HSS) tube and pipe sections.	5.5.4.1	A.3.3.1.8
Connections			
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation, and the anchorage is able to develop the least of the following: the tensile capacity of the column, the tensile capacity of the lowest-level column splice (if any), or the uplift capacity of the foundation.	5.7.3.1	A.5.3.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

17.6 STRUCTURAL CHECKLISTS FOR BUILDING TYPE S3: METAL BUILDING FRAMES

For building systems and configurations that comply with the S3 building type description in Table 3-1, the Collapse Prevention Structural Checklist in Table 17-12 shall be completed where required by Table 4-6 for Collapse Prevention Structural Performance, and the Immediate Occupancy Structural Checklist in Table 17-13 shall be completed where required by Table 4-6, for Immediate Occupancy Structural Performance. The Structural Checklist for Metal Building Frames shall not be used for a structure with a roof and wall dead load greater than 25 lb/ft^2 (1.2 kN/m^2) or a building area greater than 20,000 ft^2 (1,858 m^2). Where either limit is exceeded, the Structural Checklists for Steel Moment Frames (Type S1 or S1a) shall be used. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

Table 17-12. Collapse Prevention Structural Checklist for Building Type S3.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	BRACE AXIAL STRESS CHECK: The axial stress in the diagonals, calculated using the Quick Check procedure of Section 4.4.3.4 is less than $0.50F_y$.	5.5.4.1	A.3.3.1.2
Connections			
C NC N/A U	TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel moment frames.	5.7.2	A.5.2.2
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation with a minimum of two anchor rods and with the base plates bearing on concrete or a grout pad.	5.7.3.1	A.5.3.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the elastic moment (F_yS) of the adjoining members.	5.5.2.2.1	A.3.1.3.4
C NC N/A U	COMPACT MEMBERS: All frame elements meet compact section requirements in accordance with AISC 360, Table B4.1a.	5.5.2.2.5	A.3.1.3.8
Diaphragms			
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	ROOF PANELS: Where considered as diaphragm elements for lateral resistance, metal, plastic, or cementitious roof panels are positively attached to the roof framing to resist seismic forces.	5.7.5	A.5.5.1
C NC N/A U	WALL PANELS: Where considered as shear elements for lateral resistance, metal, fiberglass, or cementitious wall panels are positively attached to the framing and foundation to resist seismic forces.	5.7.5	A.5.5.2

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-13. Immediate Occupancy Checklist for Building Type S3.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low and Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	BRACE AXIAL STRESS CHECK: The axial stress in the diagonals, calculated using the Quick Check procedure of Section 4.4.3.4 is less than $0.50F_y$.	5.5.4.1	A.3.3.1.2
C NC N/A U	FLEXURAL STRESS CHECK: The average flexural stress in the moment-frame columns and beams, calculated using the Quick Check procedure of Section 4.4.3.9 is less than F_y.	5.5.2.2.2	A.3.1.3.3
Connections			
C NC N/A U	TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel moment frames.	5.7.2	A.5.2.2
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation with a minimum of two anchor rods and with the base plates bearing on concrete or a grout pad. The anchor rods are capable of resisting the overturning force using the Quick Check procedure of Section 4.4.3.6.	5.7.3.1	A.5.3.1
Moderate Seismicity (Complete the Following Items in Addition to the Items for Very Low and Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the elastic moment (F_yS) of the adjoining members.	5.5.2.2.1	A.3.1.3.4

continues

Table 17-13 *(Continued)*. Immediate Occupancy Checklist for Building Type S3.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Diaphragms			
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	ROOF PANELS: Where considered as diaphragm elements for lateral resistance, metal, plastic, or cementitious roof panels are positively attached to the roof framing to resist seismic forces.	5.7.5	A.5.5.1
C NC N/A U	WALL PANELS: Where considered as shear elements for lateral resistance, metal, fiberglass, or cementitious wall panels are positively attached to the framing and foundation to resist seismic forces.	5.7.5	A.5.5.2
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members or panel zones.	5.5.2.2.1	A.3.1.3.4
C NC N/A U	COMPACT MEMBERS: All frame elements meet compact section requirements in accordance with AISC 360, Table B4.1a.	5.5.2.2.5	A.3.1.3.8
C NC N/A U	BEAM PENETRATIONS: All openings in frame-beam webs are less than one-quarter of the beam depth and are located in the center half of the beams.	5.5.2.2.6	A.3.1.3.9
C NC N/A U	OUT-OF-PLANE BRACING: Beam–column joints are braced out of plane.	5.5.2.2.8	A.3.1.3.11
C NC N/A U	BOTTOM FLANGE BRACING: The bottom flanges of beams are braced out of plane.	5.5.2.2.9	A.3.1.3.12
Connections			
C NC N/A U	TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel moment frames, and the connections are able to develop the lesser of the strength of the frames or the diaphragms.	5.7.2	A.5.2.2
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation, and the anchorage is able to develop the least of the following: the tensile capacity of the column, the tensile capacity of the lowest-level column splice (if any), or the uplift capacity of the foundation.	5.7.3.1	A.5.3.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

17.7 STRUCTURAL CHECKLISTS FOR BUILDING TYPE S4: DUAL SYSTEMS WITH BACKUP STEEL MOMENT FRAMES AND STIFF DIAPHRAGMS

For building systems and configurations that comply with the S4 building type description in Table 3-1, the Collapse Prevention Structural Checklist in Table 17-14 shall be completed where required by Table 4-6 for Collapse Prevention Structural Performance, and the Immediate Occupancy Structural Checklist in Table 17-15 shall be completed where required by Table 4-6 for Immediate Occupancy Structural Performance. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

17.8 STRUCTURAL CHECKLISTS FOR BUILDING TYPES S5: STEEL FRAMES WITH INFILL MASONRY SHEAR WALLS AND STIFF DIAPHRAGMS, AND S5A: STEEL FRAMES WITH INFILL MASONRY SHEAR WALLS AND FLEXIBLE DIAPHRAGMS

For building systems and configurations that comply with the S5 or S5a building type description in Table 3-1, the Collapse Prevention Structural Checklist in Table 17-16 shall be completed where required by Table 4-6 for Collapse Prevention Structural Performance, and the Immediate Occupancy Structural Checklist in Table 17-17 shall be completed where required by

Table 17-14. Collapse Prevention Structural Checklist for Building Type S4.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of braced frames or shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1 A.3.3.1.1
C NC N/A U	DRIFT CHECK: The drift ratio of the steel moment frames acting alone, calculated using the Quick Check procedure of Section 4.4.3.1 using 25% of V_c, is less than 0.025.	5.5.2.1.2	A.3.1.3.1
C NC N/A U	COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in frame columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6 is less than $0.30F_y$.	5.5.2.1.3	A.3.1.3.2
C NC N/A U	BRACE AXIAL STRESS CHECK: The axial stress in the diagonal braces, calculated using the Quick Check procedure of Section 4.4.3.4 and neglecting the steel moment frame is less than $0.50F_y$.	5.5.4.1	A.3.3.1.2
C NC N/A U	CONCRETE BEARING WALLS: Floor and roof girders and trusses are not supported at the ends of concrete walls that are less than 10 in. (254 mm) thick. This statement only applies to framing supports located less than two times the wall thickness away from the wall end.	5.5.2.5.1	A.3.1.6.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, and neglecting the steel moment frame, is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$.	5.5.3.1.1	A.3.2.2.1
C NC N/A U	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction.	5.5.3.1.3	A.3.2.2.2
Connections			
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation with a minimum of two anchor rods and with the base plates bearing on concrete or a grout pad.	5.7.3.1	A.5.3.1
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls.	5.7.2	A.5.2.1
C NC N/A U	TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames.	5.7.2	A.5.2.2
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation.	5.7.3.4	A.5.3.5
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: For braced frames, the number of braced bays in each line is greater than 2.	5.5.1.1	A.3.3.1.1
C NC N/A U	MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members based on the specified minimum yield stress of the steel.	5.5.2.2.1	A.3.1.3.4
C NC N/A U	COMPACT MEMBERS: All moment frame and brace elements meet section requirements in accordance with AISC 360, Table B4.1a.	5.5.2.2.5	A.3.1.3.8
C NC N/A U	CONNECTION STRENGTH: All the brace connections develop the buckling capacity of the diagonals.	5.5.4.4	A.3.3.1.5
C NC N/A U	K-BRACING: The bracing system does not include K-braced bays.	5.5.4.6	A.3.3.2.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members or panel zones based on 110% of the expected yield stress of the steel per AISC 341, Section A3.2.	5.5.2.2.1	A.3.1.3.4

continues

Table 17-14 *(Continued)*. Collapse Prevention Structural Checklist for Building Type S4.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	COLUMN SPLICES: At all column splice details located in moment frames, the web and flanges of I-shaped members, or all walls of tube/box members are connected to each other with the partial penetration welds with effective throat of at least 85% of the smaller member thickness or with plates that have been bolted or welded to the columns capable of developing $A_g F_{ye}$ of the thinner flange, web or tube/box wall.	5.5.2.2.4	A.3.1.3.6
C NC N/A U	STRONG COLUMN—WEAK BEAM: The percentage of strong column–weak beam joints in each story of each line of moment frames is greater than 50%.	5.5.2.1.5	A.3.1.3.7
C NC N/A U	COMPACT MEMBERS: All moment-frame and brace elements meet section requirements in accordance with AISC 341, Table D1.1a, for moderately ductile members.	5.5.2.2.5 5.5.4.3	A.3.1.3.8
C NC N/A U	COLUMN SPLICES: All column splice details located in braced frames develop 50% of the tensile strength of the column.	5.5.4.2	A.3.3.1.3
C NC N/A U	SLENDERNESS OF DIAGONALS: All diagonal elements required to carry compression have Kl/r ratios less than 200.	5.5.4.3	A.3.3.1.4
C NC N/A U	CONNECTION STRENGTH: All the brace connections develop the yield capacity of the diagonals.	5.5.4.4	A.3.3.1.5
C NC N/A U	CHEVRON BRACING: Beams in chevron, or V-braced, bays are capable of resisting the vertical load resulting from the simultaneous yielding and buckling of the brace pairs.	5.5.4.6	A.3.3.2.3
C NC N/A U	CONCENTRICALLY BRACED FRAME JOINTS: All the diagonal braces frame into the beam–column joints concentrically.	5.5.4.8	A.3.3.2.4
C NC N/A U	COUPLING BEAMS: Coupling beams have stirrups spaced at or less than d/2, and each wall or wall segment connected to the coupling beam is supported such that it can resist shear and overturning forces in the absence of the coupling beam. This statement only applies to coupling beams with span-to-depth ratios exceeding 2-to-1.	5.5.3.2.1	A.3.2.2.3
Flexible Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the braced frames or moment frames extend less than 25% of the frame length.	5.6.1.3	A.4.1.5

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-15. Immediate Occupancy Structural Checklist for Building Type S4.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in frame columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6 is less than $0.30F_y$.	5.5.2.1.3	A.3.1.3.2
C NC N/A U	BRACE AXIAL STRESS CHECK: The axial stress in the diagonal braces, calculated using the Quick Check procedure of Section 4.4.3.4 and neglecting the steel moment frame, is less than $0.50F_y$.	5.5.4.1	A.3.3.1.2

continues

Table 17-15 *(Continued)*. Immediate Occupancy Structural Checklist for Building Type S4.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	CONCRETE BEARING WALLS: Floor and roof girders and trusses are not supported at the ends of concrete walls that are less than 10 in. (254 mm) thick. This statement only applies to framing supports located less than two times the wall thickness away from the wall end.	5.5.2.5.1	A.3.1.6.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.4.3.3 and neglecting the steel moment frame, is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$.	5.5.3.1.1	A.3.2.2.1
C NC N/A U	REINFORCING STEEL: The ratio of shear wall reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. The spacing of reinforcing steel is equal to or less than 18 in. (457 mm).	5.5.3.1.3	A.3.2.2.2
Connections			
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation with a minimum of two anchor rods and with the base plates bearing on concrete or a grout pad. The anchor rods are capable of resisting the overturning force using the Quick Check procedure of Section 4.4.3.6.	5.7.3.1	A.5.3.1
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls.	5.7.2	A.5.2.1
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation, and the dowels are able to develop the lesser of the strength of the walls or the uplift capacity of the foundation.	5.7.3.4	A.5.3.5
Low Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	DRIFT CHECK: The drift ratio of the steel moment frames acting alone, calculated using the Quick Check procedure of Section 4.4.3.1 using 25% of V_c, is less than 0.015.	5.5.2.1.2	A.3.1.3.1
C NC N/A U	REDUNDANCY: The number of lines of braced frames or shear walls in each principal direction is greater than or equal to 2. The number of braced bays in each line is greater than 3.	5.5.1.1	A.3.2.1.1 A.3.1.1.1
C NC N/A U	INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements.	5.5.2.1.1	A.3.1.2.1
Connections			
C NC N/A U	TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames, and the connections are able to develop the lesser of the strength of the frames or the diaphragms.	5.7.2	A.5.2.2
Moderate Seismicity (Complete the Following Items in Addition to the Items for Very Low and Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members based on the specified minimum yield stress of the steel.	5.5.2.2.1	A.3.1.3.4
C NC N/A U	PANEL ZONES: All panel zones have the shear capacity to resist the shear demand required to develop 0.8 times the sum of the flexural strengths of the girders framing in at the face of the column.	5.5.2.2.3	A.3.1.3.5
C NC N/A U	COLUMN SPLICES: All column splice details located in moment frames include connection of both flanges and the web, and the splice develops the strength of the column.	5.5.2.2.4	A.3.1.3.6
C NC N/A U	STRONG COLUMN–WEAK BEAM: The percentage of strong column–weak beam joints in each story of each line of moment frames is greater than 50%.	5.5.2.1.5	A.3.1.3.7
C NC N/A U	BEAM PENETRATIONS: All openings in frame-beam webs are less than one-quarter of the beam depth and are located in the center half of the beams.	5.5.2.2.6	A.3.1.3.9
C NC N/A U	GIRDER FLANGE CONTINUITY PLATES: There are girder flange continuity plates at all moment-resisting frame joints.	5.5.2.2.7	A.3.1.3.10
C NC N/A U	OUT-OF-PLANE BRACING: Beam–column joints are braced out of plane.	5.5.2.2.8	A.3.1.3.11

continues

Table 17-15 *(Continued)*. Immediate Occupancy Structural Checklist for Building Type S4.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	BOTTOM FLANGE BRACING: The bottom flanges of beams are braced out of plane.	5.5.2.2.9	A.3.1.3.12
C NC N/A U	COMPACT MEMBERS: All brace elements meet section requirements in accordance with AISC 360, Table B4.1a.	5.5.4.3	A.3.3.1.7
C NC N/A U	COLUMN SPLICES: All column splice details located in braced frames develop 100% of the tensile strength of the column.	5.5.4.2	A.3.3.1.3
C NC N/A U	SLENDERNESS OF DIAGONALS: All diagonal elements required to carry compression have Kl/r ratios less than 200.	5.5.4.3	A.3.3.1.4
C NC N/A U	CONNECTION STRENGTH: All the brace connections develop the buckling capacity of the diagonals.	5.5.4.4	A.3.3.1.5
C NC N/A U	OUT-OF-PLANE BRACING: Braced frame connections that are attached to beam bottom flanges located away from beam–column joints are braced out of plane at the bottom flange of the beams.	5.5.4.5	A.3.3.1.6
C NC N/A U	K-BRACING: The bracing system does not include K-braced bays.	5.5.4.6	A.3.3.2.1
C NC N/A U	TENSION-ONLY BRACES: Tension-only braces do not compose more than 70% of the total seismic-force-resisting capacity in structures more than two stories high.	5.5.4.7	A.3.3.2.2
C NC N/A U	CHEVRON BRACING: Beams in chevron, or V-braced, bays are capable of resisting the vertical load resulting from the simultaneous yielding and buckling of the brace pairs.	5.5.4.6	A.3.3.2.3
C NC N/A U	CONCENTRICALLY BRACED FRAME JOINTS: All the diagonal braces frame into the beam–column joints concentrically.	5.5.4.8	A.3.3.2.4
C NC N/A U	COUPLING BEAMS: Coupling beams have the capacity in shear to develop the uplift capacity of the adjacent wall or to develop the flexural capacity of the coupling beam, whichever is less. This statement only applies to coupling beams with span-to-depth ratios exceeding 2-to-1.	5.5.3.2.1	A.3.2.2.3
C NC N/A U	OVERTURNING: All shear walls have aspect ratios less than 4-to-1. Wall piers need not be considered.	5.5.3.1.4	A.3.2.2.4
C NC N/A U	CONFINEMENT REINFORCING: For shear walls with aspect ratios greater than 2-to-1, the boundary elements are confined with spirals or ties with spacing less than $8d_b$.	5.5.3.2.2	A.3.2.2.5
C NC N/A U	WALL REINFORCING AT OPENINGS: There is added trim reinforcement around all wall openings with a dimension greater than three times the thickness of the wall.	5.5.3.1.5	A.3.2.2.6
C NC N/A U	WALL THICKNESS: Thicknesses of bearing walls are not less than 1/25 the unsupported height or length, whichever is shorter, nor less than 4 in. (101.6 mm).	5.5.3.1.2	A.3.2.2.7
Diaphragms (Stiff or Flexible)			
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the braced frames or moment frames extend less than 15% of the frame length.	5.6.1.3	A.4.1.5
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
Foundation System			
C NC N/A U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$.	5.4.3.3	A.6.2.1

continues

Table 17-15 *(Continued)*. Immediate Occupancy Structural Checklist for Building Type S4.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
High Seismicity (Complete the Following Items in Addition to the Items for Very Low, Low, and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members or panel zones based on 110% of the expected yield stress of the steel per AISC 341, Section A3.2.	5.5.2.2.1	A.3.1.3.4.
C NC N/A U	COMPACT MEMBERS: All moment and braced frame columns and beams meet section requirements in accordance with AISC 341, Table D1.1a, for highly ductile members. Braced frame beams meet section requirements for moderately ductile members.	5.5.2.2.5 5.5.4.3	A.3.3.1.8
C NC N/A U	CONNECTION STRENGTH: All the brace connections develop the yield capacity of the diagonals.	5.5.4.4	A.3.3.1.5
Connections			
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation, and the anchorage is able to develop the least of the following: the tensile capacity of the column, the tensile capacity of the lowest-level column splice (if any), or the uplift capacity of the foundation.	5.7.3.1	A.5.3.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-16. Collapse Prevention Structural Checklist for Building Types S5 and S5a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3 is less than 70 lb/in.^{2} (0.48 MPa)	5.5.3.1.1	A.3.2.4.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the unreinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3 is less than 30 lb/in.^{2} (0.21 MPa) for clay units and 70 lb/in.^{2} (0.48 MPa) for concrete units. Bays with openings greater than 25% of the wall area shall not be included in A_w of Equation (4-8).	5.5.3.1.1	A.3.2.5.1
C NC N/A U	INFILL WALL CONNECTIONS: Masonry is in full contact with frame.	5.5.3.5.1 5.5.3.5.3	A.3.2.6.1
Connections			
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation with a minimum of two anchor rods and with the base plates bearing on concrete or a grout pad.	5.7.3.1	A.5.3.1
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	INFILL WALL ECCENTRICITY: The centerline of the infill masonry wall is not offset from the centerline of the steel framing by more than 25% of the wall thickness.	5.5.3.5.3	A.3.2.6.5
Connections			
C NC N/A U	TRANSFER TO INFILL WALLS: Diaphragms are connected for transfer of loads to the infill walls.	5.7.2	A.5.2.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	PROPORTIONS: The height-to-thickness ratio of the unreinforced infill walls at each story is less than 9.	5.5.3.1.2	A.3.2.6.2

continues

Table 17-16 *(Continued)*. Collapse Prevention Structural Checklist for Building Types S5 and S5a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	CAVITY WALLS: The infill walls are not of cavity construction.	5.5.3.5.2	A.3.2.6.3
Flexible Diaphragms			
C NC N/A U	CROSSTIES: There are continuous crossties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 24 ft (7.3 m) and aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.2
C NC N/A U	BLOCKED DIAPHRAGMS: All blocked wood structural panel diaphragms have horizontal spans less than 120 ft (36.5 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	CANTILEVERED WOOD DIAPHRAGMS: All cantilevered wood diaphragms that provide lateral support for concrete or masonry walls consist of wood structural panels and have a maximum cantilever length of 20 ft (6.1 m) if unblocked or 35 ft (10.7 m) if blocked, and a maximum ratio of cantilever length to diaphragm width of 1:2 if unblocked and 1:1 if blocked. In addition, the cantilevered diaphragm has a back-span length equal to or greater than the cantilevered portion.	5.6.2	A.4.2.4
C NC N/A U	NON-CONCRETE-FILLED DIAPHRAGMS: Bare steel deck diaphragms or steel deck diaphragms with fill other than reinforced structural concrete consist of horizontal spans of less than 120 ft (36.5 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. (3.1 mm) before engagement of the anchors.	5.7.1.2	A.5.1.4

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-17. Immediate Occupancy Structural Checklist for Building Types S5 and S5a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 70 lb/in.^{2} (0.48 MPa).	5.5.3.1.1	A.3.2.4.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the unreinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 30 lb/in.^{2} (0.21 MPa) for clay units and 70 lb/in.^{2} (0.48 MPa) for concrete units. Bays with openings greater than 25% of the wall area shall not be included in A_w of Equation (4-8).	5.5.3.1.1	A.3.2.5.1
C NC N/A U	INFILL WALL CONNECTIONS: Masonry is in full contact with frame.	5.5.3.5.1 5.5.3.5.3	A.3.2.6.1

continues

Table 17-17 *(Continued)*. Immediate Occupancy Structural Checklist for Building Types S5 and S5a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Connections			
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation with a minimum of two anchor rods and with the base plates bearing on concrete or a grout pad. The anchor rods are capable of resisting the overturning force using the Quick Check procedure of Section 4.4.3.6.	5.7.3.1	A.5.3.1
Low Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 12 ft (3.6 m) and aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.2 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.2
C NC N/A U	BLOCKED DIAPHRAGMS: All blocked wood structural panel diaphragms have horizontal spans less than 90 ft (27.4 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	CANTILEVERED WOOD DIAPHRAGMS: All cantilevered wood diaphragms that provide lateral support for concrete or masonry walls consist of wood structural panels and have a maximum cantilever length of 15 ft (4.6 m) if unblocked or 25 ft (7.6 m) if blocked, and a maximum ratio of cantilever length to diaphragm width of 1:2.5 if unblocked and 1:1.5 if blocked. In addition, the cantilevered diaphragm has a back-span length equal to or greater than the cantilevered portion.	5.6.2	A.4.2.4
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	REINFORCING AT WALL OPENINGS: All wall openings that interrupt rebar have trim reinforcing on all sides or are checked as unreinforced infill frames.	5.5.3.1.5	A.3.2.4.3
C NC N/A U	PROPORTIONS: The height-to-thickness ratio of the unreinforced infill walls at each story is less than 13.	5.5.3.1.2	A.3.2.6.2
C NC N/A U	CAVITY WALLS: The infill walls are not of cavity construction.	5.5.3.5.2	A.3.2.6.3
C NC N/A U	INFILL WALL ECCENTRICITY: The centerline of the infill masonry wall is not offset from the centerline of the steel framing by more than 25% of the wall thickness.	5.5.3.5.3	A.3.2.6.5
Connections			
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of loads to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms.	5.7.2	A.5.2.1
Diaphragms (Stiff or Flexible)			
C NC N/A U	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the braced frames extend less than 15% of the frame length.	5.6.1.3	A.4.1.5
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
Flexible Diaphragms			
C NC N/A U	CROSSTIES: There are continuous crossties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	NON-CONCRETE-FILLED DIAPHRAGMS: Bare steel deck diaphragms or steel deck diaphragms with fill other than reinforced concrete consist of horizontal spans of less than 40 ft (12.2 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1

continues

Table 17-17 *(Continued)*. Immediate Occupancy Structural Checklist for Building Types S5 and S5a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. (3.1 mm) before engagement of the anchors.	5.7.1.2	A.5.1.4
Foundation System			
C NC N/A U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$.	5.4.3.3	A.6.2.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	PROPORTIONS: The height-to-thickness ratio of the unreinforced infill walls at each story is less than 8.	5.5.3.1.2	A.3.2.6.2
Connections			
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation, and the anchorage is able to develop the least of the following: the tensile capacity of the column, the tensile capacity of the lowest-level column splice (if any), or the uplift capacity of the foundation.	5.7.3.1	A.5.3.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 4-6 for Immediate Occupancy Structural Performance. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

17.9 STRUCTURAL CHECKLISTS FOR BUILDING TYPE CFS1: COLD-FORMED STEEL LIGHT-FRAME BEARING WALL CONSTRUCTION, SHEAR WALL LATERAL SYSTEM

For building systems and configurations that comply with the CFS1 building type description in Table 3-1, the Collapse Prevention Structural Checklist in Table 17-18 shall be completed where required by Table 4-6 for Collapse Prevention Structural Performance, and the Immediate Occupancy Structural Checklist in Table 17-19 shall be completed where required by Table 4-6 for Immediate Occupancy Structural Performance. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

17.10 STRUCTURAL CHECKLISTS FOR BUILDING TYPE CFS2: COLD-FORMED STEEL LIGHT-FRAME BEARING WALL CONSTRUCTION, STRAP-BRACED LATERAL WALL SYSTEM

For building systems and configurations that comply with the CFS2 building type description in Table 3-1, the Collapse Prevention Structural Checklist in Table 17-20 shall be completed where required by Table 4-6 for Collapse Prevention Structural Performance, and the Immediate Occupancy Structural Checklist in Table 17-21 shall be completed where required by Table 4-6 for Immediate Occupancy Structural Performance. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

Table 17-18. Collapse Prevention Structural Checklist for Building Type CFS1.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.4.3.3 is less than the following values: Wood structural panel sheathing 1,000 lb/ft (14.6 kN/m), Steel sheet sheathing 700 lb/ft (10.2 kN/m), and All other conditions 100 lb/ft (1.5 kN/m).	5.5.3.1.1	A.3.2.8.1
C NC N/A U	STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multistory buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system.	5.5.3.7.1	A.3.2.8.2
C NC N/A U	GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard is not used for shear walls on buildings more than one story high with the exception of the uppermost level of a multistory building.	5.5.3.7.1	A.3.2.8.3
C NC N/A U	NARROW SHEAR WALLS: Narrow shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces.	5.5.3.7.1	A.3.2.8.4
C NC N/A U	WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor.	5.5.3.7.2	A.3.2.8.5
C NC N/A U	HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-1.	5.5.3.7.3	A.3.2.8.6
C NC N/A U	OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel or steel sheet shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces.	5.5.3.7.5	A.3.2.8.8
Connections			
C NC N/A U	POSTS: There is a positive connection of posts to the foundation.	5.7.3.3	A.5.3.3
C NC N/A U	SILLS (BASE TRACK): All sills or base tracks are bolted to the foundation.	5.7.3.3	A.5.3.4
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Connections			
C NC N/A U	SILL (BASE TRACK) BOLTS: Bolts are spaced at 6 ft (1.8 m) or less with acceptable edge and end distance provided for steel and concrete.	5.7.3.3	A.5.3.7
Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	SPANS: All diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels.	5.6.2	A.4.2.2
C NC N/A U	UNBLOCKED DIAPHRAGMS: All unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.2
C NC N/A U	OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-19. Immediate Occupancy Structural Checklist for Building Type CFS1.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.4.3.3 is less than the following values: Wood structural panel sheathing 1,000 lb/ft (14.6 kN/m), Steel sheet sheathing 700 lb/ft (10.2 kN/m), and All other conditions 100 lb/ft (1.5 kN/m).	5.5.3.1.1	A.3.2.8.1
C NC N/A U	STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multistory buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system.	5.5.3.7.1	A.3.2.8.2
C NC N/A U	GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard is not used for shear walls on buildings more than one story high with the exception of the uppermost level of a multistory building.	5.5.3.7.1	A.3.2.8.3
C NC N/A U	NARROW SHEAR WALLS: Narrow shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces.	5.5.3.7.1	A.3.2.8.4
C NC N/A U	WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor.	5.5.3.7.2	A.3.2.8.5
C NC N/A U	HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-2.	5.5.3.7.3	A.3.2.8.6
C NC N/A U	OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel or steel sheet shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces.	5.5.3.7.5	A.3.2.8.8
Connections			
C NC N/A U	POSTS: There is a positive connection of posts to the foundation.	5.7.3.3	A.5.3.3
C NC N/A U	SILLS (BASE TRACK): All sills or base tracks are bolted to the foundation.	5.7.3.3	A.5.3.4
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
Low, Moderate, and High Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	HOLD-DOWN ANCHORS: All shear walls have hold-down anchors attached to the end studs, constructed in accordance with acceptable construction practices.	5.5.3.7.6	A.3.2.8.9
C NC N/A U	NARROW SHEAR WALLS: Narrow shear walls with an aspect ratio greater than 1.5-to-1 are not used to resist seismic forces.	5.5.3.7.1	A.3.2.8.4
Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
C NC N/A U	SPANS: All diaphragms with spans greater than 12 ft (3.6 m) consist of wood structural panels.	5.6.2	A.4.2.2
C NC N/A U	UNBLOCKED DIAPHRAGMS: All unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.1 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.2

continues

Table 17-19 *(Continued)*. Immediate Occupancy Structural Checklist for Building Type CFS1.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	OTHER DIAPHRAGMS: Diaphragm do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	SILL (BASE TRACK) BOLTS: Sill or base track bolts are spaced at 4 ft (1.2 m) or less, with acceptable edge and end distance provided for steel and concrete.	5.7.3.3	A.5.3.7

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-20. Collapse Prevention Structural Checklist for Building Type CFS2.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of strap-braced walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	STRAP-BRACED WALLS—BRACE AXIAL STRESS CHECK: The axial stress in the diagonal straps, calculated using the Quick Check procedure of Section 4.4.3.4 is less than $0.50F_y$.	5.5.4.1	A.3.3.1.2
C NC N/A U	STRAP-BRACED WALLS—CHORD STUD AXIAL CHECK: The axial force caused by overturning plus the gravity load on the end stud is less than the nominal strength of the end stud calculated in accordance with AISI S100.	5.5.4.9.5	A.3.3.2.9
C NC N/A U	NARROW STRAP-BRACED WALLS: Narrow strap-braced walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces.	5.5.4.9.1	A.3.3.2.5
C NC N/A U	WALLS CONNECTED THROUGH FLOORS: Strap-braced walls have an interconnection between stories to transfer overturning and shear forces through the floor.	5.5.4.9.2	A.3.3.2.6
C NC N/A U	HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all strap-braced walls on the downhill slope have an aspect ratio less than 1-to-1.	5.5.4.9.3	A.3.3.2.7
Connections			
C NC N/A U	POSTS: There is a positive connection of posts to the foundation.	5.7.3.3	A.5.3.3
C NC N/A U	SILLS (BASE TRACK): All sills or base tracks are bolted to the foundation.	5.7.3.3	A.5.3.4
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Connections			
C NC N/A U	SILL (BASE TRACK) BOLTS: Bolts are spaced at 6 ft (1.8 m) or less with acceptable edge and end distance provided for steel and concrete.	5.7.3.3	A.5.3.7
C NC N/A U	STRAP-BRACE CONNECTIONS: Strap connections develop the yield capacity of the straps.	5.5.4.4	A.3.3.1.5
Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3

continues

Table 17-20 *(Continued)*. Collapse Prevention Structural Checklist for Building Type CFS2.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	SPANS: All diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels.	5.6.2	A.4.2.2
C NC N/A U	UNBLOCKED DIAPHRAGMS: All unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.2
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-21. Immediate Occupancy Structural Checklist for Building Type CFS2.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of strap-braced walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	STRAP-BRACED WALLS—BRACE AXIAL STRESS CHECK: The axial stress in the diagonal straps, calculated using the Quick Check procedure of Section 4.4.3.4 is less than $0.50F_y$.	5.5.4.1	A.3.3.1.2
C NC N/A U	STRAP-BRACED WALLS—CHORD STUD AXIAL CHECK: The axial force caused by overturning plus the gravity load on the end stud is less than the nominal strength of the end stud calculated in accordance with AISI S100.	5.5.4.9.5	A.3.3.2.9
C NC N/A U	NARROW STRAP-BRACED WALLS: Narrow strap-braced walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces.	5.5.4.9.1	A.3.3.2.5
C NC N/A U	WALLS CONNECTED THROUGH FLOORS: Strap-braced walls have an interconnection between stories to transfer overturning and shear forces through the floor.	5.5.4.9.2	A.3.3.2.6
C NC N/A U	HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all strap-braced walls on the downhill slope have an aspect ratio less than 1-to-2.	5.5.4.9.3	A.3.3.2.7
Connections			
C NC N/A U	POSTS: There is a positive connection of posts to the foundation.	5.7.3.3	A.5.3.3
C NC N/A U	SILLS (BASE TRACK): All sills or base tracks are bolted to the foundation.	5.7.3.3	A.5.3.4
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
Low, Moderate, and High Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	HOLD-DOWN ANCHORS: All strap-braced walls have hold-down anchors attached to the end studs, constructed in accordance with acceptable construction practices.	5.5.3.6.6	A.3.3.2.8
Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7

continues

Table 17-21 *(Continued)*. Immediate Occupancy Structural Checklist for Building Type CFS2.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
C NC N/A U	SPANS: All diaphragms with spans greater than 12 ft (3.6 m) consist of wood structural panels.	5.6.2	A.4.2.2
C NC N/A U	UNBLOCKED DIAPHRAGMS: All unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.1 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.2
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	SILL (BASE TRACK) BOLTS: Sill or base track bolts are spaced at 4 ft (1.2 m) or less, with acceptable edge and end distance provided for steel and concrete.	5.7.3.3	A.5.3.7
C NC N/A U	STRAP-BRACE CONNECTIONS: Strap connections develop the yield capacity of the straps.	5.5.4.4	A.3.3.1.5
C NC N/A U	STRAP-BRACE DETAILING: Strap braces are tight to the stud and attached to the intermediate studs in accordance with the requirements of AISI S400.	5.5.4.9.6	A.3.3.2.10

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

17.11 STRUCTURAL CHECKLISTS FOR BUILDING TYPE C1: CONCRETE MOMENT FRAMES

For building systems and configurations that comply with the C1 building type description in Table 3-1, the Collapse Prevention Structural Checklist in Table 17-22 shall be completed where required by Table 4-6 for Collapse Prevention Structural Performance, and the Immediate Occupancy Structural Checklist in Table 17-23 shall be completed where required by Table 4-6 for Immediate Occupancy Structural Performance. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

17.12 STRUCTURAL CHECKLIST FOR BUILDING TYPES C2: CONCRETE SHEAR WALLS WITH STIFF DIAPHRAGMS, AND C2A: CONCRETE SHEAR WALLS WITH FLEXIBLE DIAPHRAGMS

For building systems and configurations that comply with the C2 or C2a building type description in Table 3-1, the Collapse Prevention Structural Checklist in Table 17-24 shall be completed where required by Table 4-6 for Collapse Prevention Structural Performance, and the Immediate Occupancy Structural Checklist in Table 17-25 shall be completed where required by Table 4-6 for Immediate Occupancy Structural Performance. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

17.13 STRUCTURAL CHECKLISTS FOR BUILDING TYPES C3: CONCRETE FRAMES WITH INFILL MASONRY SHEAR WALLS, AND C3A: CONCRETE FRAMES WITH INFILL MASONRY SHEAR WALLS AND FLEXIBLE DIAPHRAGMS

For building systems and configurations that comply with the C3 or C3a building type description in Table 3-1, the Collapse Prevention Structural Checklist in Table 17-26 shall be completed where required by Table 4-6 for Collapse Prevention Structural Performance, and the Immediate Occupancy Structural Checklist in Table 17-27 shall be completed where required by Table 4-6 for Immediate Occupancy Structural Performance. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant

Table 17-22. Collapse Prevention Structural Checklist for Building Type C1.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.1.1.1
C NC N/A U	COLUMN AXIAL STRESS CHECK: The axial stress caused by unfactored gravity loads in columns subjected to overturning forces because of seismic demands is less than $0.20f'_c$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6 is less than $0.30f'_c$.	5.5.2.1.3	A.3.1.4.2
Connections			
C NC N/A U	CONCRETE COLUMNS: All concrete columns are doweled into the foundation with a minimum of four bars.	5.7.3.1	A.5.3.2
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of bays of moment frames in each line is greater than or equal to 2.	5.5.1.1	A.3.1.1.1
C NC N/A U	INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements.	5.5.2.1.1	A.3.1.2.1
C NC N/A U	COLUMN SHEAR STRESS CHECK: The shear stress in the concrete columns, calculated using the Quick Check procedure of Section 4.4.3.2 is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$.	5.5.2.1.4	A.3.1.4.1
C NC N/A U	FLAT SLAB FRAMES: The seismic-force-resisting system is not a frame consisting of columns and a flat slab or plate without beams.	5.5.2.3.1	A.3.1.4.3
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	PRESTRESSED FRAME ELEMENTS: The seismic-force-resisting frames do not include any prestressed or posttensioned elements where the average prestress exceeds the lesser of 700 lb/in.^{2} (4.83 MPa) or $f'_c/6$ at potential hinge locations. The average prestress is calculated in accordance with the Quick Check procedure of Section 4.4.3.8.	5.5.2.3.2	A.3.1.4.4
C NC N/A U	CAPTIVE COLUMNS: There are no columns at a level with height-to-depth ratios less than 50% of the nominal height-to-depth ratio of the typical columns at that level.	5.5.2.3.3	A.3.1.4.5
C NC N/A U	NO SHEAR FAILURES: The shear capacity of frame members is able to develop the moment capacity at the ends of the members.	5.5.2.3.4	A.3.1.4.6
C NC N/A U	STRONG COLUMN—WEAK BEAM: The sum of the moment capacity of the columns is 20% greater than that of the beams at frame joints.	5.5.2.1.5	A.3.1.4.7
C NC N/A U	BEAM BARS: At least two longitudinal top and two longitudinal bottom bars extend continuously throughout the length of each frame beam. At least 25% of the longitudinal bars provided at the joints for either positive or negative moment are continuous throughout the length of the members.	5.5.2.3.5	A.3.1.4.8
C NC N/A U	COLUMN-BAR SPLICES: All column-bar lap splice lengths are greater than $35d_b$ and are enclosed by ties spaced at or less than $8d_b$. Alternatively, column bars are spliced with mechanical couplers with a capacity of at least 1.25 times the nominal yield strength of the spliced bar.	5.5.2.3.6	A.3.1.4.9
C NC N/A U	BEAM-BAR SPLICES: The lap splices or mechanical couplers for longitudinal beam reinforcing are not located within $l_b/4$ of the joints and are not located in the vicinity of potential plastic hinge locations.	5.5.2.3.6	A.3.1.4.10
C NC N/A U	COLUMN-TIE SPACING: Frame columns have ties spaced at or less than $d/4$ throughout their length and at or less than $8d_b$ at all potential plastic hinge locations.	5.5.2.3.7	A.3.1.4.11
C NC N/A U	STIRRUP SPACING: All beams have stirrups spaced at or less than $d/2$ throughout their length. At potential plastic hinge locations, stirrups are spaced at or less than the minimum of $8d_b$ or $d/4$.	5.5.2.3.7	A.3.1.4.12

continues

Table 17-22 *(Continued)*. Collapse Prevention Structural Checklist for Building Type C1.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	JOINT TRANSVERSE REINFORCING: Beam–column joints have ties spaced at or less than $8d_b$.	5.5.2.3.8	A.3.1.4.13
C NC N/A U	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components.	5.5.2.5.2	A.3.1.6.2
C NC N/A U	FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints.	5.5.2.5.3	A.3.1.6.3
Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
Connections			
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps.	5.7.3.5	A.5.3.8

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-23. Immediate Occupancy Structural Checklist for Building Type C1.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2. The number of bays of moment frames in each line is greater than or equal to 3.	5.5.1.1	A.3.1.1.1
C NC N/A U	INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements.	5.5.2.1.1	A.3.1.2.1
C NC N/A U	COLUMN SHEAR STRESS CHECK: The shear stress in the concrete columns, calculated using the Quick Check procedure of Section 4.4.3.2 is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$.	5.5.2.1.4	A.3.1.4.1
C NC N/A U	COLUMN AXIAL STRESS CHECK: The axial stress caused by unfactored gravity loads in columns subjected to overturning demands is less than $0.13f'_c$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6 is less than $0.30f'_c$.	5.5.2.1.3	A.3.1.4.2
Connections			
C NC N/A U	CONCRETE COLUMNS: All concrete columns are doweled into the foundation, and the dowels are able to develop the tensile capacity of reinforcement in columns of the seismic-force-resisting system.	5.7.3.1	A.5.3.2
Low Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	FLAT SLAB FRAMES: The seismic-force-resisting system is not a frame consisting of columns and a flat slab or plate without beams.	5.5.2.3.1	A.3.1.4.3
C NC N/A U	PRESTRESSED FRAME ELEMENTS: The seismic-force-resisting frames do not include any prestressed or posttensioned elements where the average prestress exceeds the lesser of 700 lb/in.^{2} (4.83 MPa) or $f'_c/6$ at potential hinge locations. The average prestress is calculated in accordance with the Quick Check procedure of Section 4.4.3.8.	5.5.2.3.2	A.3.1.4.4
C NC N/A U	CAPTIVE COLUMNS: There are no columns at a level with height-to-depth ratios less than 75% of the nominal height-to-depth ratio of the typical columns at that level.	5.5.2.3.3	A.3.1.4.5
C NC N/A U	NO SHEAR FAILURES: The shear capacity of frame members is able to develop the moment capacity at the ends of the members.	5.5.2.3.4	A.3.1.4.6

continues

Table 17-23 *(Continued)*. Immediate Occupancy Structural Checklist for Building Type C1.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	STRONG COLUMN—WEAK BEAM: The sum of the moment capacity of the columns is 20% greater than that of the beams at frame joints.	5.5.2.1.5	A.3.1.4.7
C NC N/A U	BEAM BARS: At least two longitudinal top and two longitudinal bottom bars extend continuously throughout the length of each frame beam. At least 25% of the longitudinal bars provided at the joints for either positive or negative moment are continuous throughout the length of the members.	5.5.2.3.5	A.3.1.4.8
C NC N/A U	COLUMN-BAR SPLICES: All column-bar lap splice lengths are greater than $50d_b$ and are enclosed by ties spaced at or less than $8d_b$. Alternatively, column bars are spliced with mechanical couplers with a capacity of at least 1.25 times the nominal yield strength of the spliced bar.	5.5.2.3.6	A.3.1.4.9
C NC N/A U	BEAM-BAR SPLICES: The lap splices or mechanical couplers for longitudinal beam reinforcing are not located within $l_b/4$ of the joints and are not located in the vicinity of potential plastic hinge locations.	5.5.2.3.6	A.3.1.4.10
C NC N/A U	COLUMN-TIE SPACING: Frame columns have ties spaced at or less than $d/4$ throughout their length and at or less than $8d_b$ at all potential plastic hinge locations.	5.5.2.3.7	A.3.1.4.11
C NC N/A U	STIRRUP SPACING: All beams have stirrups spaced at or less than $d/2$ throughout their length. At potential plastic hinge locations, stirrups are spaced at or less than the minimum of $8d_b$ or $d/4$.	5.5.2.3.7	A.3.1.4.12
C NC N/A U	JOINT TRANSVERSE REINFORCING: Beam–column joints have ties spaced at or less than $8d_b$.	5.5.2.3.8	A.3.1.4.13
C NC N/A U	JOINT ECCENTRICITY: There are no eccentricities larger than 20% of the smallest column plan dimension between girder and column centerlines.	5.5.2.3.9	A.3.1.4.14
C NC N/A U	STIRRUP AND TIE HOOKS: The beam stirrups and column ties are anchored into the member cores with hooks of 135 degrees or more.	5.5.2.3.10	A.3.1.4.15
C NC N/A U	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components and are Compliant with the following items in this table: COLUMN-BAR SPLICES, BEAM-BAR SPLICES, COLUMN-TIE SPACING, STIRRUP SPACING, and STIRRUP AND TIE HOOKS.	5.5.2.5.2	A.3.1.6.2
C NC N/A U	FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints.	5.5.2.5.3	A.3.1.6.3
Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
Connections			
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps; the pile cap reinforcement and pile anchorage are able to develop the tensile capacity of the piles.	5.7.3.5	A.5.3.8
Moderate and High Seismicity (Complete the Following Items in Addition to the Items for Very Low and Low Seismicity)			
Foundation System			
C NC N/A U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$.	5.4.3.3	A.6.2.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-24. Collapse Prevention Structural Checklist for Building Types C2 and C2a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	CONCRETE BEARING WALLS: Floor and roof girders and trusses are not supported at the ends of concrete walls that are less than 10 in. (254 mm) thick. This statement only applies to framing supports located less than two times the wall thickness away from the wall end.	5.5.2.5.1	A.3.1.6.1
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.4.3.3 is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$.	5.5.3.1.1	A.3.2.2.1
C NC N/A U	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction.	5.5.3.1.3	A.3.2.2.2
Connections			
C NC N/A U	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls.	5.7.2	A.5.2.1
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing directly above the foundation.	5.7.3.4	A.5.3.5
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components.	5.5.2.5.2	A.3.1.6.2
C NC N/A U	FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints.	5.5.2.5.3	A.3.1.6.3
C NC N/A U	COUPLING BEAMS: Coupling beams have stirrups spaced at or less than d/2, and each wall or wall segment connected to the coupling beam is supported such that it can resist shear and overturning forces in the absence of the coupling beam. This statement only applies to coupling beams with span-to-depth ratios exceeding 2-to-1.	5.5.3.2.1	A.3.2.2.3
Diaphragms (Stiff or Flexible)			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
Flexible Diaphragms			
C NC N/A U	CROSSTIES: There are continuous crossties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 24 ft (7.3 m) and aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.2

continues

Table 17-24 *(Continued)*. Collapse Prevention Structural Checklist for Building Types C2 and C2a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	BLOCKED DIAPHRAGMS: All blocked wood structural panel diaphragms have horizontal spans less than 120 ft (36.5 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	CANTILEVERED WOOD DIAPHRAGMS: All cantilevered wood diaphragms that provide lateral support for concrete or masonry walls consist of wood structural panels and have a maximum cantilever length of 20 ft (6.1 m) if unblocked or 35 ft (10.7 m) if blocked, and a maximum ratio of cantilever length to diaphragm width of 1:2 if unblocked and 1:1 if blocked. In addition, the cantilevered diaphragm has a back-span length equal to or greater than the cantilevered portion.	5.6.2	A.4.2.4
C NC N/A U	NON-CONCRETE-FILLED DIAPHRAGMS: Bare steel deck diaphragms or steel deck diaphragms with fill other than reinforced structural concrete consist of horizontal spans of less than 120 ft (36.5 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps.	5.7.3.5	A.5.3.8

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-25. Immediate Occupancy Structural Checklist for Building Types C2 and C2a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	CONCRETE BEARING WALLS: Floor and roof girders and trusses are not supported at the ends of concrete walls that are less than 10 in. (254 mm) thick. This statement only applies to framing supports located less than two times the wall thickness away from the wall end.	5.5.2.5.1	A.3.1.6.1
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.4.3.3 is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$.	5.5.3.1.1	A.3.2.2.1
C NC N/A U	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. The spacing of reinforcing steel is equal to or less than 18 in. (457.2 mm).	5.5.3.1.3	A.3.2.2.2
Connections			
C NC N/A U	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of loads to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms.	5.7.2	A.5.2.1

continues

Table 17-25 *(Continued)*. Immediate Occupancy Structural Checklist for Building Types C2 and C2a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation, and the dowels are able to develop the lesser of the strength of the walls or the uplift capacity of the foundation.	5.7.3.4	A.5.3.5
Low Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components and are compliant with the following items in Table 17-23: COLUMN-BAR SPLICES, BEAM-BAR SPLICES, COLUMN-TIE SPACING, STIRRUP SPACING, and STIRRUP AND TIE HOOKS.	5.5.2.5.2	A.3.1.6.2
C NC N/A U	FLAT SLABS: Flat slabs or plates not part of seismic-force-resisting system have continuous bottom steel through the column joints.	5.5.2.5.3	A.3.1.6.3
C NC N/A U	COUPLING BEAMS: Coupling beams have the capacity in shear to develop the uplift capacity of the adjacent wall or to develop the flexural capacity of the coupling beam, whichever is less. This statement only applies to coupling beams with span-to-depth ratios exceeding 2-to-1.	5.5.3.2.1	A.3.2.2.3
C NC N/A U	OVERTURNING: All shear walls have aspect ratios less than 4-to-1. Wall piers need not be considered.	5.5.3.1.4	A.3.2.2.4
C NC N/A U	CONFINEMENT REINFORCING: For shear walls with aspect ratios greater than 2-to-1, the boundary elements are confined with spirals or ties with spacing less than $8d_b$.	5.5.3.2.2	A.3.2.2.5
C NC N/A U	WALL REINFORCING AT OPENINGS: There is added trim reinforcement around all wall openings with a dimension greater than three times the thickness of the wall.	5.5.3.1.5	A.3.2.2.6
C NC N/A U	WALL THICKNESS: Thicknesses of bearing walls are not less than 1/25 the unsupported height or length, whichever is shorter, nor less than 4 in. (101.6 mm).	5.5.3.1.2	A.3.2.2.7
Diaphragms (Stiff or Flexible)			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
Flexible Diaphragms			
C NC N/A U	CROSSTIES: There are continuous crossties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 12 ft (3.6 m) and aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.2 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.2
C NC N/A U	BLOCKED DIAPHRAGMS: All blocked wood structural panel diaphragms have horizontal spans less than 90 ft (27.4 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3

continues

Table 17-25 *(Continued)*. Immediate Occupancy Structural Checklist for Building Types C2 and C2a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	CANTILEVERED WOOD DIAPHRAGMS: All cantilevered wood diaphragms that provide lateral support for concrete or masonry walls consist of wood structural panels and have a maximum cantilever length of 15 ft (4.6 m) if unblocked or 25 ft (7.6 m) if blocked, and a maximum ratio of cantilever length to diaphragm width of 1:2.5 if unblocked and 1:1.5 if blocked. In addition, the cantilevered diaphragm has a back-span length equal to or greater than the cantilevered portion.	5.6.2	A.4.2.4
C NC N/A U	NON-CONCRETE-FILLED DIAPHRAGMS: Bare steel deck diaphragms or steel deck diaphragms with fill other than reinforced concrete consist of horizontal spans of less than 40 ft (12.2 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps; the pile cap reinforcement and pile anchorage are able to develop the tensile capacity of the piles.	5.7.3.5	A.5.3.8
Moderate and High Seismicity (Complete the Following Items in Addition to the Items for Very Low and Low Seismicity)			
Foundation System			
C NC N/A U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$.	5.4.3.3	A.6.2.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-26. Collapse Prevention Structural Checklist for Building Types C3 and C3a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3 is less than 70 lb/in.^{2} (0.48 MPa).	5.5.3.1.1	A.3.2.4.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the unreinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3 is less than 30 lb/in.^{2} (0.21 MPa) for clay units and 70 lb/in.^{2} (0.48 MPa) for concrete units. Bays with openings greater than 25% of the wall area shall not be included in A_w of Equation (4-8).	5.5.3.1.1	A.3.2.5.1
C NC N/A U	INFILL WALL CONNECTIONS: Masonry is in full contact with frame.	5.5.3.5.1 5.5.3.5.3	A.3.2.6.1
Connections			
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of loads to the shear walls.	5.7.2	A.5.2.1
C NC N/A U	CONCRETE COLUMNS: All concrete columns are doweled into the foundation with a minimum of four bars.	5.7.3.1	A.5.3.2
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components.	5.5.2.5.2	A.3.1.6.2

continues

Table 17-26 *(Continued)*. Collapse Prevention Structural Checklist for Building Types C3 and C3a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints.	5.5.2.5.3	A.3.1.6.3
C NC N/A U	PROPORTIONS: The height-to-thickness ratio of the unreinforced infill walls at each story is less than 9.	5.5.3.1.2	A.3.2.6.2
C NC N/A U	CAVITY WALLS: The infill walls are not of cavity construction.	5.5.3.5.2	A.3.2.6.3
C NC N/A U	INFILL WALLS: The infill walls are continuous to the soffits of the frame beams and to the columns to either side.	5.5.3.5.3	A.3.2.6.4
Diaphragms (Stiff or Flexible)			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft (2.4 m) long.	5.6.1.3	A.4.1.6
Flexible Diaphragms			
C NC N/A U	CROSSTIES: There are continuous crossties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 24 ft (7.3 m) and aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.2
C NC N/A U	BLOCKED DIAPHRAGMS: All blocked wood structural panel diaphragms have horizontal spans less than 120 ft (36.5 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	CANTILEVERED WOOD DIAPHRAGMS: All cantilevered wood diaphragms that provide lateral support for concrete or masonry walls consist of wood structural panels and have a maximum cantilever length of 20 ft (6.1 m) if unblocked or 35 ft (10.7 m) if blocked, and a maximum ratio of cantilever length to diaphragm width of 1:2 if unblocked and 1:1 if blocked. In addition, the cantilevered diaphragm has a back-span length equal to or greater than the cantilevered portion.	5.6.2	A.4.2.4
C NC N/A U	NON-CONCRETE-FILLED DIAPHRAGMS: Bare steel deck diaphragms or steel deck diaphragms with fill other than reinforced structural concrete consist of horizontal spans of less than 120 ft (36.5 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps.	5.7.3.5	A.5.3.8
C NC N/A U	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. (3.1 mm) before engagement of the anchors.	5.7.1.2	A.5.1.4

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-27. Immediate Occupancy Structural Checklists for Building Types C3 and C3a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3 is less than 70 lb/in.^{2} (0.48 MPa).	5.5.3.1.1	A.3.2.4.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the unreinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3 is less than 30 lb/in.^{2} (0.21 MPa) for clay units and 70 lb/in.^{2} (0.48 MPa) for concrete units. Bays with openings greater than 25% of the wall area shall not be included in A_w of Equation (4-8).	5.5.3.1.1	A.3.2.5.1
C NC N/A U	INFILL WALL CONNECTIONS: Masonry is in full contact with frame.	5.5.3.5.1 5.5.3.5.3	A.3.2.6.1
Connections			
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of loads to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms.	5.7.2	A.5.2.1
C NC N/A U	CONCRETE COLUMNS: All concrete columns are doweled into the foundation with a minimum of four bars, and the dowels are able to develop the tensile capacity of reinforcement in columns of the seismic-force-resisting system.	5.7.3.1	A.5.3.2
Low Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components and are Compliant with the following items in Table 17-23: COLUMN-BAR SPLICES, BEAM-BAR SPLICES, COLUMN-TIE SPACING, STIRRUP SPACING, and STIRRUP AND TIE HOOKS.	5.5.2.5.2	A.3.1.6.2
C NC N/A U	FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints.	5.5.2.5.3	A.3.1.6.3
C NC N/A U	REINFORCING AT WALL OPENINGS: All wall openings that interrupt rebar have trim reinforcing on all sides.	5.5.3.1.5	A.3.2.4.3
C NC N/A U	PROPORTIONS: The height-to-thickness ratio of the unreinforced infill walls at each story is less than 13.	5.5.3.1.2	A.3.2.6.2
C NC N/A U	CAVITY WALLS: The infill walls are not of cavity construction.	5.5.3.5.2	A.3.2.6.3
C NC N/A U	INFILL WALLS: The infill walls are continuous to the soffits of the frame beams and to the columns to either side.	5.5.3.5.3	A.3.2.6.4
Diaphragms (Stiff or Flexible)			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 4 ft (1.2 m) long.	5.6.1.3	A.4.1.6
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8

continues

Table 17-27 *(Continued)*. Immediate Occupancy Structural Checklists for Building Types C3 and C3a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Flexible Diaphragms			
C NC N/A U	CROSSTIES: There are continuous crossties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 12 ft (3.6 m) and aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.1 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.2
C NC N/A U	BLOCKED DIAPHRAGMS: All blocked wood structural panel diaphragms have horizontal spans less than 90 ft (27.4 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	CANTILEVERED WOOD DIAPHRAGMS: All cantilevered wood diaphragms that provide lateral support for concrete or masonry walls consist of wood structural panels and have a maximum cantilever length of 15 ft (4.6 m) if unblocked or 25 ft (7.6 m) if blocked, and a maximum ratio of cantilever length to diaphragm width of 1:2.5 if unblocked and 1:1.5 if blocked. In addition, the cantilevered diaphragm has a back-span length equal to or greater than the cantilevered portion.	5.6.2	A.4.2.4
C NC N/A U	NON-CONCRETE-FILLED DIAPHRAGMS: Bare steel deck diaphragms or steel deck diaphragms with fill other than reinforced concrete consist of horizontal spans of less than 40 ft (12.2 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps; the pile cap reinforcement and pile anchorage are able to develop the tensile capacity of the piles.	5.7.3.5	A.5.3.8
C NC N/A U	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. (3.1 mm) before engagement of the anchors.	5.7.1.2	A.5.1.4
Moderate Seismicity (Complete the Following Items in Addition to the Items for Very Low and Low Seismicity)			
Foundation System			
C NC N/A U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than 0.6S_a.	5.4.3.3	A.6.2.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	PROPORTIONS: The height-to-thickness ratio of the unreinforced infill walls at each story is less than 8.	5.5.3.1.2	A.3.2.6.2

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

(NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

17.14 STRUCTURAL CHECKLISTS FOR BUILDING TYPES PC1: PRECAST OR TILT-UP CONCRETE SHEAR WALLS WITH FLEXIBLE DIAPHRAGMS, AND PC1A: PRECAST OR TILT-UP CONCRETE SHEAR WALLS WITH STIFF DIAPHRAGMS

For building systems and configurations that comply with the PC1 or PC1a building type description in Table 3-1, the Collapse Prevention Structural Checklist in Table 17-28 shall be completed where required by Table 4-6 for Collapse Prevention Structural Performance, and the Immediate Occupancy Structural

Table 17-28. Collapse Prevention Structural Checklist for Building Types PC1 and PC1a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low Seismicity			
Connections			
C NC N/A U	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	WALL SHEAR STRESS CHECK: The shear stress in the precast panels, calculated using the Quick Check procedure of Section 4.4.3.3 is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$.	5.5.3.1.1	A.3.2.3.1
C NC N/A U	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction.	5.5.3.1.3	A.3.2.3.2
C NC N/A U	WALL THICKNESS: Thicknesses of bearing walls are not less than 1/40 the unsupported height or length, whichever is shorter, nor less than 4 in. (101.6 mm).	5.5.3.1.2	A.3.2.3.5
Diaphragms			
C NC N/A U	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab with a minimum thickness of 2 in. (50.8 mm).	5.6.4	A.4.5.1
Connections			
C NC N/A U	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers or top plates fastened to the walls.	5.7.1.3	A.5.1.2
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls.	5.7.2	A.5.2.1
C NC N/A U	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements.	5.7.2	A.5.2.3
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	DEFLECTION COMPATIBILITY FOR RIGID DIAPHRAGMS: Secondary components have the shear capacity to develop the flexural strength of the components.	5.5.2.5.2	A.3.1.6.2
C NC N/A U	WALL OPENINGS: The total combined width of openings and wall piers with aspect ratios greater than 2-to-1 along any perimeter wall line constitutes less than 75% of the total length of any perimeter wall.	5.5.3.3.1	A.3.2.3.3
Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	CROSSTIES IN FLEXIBLE DIAPHRAGMS: There are continuous crossties between diaphragm chords to distribute the out-of-plane wall anchorage forces into the diaphragm. Where each out-of-plane connection does not have a continuous crosstie across the entire diaphragm, these connections are developed into subdiaphragms between crossties with a maximum length-to-width ratio of 3-to-1.	5.6.1.2	A.4.1.2

continues

Table 17-28 *(Continued)*. Collapse Prevention Structural Checklist for Building Types PC1 and PC1a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 24 ft (7.3 m) and aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.2
C NC N/A U	BLOCKED DIAPHRAGMS: All blocked wood structural panel diaphragms have horizontal spans less than 120 ft (36.5 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	CANTILEVERED WOOD DIAPHRAGMS: All cantilevered wood diaphragms that provide lateral support for concrete or masonry walls consist of wood structural panels and have a maximum cantilever length of 20 ft (6.1 m) if unblocked or 35 ft (10.7 m) if blocked, and a maximum ratio of cantilever length to diaphragm width of 1:2 if unblocked and 1:1 if blocked. In addition, the cantilevered diaphragm has a back-span length equal to or greater than the cantilevered portion.	5.6.2	A.4.2.4
C NC N/A U	NON-CONCRETE-FILLED DIAPHRAGMS: Bare steel deck diaphragms or steel deck diaphragms with fill other than reinforced structural concrete consist of horizontal spans of less than 120 ft (36.5 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	MINIMUM NUMBER OF WALL ANCHORS PER PANEL: There are at least two anchors connecting each precast wall panel to the diaphragm elements.	5.7.1.4	A.5.1.3
C NC N/A U	PRECAST WALL PANELS: Precast wall panels are connected to the foundation.	5.7.3.4	A.5.3.6
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps.	5.7.3.5	A.5.3.8
C NC N/A U	GIRDERS: Girders supported by walls or pilasters have at least two ties securing the anchor bolts unless provided with independent stiff wall anchors with strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.4.2	A.5.4.2

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-29. Immediate Occupancy Structural Checklist for Building Types PC1 and PC1a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	WALL SHEAR STRESS CHECK: The shear stress in the precast panels, calculated using the Quick Check procedure of Section 4.4.3.3 is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$.	5.5.3.1.1	A.3.2.3.1
C NC N/A U	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. The spacing of reinforcing steel is equal to or less than 18 in. (457 mm).	5.5.3.1.3	A.3.2.3.2

continues

Table 17-29 *(Continued)*. Immediate Occupancy Structural Checklist for Building Types PC1 and PC1a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Diaphragms (Stiff or Flexible)			
C NC N/A U	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab with a minimum thickness of 2 in. (50.8 mm).	5.6.4	A.4.5.1
Connections			
C NC N/A U	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers or top plates fastened to the walls.	5.7.1.4	A.5.1.2
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms.	5.7.2	A.5.2.1
C NC N/A U	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements, and the dowels are able to develop the least of the shear strength of the walls, frames, or slabs.	5.7.2	A.5.2.3
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
Low Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	DEFLECTION COMPATIBILITY FOR RIGID DIAPHRAGMS: Secondary components have the shear capacity to develop the flexural strength of the components.	5.5.2.5.2	A.3.1.6.2
C NC N/A U	WALL OPENINGS: The total combined width of openings and wall piers with aspect ratios greater than 2-to-1 along any perimeter wall line constitutes less than 50% of the total length of any perimeter wall.	5.5.3.3.1	A.3.2.3.3
C NC N/A U	PANEL-TO-PANEL CONNECTIONS: Adjacent wall panels are interconnected to transfer overturning forces between panels by methods other than welded steel inserts.	5.5.3.3.3	A.3.2.3.4
C NC N/A U	WALL THICKNESS: Thicknesses of bearing walls are not less than 1/25 the unsupported height or length, whichever is shorter, nor less than 4 in. (101.6 mm).	5.5.3.1.2	A.3.2.3.5
Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	CROSSTIES FOR FLEXIBLE DIAPHRAGMS: There are continuous crossties between diaphragm chords to distribute the out-of-plane wall anchorage forces into the diaphragm. Where each out-of-plane connection does not have a continuous crosstie across the entire diaphragm, these connections are developed into subdiaphragms between crossties with a maximum length-to-width ratio of 3-to-1.	5.6.1.2	A.4.1.2
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8

continues

Table 17-29 *(Continued)*. Immediate Occupancy Structural Checklist for Building Types PC1 and PC1a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 12 ft (3.6 m) and aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.1 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.2
C NC N/A U	BLOCKED DIAPHRAGMS: All blocked wood structural panel diaphragms have horizontal spans less than 90 ft (27.4 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	CANTILEVERED WOOD DIAPHRAGMS: All cantilevered wood diaphragms that provide lateral support for concrete or masonry walls consist of wood structural panels and have a maximum cantilever length of 15 ft (4.6 m) if unblocked or 25 ft (7.6 m) if blocked, and a maximum ratio of cantilever length to diaphragm width of 1:2.5 if unblocked and 1:1.5 if blocked. In addition, the cantilevered diaphragm has a back-span length equal to or greater than the cantilevered portion.	5.6.2	A.4.2.4
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	MINIMUM NUMBER OF WALL ANCHORS PER PANEL: There are at least two anchors connecting each precast wall panel into the diaphragm elements.	5.7.1.4	A.5.1.3
C NC N/A U	PRECAST WALL PANELS: Precast wall panels are connected to the foundation, and the connections are able to develop the strength of the walls.	5.7.3.4	A.5.3.6
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps; the pile cap reinforcement and pile anchorage are able to develop the tensile capacity of the piles.	5.7.3.5	A.5.3.8
C NC N/A U	GIRDERS: Girders supported by walls or pilasters have at least two ties securing the anchor bolts unless provided with independent stiff wall anchors with strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.4.2	A.5.4.2
Moderate and High Seismicity (Complete the Following Items in Addition to the Items for Very Low and Low Seismicity)			
Foundation System			
C NC N/A U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$.	5.4.3.3	A.6.2.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Checklist in Table 17-29 shall be completed where required by Table 4-6 for Immediate Occupancy Structural Performance. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

17.15 STRUCTURAL CHECKLISTS FOR BUILDING TYPE PC2: PRECAST CONCRETE FRAMES WITH SHEAR WALLS

For building systems and configurations that comply with the PC2 building type description in Table 3-1, the Collapse Prevention Structural Checklist in Table 17-30 shall be completed where required by Table 4-6 for Collapse Prevention Structural Performance, and the Immediate Occupancy Structural Checklist in Table 17-31 shall be completed where required by Table 4-6 for Immediate Occupancy Structural Performance. Tier 1

Table 17-30. Collapse Prevention Structural Checklist for Building Type PC2.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	CONCRETE BEARING WALLS: Floor and roof girders and trusses are not supported at the ends of concrete walls that are less than 10 in. (254 mm) thick. This statement only applies to framing supports located less than two times the wall thickness away from the wall end.	5.5.2.5.1	A.3.1.6.1
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.4.3.3 is less than the greater of 100 lb/in.^2 (0.69 MPa) or $2\sqrt{f'_c}$.	5.5.3.1.1	A.3.2.2.1
C NC N/A U	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction.	5.5.3.1.3	A.3.2.2.2
Diaphragms			
C NC N/A U	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab with a minimum thickness of 2 in. (50.8 mm).	5.6.4	A.4.5.1
Connections			
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls.	5.7.2	A.5.2.1
C NC N/A U	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements.	5.7.2	A.5.2.3
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation.	5.7.3.4	A.5.3.5
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	PRECAST FRAMES: For buildings with concrete shear walls, precast concrete frame elements are not considered as primary components for resisting seismic forces.	5.5.2.4 5.5.2.5.1 5.5.2.5.2	A.3.1.5.2
C NC N/A U	PRECAST CONNECTIONS: For buildings with concrete shear walls, the connection between precast frame elements, such as chords, ties, and collectors in the seismic-force-resisting system, develops the capacity of the connected members.	5.6.1.1	A.3.1.5.3
C NC N/A U	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components.	5.5.2.5.2	A.3.1.6.2
C NC N/A U	COUPLING BEAMS: Coupling beams have stirrups spaced at or less than d/2, and each wall or wall segment connected to the coupling beam is supported such that it can resist shear and overturning forces in the absence of the coupling beam. This statement only applies to coupling beams with span-to-depth ratios exceeding 2-to-1.	5.5.3.2.1	A.3.2.2.3
Diaphragms			
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.5.3.3.1	A.4.1.4
Connections			
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps.	5.7.3.5	A.5.3.8
C NC N/A U	CORBEL BEARING: If the frame girders bear on column corbels, the length of bearing is greater than 3 in. (76 mm).	5.7.4.3	A.5.4.3
C NC N/A U	CORBEL CONNECTIONS: The frame girders are not connected to corbels with welded elements.	5.7.4.3	A.5.4.4

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-31. Immediate Occupancy Structural Checklist for Building Type PC2.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	CONCRETE BEARING WALLS: Floor and roof girders and trusses are not supported at the ends of concrete walls that are less than 10 in. (254 mm) thick. This statement only applies to framing supports located less than two times the wall thickness away from the wall end.	5.5.2.5.1	A.3.1.6.1
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.4.3.3 is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$.	5.5.3.1.1	A.3.2.2.1
C NC N/A U	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. The spacing of reinforcing steel is equal to or less than 18 in. (457 mm).	5.5.3.1.3	A.3.2.2.2
Diaphragms			
C NC N/A U	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab with a minimum thickness of 2 in. (50.8 mm).	5.6.4	A.4.5.1
Connections			
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms.	5.7.2	A.5.2.1
C NC N/A U	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements, and the dowels are able to develop the least of the shear strength of the walls, frames, or slabs.	5.7.2	A.5.2.3
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation, and the dowels are able to develop the lesser of the strength of the walls or the uplift capacity of the foundation.	5.7.3.4	A.5.3.5
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
Low Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	PRECAST FRAMES: For buildings with concrete shear walls, precast concrete frame elements are not considered as primary components for resisting seismic forces.	5.5.2.4 5.5.2.5.1 5.5.2.5.2	A.3.1.5.2
C NC N/A U	PRECAST CONNECTIONS: For buildings with concrete shear walls, the connection between precast frame elements, such as chords, ties, and collectors in the seismic-force-resisting system, develops the capacity of the connected members.	5.6.1.1	A.3.1.5.3
C NC N/A U	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components.	5.5.2.5.2	A.3.1.6.2
C NC N/A U	COUPLING BEAMS: Coupling beams have the capacity in shear to develop the uplift capacity of the adjacent wall or to develop the flexural capacity of the coupling beam, whichever is less. This statement only applies to coupling beams with span-to-depth ratios exceeding 2-to-1.	5.5.3.2.1	A.3.2.2.3
C NC N/A U	OVERTURNING: All shear walls have aspect ratios less than 4-to-1. Wall piers need not be considered.	5.5.3.1.4	A.3.2.2.4
C NC N/A U	CONFINEMENT REINFORCING: For shear walls with aspect ratios greater than 2-to-1, the boundary elements are confined with spirals or ties with spacing less than $8d_b$.	5.5.3.2.2	A.3.2.2.5

continues

Table 17-31 *(Continued)*. Immediate Occupancy Structural Checklist for Building Type PC2.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	WALL REINFORCING AT OPENINGS: There is added trim reinforcement around all wall openings with a dimension greater than three times the thickness of the wall.	5.5.3.1.5	A.3.2.2.6
C NC N/A U	WALL THICKNESS: Thickness of bearing walls is not less than 1/25 the unsupported height or length, whichever is shorter, nor less than 4 in. (101.6 mm).	5.5.3.1.2	A.3.2.3.5
Diaphragms			
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
Connections			
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps; the pile cap reinforcement and pile anchorage are able to develop the tensile capacity of the piles.	5.7.3.5	A.5.3.8
C NC N/A U	CORBEL BEARING: If the frame girders bear on column corbels, the length of bearing is greater than 3 in. (76 mm).	5.7.4.3	A.5.4.3
C NC N/A U	CORBEL CONNECTIONS: The frame girders are not connected to corbels with welded elements.	5.7.4.3	A.5.4.4
Moderate and High Seismicity (Complete the Following Items in Addition to the Items for Very Low and Low Seismicity)			
Foundation System			
C NC N/A U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$.	5.4.3.3	A.6.2.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

17.16 STRUCTURAL CHECKLISTS FOR BUILDING TYPE PC2A: PRECAST CONCRETE FRAMES WITHOUT SHEAR WALLS

For building systems and configurations that comply with the PC2a building type description in Table 3-1, the Collapse Prevention Structural Checklist in Table 17-32 shall be completed where required by Table 4-6 for Collapse Prevention Structural Performance, and the Immediate Occupancy Structural Checklist in Table 17-33 shall be completed where required by Table 4-6 for Immediate Occupancy Structural Performance. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

17.17 STRUCTURAL CHECKLISTS FOR BUILDING TYPES RM1: REINFORCED MASONRY BEARING WALLS WITH FLEXIBLE DIAPHRAGMS, AND RM2: REINFORCED MASONRY BEARING WALLS WITH STIFF DIAPHRAGMS

For building systems and configurations that comply with the RM1 or RM2 building type description in Table 3-1, the Collapse Prevention Structural Checklist in Table 17-34 shall be completed where required by Table 4-6 for Collapse Prevention Structural Performance, and the Immediate Occupancy Structural Checklist in Table 17-35 shall be completed where required by Table 4-6 for Immediate Occupancy Structural Performance. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Table 17-32. Collapse Prevention Structural Checklist for Building Type PC2a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2. The number of bays of moment frames in each line is greater than or equal to 2.	5.5.1.1	A.3.1.1.1
C NC N/A U	COLUMN SHEAR STRESS CHECK: The shear stress in the concrete columns, calculated using the Quick Check procedure of Section 4.4.3.2 is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$.	5.5.2.1.4	A.3.1.4.1
C NC N/A U	COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10f'_c$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6 is less than $0.30f'_c$.	5.5.2.1.3	A.3.1.4.2
C NC N/A U	PRECAST CONNECTION CHECK: The precast connections at frame joints have the capacity to resist the shear and moment demands calculated using the Quick Check procedure of Section 4.4.3.5.	5.5.2.4	A.3.1.5.1
Diaphragms			
C NC N/A U	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab with a minimum thickness of 2 in. (50.8 mm).	5.6.4	A.4.5.1
Connections			
C NC N/A U	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements, and the dowels are able to develop the least of the shear strength of the walls, frames, or slabs.	5.7.2	A.5.2.3
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	PRESTRESSED FRAME ELEMENTS: The seismic-force-resisting frames do not include any prestressed or posttensioned elements where the average prestress exceeds the lesser of 700 lb/in.^{2} (4.83 MPa) or $f'_c/6$ at potential hinge locations. The average prestress is calculated in accordance with the Quick Check procedure of Section 4.4.3.8.	5.5.2.3.2	A.3.1.4.4
C NC N/A U	CAPTIVE COLUMNS: There are no columns at a level with height-to-depth ratios less than 50% of the nominal height-to-depth ratio of the typical columns at that level.	5.5.2.3.3	A.3.1.4.5
C NC N/A U	JOINT REINFORCING: Beam–column joints have ties spaced at or less than $8d_b$.	5.5.2.3.8	A.3.1.4.13
C NC N/A U	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components.	5.5.2.5.2	A.3.1.6.2
Connections			
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps.	5.7.3.5	A.5.3.8
C NC N/A U	GIRDERS: Girders supported by walls or pilasters have at least two ties securing the anchor bolts unless provided with independent stiff wall anchors with strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.4.2	A.5.4.2
C NC N/A U	CORBEL BEARING: If the frame girders bear on column corbels, the length of bearing is greater than 3 in. (76 mm).	5.7.4.3	A.5.4.3
C NC N/A U	CORBEL CONNECTIONS: The frame girders are not connected to corbels with welded elements.	5.7.4.3	A.5.4.4

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-33. Immediate Occupancy Structural Checklist for Building Type PC2a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to two. The number of bays of moment frames in each line is greater than or equal to 3.	5.5.1.1	A.3.1.1.1
C NC N/A U	COLUMN SHEAR STRESS CHECK: The shear stress in the concrete columns, calculated using the Quick Check procedure of Section 4.4.3.2 is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$.	5.5.2.1.4	A.3.1.4.1
C NC N/A U	COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10f'_c$. Alternatively, the axial stresses caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6 is less than $0.30f'_c$.	5.5.2.1.3	A.3.1.4.2
C NC N/A U	PRECAST CONNECTION CHECK: The precast connections at frame joints have the capacity to resist the shear and moment demands calculated using the Quick Check procedure of Section 4.4.3.5.	5.5.2.4	A.3.1.5.1
Diaphragms			
C NC N/A U	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab with a minimum thickness of 2 in. (50.8 mm).	5.6.4	A.4.5.1
Connections			
C NC N/A U	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements, and the dowels are able to develop the least of the shear strength of the walls, frames, or slabs.	5.7.2	A.5.2.3
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
Low Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	PRESTRESSED FRAME ELEMENTS: The seismic-force-resisting frames do not include any prestressed or posttensioned elements where the average prestress exceeds the lesser of 700 lb/in.^{2} (4.83 MPa) or $f'_c/6$ at potential hinge locations. The average prestress is calculated in accordance with the Quick Check procedure of Section 4.4.3.8.	5.5.2.3.2	A.3.1.4.4
C NC N/A U	CAPTIVE COLUMNS: There are no columns at a level with height-to-depth ratios less than 75% of the nominal height-to-depth ratio of the typical columns at that level.	5.5.2.3.3	A.3.1.4.5
C NC N/A U	JOINT REINFORCING: Beam–column joints have ties spaced at or less than $8d_b$.	5.5.2.3.8	A.3.1.4.13
C NC N/A U	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components.	5.5.2.5.2	A.3.1.6.2
Diaphragms			
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
Connections			
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps; the pile cap reinforcement and pile anchorage are able to develop the tensile capacity of the piles.	5.7.3.5	A.5.3.8
C NC N/A U	GIRDERS: Girders supported by frames have at least two ties securing the anchor bolts unless provided with independent stiff wall anchors with strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.4.2	A.5.4.2

continues

Table 17-33 *(Continued)*. Immediate Occupancy Structural Checklist for Building Type PC2a.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	CORBEL BEARING: If the frame girders bear on column corbels, the length of bearing is greater than 3 in. (76 mm).	5.7.4.3	A.5.4.3
C NC N/A U	CORBEL CONNECTIONS: The frame girders are not connected to corbels with welded elements.	5.7.4.3	A.5.4.4
C NC N/A U	TRANSFER TO FRAMES: Diaphragms are connected for transfer of loads to the frames.	5.7.2	A.5.2.1
Moderate and High Seismicity (Complete the Following Items in Addition to the Items for Very Low and Low Seismicity)			
Foundation System			
C NC N/A U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$.	5.4.3.3	A.6.2.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-34. Collapse Prevention Structural Checklist for Building Types RM1 and RM2.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3 is less than 70 lb/in.^{2} (0.48 MPa).	5.5.3.1.1	A.3.2.4.1
C NC N/A U	REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 of the wall with the minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 in. (1.2 m), and all vertical bars extend to the top of the walls.	5.5.3.1.3	A.3.2.4.2
Stiff Diaphragms			
C NC N/A U	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab.	5.6.4	A.4.5.1
Connections			
C NC N/A U	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers or top plates fastened to the walls.	5.7.1.3	A.5.1.2
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls.	5.7.2	A.5.2.1
C NC N/A U	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements.	5.7.2	A.5.2.3
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation.	5.7.3.4	A.5.3.5
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1

continues

Table 17-34 *(Continued)*. Collapse Prevention Structural Checklist for Building Types RM1 and RM2.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Stiff Diaphragms			
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft (2.4 m) long.	5.6.1.3	A.4.1.6
Flexible Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	CROSSTIES: There are continuous crossties between diaphragm chords to distribute the out-of-plane wall anchorage forces into the diaphragm. Where each out-of-plane connection does not have a continuous crosstie across the entire diaphragm, these connections are developed into subdiaphragms between crossties with a maximum length-to-width ratio of 3-to-1.	5.6.1.2	A.4.1.2
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft (2.4 m) long.	5.6.1.3	A.4.1.6
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 24 ft (7.3 m) and aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.2
C NC N/A U	BLOCKED DIAPHRAGMS: All blocked wood structural panel diaphragms have horizontal spans less than 120 ft (36.5 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	CANTILEVERED WOOD DIAPHRAGMS: All cantilevered wood diaphragms that provide lateral support for concrete or masonry walls consist of wood structural panels and have a maximum cantilever length of 20 ft (6.1 m) if unblocked or 35 ft (10.7 m) if blocked, and a maximum ratio of cantilever length to diaphragm width of 1:2 if unblocked and 1:1 if blocked. In addition, the cantilevered diaphragm has a back-span length equal to or greater than the cantilevered portion.	5.6.2	A.4.2.4
C NC N/A U	NON-CONCRETE-FILLED DIAPHRAGMS: Bare steel deck diaphragms or steel deck diaphragms with fill other than reinforced structural concrete consist of horizontal spans of less than 120 ft (36.5 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. (3.1 mm) before engagement of the anchors.	5.7.1.2	A.5.1.4

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-35. Immediate Occupancy Structural Checklist for Building Types RM1 and RM2.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3 is less than 70 lb/in.^{2} (0.48 MPa).	5.5.3.1.1	A.3.2.4.1
C NC N/A U	REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 of the wall with the minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 in. (1.2 m), and all vertical bars extend to the top of the walls.	5.5.3.1.3	A.3.2.4.2
Connections			
C NC N/A U	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers or top plates fastened to the walls.	5.7.1.3	A.5.1.2
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms.	5.7.2	A.5.2.1
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation, and the dowels are able to develop the lesser of the strength of the walls or the uplift capacity of the foundation.	5.7.3.4	A.5.3.5
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
Stiff Diaphragms			
C NC N/A U	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab.	5.6.4	A.4.5.1
C NC N/A U	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements.	5.7.2	A.5.2.3
Low, Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	REINFORCING AT WALL OPENINGS: All wall openings that interrupt rebar have trim reinforcing on all sides.	5.5.3.1.5	A.3.2.4.3
C NC N/A U	PROPORTIONS: The height-to-thickness ratio of the shear walls at each story is less than 30.	5.5.3.1.2	A.3.2.4.4
Diaphragms (Stiff or Flexible)			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 4 ft (1.2 m) long.	5.6.1.3	A.4.1.6
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7

continues

Table 17-35 *(Continued)*. Immediate Occupancy Structural Checklist for Building Types RM1 and RM2.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
Flexible Diaphragms			
C NC N/A U	CROSSTIES: There are continuous crossties between diaphragm chords to distribute the out-of-plane wall anchorage forces into the diaphragm. Where each out-of-plane connection does not have a continuous crosstie across the entire diaphragm, these connections are developed into subdiaphragms between crossties with a maximum length-to-width ratio of 3-to-1.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 12 ft (3.6 m) and aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.1 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.2
C NC N/A U	BLOCKED DIAPHRAGMS: All blocked wood structural panel diaphragms have horizontal spans less than 90 ft (27.4 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	CANTILEVERED WOOD DIAPHRAGMS: All cantilevered wood diaphragms that provide lateral support for concrete or masonry walls consist of wood structural panels and have a maximum cantilever length of 15 ft (4.6 m) if unblocked or 25 ft (7.6 m) if blocked, and a maximum ratio of cantilever length to diaphragm width of 1:2.5 if unblocked and 1:1.5 if blocked. In addition, the cantilevered diaphragm has a back-span length equal to or greater than the cantilevered portion.	5.6.2	A.4.2.4
C NC N/A U	NON-CONCRETE-FILLED DIAPHRAGMS: Bare steel deck diaphragms or steel deck diaphragms with fill other than reinforced concrete consist of horizontal spans of less than 40 ft (12.2 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. (3.1 mm) before engagement of the anchors.	5.7.1.2	A.5.1.4
Moderate and High Seismicity (Complete the Following Items in Addition to the Items for Very Low and Low Seismicity)			
Foundation System			
C NC N/A U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$.	5.4.3.3	A.6.2.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

17.18 STRUCTURAL CHECKLISTS FOR BUILDING TYPES URM: UNREINFORCED MASONRY BEARING WALLS WITH FLEXIBLE DIAPHRAGMS, AND URMA: UNREINFORCED MASONRY BEARING WALLS WITH STIFF DIAPHRAGMS

For building systems and configurations that comply with the URM or URMa building type description in Table 3-1, the Collapse Prevention Structural Checklist in Table 17-36 shall be completed where required by Table 4-6 for Collapse Prevention Structural Performance, and the Immediate Occupancy Structural Checklist in Table 17-37 shall be completed where required by

Table 17-36. Collapse Prevention Structural Checklist for Building Types URM and URMa.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the unreinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 30 lb/in.^{2} (0.21 MPa) for clay units and 70 lb/in.^{2} (0.48 MPa) for concrete units.	5.5.3.1.1	A.3.2.5.1
Connections			
C NC N/A U	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers or top plates fastened to the walls.	5.7.1.3	A.5.1.2
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls.	5.7.2	A.5.2.1
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	PROPORTIONS: The height-to-thickness ratio of the shear walls at each story is less than the following: Top story of multistory building, 9; First story of multistory building, 15; and All other conditions, 13.	5.5.3.1.2	A.3.2.5.2
C NC N/A U	MASONRY LAYUP: Filled collar joints of multiwythe masonry walls have negligible voids.	5.5.3.4.1	A.3.2.5.3
Diaphragms (Stiff or Flexible)			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft (2.4 m) long.	5.6.1.3	A.4.1.6
Flexible Diaphragms			
C NC N/A U	CROSSTIES: There are continuous crossties between diaphragm chords to distribute the out-of-plane wall anchorage forces into the diaphragm. Where each out-of-plane connection does not have a continuous crosstie across the entire diaphragm, these connections are developed into subdiaphragms between crossties with a maximum length-to-width ratio of 3-to-1.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 24 ft (7.3 m) and aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.2

continues

Table 17-36 *(Continued)*. Collapse Prevention Structural Checklist for Building Types URM and URMa.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	BLOCKED DIAPHRAGMS: All blocked wood structural panel diaphragms have horizontal spans less than 120 ft (36.5 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	CANTILEVERED WOOD DIAPHRAGMS: All cantilevered wood diaphragms that provide lateral support for concrete or masonry walls consist of wood structural panels and have a maximum cantilever length of 20 ft (6.1 m) if unblocked or 35 ft (10.7 m) if blocked, and a maximum ratio of cantilever length to diaphragm width of 1:2 if unblocked and 1:1 if blocked. In addition, the cantilevered diaphragm has a back-span length equal to or greater than the cantilevered portion.	5.6.2	A.4.2.4
C NC N/A U	NON-CONCRETE-FILLED DIAPHRAGMS: Bare steel deck diaphragms or steel deck diaphragms with fill other than reinforced structural concrete consist of horizontal spans of less than 120 ft (36.5 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. (3.1 mm) before engagement of the anchors.	5.7.1.2	A.5.1.4
C NC N/A U	BEAM, GIRDER, AND TRUSS SUPPORTS: Beams, girders, and trusses supported by unreinforced masonry walls or pilasters have independent secondary columns for support of vertical loads.	5.7.4.4	A.5.4.5

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-37. Immediate Occupancy Structural Checklist for Building Types URM and URMa.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the unreinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 30 lb/in.^{2} (0.21 MPa) for clay units and 70 lb/in.^{2} (0.48 MPa) for concrete units.	5.5.3.1.1	A.3.2.5.1
Connections			
C NC N/A U	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers or top plates fastened to the walls.	5.7.1.3	A.5.1.2
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms.	5.7.2	A.5.2.1

continues

Table 17-37 *(Continued)*. Immediate Occupancy Structural Checklist for Building Types URM and URMa.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
Low, Moderate, and High Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	PROPORTIONS: The height-to-thickness ratio of the shear walls at each story is less than the following: Top story of multistory building, 9; First story of multistory building, 15; and All other conditions, 13.	5.5.3.1.2	A.3.2.5.2
C NC N/A U	MASONRY LAYUP: Filled collar joints of multiwythe masonry walls have negligible voids.	5.5.3.4.1	A.3.2.5.3
Diaphragms (Stiff or Flexible)			
C NC N/A U	DIAPHRAGM CONTINUITY: Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 4 ft (1.2 m) long.	5.6.1.3	A.4.1.6
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
Flexible Diaphragms			
C NC N/A U	CROSSTIES: There are continuous crossties between diaphragm chords to distribute the out-of-plane wall anchorage forces into the diaphragm. Where each out-of-plane connection does not have a continuous crosstie across the entire diaphragm, these connections are developed into subdiaphragms between crossties with a maximum length-to-width ratio of 3-to-1.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have horizontal spans less than 12 ft (3.6 m) and aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.1 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.2
C NC N/A U	BLOCKED DIAPHRAGMS: All blocked wood structural panel diaphragms have horizontal spans less than 90 ft (27.4 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	CANTILEVERED WOOD DIAPHRAGMS: All cantilevered wood diaphragms that provide lateral support for concrete or masonry walls consist of wood structural panels and have a maximum cantilever length of 15 ft (4.6 m) if unblocked or 25 ft (7.6 m) if blocked, and a maximum ratio of cantilever length to diaphragm width of 1:2.5 if unblocked and 1:1.5 if blocked. In addition, the cantilevered diaphragm has a back-span length equal to or greater than the cantilevered portion.	5.6.2	A.4.2.4
C NC N/A U	NON-CONCRETE-FILLED DIAPHRAGMS: Bare steel deck diaphragms or steel deck diaphragms with fill other than reinforced concrete consist of horizontal spans of less than 40 ft (12.2 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1

continues

Table 17-37 *(Continued)*. Immediate Occupancy Structural Checklist for Building Types URM and URMa.

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, steel deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. (3.1 mm) before engagement of the anchors.	5.7.1.2	A.5.1.4
C NC N/A U	BEAM, GIRDER, AND TRUSS SUPPORTS: Beams, girders, and trusses supported by unreinforced masonry walls or pilasters have independent secondary columns for support of vertical loads.	5.7.4.4	A.5.4.5

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 4-6 for Immediate Occupancy Structural Performance. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

17.19 NONSTRUCTURAL CHECKLIST

The nonstructural checklist in Table 17-38 shall be completed for combinations of Performance Levels and Level of Seismicity

Table 17-38. Nonstructural Checklist.

Status	Evaluation Statement[a,b]	Tier 2 Reference	Commentary Reference
Life Safety Systems			
C NC N/A U	**HR—not required; LS—LMH; PR—LMH.** FIRE SUPPRESSION PIPING: Fire suppression piping is anchored and braced in accordance with NFPA-13.	13.7.4	A.7.13.1
C NC N/A U	**HR—not required; LS—LMH; PR—LMH.** FLEXIBLE COUPLINGS: Fire suppression piping has flexible couplings in accordance with NFPA-13.	13.7.4	A.7.13.2
C NC N/A U	**HR—not required; LS—LMH; PR—LMH.** EMERGENCY POWER: Equipment used to power or control Life Safety systems is anchored or braced.	13.7.7	A.7.12.1
C NC N/A U	**HR—not required; LS—LMH; PR—LMH.** STAIR AND SMOKE DUCTS: Stair pressurization and smoke control ducts are braced and have flexible connections at seismic joints.	13.7.6	A.7.14.1
C NC N/A U	**HR—not required; LS—MH; PR—MH.** SPRINKLER CEILING CLEARANCE: Penetrations through panelized ceilings for fire suppression devices provide clearances in accordance with NFPA-13.	13.7.4	A.7.13.3
C NC N/A U	**HR—not required; LS—not required; PR—LMH.** EMERGENCY LIGHTING: Emergency and egress lighting equipment is anchored or braced.	13.7.9	A.7.3.1
Hazardous Materials			
C NC N/A U	**HR—LMH; LS—LMH; PR—LMH.** HAZARDOUS MATERIAL EQUIPMENT: Equipment mounted on vibration isolators and containing hazardous material is equipped with restraints or snubbers.	13.7.1	A.7.12.2
C NC N/A U	**HR—LMH; LS—LMH; PR—LMH.** HAZARDOUS MATERIAL STORAGE: Breakable containers that hold hazardous material, including gas cylinders, are restrained by latched doors, shelf lips, wires, or other methods.	13.8.3	A.7.15.1
C NC N/A U	**HR—MH; LS—MH; PR—MH.** HAZARDOUS MATERIAL DISTRIBUTION: Piping or ductwork conveying hazardous materials is braced or otherwise protected from damage that would allow hazardous material release.	13.7.3 13.7.5 13.7.6	A7.13.4 A.7.14.2

continues

Table 17-38 *(Continued)*. Nonstructural Checklist.

Status	Evaluation Statement^{a,b}	Tier 2 Reference	Commentary Reference
C NC N/A U	**HR—MH; LS—MH; PR—MH.** SHUTOFF VALVES: Piping containing hazardous material, including natural gas, has shutoff valves or other devices to limit spills or leaks.	13.7.3 13.7.5	A.7.15.3
C NC N/A U	**HR—LMH; LS—LMH; PR—LMH.** FLEXIBLE COUPLINGS: Hazardous material ductwork and piping, including natural gas piping, have flexible couplings.	13.7.3 13.7.5 13.7.6	A.7.15.4
C NC N/A U	**HR—MH; LS—MH; PR—MH.** PIPING OR DUCTS CROSSING SEISMIC JOINTS: Piping or ductwork carrying hazardous material that either crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements.	13.7.3 13.7.5 13.7.6	A.7.13.6
Partitions			
C NC N/A U	**HR—LMH; LS—LMH; PR—LMH.** UNREINFORCED MASONRY: Unreinforced masonry or hollow-clay tile partitions are braced at a spacing of at most 10 ft (3 m) in Low or Moderate Seismicity, or at most 6 ft (1.8 m) in High Seismicity.	13.6.2	A.7.1.1
C NC N/A U	**HR—LMH; LS—LMH; PR—LMH.** HEAVY PARTITIONS SUPPORTED BY CEILINGS: The tops of masonry or hollow-clay tile partitions are not laterally supported by an integrated ceiling system.	13.6.2	A.7.2.1
C NC N/A U	**HR—not required; LS—MH; PR—MH.** DRIFT: Rigid cementitious partitions are detailed to accommodate the following drift ratios: in steel moment frame, concrete moment frame, and wood frame buildings, 0.02; in other buildings, 0.005.	13.6.2	A.7.1.2
C NC N/A U	**HR—not required; LS—not required; PR—MH.** LIGHT PARTITIONS SUPPORTED BY CEILINGS: The tops of gypsum board partitions are not laterally supported by an integrated ceiling system.	13.6.2	A.7.2.1
C NC N/A U	**HR—not required; LS—not required; PR—MH.** STRUCTURAL SEPARATIONS: Partitions that cross structural separations have seismic or control joints.	13.6.2	A.7.1.3
C NC N/A U	**HR—not required; LS—not required; PR—MH.** TOPS: The tops of ceiling-high framed or panelized partitions have lateral bracing to the structure at a spacing equal to or less than 6 ft (1.8 m).	13.6.2	A.7.1.4
Ceilings			
C NC N/A U	**HR—H; LS—MH; PR—LMH.** SUSPENDED LATH AND PLASTER: Suspended lath and plaster ceilings have attachments that resist seismic forces for every 12 ft^2 (1.1 m^2) of area.	13.6.4	A.7.2.3
C NC N/A U	**HR—not required; LS—MH; PR—LMH.** SUSPENDED GYPSUM BOARD: Suspended gypsum board ceilings have attachments that resist seismic forces for every 12 ft^2 (1.1 m^2) of area.	13.6.4	A.7.2.3
C NC N/A U	**HR—not required; LS—not required; PR—MH.** INTEGRATED CEILINGS: Integrated suspended ceilings with continuous areas greater than 144 ft^2 (13.4 m^2) and ceilings of smaller areas that are not surrounded by restraining partitions are laterally restrained at a spacing no greater than 12 ft (3.6 m) with members attached to the structure above. Each restraint location has a minimum of four diagonal wires and compression struts, or diagonal members capable of resisting compression.	13.6.4	A.7.2.2
C NC N/A U	**HR—not required; LS—not required; PR—MH.** EDGE CLEARANCE: The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 (13.4 m^2) have clearances from the enclosing wall or partition of at least the following: in Moderate Seismicity, 1/2 in. (13 mm); in High Seismicity, 3/4 in. (19 mm).	13.6.4	A.7.2.4
C NC N/A U	**HR—not required; LS—not required; PR—MH.** CONTINUITY ACROSS STRUCTURE JOINTS: The ceiling system does not cross any seismic joint and is not attached to multiple independent structures.	13.6.4	A.7.2.5

continues

Table 17-38 *(Continued)*. Nonstructural Checklist.

Status	Evaluation Statement[a,b]	Tier 2 Reference	Commentary Reference
C NC N/A U	**HR—not required; LS—not required; PR—H.** EDGE SUPPORT: The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 (13.4 m^2) are supported by closure angles or channels not less than 2 in. (50.8 mm) wide.	13.6.4	A.7.2.6
C NC N/A U	**HR—not required; LS—not required; PR—H.** SEISMIC JOINTS: Acoustical tile or lay-in panel ceilings have seismic separation joints such that each continuous portion of the ceiling is no more than 2,500 ft^2 (232.3 m^2) and has a ratio of long-to-short dimension no more than 4-to-1.	13.6.4	A.7.2.7
Light Fixtures			
C NC N/A U	**HR—not required; LS—MH; PR—MH.** INDEPENDENT SUPPORT: Light fixtures that weigh more per square foot (square meter) than the ceiling they penetrate are supported independent of the grid ceiling suspension system by a minimum of two wires at diagonally opposite corners of each fixture.	13.6.4 13.7.9	A.7.3.2
C NC N/A U	**HR—not required; LS—not required; PR—H.** PENDANT SUPPORTS: Light fixtures on pendant supports are attached at a spacing equal to or less than 6 ft (1.8 m). Unbraced suspended fixtures are free to allow a 360-degree range of motion at an angle not less than 45 degrees from horizontal without contacting adjacent components. Alternatively, if fixures are rigidly supported and/or braced, they are free to move with the structure to which they are attached without damaging adjoining components. Additionally, the connection to the structure is capable of accommodating the movement without failure.	13.7.9	A.7.3.3
C NC N/A U	**HR—not required; LS—not required; PR—H.** LENS COVERS: Lens covers on light fixtures are attached with safety devices.	13.7.9	A.7.3.4
Cladding and Glazing			
C NC N/A U	**HR—MH; LS—MH; PR—MH.** CLADDING ANCHORS: Cladding components weighing more than 10 lb/ft^2 (0.48 kN/m^2) are mechanically anchored to the structure at a spacing equal to or less than the following: for Life Safety in Moderate Seismicity, 6 ft (1.8 m); for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 ft (1.2 m).	13.6.1	A.7.4.1
C NC N/A U	**HR—not required; LS—MH; PR—MH.** CLADDING ISOLATION: For steel or concrete moment-frame buildings, panel connections are detailed to accommodate a story drift ratio by the use of rods attached to framing with oversize holes or slotted holes of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicity, 0.02, and the rods have a length-to-diameter ratio of 4.0 or less.	13.6.1	A.7.4.2
C NC N/A U	**HR—MH; LS—MH; PR—MH.** MULTISTORY PANELS: For multistory panels attached at more than one floor level, panel connections are detailed to accommodate a story drift ratio by the use of rods attached to framing with oversize holes or slotted holes of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicity, 0.02, and the rods have a length-to-diameter ratio of 4.0 or less.	13.6.1	A.7.4.3
C NC N/A U	**HR—not required; LS—MH; PR—MH.** THREADED RODS: Threaded rods for panel connections detailed to accommodate drift by bending of the rod have a length-to-diameter ratio greater than 0.06 times the story height in inches (millimeters) for Life Safety in Moderate Seismicity and 0.12 times the story height in inches (millimeters) for Life Safety in High Seismicity and Position Retention in any seismicity.	13.6.1	A.7.4.8
C NC N/A U	**HR—MH; LS—MH; PR—MH.** PANEL CONNECTIONS: Cladding panels are anchored out of plane with a minimum number of connections for each wall panel, as follows: for Life Safety in Moderate Seismicity, 2 connections; for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 connections.	13.6.1.4	A.7.4.4

continues

Table 17-38 *(Continued)*. Nonstructural Checklist.

Status	Evaluation Statement[a,b]	Tier 2 Reference	Commentary Reference
C NC N/A U	**HR—MH; LS—MH; PR—MH.** BEARING CONNECTIONS: Where bearing connections are used, there is a minimum of two bearing connections for each cladding panel.	13.6.1.4	A.7.4.5
C NC N/A U	**HR—MH; LS—MH; PR—MH.** INSERTS: Where concrete cladding components use inserts, the inserts have positive anchorage or are anchored to reinforcing steel.	13.6.1.4	A.7.4.6
C NC N/A U	**HR—not required; LS—MH; PR—MH.** OVERHEAD GLAZING: Glazing panes of any size in curtain walls and individual interior or exterior panes more than 16 ft^2 (1.5 m^2) in area are laminated annealed or laminated heat-strengthened glass and are detailed to remain in the frame when cracked.	13.6.1.5	A.7.4.7
Masonry Veneer			
C NC N/A U	**HR—not required; LS—LMH; PR—LMH.** TIES: Masonry veneer is connected to the backup with corrosion-resistant ties. There is a minimum of one tie for every 2-2/3 ft^2 (0.25 m^2), and the ties have spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 36 in. (914 mm); for Life Safety in High Seismicity and for Position Retention in any seismicity, 24 in. (610 mm).	13.6.1.2	A.7.5.1
C NC N/A U	**HR—not required; LS—LMH; PR—LMH.** SHELF ANGLES: Masonry veneer is supported by shelf angles or other elements at each floor above the ground floor.	13.6.1.2	A.7.5.2
C NC N/A U	**HR—not required; LS—LMH; PR—LMH.** WEAKENED PLANES: Masonry veneer is anchored to the backup adjacent to weakened planes, such as at the locations of flashing.	13.6.1.2	A.7.5.3
C NC N/A U	**HR—LMH; LS—LMH; PR—LMH.** UNREINFORCED MASONRY BACKUP: There is no unreinforced masonry backup.	13.6.1.1 13.6.1.2	A.7.7.2
C NC N/A U	**HR—not required; LS—MH; PR—MH.** STUD TRACKS: For veneer with cold-formed steel stud backup, stud tracks are fastened to the structure at a spacing equal to or less than 24 in. (610 mm) on center.	13.6.1.1 13.6.1.2	A.7.6.1
C NC N/A U	**HR—not required; LS—MH; PR—MH.** ANCHORAGE: For veneer with concrete block or masonry backup, the backup is positively anchored to the structure at a horizontal spacing equal to or less than 4 ft (1.2 m) along the floors and roof.	13.6.1.1 13.6.1.2	A.7.7.1
C NC N/A U	**HR—not required; LS—not required; PR—MH.** WEEP HOLES: In veneer anchored to stud walls, the veneer has functioning weep holes and base flashing.	13.6.1.2	A.7.5.4
C NC N/A U	**HR—not required; LS—not required; PR—MH.** OPENINGS: For veneer with cold-formed-steel stud backup, steel studs frame window and door openings.	13.6.1.1 13.6.1.2	A.7.6.2
Parapets, Cornices, Penthouses, and Appendages			
C NC N/A U	**HR—LMH; LS—LMH; PR—LMH.** URM PARAPETS OR CORNICES: Laterally unsupported unreinforced masonry parapets or cornices have height-to-thickness ratios no greater than the following: for Life Safety in Low or Moderate Seismicity, 2.5; for Life Safety in High Seismicity and for Position Retention in any seismicity, 1.5.	13.6.5	A.7.8.1
C NC N/A U	**HR—not required; LS—LMH; PR—LMH.** CANOPIES: Canopies at building exits are anchored to the structure at a spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 10 ft (3 m); for Life Safety in High Seismicity and for Position Retention in any seismicity, 6 ft (1.8 m).	13.6.6	A.7.8.2
C NC N/A U	**HR—H; LS—MH; PR—LMH.** CONCRETE PARAPETS: Concrete parapets with height-to-thickness ratios greater than 2.5 have vertical reinforcement.	13.6.5	A.7.8.3
C NC N/A U	**HR—MH; LS—MH; PR—LMH.** APPENDAGES: Cornices, parapets, signs, and other ornamentation or appendages that extend above the highest point of anchorage to the structure or cantilever from components are reinforced and anchored to the structural system at a spacing equal to or less than 6 ft (1.8 m). This evaluation statement item does not apply to parapets or cornices covered by other evaluation statements.	13.6.6	A.7.8.4

continues

Table 17-38 *(Continued)*. Nonstructural Checklist.

Status	Evaluation Statement[a,b]	Tier 2 Reference	Commentary Reference
C NC N/A U	**HR—MH; LS—MH; PR—LMH.** PENTHOUSES: Penthouses are not used for regular occupancy and are constructed as an extension of the building frame or have a lateral-force-resisting system in each direction consistent with structural systems listed in Table 12.2-1 or Table 15.4-1 of ASCE 7.	13.6.7	A.7.8.5
C NC N/A U	**HR—MH; LS—MH; PR—LMH.** TILE ROOFS: For roofs with slopes greater than or equal to 3 vertical to 12 horizontal, roof tiles weighing more than 4 lb/ft^2 (0.05 kN/m^2) are individually secured to the roof framing or roof deck with wires, fasteners, or adhesive.	13.6.8	A.7.8.6
Masonry Chimneys			
C NC N/A U	**HR—LMH; LS—LMH; PR—LMH.** URM CHIMNEYS: Unreinforced masonry chimneys extend above the roof surface no more than the following: for Life Safety in Low or Moderate Seismicity, three times the least dimension of the chimney; for Life Safety in High Seismicity and for Position Retention in any seismicity, two times the least dimension of the chimney.	13.6.9	A.7.9.1
C NC N/A U	**HR—LMH; LS—LMH; PR—LMH.** ANCHORAGE: Masonry chimneys are anchored at each floor level, at the topmost ceiling level, and at the roof.	13.6.9	A.7.9.2
Stairs			
C NC N/A U	**HR—not required; LS—LMH; PR—LMH.** STAIR ENCLOSURES: Hollow-clay tile or unreinforced masonry walls around stair enclosures are restrained out of plane and have height-to-thickness ratios not greater than the following: for Life Safety in Low or Moderate Seismicity, 15-to-1; for Life Safety in High Seismicity and for Position Retention in any seismicity, 12-to-1.	13.6.2 13.6.10	A.7.10.1
C NC N/A U	**HR—not required; LS—LMH; PR—LMH.** STAIR DETAILS: The connection between the stairs and the structure does not rely on post-installed anchors in concrete or masonry, and the stair details are capable of accommodating the drift calculated using the Quick Check procedure of Section 4.4.3.1, for moment-frame structures or 1/2 in. (12.7 mm) for all other structures without including any lateral stiffness contribution from the stairs.	13.6.10	A.7.10.2
Contents and Furnishings			
C NC N/A U	**HR—LMH; LS—MH; PR—MH.** INDUSTRIAL STORAGE RACKS: Industrial storage racks or pallet racks more than 12 ft (3.6 m) high meet the requirements of ANSI/RMI MH 16.1 as modified by ASCE 7, Chapter 15.	13.8.1	A.7.11.1
C NC N/A U	**HR—not required; LS—H; PR—MH.** TALL NARROW CONTENTS: Contents more than 6 ft (1.8 m) high with a height-to-depth or height-to-width ratio greater than 3-to-1 are anchored to the structure or to each other.	13.8.2	A.7.11.2
C NC N/A U	**HR—not required; LS—H; PR—H.** FALL-PRONE CONTENTS: Equipment, stored items, or other contents weighing more than 20 lb (9.1 kg) whose center of mass is more than 4 ft (1.2 m) above the adjacent floor level are braced or otherwise restrained.	13.8.2	A.7.11.3
C NC N/A U	**HR—not required; LS—not required; PR—MH.** ACCESS FLOORS: Access floors more than 9 in. (229 mm) high are braced.	13.6.12	A.7.11.4
C NC N/A U	**HR—not required; LS—not required; PR—MH.** EQUIPMENT ON ACCESS FLOORS: Equipment and other contents supported by access floor systems are anchored or braced to the structure independent of the access floor.	13.7.7 13.6.12	A.7.11.5
C NC N/A U	**HR—not required; LS—not required; PR—H.** SUSPENDED CONTENTS: Items suspended without lateral bracing are free to swing from or move with the structure from which they are suspended without damaging themselves or adjoining components.	13.8.2	A.7.11.6
Mechanical and Electrical Equipment			
C NC N/A U	**HR—not required; LS—H; PR—H.** FALL-PRONE EQUIPMENT: Equipment weighing more than 20 lb (9.1 kg) whose center of mass is more than 4 ft (1.2 m) above the adjacent floor level, and which is not in-line equipment, is braced.	13.7.1 13.7.7	A.7.12.4

continues

Table 17-38 *(Continued)*. Nonstructural Checklist.

Status	Evaluation Statement^{a,b}	Tier 2 Reference	Commentary Reference
C NC N/A U	**HR—not required; LS—H; PR—H.** IN-LINE EQUIPMENT: Equipment installed in line with a duct or piping system, with an operating weight more than 75 lb (34.0 kg), is supported and laterally braced independent of the duct or piping system.	13.7.1	A.7.12.5
C NC N/A U	**HR—not required; LS—H; PR—MH.** TALL NARROW EQUIPMENT: Equipment more than 6 ft (1.8 m) high with a height-to-depth or height-to-width ratio greater than 3-to-1 is anchored to the floor slab or adjacent structural walls.	13.7.1 13.7.7	A.7.12.6
C NC N/A U	**HR—not required; LS—not required; PR—MH.** MECHANICAL DOORS: Mechanically operated doors are detailed to operate at a story drift ratio of 0.01.	13.6.11	A.7.12.7
C NC N/A U	**HR—not required; LS—not required; PR—H.** SUSPENDED EQUIPMENT: Equipment suspended without lateral bracing is free to swing from or move with the structure from which it is suspended without damaging itself or adjoining components.	13.7.1 13.7.7	A.7.12.8
C NC N/A U	**HR—not required; LS—not required; PR—H.** VIBRATION ISOLATORS: Equipment mounted on vibration isolators is equipped with horizontal restraints or snubbers and with vertical restraints to resist overturning.	13.7.1	A.7.12.9
C NC N/A U	**HR—not required; LS—not required; PR—H.** HEAVY EQUIPMENT: Floor-supported or platform-supported equipment weighing more than 400 lb (181.4 kg) is anchored to the structure.	13.7.1 13.7.7	A.7.12.10
C NC N/A U	**HR—not required; LS—not required; PR—H.** ELECTRICAL EQUIPMENT: Electrical equipment is laterally braced to the structure.	13.7.7	A.7.12.11
C NC N/A U	**HR—not required; LS—not required; PR—H.** CONDUIT COUPLINGS: Conduit greater than 2.5 in. (64 mm) trade size that is attached to panels, cabinets, or other equipment and is subject to relative seismic displacement has flexible couplings or connections.	13.7.8	A.7.12.12
Piping			
C NC N/A U	**HR—not required; LS—not required; PR—H.** FLEXIBLE COUPLINGS: Fluid and gas piping has flexible couplings.	13.7.3 13.7.5	A.7.13.2
C NC N/A U	**HR—not required; LS—not required; PR—H.** FLUID AND GAS PIPING: Fluid and gas piping is anchored and braced to the structure to limit spills or leaks.	13.7.3 13.7.5	A.7.13.4
C NC N/A U	**HR—not required; LS—not required; PR—H.** C-CLAMPS: One-sided C-clamps that support piping larger than 2.5 in. (64 mm) in diameter are restrained.	13.7.3 13.7.5	A.7.13.5
C NC N/A U	**HR—not required; LS—not required; PR—H.** PIPING CROSSING SEISMIC JOINTS: Piping that crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements.	13.7.3 13.7.5	A.7.13.6
Ducts			
C NC N/A U	**HR—not required; LS—not required; PR—H.** DUCT BRACING: Rectangular ductwork larger than 6 ft^2 (0.56 m^2) in cross-sectional area and round ducts larger than 28 in. (711 mm) in diameter are braced. The maximum spacing of transverse bracing does not exceed 30 ft (9.1 m). The maximum spacing of longitudinal bracing does not exceed 60 ft (18.3 m).	13.7.6	A.7.14.2
C NC N/A U	**HR—not required; LS—not required; PR—H.** DUCT SUPPORT: Ducts are not supported by piping or electrical conduit.	13.7.6	A.7.14.3
C NC N/A U	**HR—not required; LS—not required; PR—H.** DUCTS CROSSING SEISMIC JOINTS: Ducts that cross seismic joints or isolation planes or are connected to independent structures have couplings or other details to accommodate the relative seismic displacements.	13.7.6	A.7.14.4
Elevators			
C NC N/A U	**HR—not required; LS—H; PR—H.** RETAINER GUARDS: Sheaves and drums have cable retainer guards.	13.7.11	A.7.16.1
C NC N/A U	**HR—not required; LS—H; PR—H.** RETAINER PLATE: A retainer plate is present at the top and bottom of both car and counterweight.	13.7.11	A.7.16.2

continues

Table 17-38 *(Continued)*. Nonstructural Checklist.

Status	Evaluation Statement[a,b]	Tier 2 Reference	Commentary Reference
C NC N/A U	**HR—not required; LS—not required; PR—H.** ELEVATOR EQUIPMENT: Equipment, piping, and other components that are part of the elevator system are anchored.	13.7.11	A.7.16.3
C NC N/A U	**HR—not required; LS—not required; PR—H.** SEISMIC SWITCH: Elevators capable of operating at speeds of 150 ft/min (0.30 m/min) or faster are equipped with seismic switches that meet the requirements of ASME A17.1 (ASME 2000a) or have trigger levels set to 20% of the acceleration of gravity at the base of the structure and 50% of the acceleration of gravity in other locations.	13.7.11	A.7.16.4
C NC N/A U	**HR—not required; LS—not required; PR—H.** SHAFT WALLS: Elevator shaft walls are anchored and reinforced to prevent toppling into the shaft during strong shaking.	13.7.11	A.7.16.5
C NC N/A U	**HR—not required; LS—not required; PR—H.** COUNTERWEIGHT RAILS: All counterweight rails and divider beams are sized in accordance with ASME A17.1.	13.7.11	A.7.16.6
C NC N/A U	**HR—not required; LS—not required; PR—H.** BRACKETS: The brackets that tie the car rails and the counterweight rail to the structure are sized in accordance with ASME A17.1.	13.7.11	A.7.16.7
C NC N/A U	**HR—not required; LS—not required; PR—H.** SPREADER BRACKET: Spreader brackets are not used to resist seismic forces.	13.7.11	A.7.16.8
C NC N/A U	**HR—not required; LS—not required; PR—H.** GO-SLOW ELEVATORS: The building has a go-slow elevator system.	13.7.11	A.7.16.9

[a] Performance Level: HR = Hazards Reduced, LS = Life Safety, and PR = Position Retention.
[b] Level of Seismicity: L = Low, M = Moderate, and H = High.
Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

as required by Table 4-6. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

Compliant items shall be deemed by the design professional to satisfy the corresponding Performance Objective in the evaluation statement and shall meet all of the following conditions:

1. Supporting members relied on for compliance have complete load paths to supporting structural members;
2. Bracing members, connecting members, and supporting structural or architectural components relied on for compliance are of materials and dimensions suitable to the application; and
3. Fasteners and connectors relied on for compliance are of materials and sizes suitable to the application.

Items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation at the Life Safety Nonstructural Performance Level, an evaluation statement need not be marked Noncompliant if the noncompliance occurs only in locations where related damage would not cause severe injury or death to one or more people.

For the Hazards Reduced Nonstructural Performance Level, the evaluation statement is permitted to be found Compliant if it can be shown that the specific hazard will not endanger many people.

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CHAPTER 18
REFERENCE DOCUMENTS

18.1 CONSENSUS STANDARDS AND OTHER REFERENCE DOCUMENTS

This section contains both consensus standards and other reference documents cited within the provisions of the standard.

AAMA 501.6, *Recommended Dynamic Test Method for Determining the Seismic Drift Causing Glass Fallout from a Wall System*, American Architectural Manufacturers Association, 2009.

ACI 214.4R, *Guide for Obtaining Cores and Interpreting Compressive Strength Results*, American Concrete Institute, 2011.

ACI 228.2R, *Report on Nondestructive Test Methods for Evaluation of Concrete in Structures*, American Concrete Institute, 2013.

ACI 318, *Building Code Requirements for Structural Concrete and Commentary*, American Concrete Institute, 2019.

ACI 562R, *Code Requirements for Assessment, Repair, and Rehabilitation of Existing Concrete Structures and Commentary*, American Concrete Institute, 2016.

AISC 341, *Seismic Provisions for Structural Steel Buildings*, American Institute of Steel Construction, 2022.

AISC 342, *Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings*, American Institute of Steel Construction, 2022.

AISC 360, *Specification for Structural Steel Buildings*, American Institute of Steel Construction, 2022.

AISI S100, *North American Specification for the Design of Cold-Formed Steel Structural Members*, 2016 Edition (Reaffirmed 2020), with Supplement 2, 2020 Edition, AISI S100-16 (2020) w/S2-20, American Iron and Steel Institute, 2020.

AISI S240, *North American Standard for Cold-Formed Steel Structural Framing*, 2020 Edition, American Iron and Steel Institute, 2020.

AISI S400, *North American Standard for Seismic Design of Cold-Formed Steel Structural Systems*, 2020 Edition. American Iron and Steel Institute, 2020.

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ANSI/AWC NDS-2018, *National Design Specification (NDS) for Wood Construction – with 2018 NDS Supplement*, American Wood Council, 2017.

ANSI MH 16.1, *Design, Testing and Utilization of Industrial Steel Storage Racks*, Rack Manufacturers Institute, 2021.

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ASCE/SEI 7, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE, 2022.

ASCE/SEI 31-03, *Seismic Evaluation of Existing Buildings*, ASCE, 2003.

ASCE/SEI 41-06, *Seismic Rehabilitation of Existing Buildings*, American Society of Civil Engineers, 2007.

ASCE/SEI 41-13, *Seismic Rehabilitation of Existing Buildings*, ASCE, 2014.

ASCE/SEI 41-17, *Seismic Rehabilitation of Existing Buildings*, ASCE, 2017.

ASME B31, *Code for Pressure Piping*, American Society of Mechanical Engineers, 2000.

ASTM A7, *Specification for Steel for Bridges and Buildings*, ASTM International, 1939–1967.

ASTM A9, *Specification for Steel Buildings*, ASTM International, 1933.

ASTM A9-33T, *Tentative Specifications for Steel Buildings*, ASTM International, 1933.

ASTM A15, *Specification for Billet-Steel Bars for Concrete Reinforcement*, ASTM International, 1911.

ASTM A16, *Specification for Rail-Steel Bars of Concrete Reinforcement*, ASTM International, 1913.

ASTM A36/A36M, *Standard Specification for Carbon Structural Steel*, ASTM International, 2019.

ASTM A61, *Specification for Deformed Rail Steel Bars for Concrete Reinforcement with 60,000 psi Minimum Yield Strength*, ASTM International, 1963.

ASTM A140-32T, *Specification for Steel Bridges and Buildings*, ASTM International, 1932.

ASTM A160, *Specification for Axle-Steel Bars for Concrete Reinforcement*, ASTM International, 1936.

ASTM A185, *Standard Specification for Steel Welded Wire Reinforcement, Plain, for Concrete*, ASTM International, 1936.

ASTM A245, *Specification for Flat-Rolled Carbon Steet Sheets of Structural Quality*, ASTM International, 1960.

ASTM A370, *Standard Test Methods and Definitions for Mechanical Testing of Steel Products*, ASTM International, 2021.

ASTM A408, *Specification for Special Large Size Deformed Billet-Steel Bars for Concrete Reinforcement*, ASTM International, 1967.

ASTM A416/A416M, *Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete*, ASTM International, 2015.

ASTM A421/A421M, *Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete*, ASTM International, 2015.

ASTM A431, *Specification for High-Strength Deformed Billet-Steel Bars for Concrete Reinforcement with 75,000 psi Minimum Yield Strength*, ASTM International, 1957.

ASTM A432, *Specification for Deformed Billet Steel Bars for Concrete Reinforcement with 60,000 psi Minimum Yield Point*, ASTM International, 1959.

ASTM A497, *Specification for Steel Welded Wire Reinforcement, Deformed, for Concrete*, ASTM International, 1964.

ASTM A500/A500M, *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes*, ASTM International, 2021a.

ASTM A572/A572M (formerly A441/A441M), *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*, ASTM International, 2021e1.

ASTM A606/A606M, *Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance*, ASTM International, 2018.

ASTM A615/A615M, *Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement*, ASTM International, 2022.

ASTM A616, *Standard Specification for Rail-Steel Deformed and Plain Bars for Concrete Reinforcement*, ASTM International, 1968.

ASTM A617, *Standard Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement*, ASTM International, 1968.

ASTM A653/A653M, *Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process*, ASTM International, 2022.

ASTM A706, *Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement*, ASTM International, 2009.

ASTM A722/A722M, *Standard Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete*, ASTM International, 2015.

ASTM A792/A792M, *Standard Specifications for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process*, ASTM International, 2022.

ASTM A875/A875M *Standard Specification for Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the Hot-Dip Process*, ASTM International, 2022.

ASTM A955, *Standard Specification for Deformed and Plain Stainless Steel Bars for Concrete Reinforcement*, ASTM International, 1996.

ASTM A992/A992M, *Standard Specification for Structural Steel Shapes*, ASTM International, 2022.

ASTM A1003, *Standard Specification for Steel Sheet, Carbon, Metallic- and Nonmetallic-Coated for Cold-Formed Framing Members*, ASTM International, 2015 (reaffirmed 2021).

ASTM A1008, *Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, Required Hardness, Solution Hardened, and Bake Hardenable*, ASTM International, 2021a.

ASTM A1011, *Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, and Ultra-High Strength*, ASTM International, 2018a.

ASTM C34, *Standard Specifications for Structural Clay Load-Bearing Tile*, ASTM International, 2013.

ASTM C39/C39M, *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*, ASTM International, 2015.

ASTM C42/C42M, *Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*, ASTM International, 2013.

ASTM C140, *Standard Test Methods for Sampling and Testing Concrete Masonry Units and Related Units*, ASTM International, 2016.

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APPENDIX A

GUIDELINES FOR DEFICIENCY-BASED PROCEDURES

A.1 GENERAL

This appendix chapter provides commentary to the checklists used for the Tier 1 screening in Chapter 4. This commentary, which is referenced from the checklists contained in Chapter 17, includes each checklist statement, followed by commentary on the potential deficiency represented by the checklist statement and considerations for mitigation of the deficiency. This checklist commentary can also be used for guidance in the further evaluation and potential retrofit of identified deficiencies using the Tier 2 deficiency-based evaluation and retrofit procedures in Chapter 5.

The deficiencies identified by a Tier 1 evaluation at the Collapse Prevention Structural Performance Level can generally be classified by risk of collapse based on post-earthquake observation of buildings with similar deficiencies. Where major structural irregularities and poor detailing conditions are both present, this could lead to an overall critical deficiency. When judging whether a building will likely require a retrofit based solely on the Tier 1 deficiencies or estimating the likelihood that a Tier 1 noncompliant building may eventually pass a Tier 2 or Tier 3 evaluation, it may be helpful to consider the relative risk of global collapse posed by the noncompliant conditions.

In general, deficiencies most critical to the overall stability of buildings with the potential for global collapse include

- Lack of proper substantial load path with connections able to develop member capacities for the primary seismic-force-resisting elements
- Major irregularities
  - Vertical irregularities
  - Extreme torsion
  - Weak or Soft stories
- Substantial overstress of primary seismic-force-resisting elements
- Lack of redundancy when combined with major irregularities
- Poor detailing resulting in brittle primary seismic-force-resisting elements or relative proportioning of primary seismic-force-resisting elements
  - Inadequate column ties
  - Inadequate brace connections leading to possible story mechanism due to widespread likelihood of member or connection fracture
  - Inadequate column strength relative to beams

Deficiencies generally related to potential for local collapse include

- Lack of load path for secondary elements
- Poor detailing of items at risk for local collapse
  - Inadequate beam ties
  - Lack of continuous beam reinforcing bars through supports
  - Lack of adequate beam–column joint reinforcing
  - Inadequate punching shear capacity of flat plates
  - Shear critical gravity items
  - Nonductile beam–column moment connections
  - Nonductile brace connections
  - Anchorage of masonry/concrete walls to flexible diaphragms is inadequate or lacking altogether

The risk to global or partial collapse related to geotechnical site hazards varies widely based on the severity of the hazard. Surface fault rupture, slope failure, and liquefaction have caused major structural damage in previous earthquakes, but it is difficult to predict if these conditions will be present at a specific building site or whether the effects will be severe.

When a major irregularity condition is present in combination with failure of the Quick Check of lateral strength on the main lateral elements it may be useful to conduct a supplemental check on the strength of the lateral resisting elements to assist in judging how likely a building is to pass a Tier 2 evaluation.

For example, for the Quick Checks of lateral strength, the shear demand in a story can be checked using the m-factors and expected material strengths used in a Tier 2 evaluation without conducting a full Tier 2 assessment. If the structure is still deficient using this check, there is a low likelihood of passing a Tier 2 evaluation and retrofit is likely required.

For consideration of nonstructural hazards, these generally do not result in high potential for global or local collapse of the structure; however, nonstructural hazards can pose significant risk to the safety of people within or around the exterior of a building or to the ability of people to safely exit a building. Significant risks include heavy, unbraced components such as unreinforced masonry parapets, exterior appurtenances, slender reinforced masrony partitions, tall unbraced storage racks, and interior plaster ceilings. The nonstructural checklist items required for the Hazards Reduced Performance Level is another resource for identifying the nonstructural components generally assumed to pose the most significant risk.

Additional commentary on the specific requirements for the Tier 2 analysis procedures is provided in Chapter C5.

The appendix is organized as follows:

- A.2 Procedures for Building Systems,
- A.3 Procedures for Seismic-Force-Resisting Systems,
- A.4 Procedures for Diaphragms,
- A.5 Procedures for Connections,
- A.6 Procedures for Geologic Site Hazards and Foundations, and
- A.7 Procedures for Nonstructural Components.

A.2 PROCEDURES FOR BUILDING SYSTEMS

This section provides guidelines for using the Tier 1 building systems checklists and the Tier 2 deficiency-based evaluation and retrofit procedures for all building systems: general, configuration, and condition of the materials.

A.2.1 General

A.2.1.1 Load Path *The structure contains a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements to the foundation.*

There must be a complete seismic-force-resisting system that forms a continuous load path between the foundation, all diaphragm levels, and all portions of the building for proper seismic performance. The general load path is as follows: seismic forces originating throughout the building are delivered through structural connections to horizontal diaphragms; the diaphragms distribute these forces to the vertical elements of the seismic-force-resisting system, such as shear walls and frames; the vertical elements transfer the forces into the foundation; and the foundation transfers the forces into the supporting soil. Compliance with this statement indicates only the existence of a complete load path and that all elements and connections within the load path appear to be detailed for transferring seismic forces. The adequacy of the load path is checked in subsequent statements.

If there is a discontinuity in the load path, the building is unable to resist seismic forces regardless of the strength of the existing elements. Mitigation with elements or connections needed to complete the load path is necessary to achieve the selected performance level. The design professional should be watchful for gaps in the load path. Examples would include a shear wall that does not extend to the foundation, a missing shear transfer connection between a diaphragm and vertical element, a discontinuous chord at a diaphragm notch, or a missing collector.

In cases where there is a structural discontinuity, a load path may exist, but it may be a very undesirable one. At discontinuous shear walls, for example, the diaphragm may transfer the forces to frames not intended to be part of the seismic-force-resisting system. Although not ideal, the load path is compliant, and it may be possible to show that the load path is acceptable. Another compliant load path that may be undesirable is where seismic forces are transferred between seismic-force-resisting elements through friction.

Load path discontinuities can be mitigated by adding components to complete the load path. This method may require adding new, well-founded shear walls or frames to fill gaps in existing shear walls or frames that are not carried continuously to the foundation. Alternatively, it may require the addition of components throughout the building to pick up forces from diaphragms that have no path into existing vertical elements.

A.2.1.2 Adjacent Buildings *The clear distance between the building being evaluated and any adjacent building is greater than the ratios of the height of the shorter building shown in Table A-1.*

Table A-1. Ratio between Heights of Two Buildings to Determine Clear Distance.

Seismicity	Collapse Prevention	Immediate Occupancy
Very Low	N/A	0.15%
Low	0.25%	0.5%
Moderate	0.5%	1.0%
High	1.5%	3.0%

Buildings are often built right up to property lines to make maximum use of space, and historically, buildings have been designed as if the adjacent buildings do not exist. As a result, the buildings may impact each other, or pound, during an earthquake. Building pounding can alter the dynamic response of both buildings and impart additional inertial forces on both structures.

Where one or both buildings have setbacks, the minimum separation should be evaluated based on the common height between the two buildings. Above the level of the setback, the separation should be evaluated based on the total height of the shorter building.

Buildings that are the same height and have matching floors exhibit similar dynamic behavior. If the buildings pound, floors impact other floors, so damage caused by pounding is usually limited to nonstructural components. Where the floors of adjacent buildings are at different elevations, floors impact the columns of the adjacent building and can cause structural damage (Figure A-1). Where the buildings are of different heights, the shorter building can act as a buttress for the taller building. The shorter building receives an unexpected load, and the taller building suffers from a major stiffness discontinuity that alters its dynamic response (Figure A-2). Because neither building is necessarily designed for these conditions, there is a potential for extensive damage and possible collapse.

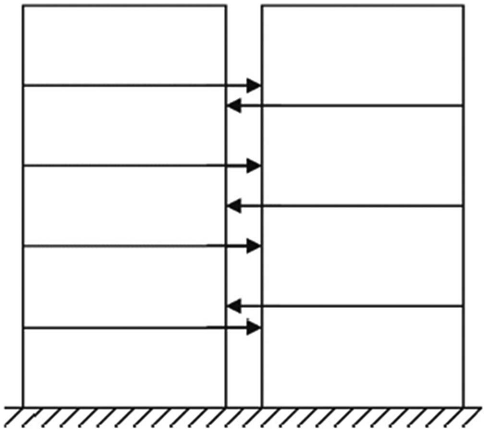


Figure A-1. Unmatching floors.

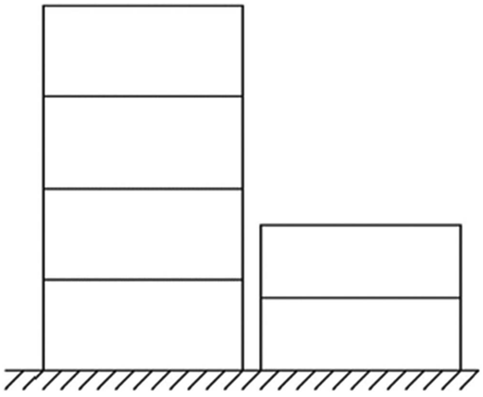


Figure A-2. Buildings of different heights.

Many buildings that are built tight to each other appear to survive earthquakes by acting as a solid block. However, the end buildings of the block may have pronounced pounding. An example of this condition was the downtown area of San Francisco during the Loma Prieta earthquake. End-of-block buildings with unmatching floors have the greatest Life Safety concern.

A criterion for building separation was developed for the third edition of FEMA P-154 *Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook* (FEMA 2015a) and described in FEMA P-155 *Rapid Visual Screening of Buildings for Potential Seismic Hazards: Supporting Documentation* (FEMA 2015b). The separation requirements for Collapse Prevention recognize that some pounding between buildings may occur but would not be sufficient to cause a collapse condition. The separation requirements for Immediate Occupancy are taken as two times the separation requirements for Collapse Prevention.

Noncompliant separations must be checked using calculated drifts for both buildings. The square root sum of squares (SRSS) combination is used because of the low probability that maximum drifts in both buildings will occur simultaneously and out of phase. Where information on the adjacent building is not available, conservative estimates for drift should be made in the evaluation.

The potential hazard of the adjacent building also must be evaluated. If a neighbor building is a potential collapse hazard, this fact must be reported.

Stiffening elements (typically braced frames or shear walls) can be added to one or both buildings to reduce the expected drifts to acceptable levels. With separate structures in a single building complex, it may be possible to tie them together structurally to force them to respond as a single structure. The relative stiffnesses of each and the resulting force interactions must be determined to ensure that additional deficiencies are not created. Pounding can also be eliminated by demolishing a portion of one building to increase the separation.

A.2.1.3 Mezzanines *Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure.*

It is common for mezzanines to lack a well-defined seismic-force-resisting system. Often, mezzanines are added on by the building owner after the original construction of the building. Mezzanines may be partially attached to the structural framing of the main building, in which case the lateral bracing for the mezzanine may partially rely on the building's seismic-force-resisting system and may require additional lateral bracing. Unbraced mezzanines can be a potential collapse hazard and should be checked for stability.

Seismic-force-resisting elements must be present in both directions to provide bracing. Where the mezzanine is attached to the main structure, the supporting elements of the main structure should be evaluated, considering both the magnitude and location of the additional forces imparted by the mezzanine.

If the load path is incomplete or nonexistent, mitigation with elements or connections needed to complete the load path is necessary to achieve the selected performance level.

Diagonal braces, moment frames, or shear walls can be added at or near the perimeter of the mezzanine where bracing elements are missing to provide a complete and balanced seismic-force-resisting system that meets the requirements of this standard.

A.2.2 Configuration

A.2.2.1 General Good details and construction quality are of secondary value if a building has an odd shape that was not properly considered in the design. Although a building with an irregular configuration may be designed to meet all code requirements, irregular buildings generally do not perform as well as regular buildings in an earthquake. Typical building configuration deficiencies include an irregular geometry, a weakness in a given story, a concentration of mass, or a discontinuity in the seismic-force-resisting system.

Vertical irregularities are defined in terms of strength, stiffness, geometry, and mass. These quantities are evaluated separately, but they are related and may occur simultaneously. For example, the frame in Figure A-3 has a tall first story. It can be a weak story, a soft story, or both, depending on the relative strength and stiffness of this story and the stories above.

One of the basic goals in the design of a building is efficient use of materials such that all members are stressed about equally. In seismic design, this goal is modified so that stresses within groups of members are about the same. For example, in moment frames (as discussed in Section A.3.1), it is desirable to have the beams weaker than the columns but to have all the beams at the same stress level. In such a design, the members yield at about the same level of seismic forces; there is no single weak link. Code provisions regarding vertical irregularities are intended to achieve this result. Significant irregularities that would cause damage to be concentrated in certain areas require special treatment.

Horizontal irregularities involve the horizontal distribution of seismic forces to the resisting frames or shear walls. Irregularities in the shape of the diaphragm itself (i.e., diaphragms that are L-shaped or have notches) are discussed in Section A.4.

New vertical seismic-force-resisting elements can be provided to eliminate the vertical irregularity. For weak stories, soft stories, and vertical discontinuities, new elements of the same type can be added as needed.

The effects of plan irregularities that create torsion can be eliminated with the addition of seismic-force-resisting bracing elements that support all major diaphragm segments in a balanced manner. Although it is possible in some cases to allow the irregularity to remain and instead strengthen those structural components that are overstressed by its existence, this provision does not directly address the problem and requires the use of the Tier 3 systematic retrofit procedure.

A.2.2.2 Weak Story *The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above.*

The story strength is the total strength of all the seismic-force-resisting elements in a given story for the direction under consideration. It is the shear capacity of columns or shear walls

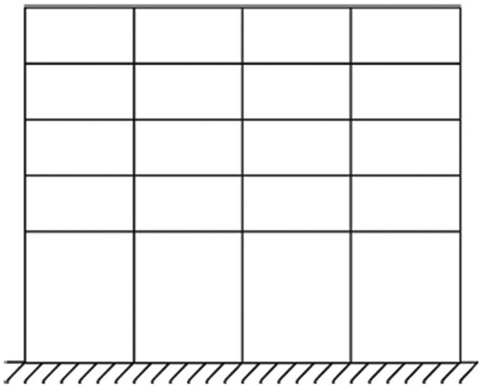


Figure A-3. Tall story.

or the horizontal component of the capacity of diagonal braces. If the columns are flexure controlled, the shear strength is the shear corresponding to the flexural strength. Weak stories are usually found where vertical discontinuities exist or where member size or reinforcement has been reduced. It is necessary to calculate the story strengths and compare them. The result of a weak story is a concentration of inelastic activity that may result in the partial or total collapse of the story.

In general an examination of the building elevations can determine if a weak story exists without the need for calculation. A reduction in the number or length of seismic-force-resisting elements or a change in the type of seismic-force-resisting system is an obvious indication that a weak story might exist. A gradual reduction of seismic-force-resisting elements as the building increases in height is typical and is not considered a weak story condition.

A dynamic analysis should be performed to determine if there are unexpectedly high seismic demands at locations of strength discontinuities. Compliance can be achieved if the elements of the weak story can be shown to have adequate capacity near-elastic levels.

A.2.2.3 Soft Story *The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above.*

This condition commonly occurs in commercial buildings with open fronts at ground-floor storefronts and hotels or office buildings with particularly tall first stories. Figure A-3 shows an example of a tall story. Such cases are not necessarily soft stories because the tall columns may have been designed with appropriate stiffness, but they are likely to be soft stories if they have been designed without consideration for story drift. Soft stories usually are revealed by an abrupt change in story drift. In general an examination of the building elevations can determine if a soft story exists without the need for calculation. A tall story or a change in the type of seismic-force-resisting system is an obvious indication that a soft story might exist. A gradual reduction of seismic-force-resisting elements as the building increases in height is typical and is not considered a soft story condition. Another simple first step might be to plot and compare the story drifts, as indicated in Figure A-4, if analysis results happen to be available.

The difference between "soft" and "weak" stories is the difference between stiffness and strength. A column may be limber but strong or stiff but weak. A change in column size can affect strength and stiffness, and both need to be considered.

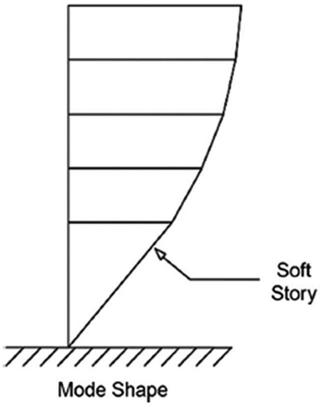


Figure A-4. Soft story.

A dynamic analysis should be performed to determine if there are unexpectedly high seismic demands at locations of stiffness discontinuities.

A.2.2.4 Vertical Irregularities *All vertical elements in the seismic-force-resisting system are continuous to the foundation.*

Vertical discontinuities are usually detected by visual observation. The most common example is a discontinuous shear wall or braced frame. The element is not continuous to the foundation; rather, it stops at an upper level. The shear at this level is transferred through the diaphragm to other resisting elements below. This force transfer can be accomplished through a strut if the elements are on the same plane (Figure A-5) or through a connecting diaphragm if the elements are not in the same plane (Figure A-6). In either case, the overturning forces that develop in the element continue down through the supporting columns.

This issue is a local strength and ductility problem below the discontinuous elements, not a global story strength or stiffness

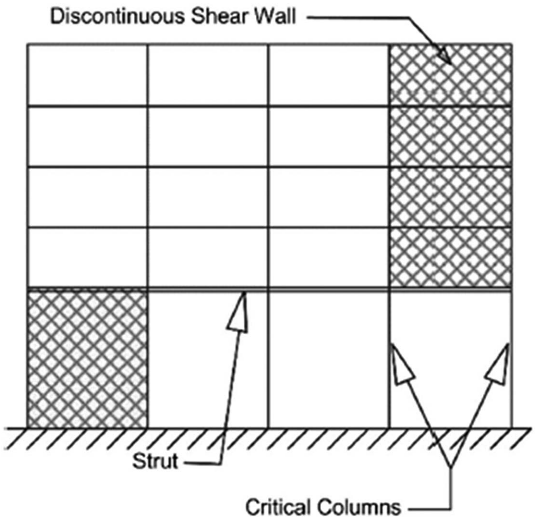


Figure A-5. Vertical discontinuity in plane.

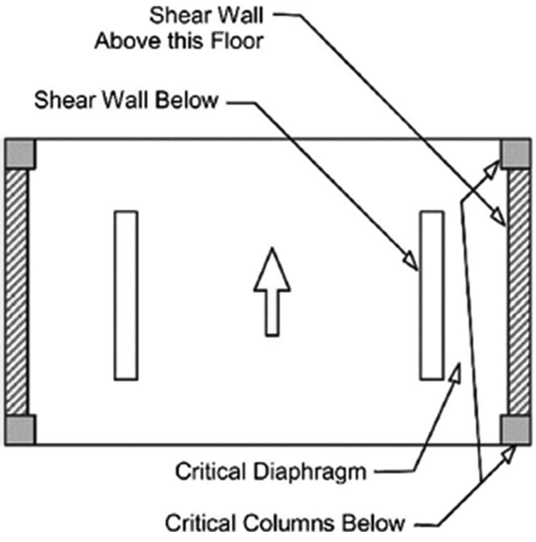


Figure A-6. Vertical discontinuity out of plane.

irregularity. The concern is that the wall or braced frame may have more shear capacity than was considered in the design. These capacities impose overturning forces that could overwhelm the columns. Although the strut or connecting diaphragm may be adequate to transfer the shear forces to adjacent elements, the columns that support vertical loads are the most critical. Moment frames can have the same kind of discontinuity.

Compliance can be achieved if an adequate load path exists to transfer seismic force and if the supporting columns can be demonstrated to have adequate capacity to resist the overturning forces generated by the shear capacity of the discontinuous elements.

A.2.2.5 Geometry *There are no changes in horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding 1-story penthouses and mezzanines.*

Geometric irregularities are usually detected in an examination of the story-to-story variation in the dimensions of the seismic-force-resisting system (Figure A-7). A building with upper stories set back from a broader base structure is a common example. Another example is a story in a high-rise that is set back for architectural reasons. The irregularity of concern is in the dimensions of the seismic-force-resisting system, not in the dimensions of the envelope of the building, and, as such, it may not be obvious.

Geometric irregularities affect the dynamic response of the structure and may lead to unexpected higher mode effects and concentrations of demand. A dynamic analysis should be performed to more accurately calculate the distribution of seismic forces. One-story penthouses need not be considered except for the added mass.

A.2.2.6 Mass *There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered.*

Mass irregularities can be detected by comparison of the story weights (Figure A-8). The effective mass consists of the dead load of the structure tributary to each level, plus the actual weights of partitions and permanent equipment at each floor. Buildings are typically designed for primary mode effects. The validity of this approximation is dependent on the vertical distribution of mass and the stiffness in the building. Mass irregularities affect the dynamic response of the structure and may lead to unexpected higher mode effects and concentrations of demand.

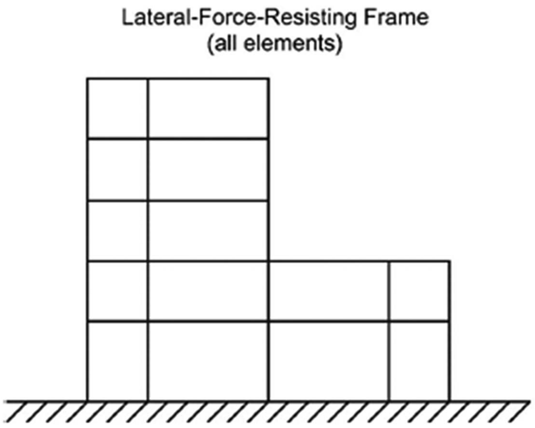


Figure A-7. Geometric irregularities.

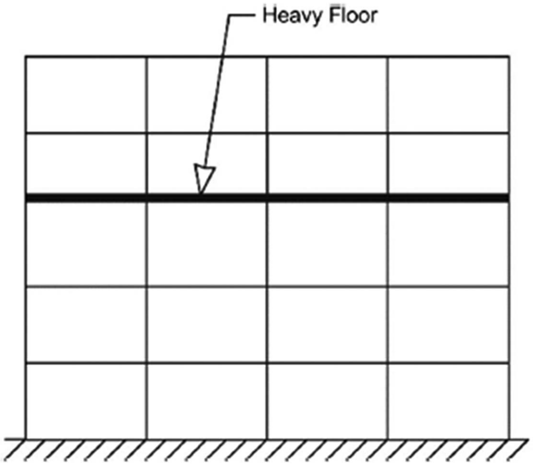


Figure A-8. Heavy floor.

A dynamic analysis should be performed to more accurately calculate the distribution of seismic forces. Light roofs and penthouses need not be considered.

A.2.2.7 Torsion *The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. This statement does not apply to buildings with flexible diaphragms.*

Wherever there is significant torsion in a building with stiff diaphragms, the concern is for additional seismic demands and lateral drifts imposed on the vertical elements by rotation of the diaphragm. Buildings can be designed to meet code forces, including torsion, but buildings with severe torsion are less likely to perform well in earthquakes. It is best to provide a balanced system at the start, rather than design torsion into the system.

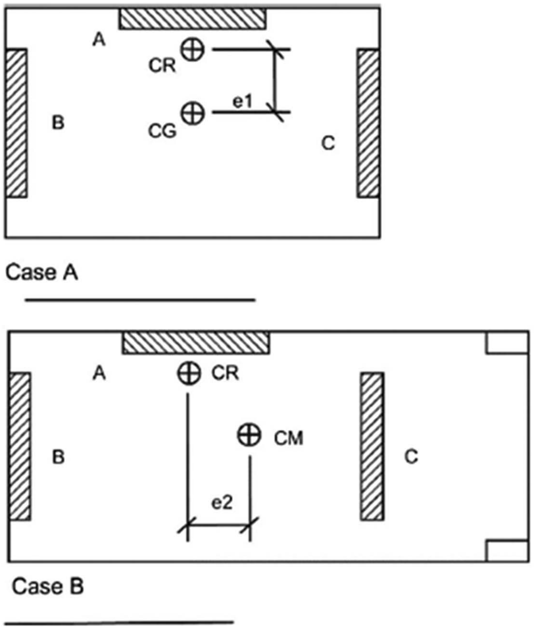
Buildings with flexible diaphragms are less susceptible to poor performance owing to torsion because the lateral forces tend to be distributed to the vertical elements of the seismic-force-resisting system based on tributary diaphragm area rather than relative rigidity of the vertical elements.

One concern is for columns that support the diaphragm, especially if the columns are not intended to be part of the seismic-force-resisting system. The columns are forced to drift laterally with the diaphragm, inducing lateral forces and P-Δ effects. Such columns often have not been designed to resist these movements.

Another concern is the strength of the vertical elements of the seismic-force-resisting system that might experience additional seismic demands caused by torsion.

In the Case A building shown in Figure A-9, the center of gravity is near the center of the diaphragm, while the center of rigidity is also near the centerline but close to Wall A. Under longitudinal loading, the eccentricity, $e1$, between the center of gravity (center of the applied seismic force) and the center of rigidity (center of resistance) causes a torsional moment. The entire seismic force is resisted directly by Wall A, and the torsional moment is resisted by a couple consisting of equal and opposite forces in Walls B and C. These two walls have displacements in opposite directions, and the diaphragm rotates.

These are simple cases for analysis and design, and if the systems are designed and detailed properly, they should perform well. With the ample portions suggested by the length of the walls in Figure A-9, stresses are low and there is little rotation of the diaphragm. The hazard appears where the diaphragm, and



Case A

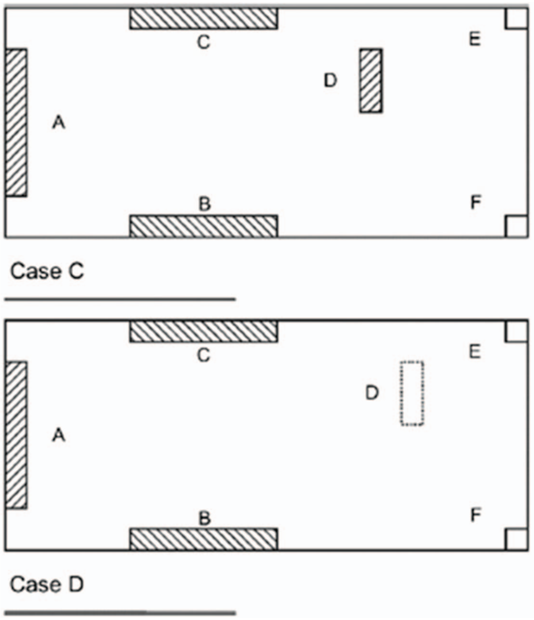
Case B

Figure A-9. Torsion: Cases A and B.

consequently, the diaphragm stresses, become large; where the stiffness of the walls is reduced; or where the walls have substantial differences in stiffness.

The Case C building (Figure A-10) has a more serious torsional condition than the ones in Figure A-9. Wall A has much greater rigidity than Wall D, as indicated by their relative lengths.

For transverse loading, the center of rigidity is close to Wall A, and there is a significant torsional movement. Walls B, C, and D, although strong enough for design forces, have little rigidity, and that allows substantial rotation of the diaphragm. There are two concerns here. First, because of the rotation of the diaphragm, there is a displacement at E and F that induces side-sway moments in the columns that may not have been recognized in the design. Their failure could lead to a collapse. Second, the stability of the building under transverse loading depends on Wall D. The Case D building (Figure A-10) is shown with Wall D failed. The remaining walls, A, B, and C, are in Figure A-9, and now there is a very large eccentricity that may cause Walls B and C to fail. This is also an example of a building that lacks redundancy.



Case C

Case D

Figure A-10. Torsion: Cases C and D.

A.3 PROCEDURES FOR SEISMIC-FORCE-RESISTING SYSTEMS

This section provides guidelines for using the Tier 1 checklists and the Tier 2 deficiency-based evaluation and retrofit guidelines that apply to seismic-force-resisting systems: moment frames, shear walls, and braced frames.

A.3.1 Moment Frames Moment frames develop their resistance to forces primarily through the flexural strength of the beam and column elements.

In an earthquake, a frame with suitable proportions and details can develop plastic hinges that absorb energy and allow the frame to survive actual displacements that are larger than calculated in an elastic-based design.

In "special" moment frames, the ends of beams and columns, being the locations of maximum seismic moment, are designed to sustain inelastic behavior associated with plastic hinging over many cycles and load reversals.

Frames without special seismic detailing depend on the reserve strength inherent in the design of the members. The basis of this reserve strength is the load factors in strength design or the factors of safety in working-stress design. Such frames are called "ordinary" moment frames. For ordinary moment frames, failure usually occurs because of a sudden brittle mechanism, such as shear failure in concrete members.

For evaluations using this standard, it is not necessary to determine the type of frame (e.g., "special") in the building. The performance issue is addressed by appropriate acceptance criteria in the specified procedures. The fundamental requirements for all ductile moment frames are the following:

They should have sufficient strength to resist seismic demands,

- They should have sufficient strength to resist seismic demands,
- They should have sufficient stiffness to limit interstory drift,
- Beam–column joints should have the shear capacity to resist the shear demand and to develop the strength of the connected members,
- Elements should be able to form plastic hinges that have the ductility to sustain the rotations to which they are subjected, and
- Beams should develop hinges before the columns at locations distributed throughout the structure (the strong column–weak beam concept).

These items are covered in more detail in the evaluation statements that follow.

The combined action of gravity loads and seismic forces are expected to cause the formation of plastic hinges in the structure. However, a concentration of plastic hinge formation at undesirable locations can severely undermine the stability of the structure. For example, the lower part of Figure A-11 shows a story mechanism in which hinges form at the tops and bottoms of all the columns in a particular story. This condition results in a concentration of ductility demand and displacement in a single story that can lead to collapse.

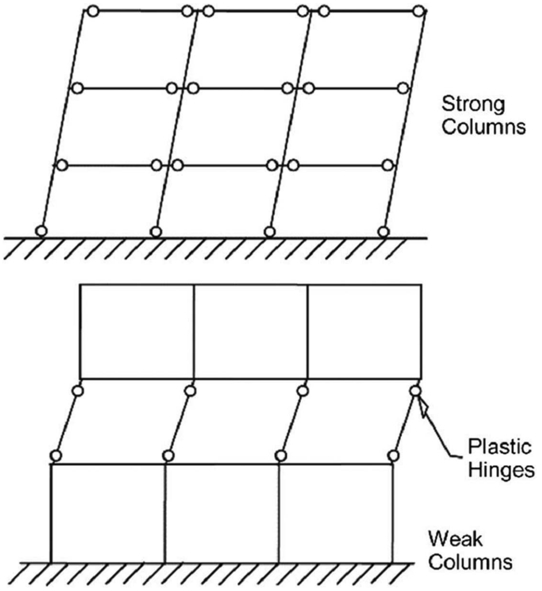


Figure A-11. Plastic hinge formation.

In a strong column situation (the upper part of Figure A-11), the beams hinge first, yielding is distributed throughout the structure, and the ductility demand is more dispersed.

A.3.1.1 General

A.3.1.1.1 Redundancy The number of lines of moment frames in each direction is greater than or equal to 2. The number of bays of moment frames in each line is greater than or equal to 2 for Life Safety and 3 for Immediate Occupancy.

Redundancy is a fundamental characteristic of seismic-force-resisting systems with superior seismic performance. Redundancy in the structure ensures that if an element in the seismic-force-resisting system fails for any reason, there is another element present that can provide seismic force resistance. Redundancy also provides multiple locations for potential yielding, distributing inelastic activity throughout the structure and improving ductility and energy absorption. Typical characteristics of redundancy include multiple lines of resistance to distribute the seismic forces uniformly throughout the structure and multiple bays in each line of resistance to reduce the shear and axial demands on any one element (Figure A-12).

A distinction should be made between redundancy and adequacy. For the purpose of this standard, redundancy is intended to mean simply "more than one." That is not to say that for large buildings two elements is adequate, or for small buildings one is not enough. Separate evaluation statements are present in the standard to determine the adequacy of the elements provided.

Where redundancy is not present in the structure, an analysis that demonstrates the adequacy of the seismic-force-resisting elements is required.

The most prudent retrofit strategy for a building without redundancy is to add new seismic-force-resisting elements in locations where the failure of a few components would cause an instability in the building. The added seismic-force-resisting elements should be of the same stiffness as the elements they are supplementing. It is not generally satisfactory just to strengthen a nonredundant element (such as by adding cover plates to a slender brace) because its failure would still result in an instability.

A.3.1.2 Moment Frames with Infill Walls Infill walls used for partitions, cladding, or shaft walls that enclose stairs and elevators should be isolated from the frames. If not isolated, they alter the response of the frames and change the behavior of the entire structural system. Lateral drifts of the frame induce forces on walls that interfere with this movement. Cladding connections must allow for this relative movement. Stiff infill walls confined by the frame develop compression struts that impart forces to the frame and cause damage to the walls. This phenomenon is particularly important around stairs or other means of egress from the building.

A.3.1.2.1 Interfering Walls All concrete and masonry infill walls placed in moment frames are isolated from structural elements.

Where an infill wall interferes with the moment frame, the wall becomes an unintended part of the seismic-force-resisting system. Typically these walls are not designed and detailed to participate in the seismic-force-resisting system, and they may be subject to significant damage. The amount of isolation must be able to accommodate the interstory drift of the moment frame.

Interfering walls should be checked for forces induced by the frame, particularly where damage to these walls can lead to falling hazards near means of egress. The frames should be checked for forces induced by contact with the walls, particularly if the walls are not full height or do not completely fill the bay.

It is impossible to simultaneously satisfy this section and Section A.3.2.6.1, which covers infill walls that are intended to be part of the seismic-force-resisting system.

A.3.1.3 Steel Moment Frames The following are characteristics of steel moment frames that have demonstrated acceptable seismic performance:

- The beam end connections develop the plastic moment capacity of the beam or panel zone,
- There is a high level of redundancy in the number of moment connections,
- The column web has sufficient strength to sustain the stresses in the beam–column joint,

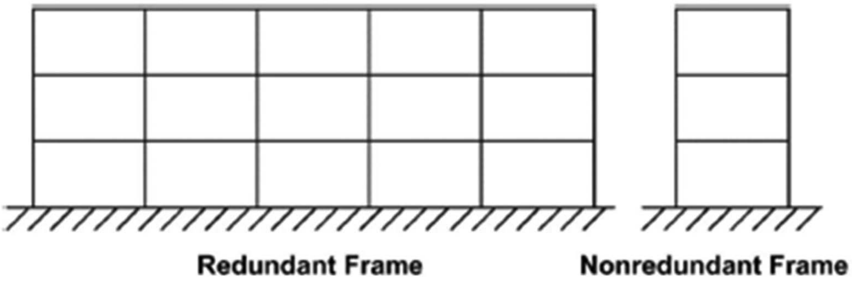


Figure A-12. Redundancy along a line of moment frame.

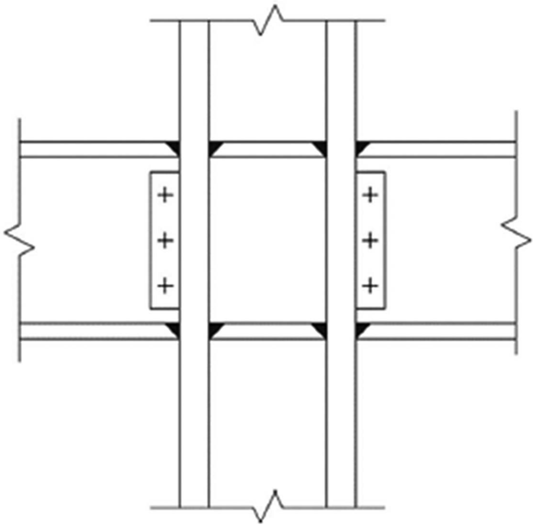


Figure A-13. Pre-Northridge-type connection.

- The lower flanges have lateral bracing sufficient to maintain stability of the frame, and
- There is flange continuity through the column.

Before the 1994 Northridge earthquake, steel moment-frame connections generally consisted of complete penetration flange welds and a bolted or welded shear tab connection at the web (Figure A-13). This type of connection, which was an industry standard from 1970 to 1995, was thought to be ductile and capable of developing the full capacity of the beam sections. However, a large number of buildings experienced extensive brittle damage to this type of connection during the Northridge earthquake. As a result, an emergency code change was made to the 1994 *Uniform Building Code* (ICBO 1994) to remove the prequalification of this type of connection. For a full discussion of these connections, please refer to FEMA 351 (2000b) and FEMA 355D (2000e).

A.3.1.3.1 Drift Check The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 4.4.3.1, is less than 0.030 for Collapse Prevention and 0.015 for Immediate Occupancy.

Moment frames are more flexible than shear wall or braced frame structures. This flexibility can lead to large interstory drifts that may potentially cause extensive structural and nonstructural damage to welded beam–column connections, partitions, and cladding. Drifts also may induce large P-Δ demands and pounding where adjacent buildings are present.

For Building Type S-4 (Dual Systems), the drift check is performed using 25% of the computed seismic forces.

An analysis of noncompliant frames is required to demonstrate the adequacy of frame elements subjected to excessive lateral drifts.

The most direct mitigation approach is to add properly placed and distributed stiffening elements—new moment frames, braced frames, or shear walls—that can reduce the story drifts to acceptable levels. Alternatively, the addition of energy dissipation devices to the system may reduce the drift, although these are outside the scope of the deficiency-based retrofit method.

A.3.1.3.2 Column Axial Stress Check The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$. *Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6, is less than* $0.30F_y$.

Columns that carry a substantial amount of gravity load may have limited additional capacity to resist seismic forces. Where axial forces caused by seismic overturning moments are added, the columns may buckle in a nonductile manner because of excessive axial compression.

The alternative calculation of overturning stresses caused by seismic forces alone is intended to provide a means of identifying frames that are likely to be adequate: frames with high gravity loads but small seismic overturning forces.

Where both demands are large, the combined effect of gravity and seismic forces must be calculated to demonstrate compliance.

A.3.1.3.3 Flexural Stress Check The average flexural stress in the moment-frame columns and beams, calculated using the Quick Check procedure of Section 4.4.3.9, is less than F_y. *Columns need not be checked if the strong column–weak beam checklist item is compliant.*

The flexural stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building. Although most steel moment-frame behavior is controlled by drift, there may be some configurations that do not have adequate strength.

A.3.1.3.4 Moment-Resisting Connections All moment connections are able to develop the strength of the adjoining members based on the specified minimum yield stress of steel for moderate seismicity and the strength of the adjoining members or panel zones based on 110% of the expected yield stress of the steel in accordance with AISC 341, Section A3.2 for high seismicity.

See Section A.3.1.3 for a general discussion of moment-frame connections. For this standard, the Tier 1 evaluation statement is effectively considered noncompliant for full-penetration flange welds subject to higher cyclic demands. A more detailed analysis is required to determine the adequacy of these moment-resisting connections.

Adding a stiffer seismic-force-resisting system (e.g., braced frames or shear walls) can reduce the expected rotation demands. Connections can be modified by adding flange cover plates, vertical ribs, haunches, or brackets, or removing beam flange material to initiate yielding away from the connection location (e.g., via a pattern of drilled holes or the cutting out of flange material). Partial-penetration splices, which may become more vulnerable for conditions where the beam–column connections are modified to be more ductile, can be modified by adding plates and/or welds. Adding continuity plates alone is not likely to enhance the connection performance significantly. Moment-resisting connection capacity can be increased by adding cover plates or haunches or by using other techniques as stipulated in FEMA 351 (2000b).

A.3.1.3.5 Panel Zones All panel zones have the shear capacity to resist the shear demand required to develop 0.8 times the sum of the flexural strengths of the girders framing in at the face of the column.

Panel zones with thin webs may yield or buckle before developing the capacity of the adjoining members, reducing the inelastic performance and ductility of the moment frames.

Where panel zones cannot develop the strength of the beams, compliance can be demonstrated by checking the panel zones for actual shear demands.

Refer to Section A.3.1.3.4 for additional guidelines for retrofitting moment-frame connections.

A.3.1.3.6 Column Splices For Collapse Prevention, at all column splice details located in moment frames, the web and flanges *of I-shaped members, or all walls of tube/box members are connected to each other with the partial-penetration welds with effective throat of at least 85% of the smaller member thickness or with plates that have been bolted or welded to the columns capable of developing $A_g F_{ye}$ of the thinner flange, web or tube/box wall. For Immediate Occupancy, all column splice details located in moment frames include connection of both flanges and the web, and the splice develops the strength of the column.*

The lack of a substantial connection at the splice location may lead to separation of the spliced sections and misalignment of the columns, resulting in loss of vertical support and partial or total collapse of the building. Tests on partial-penetration weld splices have shown limited ductility.

In addition, column splice fracture was documented in the 1995 Kobe Earthquake, which contributed to significant building damage and collapse in some extreme cases.

An inadequate connection also reduces the effective capacity of the column. Splices should be checked against calculated demands to demonstrate compliance.

Refer to Section A.3.1.3.4 for additional guidelines for retrofitting moment-frame connections.

A.3.1.3.7 Strong Column–Weak Beam The percentage of strong column–weak beam joints in each story of each line of moment-resisting frames is greater than 50%.

Where columns are not strong enough to force hinging in the beams, column hinging can lead to story mechanisms and a concentration of inelastic activity at a single level. Excessive story drifts may result in instability of the frame caused by P-Δ effects. Good postelastic behavior consists of yielding distributed throughout the frame. A story mechanism limits forces in the levels above, preventing the upper levels from yielding.

If it can be demonstrated that noncompliant columns are strong enough to resist calculated demands with sufficient overstrength, acceptable behavior can be expected.

Steel plates can be added to increase the strength of the steel columns to beyond that of the beams to eliminate this issue. Stiffening elements (e.g., braced frames, shear walls, or additional moment frames) can be added throughout the building to reduce the expected frame demands.

A.3.1.3.8 Compact Members All frame elements meet section requirements in accordance with AISC 341, Table D1.1a, for "moderately ductile" members for Collapse Prevention and for "highly ductile" members for Immediate Occupancy, except for Building Type S-3, where frame elements meet compact section requirements in accordance with AISC 360, Table B4.1a.

Noncompact frame elements may experience premature local buckling before development of their full moment capacities. Members that do not meet these criteria may experience premature local buckling before development of their full moment capacities. This problem can lead to poor inelastic behavior and ductility.

The adequacy of the frame elements can be demonstrated by a Tier 2 evaluation using reduced *m*-factors in consideration of reduced capacities for noncompact sections.

Noncompact members can be eliminated by adding appropriate steel plates. Stiffening elements (e.g., braced frames, shear walls, or additional moment frames) can be added throughout the building to reduce the expected frame demands.

A.3.1.3.9 Beam Penetrations. All openings in frame-beam webs are less than one-quarter of the beam depth and are located in the center half of the beams.

Members with large beam penetrations may fail in shear before the development of their full moment capacity, resulting in poor inelastic behavior and ductility.

The critical section is at the penetration with the highest shear demand. Shear transfer across the web opening induces secondary moments in the beam sections above and below the opening that must be considered in the analysis.

Eliminating or properly reinforcing large member penetrations develops the demanded strength and deformations. Stiffening elements (e.g., braced frames, shear walls, or additional moment frames) can be added throughout the building to reduce the expected frame demands.

A.3.1.3.10 Girder Flange Continuity Plates. There are girder flange continuity plates at all moment-resisting-frame joints.

The lack of girder flange continuity plates may lead to a premature failure at the column web or flange at the joint. Beam flange forces are transferred to the column web through the column flange, resulting in a high-stress concentration at the base of the column web. The presence of continuity plates, however, transfers the beam flange forces along the entire length of the column web.

Adequate force transfer without continuity plates depends on the strength and stiffness of the column flange in weak-way bending.

Refer to Section A.3.1.3.4 for additional guidelines for retrofitting moment-frame connections.

A.3.1.3.11 Out-of-Plane Bracing. Beam–column joints are braced out of plane.

Columns without proper bracing may buckle prematurely out of plane before the strength of the joint can be developed. This buckling limits the ability of the frame to resist seismic forces.

The combination of axial load and moment on the columns results in higher compression forces in one of the column flanges. The tendency for highly loaded joints to twist out of plane is caused by compression buckling of the critical column compression flange.

Compliance can be demonstrated if the column section can provide adequate lateral restraint for the joint between points of lateral support.

Lateral bracing in the form of new steel components can be added to reduce member unbraced lengths to within the limits prescribed. Stiffening elements (e.g., braced frames, shear walls, or additional moment frames) can be added throughout the building to reduce the expected frame demands.

A.3.1.3.12 Bottom-Flange Bracing. The bottom flanges of beams are braced out of plane.

Beam flanges in compression require out-of-plane bracing to prevent lateral torsional buckling. Buckling occurs before the full strength of the beam is developed, and the ability of the frame to resist seismic forces is limited.

Top flanges are typically braced by connection to the diaphragm. Bottom-flange bracing occurs at discrete locations, such as at connection points for supported beams. The spacing of bottom-flange bracing may not be close enough to prevent premature lateral torsional buckling where seismic forces induce large compression forces in the bottom flange.

A.3.1.4 Concrete Moment Frames Concrete moment-frame buildings typically are more flexible than shear wall buildings. This flexibility can result in large interstory drifts that may lead to extensive nonstructural damage and P-Δ effects. If a concrete column has a capacity in shear that is less than the shear associated with the flexural capacity of the column, brittle column shear failure may occur and result in collapse. This condition is common in buildings in zones of moderate seismicity and in older buildings in zones of high seismicity.

The columns in these buildings often have ties at standard spacing equal to the depth of the column, whereas current ACI 318 code maximum spacing for shear reinforcing is much smaller. The following are the characteristics of concrete moment frames that have demonstrated acceptable seismic performance:

- Brittle failure is prevented by providing a sufficient number of beam stirrups, column ties, and joint ties to ensure that the shear capacity of all elements exceeds the shear associated with flexural capacity;
- Concrete confinement is provided by beam stirrups and column ties in the form of closed hoops with 135 degree hooks at locations where plastic hinges are expected to occur;
- Overall performance is enhanced by long lap splices that are restricted to favorable locations and protected with additional transverse reinforcement; and
- The strong column–weak beam requirement is achieved by suitable proportioning of the members and their longitudinal reinforcing.

Older frame systems that are lightly reinforced, precast concrete frames, and flat slab frames usually do not meet the detail requirements for ductile behavior. Adding properly placed and distributed stiffening elements, such as shear walls or braced frames, can fully supplement the moment-frame system with a new seismic-force-resisting system. For eccentric joints, columns and/or beams may be jacketed to reduce the effective eccentricity. Jackets may also be provided for shear-critical columns. It must be verified that this new system sufficiently reduces the frame shears and story drifts to acceptable levels.

A.3.1.4.1 Column Shear Stress Check. The shear stress in the concrete columns, calculated using the Quick Check procedure of Section 4.4.3.2, is less than the greater of 100 lb/in.^{2} *(0.69 MPa) or* $2\sqrt{f'_c}$

The shear stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building.

A.3.1.4.2 Column Axial Stress Check. The axial compressive stress caused by unfactored gravity loads in columns subjected to overturning demands is less than $0.20f'_c$ *for Collapse Prevention and* $0.13f'_c$ *for Immediate Occupancy for cast-in-place moment frames and* $0.10f'_c$ *for precast moment frames without shear walls. Alternatively, the axial compressive stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6, is less than* $0.30f'_c$.

Columns that carry a substantial amount of gravity load may have limited additional capacity to resist seismic forces. Where axial forces caused by seismic overturning moments are added, the columns may crush in a nonductile manner because of excessive axial compression.

The alternative calculation of overturning stresses caused by seismic forces alone is intended to provide a means of identifying frames that are likely to be adequate: frames with high gravity loads but small seismic overturning forces.

Where both demands are large, the combined effect of gravity and seismic forces must be calculated to demonstrate compliance.

A.3.1.4.3 Flat Slab Frames. The seismic-force-resisting system is not a frame consisting of columns and a flat slab or plate without beams.

The concern is the transfer of the shear and bending forces between the slab and column, which could result in a punching shear failure and partial collapse. The flexibility of the seismic-force-resisting system increases as the slab cracks.

Continuity of some bottom reinforcement through the column joint assists in the transfer of forces and provides some resistance to collapse by catenary action in the event of a punching shear failure.

A.3.1.4.4 Prestressed Frame Elements. The seismic-force-resisting frames do not include any prestressed or posttensioned elements where the average prestress exceeds the lesser of 700 lb/in.^{2} *(4.83 MPa) or* $f'_c/6$ *at potential hinge locations. The average prestress is calculated in accordance with the Quick Check procedure of Section 4.4.3.8.*

Frame elements that are prestressed or posttensioned may not behave in a ductile manner. The concern is the inelastic behavior of prestressed elements.

A.3.1.4.5 Captive Columns. There are no columns at a level with height-to-depth ratios less than 50% of the nominal height-to-depth ratio of the typical columns at that level for Collapse Prevention and 75% for Immediate Occupancy.

Captive columns tend to attract seismic forces because of high stiffness relative to other columns in a story. Significant damage has been observed in parking structure columns adjacent to ramping slabs, even in structures with shear walls. Captive column behavior also may occur in buildings with clerestory windows or in buildings with partial height masonry infill panels.

If not adequately detailed, the columns may suffer a nonductile shear failure, which may result in partial collapse of the structure.

A captive column that can develop the shear capacity to develop the flexural strength over the clear height has some ductility to prevent sudden nonductile failure of the vertical support system.

Columns may be jacketed with steel, fiber-reinforced polymer (FRP), or concrete such that they can resist the expected forces and drifts. Alternatively, the expected story drifts can be reduced throughout the building by infilling openings or adding shear walls.

A.3.1.4.6 No Shear Failures. The shear capacity of frame members is able to develop the moment capacity at the ends of the members.

If the shear capacity of a member is reached before the moment capacity, there is a potential for a sudden nonductile failure of the member, leading to collapse.

Members that cannot develop the flexural capacity in shear should be checked for adequacy against calculated shear demands. For columns, the shear capacity is affected by the axial loads and should be based on the most critical combination of axial load and shear.

A.3.1.4.7 Strong Column–Weak Beam. The sum of the moment capacity of the columns is 20% greater than that of the beams at frame joints.

Where columns are not strong enough to force hinging in the beams, column hinging can lead to story mechanisms and a concentration of inelastic activity at a single level. Excessive story drifts may result in instability of the frame caused by P-Δ effects. Good postelastic behavior consists of yielding distributed throughout the frame. A story mechanism limits forces in the levels above, preventing the upper levels from yielding.

If it can be demonstrated that noncompliant columns are strong enough to resist calculated demands with sufficient overstrength, acceptable behavior can be expected. A Tier 2 evaluation with reduced *m*-factors can be used to check the columns at near-elastic levels.

A.3.1.4.8 Beam Bars. At least two longitudinal top and two longitudinal bottom bars extend continuously throughout the length of each frame beam. At least 25% of the longitudinal bars provided at the joints for either positive or negative moment are continuous throughout the length of the members.

The requirement for two continuous bars is a Collapse Prevention measure. In the event of complete beam failure, continuous bars prevent total collapse of the supported floor, holding the beam in place by catenary action.

Previous construction techniques used bent-up longitudinal bars as reinforcement. These bars transitioned from bottom to top reinforcement at the gravity load inflection point. Some amount of continuous top and bottom reinforcement is desired because moments caused by seismic forces can shift the location of the inflection point.

Because noncompliant beams are vulnerable to collapse, the beams are required to resist demands at an elastic level. Continuous slab reinforcement adjacent to the beam may be considered as continuous top reinforcement.

A.3.1.4.9 Column-Bar Splices. All column-bar lap splice lengths are greater than $35d_b$ for Collapse Prevention and $50d_b$ for Immediate Occupancy and are enclosed by ties spaced at or less than $8d_b$. Alternatively, column bars are spliced with mechanical couplers with a capacity of at least 1.25 times the nominal yield strength of the spliced bar.

Located just above the floor level, column-bar splices are typically located in regions of potential plastic hinge formation. Short splices are subject to sudden loss of bond. Widely spaced ties can result in a spalling of the concrete cover and loss of bond. Splice failures are sudden and nonductile.

Columns with noncompliant lap splices can be checked using Tier 2 with reduced *m*-factors to account for this potential lack of ductility. If the members have sufficient capacity, the demands on the splices are less likely to exceed the capacity of the bond.

A.3.1.4.10 Beam-Bar Splices. The lap splices or mechanical couplers for longitudinal beam reinforcing are not located within $L_b/4$ of the joints and are not located in the vicinity of potential plastic hinge locations.

Lap splices located at the ends of beams and in the vicinity of potential plastic hinges may not be able to develop the full moment capacity of the beam as the concrete degrades during multiple cycles.

Beams with noncompliant lap splices can be checked using Tier 2 with reduced *m*-factors to account for this potential lack of ductility. If the members have sufficient capacity, the demands are less likely to cause degradation and loss of bond between concrete and the reinforcing steel.

A.3.1.4.11 Column-Tie Spacing. Frame columns have ties spaced at or less than d/4 throughout their length and at or less than $8d_b$ at all potential plastic hinge locations.

Widely spaced ties reduce the ductility of the column, and the column may not be able to maintain full moment capacity through several cycles. Columns with widely spaced ties have limited shear capacity, and nonductile shear failures may result.

Elements with noncompliant confinement can be checked using Tier 2 with reduced *m*-factors to account for this potential lack of ductility.

A.3.1.4.12 Stirrup Spacing. All beams have stirrups spaced at or less than d/2 throughout their length. At potential plastic hinge locations, stirrups are spaced at or less than the minimum of $8d_b$ or d/4.

Widely spaced stirrups reduce the ductility of the beam, and the beam may not be able to maintain full moment capacity through several cycles. Beams with widely spaced stirrups have limited shear capacity, and nonductile shear failures may result.

Elements with noncompliant confinement can be checked using Tier 2 with reduced *m*-factors to account for this potential lack of ductility.

A.3.1.4.13 Joint Transverse Reinforcing. Beam–column joints have ties spaced at or less than $8d_b$.

Beam–column joints without shear reinforcement may not be able to develop the strength of the connected members, leading to nonductile failure of the joint. Perimeter columns are especially vulnerable because the confinement of joint is limited to three sides (along the exterior) or two sides (at a corner). Joints have more capacity if transverse beams exist on both sides of the joint.

A.3.1.4.14 Joint Eccentricity. There are no eccentricities larger than 20% of the smallest column plan dimension between girder and column centerlines.

Joint eccentricities can result in high torsional demands on the joint area, which result in higher shear stresses. The smallest column plan dimension should be calculated for the column at each joint under consideration.

A.3.1.4.15 Stirrup and Tie Hooks. The beam stirrups and column ties are anchored into the member cores with hooks of 135 degrees or more.

To be fully effective, stirrups and ties must be anchored into the confined core of the member. Ninety-degree hooks that are anchored within the concrete cover are unreliable if the cover spalls during plastic hinging. The amount of shear resistance and confinement are reduced if the stirrups and ties are not well anchored.

Elements with noncompliant confinement can be checked using Tier 2 with reduced *m*-factors to account for this potential lack of ductility.

A.3.1.5 Precast Concrete Moment Frames The development of a competent load path is extremely critical in these buildings. If the connections have sufficient strength so that yielding first occurs in the members rather than in the connections, the building should be evaluated as a moment-frame system, Type C1.

A.3.1.5.1 Precast Connection Check. The precast connections at frame joints have the capacity to resist the shear and moment demands calculated using the Quick Check procedure of Section 4.4.3.5.

Precast frame elements may have sufficient strength to meet seismic force requirements, but connections often cannot develop the strength of the members and may be subject to premature nonductile failures. Failure mechanisms may include fractures in the welded connections between inserts, pullout of embeds, and spalling of concrete.

Because full member capacities cannot be realized, the behavior of this system is entirely dependent on the performance of the connections.

A.3.1.5.2 Precast Frames. For buildings with concrete shear walls, precast concrete frame elements are not considered as primary components for resisting seismic forces.

Precast frame elements may have sufficient strength to meet seismic force requirements, but connections often cannot develop the strength of the members and may be subject to premature nonductile failures. Failure mechanisms may include fractures in the welded connections between inserts, pullout of embeds, and spalling of concrete.

Because full member capacities cannot be realized, the behavior of this system is entirely dependent on the performance of the connections.

A.3.1.5.3 Precast Connections. For buildings with concrete shear walls, the connection between precast frame elements, such as chords, ties, and collectors in the seismic-force-resisting system, develops the capacity of the connected members.

Precast frame elements may have sufficient strength to meet seismic force requirements, but connections often cannot develop the strength of the members and may be subject to premature nonductile failures. Failure mechanisms may include fractures in the welded connections between inserts, pullout of embeds, and spalling of concrete.

Because full member capacities cannot be realized, the behavior of this system is entirely dependent on the performance of the connections.

The connections of chords, ties, and collectors can be upgraded to increase strength and/or ductility, providing alternative load paths for seismic forces. Upgrading can be achieved by such methods as adding confinement ties or increasing embedment. Shear walls can be added to reduce the demand on connections.

A.3.1.6 Frames Not Part of the Seismic-Force-Resisting System This section deals with secondary components consisting of frames that were not designed to be part of the seismic-force-resisting system. These are basic structural frames of steel or concrete that are designed for gravity loads only. Shear walls or other vertical elements provide the resistance to seismic forces. In actuality, however, all frames act as part of the seismic-force-resisting system. Lateral drifts of the building induce forces in the beams and columns of the secondary frames. Furthermore, in the event that the primary elements fail, the secondary frames become the primary seismic-force-resisting components of the building.

If the walls are concrete (infilled in steel frames or monolithic in concrete frames), the building should be treated as a concrete shear wall building (Types C2 or C2a) with the frame columns as boundary elements. If the walls are masonry infills, the frames should be treated as steel or concrete frames with infill walls of masonry (Types S5, S5a, C3, or C3a). Research is continuing on the behavior of infill frames. Seismic forces are resisted by compression struts that develop in the masonry infill and induce forces on the frame elements eccentric to the joints.

The concern for secondary frames is the potential loss of vertical-load-carrying capacity caused by excessive deformations and P-Δ effects.

A.3.1.6.1 Concrete Bearing Walls Floor and roof girders and trusses are not supported at the ends of concrete walls that are less than 10 in. (254 mm) thick. This statement only applies to framing supports located less than two times the wall thickness away from the wall end.

Concrete bearing walls are commonly used to support gravity loads from floor and roof framing members. However, in older construction the ends of bearing walls, where a column or pilaster is not present, may not have sufficient thickness or confinement to effectively support major floor or roof framing members supported by the wall end. During an earthquake, the ends of relatively thin or unconfined shear walls might become damaged by seismic forces, limiting their ability to support vertical loads. Loss of vertical support may lead to partial collapse.

Wall that are 10 in. (254 mm) thick or greater will often have either two curtains of reinforcement or are judged thick enough to have a low risk of failure owing to out-of-plane loading or local bearing conditions. In accordance with the checklist statement, the statement is intended to apply only to concrete walls; framing supported by a column or pilaster located at the end of a concrete wall is considered to be compliance with the checklist statement. Compliance can be demonstrated if the wall is judged adequate for combined vertical and seismic forces.

A.3.1.6.2 Deflection Compatibility Secondary components have the shear capacity to develop the flexural strength of the components and for Immediate Occupancy are compliant with the following items: COLUMN-BAR SPLICES, BEAM-BAR SPLICES, COLUMN-TIE SPACING, STIRRUP SPACING, and STIRRUP AND TIE HOOKS.

Frame components, especially columns, that are not specifically designed to participate in the seismic-force-resisting system still undergo displacements associated with overall seismic interstory drifts. If the columns are located some distance away from the seismic-force-resisting elements, the added deflections caused by semirigid floor diaphragms increase the drifts. Stiff columns, designed for potentially high gravity loads, may develop significant bending moments because of the imposed drifts. The moment or axial force interaction may lead to a nonductile failure of the columns and a collapse of the building.

Vertical seismic-force-resisting elements can be added to decrease the drift demands on the columns, or the ductility of the columns can be increased. Jacketing the columns with steel, FRP, or concrete is one approach to increase their ductility.

A.3.1.6.3 Flat Slabs Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints.

Flat slabs not designed to participate in the seismic-force-resisting system may still experience seismic forces because of displacements associated with overall building drift. The concern is the transfer of the shear and bending forces between the slab and column, which could result in a punching shear failure.

A problem with some slabs can occur when a small section of slab exists between two adjacent shear walls or braced frames. The section of slab can act as a coupling beam, even though it was not intended to do so. This action can result in excessive damage to the slab and loss of vertical slab support if the slab is not properly detailed. Thin slabs and those with long spans have less tendency to act as coupling beams and would attract less force.

Continuity of some bottom reinforcement through the column joint assists in the transfer of forces and provides some resistance to collapse by catenary action in the event of a punching shear failure (Figure A-14). Bars can be considered continuous if they have proper lap splices, have mechanical couplers, or are developed beyond the support.

A.3.2 Shear Walls

A.3.2.1 General In the analysis of shear walls, it is customary to consider the shear taken by the length of the wall and the flexure taken by vertical reinforcement added at each end, much as flexure in diaphragms is designed to be taken by chords at the edges. Squat walls that are long compared with their height are dominated by shear behavior. Flexural forces require only a slight local reinforcement at each end. Slender walls that are tall compared with their length are usually dominated by flexural behavior and may require substantial boundary elements at each end.

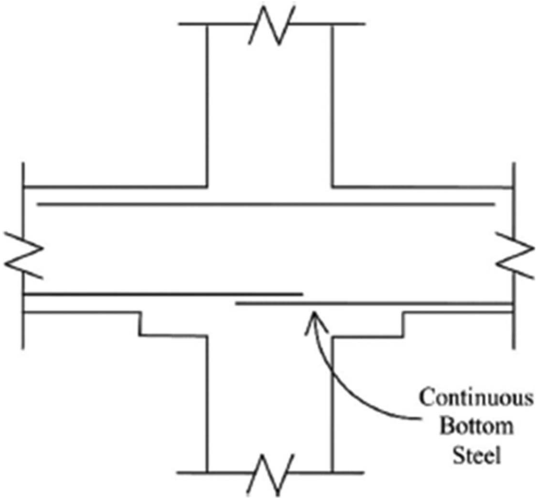


Figure A-14. Continuous bottom steel.

It is a good idea to sketch a complete free-body diagram of the wall (as indicated in Figure A-15) so that no forces are inadvertently neglected. An error often made in the design of wood shear walls is to treat the walls one story at a time, considering only the shear force in the wall and overlooking the accumulation of overturning forces from the stories above.

Where the earthquake direction being considered is parallel to a shear wall, the wall develops in-plane shear and flexural forces as previously described. Where the earthquake direction is perpendicular to a shear wall, the wall contributes little to the seismic force resistance of the building and the wall is subjected to out-of-plane forces. This section addresses the in-plane behavior of shear walls. Out-of-plane strength and anchorage of shear walls to the structure is addressed in Section A.5.

Solid shear walls usually have sufficient strength, although they may be lightly reinforced. Problems with shear wall systems arise where walls are not continuous to the foundation or where numerous openings break the walls up into small piers with limited shear and flexural capacity.

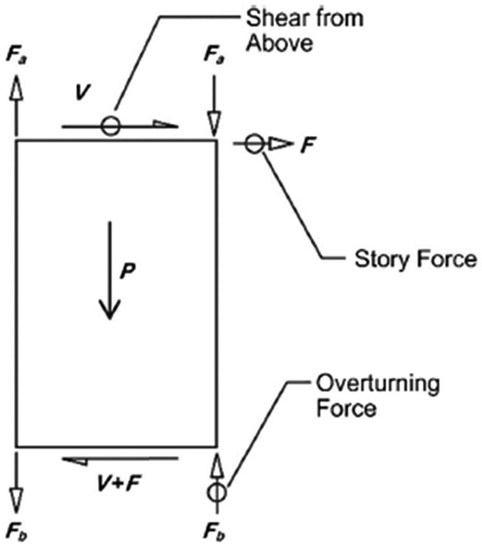


Figure A-15. Wall free-body diagram.

Figure A-16. Redundancy in shear walls.

A.3.2.1.1 Redundancy The number of lines of shear walls in each direction is greater than or equal to 2.

Refer to Section A.3.1.1.1 for commentary related to redundancy. Figure A-16 illustrates an example of redundancy for shear wall buildings in which there are multiple lines of resistance to distribute the seismic forces uniformly throughout the structure and multiple bays in each line of resistance to reduce the shear and flexure demands on any one element.

A.3.2.2 Concrete Shear Walls In highly redundant buildings with many long walls, stresses in concrete shear walls are usually low. In less redundant buildings with large openings and slender walls, the stresses can be high. In the ultimate state, where overturning forces are at their highest, a thin wall may fail in buckling along the compression edge, or it may fail in tension along the tension edge. Tension failures may consist of slippage in bar lap splices, or bar yield and fracture if adequate lap splices have been provided.

In the past, designs have been based on liberal assumptions about compression capacity and have simply packed vertical rebar into the ends of the walls to resist the tensile forces. Recent codes, recognizing the importance of boundary members, have special requirements for proportions, bar splices, and transverse reinforcement. Examples of boundary members with varying amounts of reinforcing are shown in Figure A-17. Existing buildings often do not have these elements, and the acceptance criteria are designed to allow for this.

Another development in recent codes is the requirement to provide shear strength compatible with the flexural capacity of the wall to ensure ductile flexural yielding before brittle shear failure. Long continuous walls and walls with embedded steel or large boundary elements can have high flexural capacities with the potential to induce correspondingly high shear demands that are over and above the minimum design shear demands.

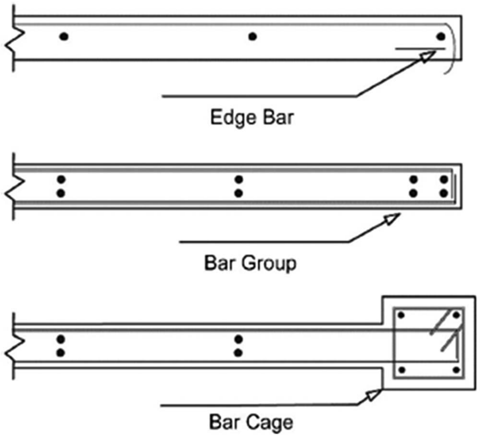


Figure A-17. Boundary elements.

A.3.2.2.1 Shear Stress Check. The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the greater of 100 lb/in.^{2} *(0.69 MPa) or* $2\sqrt{f'_c}$.

The shear stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building.

For Building Type S-4 (Dual System), the backup moment frame is neglected when determining the shear stresses on the shear walls.

New shear walls can be provided and/or the existing walls can be strengthened to satisfy seismic demand criteria. New and strengthened walls must form a complete, balanced, and properly detailed seismic-force-resisting system for the building. Special care is needed to ensure that the connection of the new walls to the existing diaphragm is appropriate and of sufficient strength such that yielding first occurs in the wall. All shear walls must have sufficient shear and overturning resistance to meet the load criteria of this standard.

A.3.2.2.2 Reinforcing Steel The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. In addition, for Immediate Occupancy the spacing of reinforcing steel is equal to or less than 18 in. (457 mm).

If the walls do not have sufficient reinforcing steel, they have limited capacity in resisting seismic forces. The wall also behaves in a nonductile manner for inelastic forces. The minimum reinforcing ratios are based on the ACI requirements for general wall reinforcing that have been applicable for many years. These limits are applicable for walls with No. 5 or smaller reinforcing bars horizontally and vertically.

Shear walls can be strengthened by infilling openings, applying FRP, or by thickening the walls; for examples, see FEMA 547 (2006).

A.3.2.2.3 Coupling Beams For Collapse Prevention, coupling beams have stirrups spaced at or less than d/2, and each wall or wall segment connected to the coupling beam is supported such that it can resist shear and overturning forces in the absence of the coupling beam. Ffor Immediate Occupancy, coupling beams have the capacity in shear to develop the uplift capacity of the adjacent wall or to develop the flexural capacity of the coupling beam, whichever is less. This statement only applies to coupling beams with span-to-depth ratios exceeding 2-to-1.

Coupling beams with sufficient strength and stiffness can increase the lateral stiffness of the system significantly beyond the stiffnesses of the independent walls. When the walls deflect laterally, large moments and shears are induced in the coupling beams as they resist the imposed deformations. Coupling beams also link the coupled walls for overturning resistance (Figure A-18).

This checklist statement only applies to coupling beams with horizontal span–to–vertical depth ratios exceeding 2-to-1 based on element proportioning where beam behavior is more dominant. Elements with lower ratios (either short spans or deep elements) will act more like wall panels rather than beams. There is no checklist statement for these deeper wall panel elements.

Coupling beam reinforcement is often inadequate for the demands that can be induced by the movement of the coupled walls. Seismic forces may damage and degrade the beams so severely that the system degenerates into a pair of independent walls. This degeneration changes the distribution of overturning forces, which may result in potential stability problems for the independent walls. The boundary reinforcement also may be inadequate for flexural demands if the walls act independently.

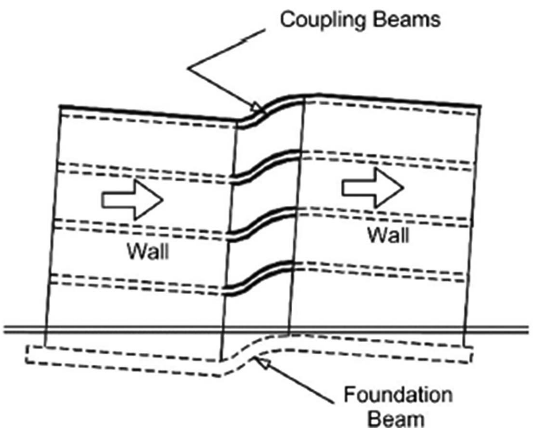


Figure A-18. Coupled walls.

If the beams are lightly reinforced, their degradation could result in falling debris that is a potential Life Safety hazard, especially at locations of egress.

Degradation of the strength and stiffness of coupling beams causes the two wall segments on either end of the coupling beam to act more as independent walls. Therefore, these walls must have support for vertical loads at each end of the wall to resist vertical loads caused by overturning.

To eliminate the need to rely on the coupling beam, the walls may be strengthened as required. The beam could be jacketed only as a means of controlling debris. If possible, the opening that defines the coupling beam could be infilled.

A.3.2.2.4 Overturning All shear walls have aspect ratios less than 4-to-1. Wall piers need not be considered.

Tall, slender shear walls may have limited overturning resistance. Displacements at the top of the building are greater than anticipated if overturning forces are not properly resisted.

Often sufficient resistance can be found in immediately adjacent bays if a load path is present to activate the adjacent column dead loads.

Lengthening or adding shear walls can reduce overturning demands; increasing the length of footings captures additional building dead load.

A.3.2.2.5 Confinement Reinforcing For shear walls with aspect ratios greater than 2-to-1, the boundary elements are confined with spirals or ties with spacing less than $8d_b$.

Fully effective shear walls require boundary elements to be properly confined with closely spaced ties (Figure A-17). Degradation of the concrete in the vicinity of the boundary elements can result in buckling of rebar in compression and failure of lap splices in tension. Nonductile failure of the boundary elements leads to reduced capacity to resist overturning forces.

Splices at boundary elements may be improved by welding bars together after exposing them.

A.3.2.2.6 Wall Reinforcing at Openings There is added trim reinforcement around all openings with a dimension greater than three times the thickness of the wall.

Conventional trim steel is adequate only for small openings (Figure A-19). Large openings cause significant shear and flexural stresses in the adjacent piers and spandrels. Inadequate reinforcing steel around these openings leads to strength deficiencies, nonductile performance, and degradation of the wall.

Shear walls with inadequate reinforcement at openings can be strengthened by infilling openings or by thickening the walls; for examples, see FEMA 547 (2006).

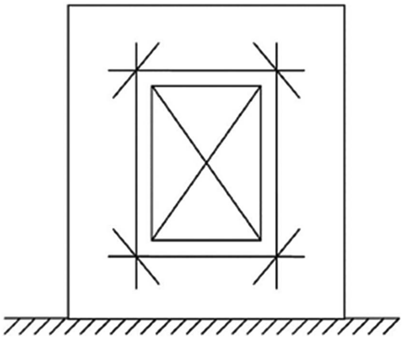


Figure A-19. Conventional trim steel.

A.3.2.2.7 Wall Thickness Thicknesses of bearing walls is not less than 1/25 the unsupported height or length, whichever is shorter, nor less than 4 in. (101 mm).

Slender bearing walls may have limited capacity for vertical loads and higher potential for damage because of out-of-plane forces and magnified moments.

A.3.2.3 Precast Concrete Shear Walls Precast concrete shear walls are constructed in segments that are usually interconnected by embedded steel elements. These connections usually possess little ductility but are important to the overall behavior of the wall assembly. Interconnection between panels increases the overturning capacity by transferring overturning demands to end panels. Panel connections at the diaphragm are often used to provide continuous diaphragm chords. Failure of these connections reduces the capacity of the system.

A.3.2.3.1 Shear Stress Check The shear stress in the precast panels, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$.

The shear stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building.

A.3.2.3.2 Reinforcing Steel. The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. In addition, for Immediate Occupancy, the spacing of reinforcing steel is equal to or less than 18 in. (457 mm).

If the walls do not have sufficient reinforcing steel, they have limited capacity in resisting seismic forces. The wall also behaves in a nonductile manner for inelastic forces.

In tilt-up construction, the reinforcement ratios are typically reversed because the principal direction of bending is vertical rather than horizontal.

A.3.2.3.3 Wall Openings The total combined width of openings and wall piers with aspect ratios greater than 2-to-1 along any perimeter wall line constitutes less than 75% of the total length of any perimeter wall for Collapse Prevention and 50% for Immediate Occupancy.

In tilt-up construction, typical wall panels are often of sufficient length that special detailing for collector elements, shear transfer, and overturning resistance is not provided. Perimeter walls that are substantially open, such as at loading docks, have limited wall length to resist seismic forces and may be subject to overturning or shear transfer problems that were not accounted for in the original design.

Walls are compliant if an adequate load path for shear transfer, collector forces, and overturning resistance can be demonstrated. Infilling openings or adding shear walls in the plane of the open bays can reduce demand on the connections and eliminate frame action.

A.3.2.3.4 Panel-to-Panel Connections Adjacent wall panels are interconnected to transfer overturning forces between panels by methods other than steel welded inserts.

Welded steel inserts can be brittle and may not be able to transfer the overturning forces between panels. Latent stresses may be present because of shrinkage and temperature effects. Brittle failure may include weld fracture, pullout of the embedded anchors, or spalling of the concrete.

Failure of these connections results in separation of the wall panels and a reduction in overturning resistance.

Appropriate retrofit solutions are outlined in FEMA 547 (2006).

Interpanel connections with inadequate capacity can be strengthened by adding steel plates across the joint or by providing a continuous wall by exposing the reinforcing steel in the adjacent units and providing ties between the panels and patching with concrete. Providing steel plates across the joint is typically the most cost-effective approach, although care must be taken to ensure adequate anchor bolt capacity by providing adequate edge distances; see also FEMA 547 (2006).

A.3.2.3.5 Wall Thickness Thicknesses of bearing walls are not less than 1/40 for Collapse Prevention or 1/25 for Immediate Occupancy of the unsupported height or length, whichever is shorter, nor less than 4 in. (101 mm).

Slender bearing walls may have limited capacity for vertical loads and higher potential for damage caused by out-of-plane forces and magnified moments.

A.3.2.4 Reinforced Masonry Shear Walls

A.3.2.4.1 Shear Stress Check The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 70 lb/in.^{2} (0.48 MPa).

The shear stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building. For partially grouted walls, the effective net section should be used in calculating the shear stress.

To meet the Performance Objectives of this standard, new walls can be provided or the existing walls can be strengthened as needed. New and strengthened walls must form a complete, balanced, and properly detailed seismic-force-resisting system for the building. Special care is needed to ensure that the connection of the new walls to the existing diaphragm is appropriate and of sufficient strength to deliver the actual seismic forces or force yielding in the wall. All shear walls must have sufficient shear and overturning resistance.

A.3.2.4.2 Reinforcing Steel The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 of the wall with the minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 in. (1,220 mm), and all vertical bars extend to the top of the walls.

If the walls do not have sufficient reinforcing steel, they have limited capacity in resisting seismic forces. The wall also behaves in a nonductile manner for inelastic forces.

Nondestructive methods should be used to locate reinforcement, and selective demolition should be used if necessary to determine the size and spacing of the reinforcing. If it cannot be verified that the wall is reinforced in accordance with the minimum requirements, then the wall should be assumed to be unreinforced and the procedures for unreinforced masonry (URM) should be followed.

A.3.2.4.3 Reinforcing at Wall Openings All wall openings that interrupt rebar have trim reinforcing on all sides.

Conventional trim steel is adequate only for small openings. Large openings cause significant shearing and flexural stresses in the adjacent piers and spandrels. Inadequate reinforcing steel around these openings leads to strength deficiencies, nonductile performance, and degradation of the wall.

The presence and location of reinforcing steel at openings may be established using nondestructive or destructive methods at selected locations to verify the size and location of the reinforcing, or using both methods. Reinforcing must be provided at all openings as required to meet the standard criteria. Steel plates may be bolted to the surface of the section as long as the bolts are sufficient to yield the steel plate.

A.3.2.4.4 Proportions The height-to-thickness ratio of the shear walls at each story is less than 30.

Slender bearing walls may have limited capacity for vertical loads and higher potential for damage caused by out-of-plane forces and magnified moments.

Walls with insufficient thickness could be strengthened either by increasing the thickness of the wall or by adding a well-detailed strong-back system. The thickened wall must be detailed in a manner that fully interconnects the wall over its full height. The strong-back system must be designed for strength; connected to the structure in a manner so that it (1) develops the full yield strength of the strong back and (2) connects to the diaphragm in a manner that distributes the load into the diaphragm; and has sufficient stiffness to ensure that the components can perform in a compatible and acceptable manner. The stiffness of the bracing should limit the out-of-plane deflections to acceptable levels such as $L/600$ to $L/900$.

A.3.2.5 Unreinforced Masonry Shear Walls

A.3.2.5.1 Shear Stress Check The shear stress in the unreinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 30 lb/in.^{2} (0.21 MPa) for clay units and 70 lb/in.^{2} (0.48 MPa) for concrete units. For infill frames, bays with openings greater than 25% of the wall area cannot be included in A_w of Equation (4-8).

The shear stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building. For concrete units, the effective net shear area should be used in calculating the shear stress.

For masonry infill walls in frames, the behavior of bays with openings is complex. Multiple compression struts form in these perforated infills and induce forces on the surrounding frame, and the contribution of the perforated masonry infills is not simply predicted. Openings in the seismic-force-resisting walls could be infilled as needed to meet the standard stress check. If supplemental strengthening is required, it should be designed using the Tier 3 systematic retrofit procedure in accordance with Chapter 6. Walls that do not meet the masonry layup requirements should not be considered as seismic-force-resisting elements and should be specially supported for out-of-plane forces.

A.3.2.5.2 Proportions The height-to-thickness ratio of the shear walls at each story is less than the following:

Top story of multistory building	*9*
First story of multistory building	*15*
All other conditions	*13*

Slender unreinforced masonry bearing walls with large height-to-thickness ratios have a potential for damage caused by out-of-plane forces that may result in falling hazards and potential collapse of the structure.

Refer to Section A.3.2.4.4 for commentary regarding potential strengthening measures.

A.3.2.5.3 Masonry Layup Filled collar joints of multiwythe masonry walls have negligible voids.

Where walls have poor collar joints, the inner and outer wythes act independently. The walls may be inadequate to resist out-of-plane forces because of a lack of composite action between the inner and outer wythes.

Mitigation to provide out-of-plane stability and anchorage of the wythes may be necessary to achieve the selected performance level.

Walls that do not meet the masonry layup requirements should not be considered as seismic-force-resisting elements and should be specially supported for out-of-plane forces.

A.3.2.6 Infill Walls in Frames

A.3.2.6.1 Infill Wall Connections Masonry is in full contact with the frame.

Performance of frame buildings with masonry infill walls is dependent on the interaction between the frame and infill panels. In-plane seismic force resistance is provided by a compression strut developing in the infill panel that extends diagonally between corners of the frame. If gaps exist between the frame and infill, this strut cannot be developed (Figure A-20). If the infill panels separate from the frame because of out-of-plane forces, the strength and stiffness of the system are determined by the properties of the bare frame, which may not be detailed to resist seismic forces. Severe damage or partial collapse caused by excessive drift and P-Δ effects may occur.

A positive connection is needed to anchor the infill panel for out-of-plane forces. In this case, a positive connection can consist of a fully grouted bed joint in full contact with the frame or complete encasement of the frame by the brick masonry. The mechanism for out-of-plane resistance of infill panels is discussed in Section A.3.2.6.2.

If the connection is nonexistent, mitigation with adequate connection to the frame is necessary to achieve the selected performance level.

It is impossible to simultaneously satisfy this section and Section A.3.1.2.1, which covers moment frames with infills not intended to be part of the seismic-force-resisting system.

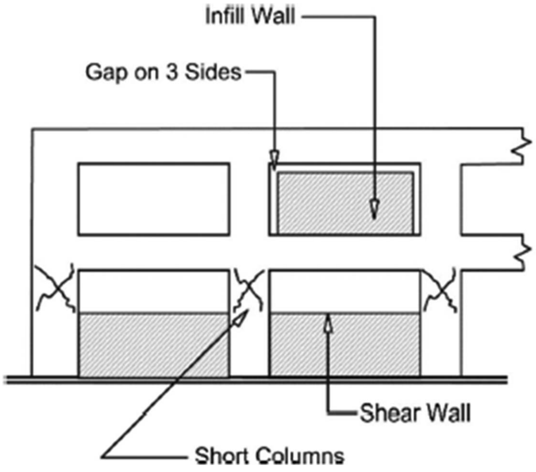


Figure A-20. Infill wall.

A.3.2.6.2 Proportions The height-to-thickness ratio of the unreinforced infill walls at each story is less than 9.0 for Collapse Prevention in levels of high seismicity, 13.0 for Immediate Occupancy in levels of moderate seismicity, and 8.0 for Immediate Occupancy in levels of high seismicity.

Slender masonry infill walls with large height-to-thickness ratios have a potential for damage caused by out-of-plane forces. Failure of these walls out of plane results in falling hazards and degradation of the strength and stiffness of the seismic-force-resisting system.

The out-of-plane stability of infill walls is dependent on many factors, including flexural strength of the wall and confinement provided by the surrounding frame. If the infill is unreinforced, the flexural strength is limited by the flexural tension capacity of the material. The surrounding frame provides confinement, induces infill thrust forces, and develops arching action against out-of-plane forces. The height-to-thickness limits in the evaluation statement are based on arching action models that exceed any plausible acceleration levels in various levels of seismicity.

Further investigation of noncompliant infill panels requires a Tier 3 systematic evaluation.

A.3.2.6.3 Cavity Walls The infill walls are not of cavity construction.

Where the infill walls are of cavity construction, the inner and outer wythes act independently because of a lack of composite action, increasing the potential for damage from out-of-plane forces. Failure of these walls out of plane results in falling hazards and degradation of the strength and stiffness of the seismic-force-resisting system.

A.3.2.6.4 Infill Walls The infill walls are continuous to the soffits of the frame beams and to the columns to either side.

Discontinuous infill walls occur where full bay windows or ventilation openings are provided between the top of the infill and the bottom soffit of the frame beams. The portion of the column above the infill is a short captive column that may attract large shear forces because of increased stiffness relative to other columns (Figure A-20). Partial infill walls also develop compression struts with horizontal components that are highly eccentric to the beam–column joints. If not adequately detailed, concrete columns may suffer a nonductile shear failure, which may result in partial collapse of the structure. Because steel columns are not subject to the same kind of brittle failure, this is not generally considered a concern in steel frame infill buildings.

A column that can develop the shear capacity to develop the flexural strength over the clear height above the infill has some ductility to prevent sudden catastrophic failure of the vertical support system.

Except where it can be shown that the column is adequate, the partial infill wall should be isolated from the boundary columns to avoid a “short column” effect. In sizing the gap between the wall and the columns, the anticipated story drift must be considered.

A.3.2.6.5 Infill Wall Eccentricity The centerline of the infill masonry wall is not offset from the centerline of the steel framing by more than 25% of the wall thickness.

An eccentricity between the infill wall and the centerline of the steel framing can induce forces in the steel framing for which the steel framing may not be adequate. Also, large eccentricities can inhibit masonry strut formation.

A.3.2.7 Walls in Wood-Frame Buildings

A.3.2.7.1 Shear Stress Check The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the following values:

Structural panel sheathing	*1,000 lb/ft (14.6 kN/m)*
Diagonal sheathing	*700 lb/ft (10.2 kN/m)*
Straight sheathing	*100 lb/ft (1.5 kN/m)*
All other conditions	*100 lb/ft (1.5 kN/m)*

The shear stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building. The transfer of shear and overturning to the foundation also should be evaluated. The structural panel sheathing Quick Check capacity assumes that the wall is constructed adequately and in fair condition. Capacities should be reduced to account for deterioration or overdriven fasteners.

Walls may be added or existing openings may be filled. Alternatively, the existing walls and connections can be strengthened. The walls should be distributed across the building in a balanced manner to reduce the shear stress for each wall. Replacing heavy materials such as tile roofing with lighter materials also reduces shear stress.

A.3.2.7.2 Stucco (Exterior Plaster) Shear Walls Multistory buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system.

Exterior stucco walls are often used (intentionally and unintentionally) for resisting seismic forces. Stucco is relatively stiff but brittle, and the shear capacity is limited. Building movements caused by differential settlement, temperature changes, and earthquake or wind forces can cause cracking in the stucco and loss of lateral strength. Seismic force resistance is unreliable because sometimes the stucco delaminates from the framing and the system is lost. Multistory buildings should not rely on stucco walls as the primary seismic-force-resisting system.

For strengthening or repair, the stucco should be removed, a wood structural panel shear wall should be added, and new stucco should be applied. The wood structural panel should be the manufacturer’s recommended thickness for the installation of stucco. The new stucco should be installed in accordance with building code requirements for waterproofing. Walls should be sufficiently anchored to the diaphragms and foundations.

A.3.2.7.3 Gypsum Wallboard or Plaster Shear Walls Interior plaster or gypsum wallboard is not used for shear walls on buildings more than one story high with the exception of the uppermost level of a multistory building.

Gypsum wallboard or gypsum plaster sheathing tends to be easily damaged by differential foundation movement or earthquake ground motions.

Although the capacity of these walls is low, most residential buildings have numerous walls constructed with plaster or gypsum wallboard. As a result, plaster and gypsum wallboard walls may provide adequate resistance to moderate earthquake shaking.

One problem that can occur is incompatibility with other seismic-force-resisting elements. For example, narrow plywood shear walls are more flexible than long stiff plaster walls; as a result, the plaster or gypsum walls take all the seismic demand until they fail, and then the plywood walls start to resist the seismic forces. In multistory buildings, plaster or gypsum wallboard walls should not be used for shear walls except in the top story.

Plaster and gypsum wallboard can be removed and replaced with structural panel shear wall as required, and the new shear walls can be covered with gypsum wallboard.

A.3.2.7.4 Narrow Wood Shear Walls Narrow wood shear walls with an aspect ratio greater than 2-to-1 for Collapse Prevention and Immediate Occupancy in very low seismicity or 1.5-to-1 for Immediate Occupancy in low, moderate, or high seismicity are not used to resist seismic forces.

Narrow shear walls are highly stressed and subject to severe deformations that reduce the capacity of the walls. Most of the damage occurs at the base and consists of sliding of the sill plate and deformation of hold-down anchors where present. As the deformation continues, the plywood pulls up on the sill plate, causing splitting. Splitting of the end studs at the bolted attachment of hold-down anchors is also common. The aspect ratio for wood walls is the story height to wall length.

Where narrow shear walls lack capacity, they should be replaced with shear walls with a height-to-width aspect ratio of 2:1 or less. These replacement walls must have sufficient strength, including being adequately connected to the diaphragm and sufficiently anchored to the foundation for shear and overturning forces.

A.3.2.7.5 Walls Connected through Floors Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor.

In platform construction, wall framing is discontinuous at floor levels. The concern is that this discontinuity might prevent shear and overturning forces from being transferred between shear walls in adjacent stories.

Mitigation with elements or connections needed to complete the load path is necessary to achieve the selected performance level.

A.3.2.7.6 Hillside Site For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-1 for Collapse Prevention and 1-to-2 for Immediate Occupancy.

Buildings on a sloping site experience significant torsion during an earthquake. Taller walls on the downhill slope are more flexible than the supports on the uphill slope. Therefore, significant displacement and racking of the shear walls on the downhill slope occur. If the walls are narrow, significant damage or collapse may occur.

A.3.2.7.7 Cripple Walls Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels.

Cripple walls are short stud walls that enclose a crawl space between the first floor and the ground. Often there are no other walls at this level, and these walls have no stiffening elements other than architectural finishes. If this sheathing fails, the building experiences significant damage and, in the extreme case, may fall off its foundation. To be effective, all exterior cripple walls below the first-floor level should have adequate shear strength, stiffness, and proper connection to the floor and foundation. Cripple walls that change height along their length, such as along sloping walls on hillside sites, do not have a uniform distribution of shear along the length of the wall because of the varying stiffness. These walls may be subject to additional damage on the uphill side because of concentration of shear demand.

Mitigation with shear elements needed to complete the load path is necessary to achieve the selected performance level.

Where bracing is inadequate, new wood structural panel sheathing can be added to the cripple wall studs. The top edge of the wood structural panel is nailed to the floor framing, and the bottom edge is nailed into the sill plate; for an example, see FEMA 547 (2006). The cripple wall should not change height along its length (the stepped top of foundation). If it does, the shorter portion of the cripple wall carries the majority of the shear and significant torsion occurs in the foundation. Added wood structural panel sheathing must have adequate strength and stiffness to reduce torsion to an acceptable level. Also, it should be verified that the sill plate is properly anchored to the foundation. If anchor bolts are lacking or insufficient, additional anchor bolts should be installed. Blocking or framing clips may be needed to connect the cripple wall bracing to the floor diaphragm or the sill plate.

A.3.2.7.8 Openings Walls with openings greater than 80% of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces.

Walls with large openings, such as garage doors, may have little or no resistance to shear and overturning forces. They must be specially detailed to resist these forces or braced to other parts of the structure with collectors, such as metal straps, developed into the adjacent construction. Special detailing and collectors are not part of conventional construction procedures. Lack of this bracing can lead to collapse of the wall.

Local shear transfer stresses can be reduced by distributing the forces from the diaphragm. Chords and/or collector members can be provided to collect and distribute shear from the diaphragm to the shear wall or bracing; for an example, see FEMA 547 (2006). Alternatively, the opening can be closed off by adding a new wall with wood structural panel sheathing.

A.3.2.7.9 Hold-Down Anchors All shear walls have hold-down anchors attached to the end studs constructed in accordance with acceptable construction practices.

Buildings without hold-down anchors may be subject to significant damage caused by uplift and racking of the shear walls. Properly constructed hold-downs must connect the floors together and activate the weight of the foundation. They must be tightly connected to the boundary element in a manner such that the deformation of the shear wall does not destroy the integrity of the hold-downs. Building drawings and manufacturers' recommendations are helpful in determining the adequacy of the hold-downs.

This condition is not considered a Life Safety concern and only needs to be examined for the Immediate Occupancy Performance Level.

If the walls are not bolted to the foundation or if the bolting is inadequate, bolts can be installed through the sill plates at regular intervals; for example, see FEMA 547 (2006). If the crawl space is not deep enough for vertical holes to be drilled through the sill plate, the installation of connection plates or angles may be a practical alternative; for example, see FEMA 547 (2006). Sheathing and additional nailing can be added where walls lack proper nailing or connections. Where the existing connections are inadequate, adding clips or straps delivers seismic forces to the walls and to the foundation sill plate.

A.3.2.8 Cold-Formed Steel Light-Frame Construction, Shear Wall Systems

A.3.2.8.1 Shear Stress Check The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the following values:

Wood structural panel sheathing	*1,000 lb/ft (14.6 kN/m)*
Steel sheet sheathing	*700 lb/ft (10.2 kN/m)*
All other conditions	*100 lb/ft (1.5 kN/m)*

The shear stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building. The transfer of shear and overturning to the foundation also should be evaluated. The wood structural panel sheathing and steel sheet sheathing Quick Check capacity assumes that the wall is constructed adequately and in fair condition. Capacities should be reduced to account for deterioration or overdriven or stripped fasteners in wood structural panel sheathing, or stripped fasteners in steel sheet sheathing. Typically, stripped screws in shear are only considered effective if the number of stripped screws does not exceed 25% of the total number of screws.

Walls may be added or existing openings may be filled. Alternatively, the existing walls and connections can be strengthened. The walls should be distributed across the building in a balanced manner to reduce the shear stress for each wall. Replacing heavy materials such as tile roofing with lighter materials also reduces shear stress.

A.3.2.8.2 Stucco (Exterior Plaster) Shear Walls Multistory buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system.

Exterior stucco walls are often used (intentionally and unintentionally) for resisting seismic forces. Stucco is relatively stiff but brittle, and the shear capacity is limited. Building movements caused by differential settlement, temperature changes, and earthquake or wind forces can cause cracking in the stucco and loss of lateral strength. Seismic force resistance is unreliable because sometimes the stucco delaminates from the framing and the system is lost. Multistory buildings should not rely on stucco walls as the primary seismic-force-resisting system.

For strengthening or repair, the stucco should be removed, a wood structural panel or steel sheet–sheathed shear wall should be added, and new stucco should be applied. Wood structural panel should be the manufacturer's recommended thickness for the installation of stucco. Steel sheet–sheathed shear walls require the addition of wood, gypsum, or similar panels over the steel sheet to facilitate installation of stucco. The new stucco should be installed in accordance with building code requirements for waterproofing. Walls should be sufficiently anchored to the diaphragms and foundations.

A.3.2.8.3 Gypsum Wallboard or Plaster Shear Walls Interior plaster or gypsum wallboard are not used as shear walls on buildings more than one story high with the exception of the uppermost level of a multistory building.

Gypsum wallboard or gypsum plaster sheathing tends to be easily damaged by differential foundation movement or earthquake ground motions.

Although the capacity of these walls is low, most residential buildings have numerous walls constructed with plaster or gypsum wallboard. As a result, plaster and gypsum wallboard walls may provide adequate resistance to moderate earthquake shaking.

One problem that can occur is incompatibility with other seismic-force-resisting elements. For example, narrow wood structural panel or steel sheet–sheathed shear walls are more flexible than long stiff plaster walls; as a result, the plaster or gypsum walls take all the seismic demand until they fail, and then the wood structural panel or steel sheet–sheathed shear walls start to resist the seismic forces. In multistory buildings, plaster or gypsum wallboard walls should not be used for shear walls except in the top story.

Plaster and gypsum wallboard can be removed and replaced with wood structural panel or steel sheet–sheathed shear walls as required, and the new shear walls can be covered with gypsum wallboard.

A.3.2.8.4 Narrow Cold-Formed Steel Wood Structural Panel or Steel Sheet–Sheathed Shear Walls Narrow wood structural panel or steel sheet–sheathed shear walls with an aspect ratio greater than 2-to-1 for Life Safety and Immediate Occupancy in very low seismicity or 1.5-to-1 for Immediate Occupancy in low, moderate, or high seismicity are not used to resist seismic forces.

Narrow shear walls are highly stressed and subject to severe deformations that reduce the capacity of the walls. Most of the damage occurs at the base and consists of sliding of the base track and deformation of hold-down anchors where present. As the deformation continues, the wood structural panel or steel sheet pulls up on the base track, causing deformation or yielding. The aspect ratio for shear walls is the story height to wall length.

Where narrow shear walls lack capacity, they should be replaced with shear walls with a height-to-width aspect ratio of 2-to-1 or less. These replacement walls must have sufficient strength, be connected to the diaphragm, and be anchored to the foundation to accommodate expected shear and overturning forces.

A.3.2.8.5 Walls Connected through Floors Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor.

In platform construction, wall framing is discontinuous at floor levels. The concern is that this discontinuity might prevent shear and overturning forces from being transferred between shear walls in adjacent stories.

Mitigation with elements or connections needed to complete the load path is necessary to achieve the selected performance level.

A.3.2.8.6 Hillside Site For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-1 for Life Safety and 1-to-2 for Immediate Occupancy.

Buildings on a sloping site experience significant torsion during an earthquake. Taller walls on the downhill slope are more flexible than the supports on the uphill slope. Therefore, significant displacement and racking of the shear walls on the downhill slope occur. If the walls are narrow, significant damage or collapse may occur.

A.3.2.8.7 Cripple Walls Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels or steel sheets.

Cripple walls are short stud walls that enclose a crawl space between the first floor and the ground. Often there are no other walls at this level, and these walls have no stiffening elements other than architectural finishes. If this sheathing fails, the building experiences significant damage and, in the extreme case, may fall off its foundation. To be effective, all exterior cripple walls below the first-floor level should have adequate shear strength, stiffness, and proper connection to the floor and foundation. Cripple walls that change height along their length, such as along sloping walls on hillside sites, do not have a uniform distribution of shear along the length of the wall because of the varying stiffness. These walls may be subject to additional damage on the uphill side because of concentration of shear demand.

Mitigation with shear elements needed to complete the load path is necessary to achieve the selected performance level.

Where bracing is inadequate, new wood structural panel or steel sheet sheathing can be added to the cripple wall studs. The top edge of the sheathing should be fastened to the floor framing and the bottom edge fastened into the base track. The cripple wall should not change height along its length (the stepped top of foundation). If it does, the shorter portion of the cripple wall carries the majority of the shear and significant torsion occurs in the foundation. Added wood structural panel or steel sheet sheathing must have adequate strength and stiffness to reduce torsion to an acceptable level. Also, it should be verified that the base track is properly anchored to the foundation. If anchor bolts are lacking or insufficient, additional anchor bolts should be installed. Blocking or framing clips may be needed to connect the cripple wall bracing to the floor diaphragm.

A.3.2.8.8 Openings Walls with openings greater than 80% of the length are braced with wood structural panel or steel sheet–sheathed shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces.

Walls with large openings, such as garage doors, may have little or no resistance to shear and overturning forces. They must be specially detailed to resist these forces or braced to other parts of the structure with collectors, such as metal straps, developed into the adjacent construction. Special detailing and collectors are not part of conventional construction procedures. Lack of this bracing can lead to collapse of the wall.

Local shear transfer stresses can be reduced by distributing the forces from the diaphragm. Chords and/or collector members can be provided to collect and distribute shear from the diaphragm to the shear wall or bracing. Alternatively, the opening can be closed off by adding a new wall with wood structural panel or steel sheet sheathing.

A.3.2.8.9 Hold-Down Anchors All shear walls have hold-down anchors attached to the end studs, constructed in accordance with acceptable construction practices.

Buildings without hold-down anchors may be subject to significant damage caused by uplift and racking of the shear walls. Properly constructed hold-downs must connect the floors together and activate the weight of the foundation. They must be tightly connected to the boundary element in a manner such that the deformation of the shear wall does not destroy the integrity of the hold-downs. Building drawings and manufacturers' recommendations are helpful in determining the adequacy of the hold-downs.

This condition is not considered a Life Safety concern and only needs to be examined for the Immediate Occupancy Performance Level.

If the walls are not bolted to the foundation or if the bolting is inadequate, bolts can be installed through the base tracks at regular intervals; if the crawl space is not deep enough for vertical holes to be drilled through the base track, the installation of connection plates or angles may be a practical alternative. Sheathing and additional fastening can be added where walls lack proper fastening. Where the existing connections are inadequate, adding clips or straps delivers seismic forces to the walls and to the foundation base track.

A.3.3 Braced Frames Braced frames develop their seismic force resistance through axial forces developed in the diagonal bracing members. The braces induce forces in the associated beams and columns, and all are subjected to stresses that are primarily axial. Where the braces are eccentric to beam–column joints, members are subjected to shear and flexure in addition to axial forces. A portal frame with knee braces near the frame joints is one example.

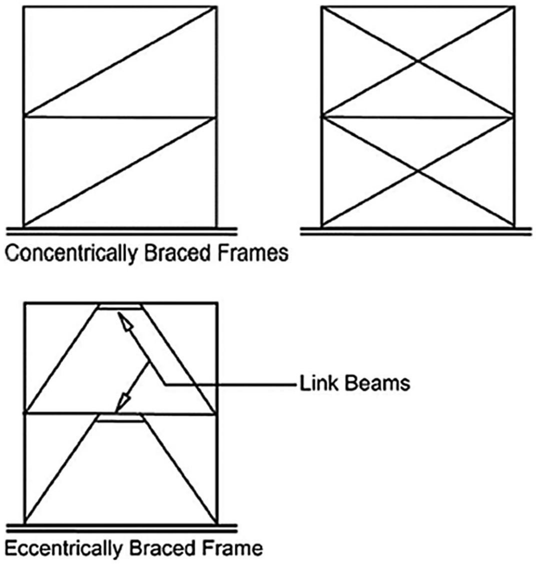


Figure A-21. Braced frames.

Braced frames are classified as either concentrically braced frames or eccentrically braced frames (Figure A-21). Concentrically braced frames (Section A.3.3.2) have braces that frame into beam–column joints or concentric connections with other braces. Minor connection eccentricities may be present and are accounted for in the design. Eccentrically braced frames (Section A.3.3.3) have braces that are purposely located away from joints and connections that are intended to induce shear and flexure demands on the members. The eccentricity is intended to force a concentration of inelastic activity at a predetermined location that will control the behavior of the system. Modern eccentrically braced frames are designed with strict controls on member proportions and special out-of-plane bracing at the connections to ensure that the frame behaves as intended.

If the strength of the braced frames is inadequate, more braced bays or shear wall panels can be added. The resulting seismic-force-resisting system must form a well-balanced system of braced frames that do not fail at their joints, are properly connected to the floor diaphragms, and whose failure mode is yielding of braces rather than overturning.

Diagonals with inadequate stiffness should be strengthened using supplemental steel plates or replaced with a larger and/or different type of section. Global stiffness can be increased by the addition of braced bays or shear wall panels.

A.3.3.1 General

A.3.3.1.1 Redundancy The number of lines of braced frames in each principal direction is greater than or equal to 2 for Collapse Prevention and Immediate Occupancy. The number of braced bays in each line is greater than 2 for Collapse Prevention and 3 for Immediate Occupancy.

Refer to Section A.3.1.1.1 for commentary related to redundancy.

A.3.3.1.2 Brace Axial Stress Check The axial stress in the diagonals, calculated using the Quick Check procedure of Section 4.4.3.4, is less than $0.50F_y$.

The axial stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building.

For Building Type S-4 (Dual System), the backup moment frame is neglected when determining the axial stresses on the braced frame diagonals.

A.3.3.1.3 Column Splices All column splice details located in braced frames develop 50% of the tensile strength of the column for Collapse Prevention and 100% of the tensile strength of the column for Immediate Occupancy.

Columns in braced frames may be subject to large tensile forces. A connection that is unable to resist this tension may limit the ability of the frame to resist seismic forces. Columns may uplift and slide off bearing supports, resulting in unexpected damage to the frame elements.

Column splices can be strengthened by adding plates and welds to ensure that they are strong enough to develop the connected components. Demands on the existing elements can be reduced by adding braced bays or shear wall panels.

A.3.3.1.4 Slenderness of Diagonals All diagonal elements required to carry compression have Kl/r ratios less than 200.

Code design requirements allow compression diagonal braces to have *Kl/r* ratios of up to 200. Research has shown that frames with slender braces designed for compression strength behave well because of the overstrength inherent in their tension capacity. The research also has shown that the postbuckling cyclic fracture life of bracing members generally increases with an increase in slenderness ratio. An upper limit is provided to preclude dynamic effects associated with extremely slender braces. For more discussion, see AISC 341 commentary (2022a).

A.3.3.1.5 Connection Strength All the brace connections develop the buckling capacity of the diagonals for moderate seismicity and the yield capacity of the diagonals for high seismicity.

Because connection failures are usually nonductile, it is more desirable to have inelastic behavior in the members.

Braced frame connections can be strengthened by adding plates and welds to ensure that they are strong enough to develop the connected components. Connection eccentricities that reduce component capacities can be eliminated, or the components can be strengthened to the required level by the addition of properly placed plates. Demands on the existing elements can be reduced by adding braced bays or shear wall panels.

A.3.3.1.6 Out-of-Plane Bracing Braced frame connections attached to beam bottom flanges located away from beam–column joints are braced out of plane at the bottom flange of the beams.

Brace connections at beam bottom flanges that do not have proper bracing may have limited ability to resist seismic forces. Out-of-plane buckling may occur before the strength of the brace is developed. Connections to beam top flanges are braced by the diaphragm, so V-bracing need not be considered.

This statement is intended to target chevron-type bracing, where braces intersect the beam from below at a location well away from a column. Here, only the beam can provide out-of-plane stability for the connection. At beam–column joints, the continuity of the column provides stability for the connection.

To demonstrate compliance, the beam is checked for the strength required to provide out-of-plane stability using the 2% rule.

A.3.3.1.7 Compact Members For moderate seismicity, all brace elements meet section requirements in accordance with AISC 360, Table B4.1a. For Collapse Prevention in high seismicity, all brace elements meet section requirements in accordance with AISC 341, Table D1.1a, for "moderately ductile" members. For Immediate Occupancy in high seismicity, all column and brace elements meet section requirements in accordance with AISC 341, Table D1.1, for "highly ductile" members, and braced frame beams meet the AISC 341, Table D1.1, requirements for "moderately ductile" members.

Noncompact brace elements may experience premature local buckling before development of their full capacities. Braces are assessed per the section requirements in accordance with AISC 341 (2022a) or AISC 360 (2022b), depending on the Level of Seismicity and Performance Level. Additionally, column and beam compactness is desirable for Immediate Occupancy performance. The width-to-thickness ratios of compression elements have been set to minimize the detrimental effects of localized buckling and subsequent fracture during repeated inelastic cycles.

The adequacy of the frame elements can be demonstrated using Tier 2 with reduced *m*-factors in consideration of reduced capacities for noncompact sections.

Noncompact members can be eliminated by adding appropriate steel plates. Stiffening elements (e.g., braced frames, shear walls, or additional moment frames) can be added throughout the building to reduce the expected frame demands.

A.3.3.1.8 Net Area The brace effective net area is not less than the brace gross area for hollow structural section (HSS) tube and pipes sections.

The concern is premature net section fracture of the brace at the connection. ASTM A53 (2022) or ASTM A500 (2021) braces (e.g., pipe braces or square, rectangular, or round hollow structural section braces), where the overslot of the brace required for erection may result in a reduced section. If this section is left unreinforced, net section fracture is the governing limit state and brace ductility may be significantly reduced.

Reinforcement may be provided in the form of steel plates welded to the tube, increasing the effective area at the reduced brace section.

A.3.3.2 Concentrically Braced Frames Common types of concentrically braced frames are shown in Figure A-22.

Braces can consist of light tension-only rod bracing, double angles, pipes, tubes, or heavy wide-flange sections.

Concrete braced frames are rare and are not permitted in some jurisdictions because it is difficult to detail the joints with the kind of reinforcing that is required for ductile behavior.

A.3.3.2.1 K-Bracing The bracing system does not include K-braced bays.

In K-brace configurations, diagonal braces intersect the column between floor levels (Figure A-22). Where the compression brace buckles, the column is loaded with the horizontal component of the adjacent tension brace. This loading induces large midheight demands that can jeopardize the stability of the column and vertical support of the building.

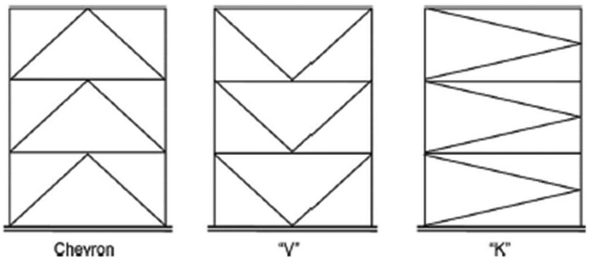


Figure A-22. Bracing types.

In most cases, columns have not been designed to resist this force. The risk to the vertical support system makes this an undesirable bracing configuration.

Horizontal girts can be added as needed to support the tension brace when the compression brace buckles, or the bracing can be revised to another system throughout the building. The column components can be strengthened with cover plates to provide them with the capacity to fully develop the unbalanced forces created by tension brace yielding.

A.3.3.2.2 Tension-Only Braces Tension-only braces do not comprise more than 70% of the total seismic-force-resisting capacity in structures more than two stories high [except in light-frame cold-formed steel structures using strap-braced walls (CFS2)].

Tension-only brace systems may allow the brace to deform with large velocities during cyclic response after tension yielding cycles have occurred. Limited energy dissipation and premature fracture can significantly reduce the strength, increase the building displacements, and jeopardize the performance of the framing system.

Tension-only diagonals with inadequate strength can be strengthened using supplemental steel plates or replaced with a larger and/or different type of section. Global strength can be increased by the addition of braced bays or shear wall panels.

A.3.3.2.3 Chevron Bracing Beams in chevron, or V-braced, bays are capable of resisting the vertical load resulting from the simultaneous yielding and buckling of the brace pairs.

In chevron- and V-brace configurations, diagonal braces intersect the beam between columns (Figure A-22). When the compression brace buckles, the beam is loaded with the vertical component of the adjacent tension brace. This configuration induces large midspan demands on the beam, resulting in structural damage to the beam.

Columns can be added as needed to support the tension brace when the compression brace buckles, or the bracing can be revised to another system throughout the building. The beam components can be strengthened with cover plates to provide them with the capacity to fully develop the unbalanced forces created by tension brace yielding.

A.3.3.2.4 Concentrically Braced Frame Joints All the diagonal braces frame into the beam–column joints concentrically.

Frames that have been designed as concentrically braced frames may have local eccentricities within the joint. A local eccentricity is where the lines of action of the bracing members do not intersect the centerline of the connecting members. These eccentricities induce additional flexural and shear stresses in the members that may not have been accounted for in the design. Excessive eccentricity can cause premature yielding of the connecting members or failures in the connections, thereby reducing the strength of the frames.

A.3.3.2.5 Narrow Strap-Braced Walls Narrow strap-braced walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces.

Cold-formed steel strap-braced walls with aspect ratios greater than 2-to-1 can generate considerable flexural stresses in the chord studs, which may not have been considered in the design. These flexural stresses can result in premature failure of the chord studs.

A.3.3.2.6 Walls Connected Through Floors Strap-braced walls have an interconnection between stories to transfer overturning and shear forces through the floor.

In platform construction, wall framing is discontinuous at floor levels. The concern is that this discontinuity might prevent shear and overturning forces from being transferred between cold-formed steel strap-braced walls in adjacent stories.

Mitigation with elements or connections needed to complete the load path is necessary to achieve the selected performance level.

A.3.3.2.7 Hillside Site For structures that are taller on at least one side by more than one-half story because of a sloping site, all strap-braced walls on the downhill slope have an aspect ratio less than 1-to-1 for Life Safety and 1-to-2 for Immediate Occupancy.

Buildings on a sloping site experience significant torsion during an earthquake. Taller walls on the downhill slope are more flexible than the supports on the uphill slope. Therefore, significant displacement and racking of the cold-formed steel strap-braced walls on the downhill slope occur. If the walls are narrow, significant damage or collapse may occur.

A.3.3.2.8 Hold-Down Anchors All strap-braced walls have hold-down anchors attached to the end studs, constructed in accordance with acceptable construction practices.

Buildings without hold-down anchors may be subject to significant damage caused by uplift and racking of the cold-formed steel strap-braced walls. Properly constructed hold-downs must connect the floors together and activate the weight of the foundation. They must be tightly connected to the boundary element in a manner such that the deformation of the cold-formed steel strap-braced wall does not destroy the integrity of the hold-downs. Building drawings and manufacturers' recommendations are helpful in determining the adequacy of the hold-downs.

This condition is not considered a Life Safety concern and only needs to be examined for the Immediate Occupancy Performance Level.

If the walls are not bolted to the foundation or if the bolting is inadequate, bolts can be installed through the base tracks at regular intervals; if the crawl space is not deep enough for vertical holes to be drilled through the base track, the installation of connection plates or angles may be a practical alternative. Sheathing and additional fastening can be added where walls lack proper fastening. Where the existing connections are inadequate, adding clips or straps delivers seismic forces to the walls and to the foundation base track.

A.3.3.2.9 Strap-Braced Walls—Chord Stud Axial Check The axial force caused by overturning plus the gravity load on the end stud is less than the nominal strength of the end stud calculated in accordance with AISI S100.

In strap-braced walls, the end stud that the brace is attached to is subjected to significant axial force demands. Cold-formed steel light-frame studs can fail in compression owing to either global or local buckling of the section, compromising the performance of the system. Therefore the axial force in the end stud caused by overturning of the wall system, which may be calculated using the braced frame overturning quick check equation or from first-principals, plus the gravity load force in the end stud should be checked against the nominal capacity of the stud, expressed in terms of strength to correlate to validate the quick check.

A.3.3.2.10 Strap-Brace Detailing Strap braces shall be tight to the stud and attached to the intermediate studs per the requirements of AISI S400.

In strap-braced walls, the performance of the system depends on the strap being tight to the framing. AISI S400 (2020c) has requirements for tightness of the strap and how the strap should be attached to the intermediate studs to provide that tightness.

A.3.3.3 Eccentrically Braced Frames Eccentrically braced frames have braces that are purposely located away from joints and connections that are intended to induce shear and flexure demands on the members. The eccentricity is intended to force a concentration of inelastic activity at a predetermined location that controls the behavior of the system. Modern eccentrically braced frames are designed with strict controls on member proportions and special out-of-plane bracing at the connections to ensure that the frame behaves as intended.

The eccentrically braced frame is recognizable by a diagonal with one end significantly offset from the joints (Figure A-23). As with any braced frame, the function of the diagonal is to provide stiffness and transmit seismic forces from the upper to the lower level. The unique feature of eccentrically braced frames is an offset zone in the beam, called the "link." The link is specially detailed for controlled yielding. This detailing is subject to very specific requirements, so an ordinary braced frame that happens to have an offset zone that looks like a link may not necessarily behave like an eccentrically braced frame.

An eccentrically braced frame has the following essential features:

- There is a link beam at one end of each brace;
- The length of the link beam is limited to control shear deformations and rotations because of flexural yielding at the ends of the link;
- The brace and the connections are designed to develop forces consistent with the strength of the link;
- Where one end of a link beam is connected to a column, the connection is a full moment connection; and
- Lateral bracing is provided to prevent out-of-plane beam displacements that would compromise the intended action.

In most cases where eccentrically braced frames are used, the frames compose the entire seismic-force-resisting system. In some tall buildings, eccentrically braced frames have been added as stiffening elements to help control drift in steel moment frames.

There are no evaluation statements for eccentrically braced frames because their history is so short, but the engineer is alerted to their possible presence in a building. For guidance in dealing with eccentrically braced frames, the evaluating engineer is referred to AISC 341 and AISC 342 (2022c). Some of the engineers familiar with current research designed eccentrically braced frames before the initial AISC provisions were finalized in the 1990s. These frames may not satisfy all of the detailing requirements present in the current code. Any frame that was clearly designed to function as a proper eccentrically braced frame should be recognized and evaluated with due regard for any possible shortcomings that affect the intended behavior. Acceptance criteria for using the Tier 2 and Tier 3 procedures for eccentrically braced frames are provided in Chapter 9.

A.4 PROCEDURES FOR DIAPHRAGMS

This section provides guidelines for using the Tier 1 checklists and the Tier 2 deficiency-based evaluation and retrofit procedures that apply to diaphragms: general, wood, metal deck, concrete, precast concrete, horizontal bracing, and other diaphragms.

Diaphragms are horizontal elements that distribute seismic forces to the vertical elements of the seismic-force-resisting system. They also provide lateral support for walls and parapets. Diaphragm forces are derived from the self-weight of the diaphragm and the weight of the elements and components that depend on the diaphragm for lateral support. Any roof, floor, or ceiling can participate in the distribution of seismic forces to vertical elements up to the limit of its strength. The degree to which it participates depends on relative stiffness and on connections. To function as diaphragms, horizontal elements must be interconnected to transfer shear, with connections that have some degree of stiffness. An array of loose elements, such as ceiling tiles or metal deck panels attached to beams with wind clips, does not qualify.

A.4.1 General It is customary to analyze diaphragms using a beam analogy. The floor, which is analogous to the web of a wide-flange beam, is assumed to carry the shear. The edge of the floor, which could be a spandrel or wall, is analogous to the flange and is assumed to carry the flexural stress. A free-body diagram of these elements is shown in Figure A-24. The diaphragm chord can consist of a line of edge beams that are connected to the floor or reinforcing in the edge of a slab or in a spandrel. Examples of chords are shown in Figure A-25.

Two essential requirements for the chord are continuity and connection with the slab. Almost any building with an edge beam has a potential diaphragm chord. Even if designed for vertical

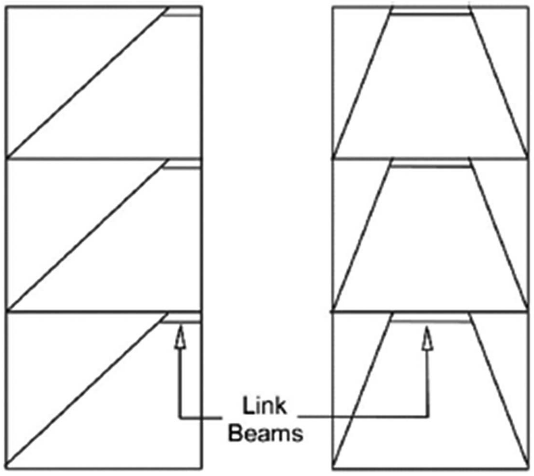


Figure A-23. Eccentrically braced frames.

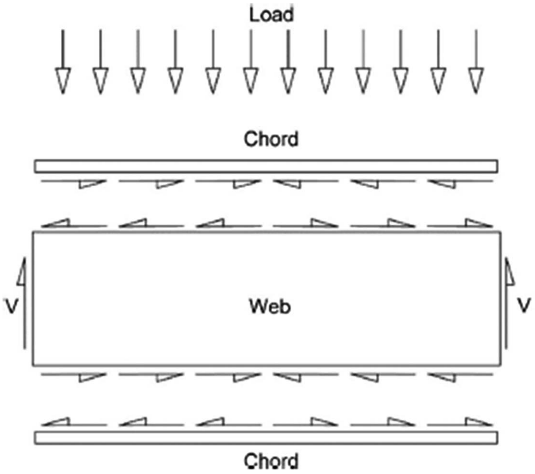


Figure A-24. Diaphragm as a beam.

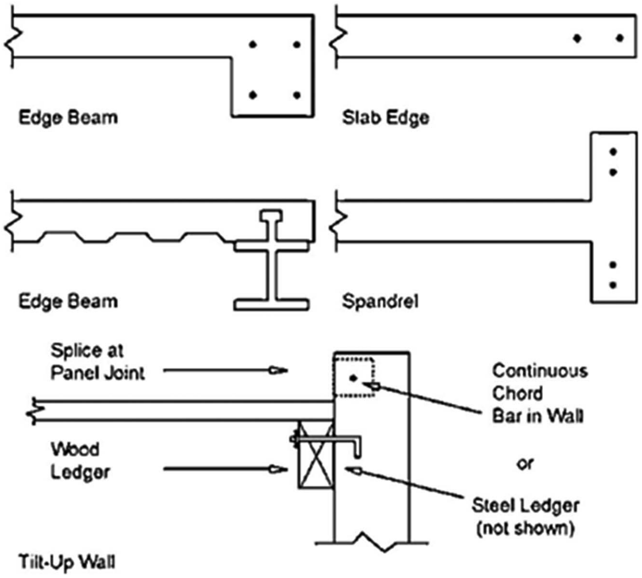


Figure A-25. Chord sections.

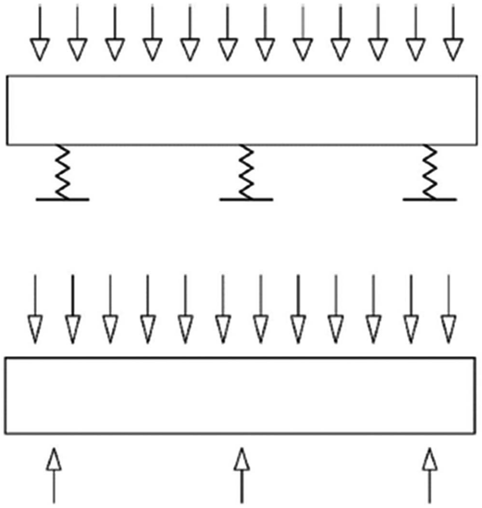


Figure A-26. Rigid and flexible diaphragm.

loads only, the beam end connections probably have some capacity to develop horizontal forces through the column.

The force in the chord is customarily determined by dividing the beam moment in the diaphragm by the depth of the diaphragm. This step yields an upper bound on the chord force because it assumes elastic beam behavior in the diaphragm and neglects bending resistance provided by any other components of the diaphragm. A lack of diaphragm damage in post-earthquake observations provides some evidence that certain diaphragms may not require specific chords as determined by the beam analogy. For the purpose of this standard, the absence of chords is regarded as a deficiency that warrants further evaluation. Consideration may be given to the available evidence regarding the suitability of the beam analogy and the need for defined chords in the building being evaluated.

Consistent with the beam analogy, a stair or skylight opening may weaken the diaphragm just as a web opening for a pipe may weaken a beam. An opening at the edge of a floor may weaken the diaphragm just as a notch in a flange weakens a beam.

An important characteristic of diaphragms is flexibility, or its opposite, rigidity. In seismic design, rigidity means relative rigidity. Of importance is the in-plane rigidity of the diaphragm relative to the walls or frame elements that transmit the seismic forces to the ground (Figure A-26). A concrete floor is relatively rigid compared with steel moment frames, whereas a metal deck roof is relatively flexible compared with concrete or masonry walls. Wood diaphragms are generally treated as flexible, but consideration must be given to rigidity of the vertical elements. Wood diaphragms may not be flexible compared with wood shear wall panels in a given building.

Another consideration is continuity over intermediate supports. In a three-bay building, for example, the diaphragm has three spans and four supports. If the diaphragm is relatively rigid, the chords should be continuous over the supports like flanges of a continuous beam over intermediate supports. If the diaphragm is flexible, it may be designed as a simple beam spanning between walls without consideration of continuity of the chords. In the latter case, the design professional should remember that the diaphragm is really continuous and that this continuity is simply being neglected.

Figure A-27 shows a diaphragm of two spans that may or may not be continuous over the intermediate support. If chord continuity is developed at the points marked X, these points are the locations of maximum chord force. If chord continuity is not provided at X, the spans act as two simple beams. The maximum chord force occurs at the middle of each span, at the points marked Y. The end rotations of the two spans may cause local damage at points X.

Finally, there must be an adequate mechanism for the transfer of diaphragm shear forces to the vertical elements. This topic is addressed in detail in Section A.5. An important element related to diaphragm force transfer is the collector, or drag strut. In Figure A-27, a member is added to collect the diaphragm shear and drag it into the short intermediate shear wall. The presence of a collector averts a concentration of stress in the diaphragm at the short shear wall. Collectors must be continuous across any interrupting elements such as perpendicular beams and must be adequately connected to the shear wall to deliver forces into the wall.

In buildings of more than one story, the design professional must consider the effect of flexible diaphragms on walls perpendicular to the direction of seismic force under consideration.

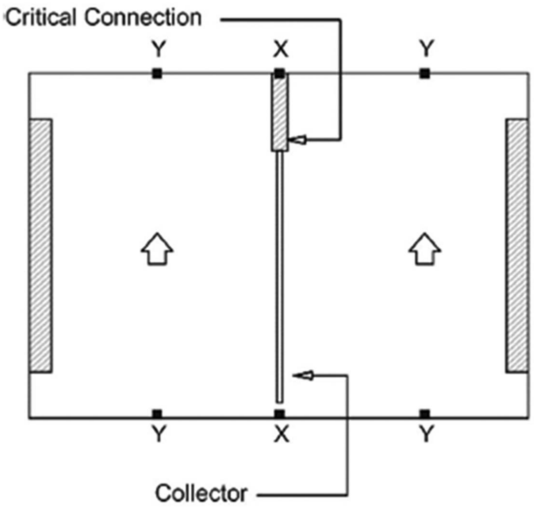


Figure A-27. Collector.

A.4.1.1 Diaphragm Continuity *Floor and roof diaphragms do not have expansion joints or vertical offsets, such as split levels, sawtooth, or clerestory configurations.*

Split-level floors and roofs, or diaphragms interrupted by expansion joints, create discontinuities in the diaphragm. This condition is common in ramped parking structures and in buildings with roof geometry consisting of sawtoothed layouts, clerestories, or other vertical offsets that reduce the continuity in the horizontal plane of the diaphragm. It is a problem unless special details are used or seismic-force-resisting elements are provided at the vertical offset of the diaphragm or on both sides of the expansion joint or discontinuity. Such a discontinuity may cause the diaphragm to function as a cantilever element or three-sided diaphragm. If the diaphragm is not supported on at least three sides by seismic-force-resisting elements, torsional forces in the diaphragm may cause it to become unstable. In both the cantilever and three-sided cases, increased lateral deflection in the discontinuous diaphragm may cause increased damage to, or collapse of, the supporting elements.

If the load path is incomplete, mitigation with elements or connections required to complete the load path is necessary to achieve the selected performance level.

The diaphragm discontinuity could be eliminated by adding new vertical elements at the diaphragm offset or the expansion joint; see FEMA 547 (2006). In some cases, special details may be used to transfer shear across an expansion joint—while still allowing the expansion joint to function—thus eliminating a diaphragm discontinuity.

A.4.1.2 Crossties *There are continuous crossties between diaphragm chords to distribute the out-of-plane wall anchorage forces into the diaphragm. Where each out-of-plane connection does not have a continuous crosstie across the entire diaphragm, these connections are developed into subdiaphragms between crossties with a maximum length-to-width ratio of 3-to-1.*

Continuous crossties between diaphragm chords are needed to develop out-of-plane wall forces into the diaphragm (Figure A-28). The crossties should have a positive and direct connection to the walls to keep the walls from separating from the building. The connection of the crosstie to the wall, and connections within the crosstie, must be detailed so that cross-grain bending or cross-grain tension does not occur in any wood member (see Section A.5.1.2).

Subdiaphragms with a maximum aspect ratio of 3:1 may be used between continuous crossties to reduce the number and length of additional crossties. See Figure A-28 for additional information on subdiaphragms.

New crossties and wall connections can be added to resist the required out-of-plane wall forces and distribute these forces through the diaphragm. New strap plates and/or rod connections can be used to connect existing framing members together so that they function as a crosstie in the diaphragm.

A.4.1.3 Roof Chord Continuity *All chord elements are continuous, regardless of changes in roof elevation.*

Diaphragms with discontinuous chords are more flexible and experience more damage around the perimeter than properly detailed diaphragms. Vertical offsets or elevation changes in a diaphragm often cause a chord discontinuity (Figure A-29). To provide continuity, the following elements are required: a continuous chord element; seismic force resistance in plane X to connect the offset portions of the diaphragm; seismic force resistance in plane Y to develop the sloping diaphragm into the chord; and vertical supports (posts) to resist overturning forces generated by plane X.

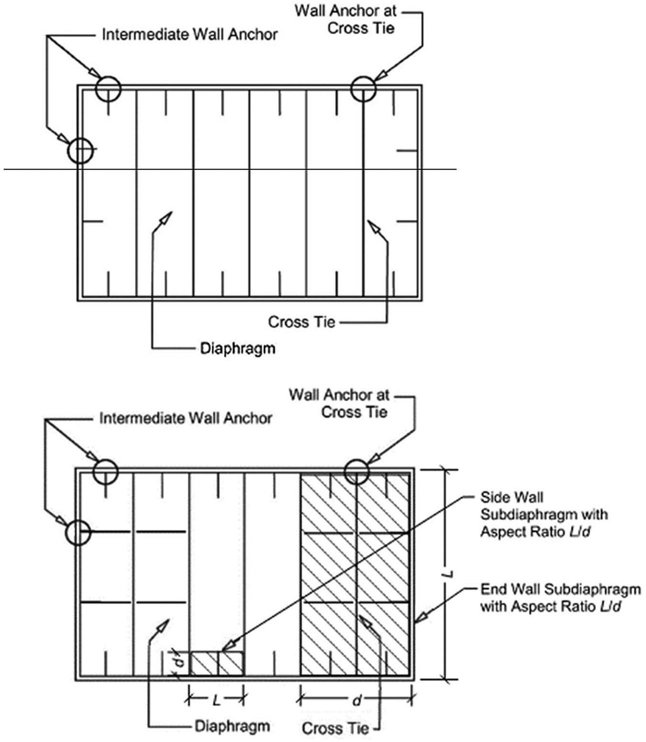


Figure A-28. Crossties.

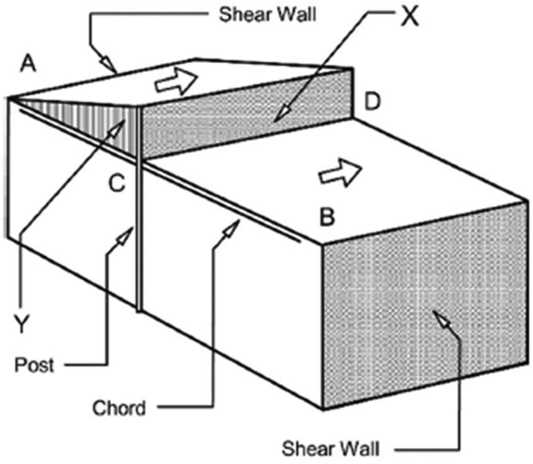


Figure A-29. Roof chord continuity.

If the load path is incomplete, mitigation with elements or connections required to complete the load path is necessary to achieve the selected performance level.

If members such as edge joists, blocking, or wall top plates have the capacity to function as chords but lack connection, adding nailed or bolted continuity splices provides a continuous diaphragm chord. New continuous steel or wood chord members can be added to the existing diaphragm where existing members lack sufficient capacity or no chord exists. New chord members can be placed at either the underside or topside of the diaphragm. In some cases, new vertical elements can be added to reduce the diaphragm span and stresses on any existing chord members. Refer to FEMA 547 (2006) and ATC-7 (1981).

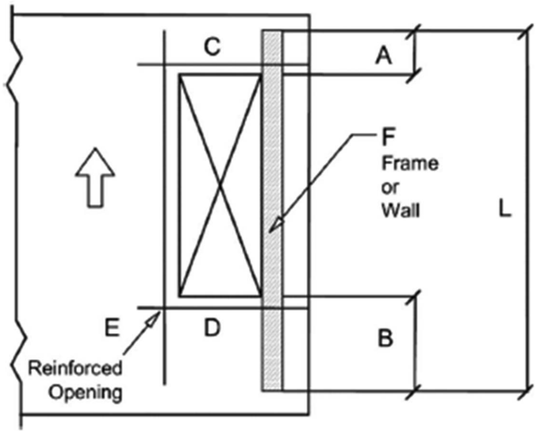


Figure A-30. Opening at exterior wall.

New chord connections should not be detailed such that they are the weakest component in the chord.

A.4.1.4 ***Openings at Shear Walls*** *Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length for Collapse Prevention and 15% of the wall length for Immediate Occupancy.*

Large openings at shear walls significantly limit the ability of the diaphragm to transfer seismic forces to the wall (Figure A-30). This limitation can have a compounding effect if the opening is near one end of the wall and divides the diaphragm into small segments with limited stiffness that are ineffective in transferring shear to the wall. This opening might have the net effect of a much larger opening. Large openings also may limit the ability of the diaphragm to provide out-of-plane support for the wall.

The presence of drag struts developed into the diaphragm beyond the wall helps mitigate this effect.

New diaphragm ties or chords can be added around the perimeter of existing openings to distribute tension and compression forces along the diaphragm. The existing sheathing should be nailed to the new diaphragm ties or chords. In some cases, it may also be necessary to (1) increase the shear capacity of the diaphragm adjacent to the opening by overlaying the existing diaphragm with a wood structural panel or (2) decrease the demand on the diaphragm by adding new vertical elements near the opening.

A.4.1.5 ***Openings at Frames*** *Diaphragm openings immediately adjacent to the moment frames or braced frames extend less than 25% of the frame length for Collapse Prevention and 15% of the frame length for Immediate Occupancy.*

Large openings at moment frames or braced frames significantly limit the ability of the diaphragm to transfer seismic forces to the frame. This limitation can have a compounding effect if the opening is near one end of the frame and divides the diaphragm into small segments with limited stiffness that are ineffective in transferring shear to the frame. This opening might have the net effect of a much larger opening.

The presence of drag struts developed into the diaphragm beyond the frame helps mitigate this effect.

Refer to Section A.4.1.4 for additional retrofit guidelines.

A.4.1.6 ***Openings at Exterior Masonry Shear Walls*** *Diaphragm openings immediately adjacent to exterior masonry walls are not greater than 8 ft (2.4 m) long for Collapse Prevention and 4 ft (1.2 m) long for Immediate Occupancy.*

Large openings at exterior masonry walls limit the ability of the diaphragm to provide out-of-plane support for the wall.

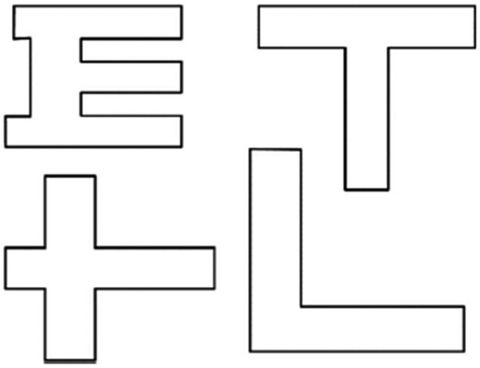


Figure A-31. Plan irregularities.

The presence of drag struts developed into the diaphragm beyond the wall helps mitigate this effect.

Refer to Section A.4.1.4 for additional retrofit guidelines.

A.4.1.7 ***Plan Irregularities*** *There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.*

Diaphragms with plan irregularities such as extending wings, plan insets, or E-, T-, X-, L-, or C-shaped configurations have reentrant corners where large tensile and compressive forces can develop (Figure A-31). Chords and collectors in the diaphragm may not have sufficient strength at these reentrant corners to resist these tensile forces. Local damage may occur (Figure A-32). Chord reinforcing is typically required to be developed at the reentrant corner. In some cases, the chord may be connected directly to a seismic-force-resisting element rather than developed into the diaphragm.

New chords with sufficient strength to resist the required force can be added at the reentrant corner. If a vertical seismic-force-resisting element exists at the reentrant corner, a new collector component should be installed in the diaphragm to reduce tensile and compressive forces at the reentrant corner. The same basic materials used in the diaphragm should be used for the chord.

A.4.1.8 ***Diaphragm Reinforcement at Openings*** *There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.*

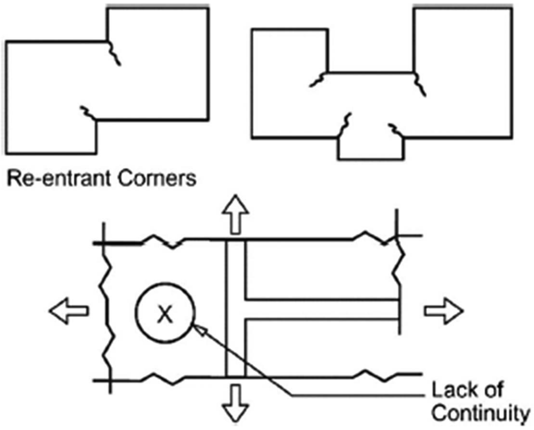


Figure A-32. Reentrant corners.

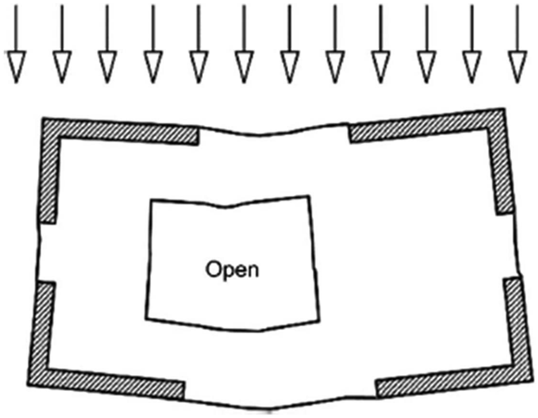


Figure A-33. Diagraphm opening.

Openings in diaphragms increase shear stresses and induce secondary moments in the diaphragm segments adjacent to the opening. Tension and compression forces are generated along the edges of these segments by the secondary moments and must be resisted by chord elements in the subdiaphragms around the openings.

Openings that are small relative to the diaphragm dimensions may have only a negligible impact. Openings that are large relative to the diaphragm dimensions can substantially reduce the stiffness of the diaphragm and induce large forces around the openings (Figure A-33).

Refer to Section A.4.1.4 for additional retrofit guidelines.

A.4.2 Wood Diaphragms

A.4.2.1 Straight Sheathing *All straight-sheathed diaphragms have horizontal spans less than 24 ft (7.3 m) and aspect ratios less than 2-to-1 for Collapse Prevention and have horizontal spans less than 12 ft (3.6 m) and aspect rations less than 1-to-1 for Immediate Occupancy in the direction being considered.*

Straight-sheathed diaphragms are flexible and weak relative to other types of wood diaphragms. Shear capacity is provided by a force couple between nails in the individual boards of the diaphragm and the supporting framing. Because of the limited strength and stiffness of these diaphragms, they are most suitable in applications with limited demand, such as in levels of low seismicity.

In levels of moderate and high seismicity, the span and aspect ratio of straight-sheathed diaphragms are limited to minimize shear demands. The aspect ratio (span/depth) must be calculated for the direction being considered.

Compliance can be achieved if the diaphragm has adequate capacity for the demands in the building being evaluated and vertical load-carrying elements can be shown to have adequate capacity at maximum diaphragm deflections.

Where the diaphragm does not have at least two nails through each board into each of the supporting members and the lateral drift and/or shear demands on the diaphragm are not excessive, the shear capacity and stiffness of the diaphragm can be increased by adding nails at the sheathing boards. This method of upgrade is most often suitable in areas of low seismicity. In other cases, a new wood structural panel should be placed over the existing straight sheathing, and the joints of the wood structural panels should be placed so that they are near the center of the sheathing boards or at a 45 degree angle to the joints between sheathing boards. Refer to FEMA 547 (2006) and ATC-7 (1981) for additional information.

New vertical elements can be added to reduce the diaphragm span-to-depth ratio. The reduction of the diaphragm span-to-depth ratio also reduces the lateral deflection and shear demand in the diaphragm. Typical construction details and methods are discussed in FEMA 547 (2006) FEMA 547 (2006).

A.4.2.2 Diagonally Sheathed and Unblocked Diaphragms *All diagonally sheathed or unblocked wood panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1 for Collapse Prevention and have horizontal spans less than 30 ft (9.2 m) and aspect ratios less than or equal to 3-to-1 for Immediate Occupancy.*

Long-span diaphragms often experience large lateral deflections and diaphragm shear demands. Large deflections in the diaphragm can result in increased damage or collapse of elements laterally supported by the diaphragm. Excessive diaphragm shear demands cause damage and reduced stiffness in the diaphragm.

Wood structural panel diaphragms may not have blocking below unsupported panel edges. Blocking may be necessary at diaphragm boundaries to prevent premature failure caused by joist rolling. The shear capacity of diagonally sheathed or unblocked diaphragms is less than that of fully blocked wood structural panel diaphragms because of the limited ability for direct shear transfer at unsupported panel edges. The span and aspect ratio of diaphragms is limited to minimize shear demands. The aspect ratio (span/depth) must be calculated for the direction being evaluated.

Compliance can be demonstrated if the diaphragm can be shown to have adequate capacity for the demands in the building being evaluated and vertical load-carrying elements can be shown to have adequate capacity at maximum deflection.

The shear capacity of unblocked diaphragms can be improved by adding new blocking and fastening at the unsupported panel edges. Placing a new wood structural panel over the existing diaphragm increases the shear capacity. Both of these methods require the partial or total removal of existing flooring or roofing to place and fasten the new overlay or fasten the existing panels to the new blocking. Strengthening of the diaphragm is usually not necessary at the central area of the diaphragm where shear is low. In certain cases where the design forces are low, it may be possible to increase the shear capacity of unblocked diaphragms with sheet metal plates stapled on the underside of the existing wood panels. These plates and staples must be designed for all related shear and torsion caused by the details related to their installation.

New vertical elements can be added to reduce the diaphragm span. The reduction of the diaphragm span also reduces the lateral deflection and shear demand in the diaphragm. However, adding new vertical elements results in a different distribution of shear demands. Additional blocking, nailing, or other retrofit measures may need to be provided at these areas, as indicated in FEMA 172 (1992a), Section 3.4.

A.4.2.3 Blocked Diaphragms *All blocked wood structural panel diaphragms have horizontal spans less than 120 ft (36.5 m) for Collapse Prevention and less than 90 ft (27.4 m) for Immediate Occupancy and have aspect ratios less than or equal to 4-to-1.*

Long-span diaphragms often experience large lateral deflections and diaphragm shear demands. Large deflections in the diaphragm can result in increased damage or collapse of elements laterally supported by the diaphragm. Excessive diaphragm shear demands cause damage and reduced stiffness in the diaphragm.

Compliance can be demonstrated if the diaphragm can be shown to have adequate capacity for the demands in the building being evaluated and vertical load-carrying elements can be shown to have adequate capacity at maximum deflection.

The shear capacity of blocked diaphragms can be improved with additional fasteners or placing additional new wood structural panels over the existing diaphragm increases the shear capacity. Both of these methods may require the partial or total removal of existing flooring or roofing to place and fasten the new overlay or fasten the existing panels to the existing or new blocking. Strengthening of the diaphragm is usually not necessary at the central area of the diaphragm where shear is low.

New vertical elements can be added to reduce the diaphragm span. The reduction of the diaphragm span also reduces the lateral deflection and shear demand in the diaphragm. However, adding new vertical elements results in a different distribution of shear demands. Additional blocking, nailing, or other retrofit measures may need to be provided at these areas, as indicated in FEMA 172 (1992a), Section 3.4.

A.4.2.4 Cantilevered Wood Diaphragms *All cantilevered diaphragms that provide lateral support for concrete or masonry walls consist of wood structural panels and have a maximum cantilever length of 20 ft (6.1 m) if unblocked or 35 ft (10.7 m) if fully blocked for Collapse Prevention and 15 ft (4.6 m) if unblocked or 25 ft (7.6 m) if fully blocked for Immediate Occupancy, and a maximum ratio of cantilever length to diaphragm width of 1:2 if unblocked and 1:1 if blocked for Collapse Prevention and 1:2.5 if unblocked and 1:1.5 if blocked for Immediate Occupancy. In addition, the cantilevered diaphragm has a back span length equal to or greater than the cantilevered portion.*

Cantilevered diaphragms can have large lateral deflections and diaphragm shear demands. Large deflections in the diaphragm can result in increased damage or collapse of elements laterally supported by the diaphragm. Excessive diaphragm shear demands cause damage and reduced stiffness in the diaphragm. Short back spans can have high shear demands throughout their length relative to the cantilevered portion.

Compliance can be demonstrated if the diaphragm can be shown to have adequate capacity for the demands in the building being evaluated and vertical load-carrying elements can be shown to have adequate capacity at maximum deflection.

The shear capacity of blocked diaphragms can be improved with additional fasteners or placing additional new wood structural panels over the existing diaphragm increases the shear capacity. Both of these methods require the partial or total removal of existing flooring or roofing to place and fasten the new overlay or fasten the existing panels to the new blocking.

A.4.3 Metal Deck Diaphragms Bare metal deck can be used as a roof diaphragm where the individual panels are adequately fastened to the supporting framing. The strength of the diaphragm depends on the profile and gauge of the deck and the layout and size of the welds or fasteners. Allowable shear capacities for metal deck diaphragms are usually obtained from approved test data and analytical work developed by the industry.

Metal decks used in floors generally have concrete fill. In cases with structural concrete fill, the metal deck is considered to be a concrete form and the diaphragm is treated as a reinforced concrete diaphragm. In some cases, however, the concrete fill is not structural. It may be a topping slab or an insulating layer that is used to encase conduits or provide a level wearing surface. This type of construction is considered to be an untopped metal deck diaphragm with a capacity determined by the metal deck alone. Nonstructural topping, however, is somewhat beneficial and has a stiffening effect on the metal deck.

Metal deck diaphragm behavior is limited by buckling of the deck and by the attachment to the framing. Weld quality can be an issue because welding of light-gauge material requires special consideration. Care must have been taken during original construction to ensure that the weld has proper fusion to the framing but did not burn through the deck material.

Concrete-filled metal decks generally make excellent diaphragms and usually are not a problem as long as the basic requirements for chords, collectors, and reinforcement around openings are met. However, the evaluating engineer should look for conditions that can weaken the diaphragm, such as troughs, gutters, and slab depressions that can have the effect of short-circuiting the system or of reducing the system to the bare deck.

A.4.3.1 Non-Concrete-Filled Diaphragms *Bare steel deck diaphragms or steel deck diaphragms with fill other than reinforced structural concrete consist of horizontal spans of less than 120 ft (36.5 m) for Collapse Prevention and less than 40 ft (12.2 m) for Immediate Occupancy and have aspect ratios less than 4-to-1.*

Steel deck diaphragms that are either untopped or have topping consisting of nonstructural concrete fill or similar toppings have limited strength and stiffness. Long-span diaphragms with large aspect ratios often experience large lateral deflections and high diaphragm shear demands. This situation is especially true for aspect ratios greater than 4-to-1.

In levels of moderate and high seismicity, the span and aspect ratio of untopped steel deck diaphragms are limited to minimize shear demands. The aspect ratio (span/depth) must be calculated for the direction being considered.

Compliance can be achieved if the diaphragm has adequate capacity for the demands in the building being evaluated.

A.4.4 Concrete Diaphragms Concrete slab diaphragm systems have demonstrated good performance in past earthquakes. Building damage is rarely attributed to a failure of the concrete diaphragm itself, but rather to failure in related elements in the load path, such as collectors or connections between diaphragms and vertical elements. These issues are addressed elsewhere in this standard. The design professional should assess concrete diaphragms for general evaluation statements that address configuration, irregularities, openings, and load path. The design professional also should carefully assess pan joist systems and other systems that have thin slabs.

A.4.5 Precast Concrete Diaphragms Precast concrete diaphragms consist of horizontal precast elements that may or may not have a cast-in-place topping slab. Precast elements may be precast planks laid on top of framing or precast T-sections that consist of both the framing and the diaphragm surface cast in one piece.

Because of the brittle nature of the connections between precast elements, special attention should be paid to eccentricities, adequacy of welds, and length of embedded bars. If a topping slab is provided, it should be capable of taking all the shear. Welded steel connections between precast elements, with low rigidity relative to the concrete topping, do not contribute significantly to the strength of the diaphragm where a topping slab is present.

A.4.5.1 Topping Slab *Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab with a minimum thickness of 2 in. (51 mm).*

Precast concrete diaphragm elements may be interconnected with welded steel inserts. These connections are susceptible to sudden failure such as weld fracture, pullout of the embedment, or spalling of the concrete. Precast concrete diaphragms without topping slabs may be susceptible to damage unless they were specifically detailed with connections capable of yielding or of developing the strength of the connected elements.

In precast construction, topping slabs may have been poured between elements without consideration for providing continuity. The topping slab may not be fully effective if it is interrupted at interior walls. The presence of dowels or continuous reinforcement is needed to provide continuity.

Where the topping slab is not continuous, an evaluation considering the discontinuity is required to ensure a complete load path for shear transfer, collectors, and chords.

A.4.6 Horizontal Bracing Horizontal bracing usually is found in industrial buildings. These buildings often have very little mass, so wind considerations govern over seismic considerations. The wind design is probably adequate if the building shows no signs of distress. If bracing is present, the design professional should look for a complete load path with the ability to collect all tributary forces and deliver them to the walls or frames. Horizontal rod bracing should be investigated for eccentricities at the connections and sagging or looseness in the rods.

A.4.7 Other Diaphragms

A.4.7.1 Other Diaphragms *Diaphragms do not consist of a system other than wood, metal steel deck, concrete, or horizontal bracing.*

In some codes and standards, there are procedures and allowable diaphragm shear capacities for diaphragms not covered by this standard. Examples include thin planks and gypsum toppings, but these systems are brittle and have limited strength. As such, they may not be desirable elements in the seismic-force-resisting system. Another example is standing seam roofs or other metal roof systems that are designed to move to minimize thermal stresses. For seismic loading in certain directions, such roofs may not provide a diaphragm load path.

The design professional should be watchful for systems that look like diaphragms but may not have the strength, stiffness, or interconnection between elements necessary to perform the intended function.

A.5 PROCEDURES FOR CONNECTIONS

This section provides guidelines for using the Tier 1 checklists and the Tier 2 deficiency-based evaluation and retrofit procedures that apply to structural connections: anchorage for normal forces, shear transfer, vertical components, interconnection of elements, and panel connections.

A.5.1 Anchorage for Normal Forces

A.5.1.1 Wall Anchorage *Exterior concrete or masonry walls, which are dependent on the diaphragm for lateral support, are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.*

Bearing walls that are not positively anchored to the diaphragms may separate from the structure, causing partial collapse of the floors and roof. Nonbearing walls that separate from the structure may represent a significant falling hazard. The hazard amplifies with the height above the building base. Amplification of the ground motion used to estimate the wall anchorage forces depends on the type and configuration of both the walls and the diaphragms, as well as the type of soil. Anchorage forces must be fully developed into the diaphragm to prevent pullout failure of the anchor or local failure of the diaphragm (Figure A-34).

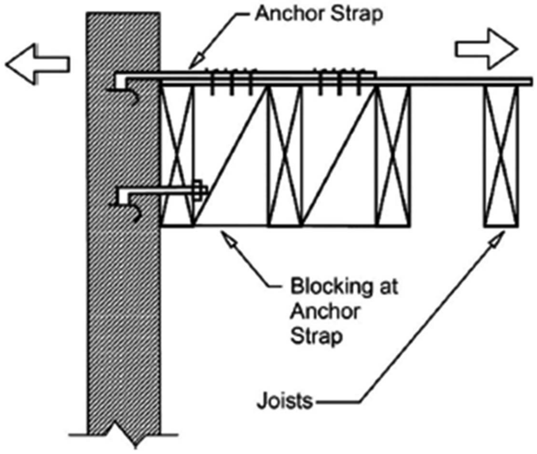


Figure A-34. Wall anchorage.

If the anchorage is nonexistent, mitigation with elements or connections needed to anchor the walls to the diaphragms is necessary to achieve the selected performance level.

To account for identified deficiencies, wall anchors can be added. Complications that may result from inadequate anchorage include cross-grain tension in wood ledgers or failure of the diaphragm-to-wall connection caused by (1) insufficient strength, number, or stability of anchors; (2) inadequate embedment of anchors; (3) inadequate development of anchors and straps into the diaphragm; (4) deformation of anchors and their fasteners that permit diaphragm boundary connection pullout; or (5) failure of wood ledgers or top plates in cross-grain tension or bending.

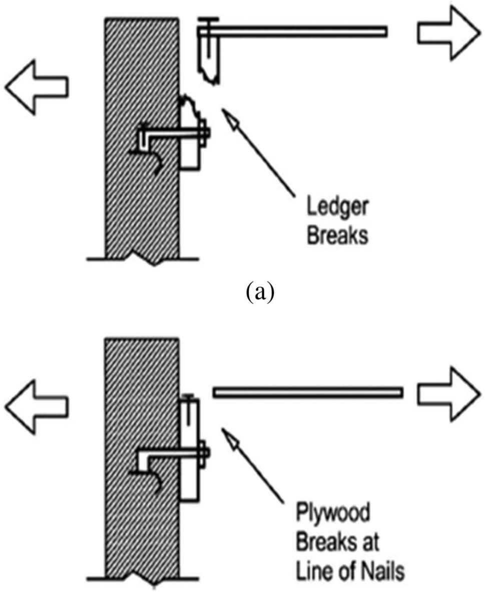
Existing anchors should be tested to determine load capacity and deformation potential, including fastener slip, according to the requirements in this standard. Special attention should be given to the testing procedure to maintain a high level of quality control. Additional anchors should be provided as needed to supplement those that fail the test, as well as those needed to meet the criteria of this standard. The quality of the retrofit depends greatly on the quality of the performed tests.

A.5.1.2 Wood Ledgers *The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers or top plates fastened to the top of walls.*

Wood members in general have very little resistance to tension applied perpendicular to grain. Connections that rely on cross-grain bending in wood ledgers induce tension perpendicular to grain. Failure caused by cross-grain bending results in the ledger breaking (Figure A-35a). Another significant failure mode caused by inadequate wall anchorage is the sheathing breaking at the line of nails (Figure A-35b). Failure of such connections is sudden and nonductile and can result in loss of bearing support and partial collapse of the floors and roof.

Wall anchorage in concrete and masonry buildings with wood diaphragms was commonly detailed with wood ledgers and top plates in areas of high seismicity before the mid-1990s. These types of details, which resulted in cross-grain bending of the ledger or the top plate, were permitted by building codes of that vintage. Post-earthquake observations and subsequent research have demonstrated the vulnerabilities with these types of details.

Mitigation with elements or connections needed to provide wall anchorage without inducing cross-grain bending is necessary to achieve the selected performance level.



(a)

(b)

Figure A-35. Wood ledgers.

A.5.1.3 Minimum Number of Wall Anchors Per Panel *There are at least two anchors connecting each precast wall panel into the diaphragm elements.*

At least two connections between each panel and the diaphragm are required for basic stability of the wall panel for out-of-plane forces. Many connection configurations are possible, including one anchor supporting two adjacent panels.

A single anchor, or line of anchors, near the panel center of mass should be evaluated for an accidental eccentricity of 5% of the critical panel dimension, as a minimum.

A.5.1.4 Stiffness of Wall Anchors *Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. (3 mm) before engagement of the anchors.*

The concern is that flexibility or slip in wall anchorage connections requires relative movement between the wall and structure before the anchor is engaged. This relative movement can induce forces in elements not intended to be part of the load path for out-of-plane forces. It can be enough to cause a loss of bearing at vertical supports, or it can induce cross-grain bending in wood ledger connections.

Compliance can be demonstrated if the movement has no detrimental effect on the connections. Forces generated by any additional eccentricity at bearing supports should be considered.

A.5.2 Shear Transfer The transfer of diaphragm shears into shear walls and frames is a critical element in the load path for seismic force resistance. If the connection is inadequate or nonexistent, the ability of the walls and frames to receive seismic forces is limited and the overall seismic force resistance of the building is reduced.

A.5.2.1 Transfer to Shear Walls or Concrete and Infill Walls *Diaphragms are connected for transfer of seismic forces to the shear walls for Collapse Prevention, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms for Immediate Occupancy.*

The floor or roof diaphragms must be connected to the shear walls or concrete to provide a complete load path for the transfer of diaphragm shear forces to the walls or frames. Where the wall or frame does not extend the full depth of the diaphragm, this connection may include collectors or drag struts. Collectors and drag struts must be continuous across intersecting framing members and must be adequately connected to the wall to deliver high tension and compression forces at a concentrated location.

In the case of frame buildings with infill walls (Building Types S5, S5a, C3, and C3a), the seismic performance is dependent on the interaction between the frame and infill, and the behavior is more like that of a shear wall building. The load path between the diaphragms and the infill panels is most likely through the frame elements, which also may act as drag struts and collectors. In this case, the evaluation statement is addressing the connection between the diaphragm and the frame elements.

If the connection is nonexistent, mitigation with elements or connections needed to transfer diaphragm shear to the shear walls is necessary to achieve the selected performance level.

Collector members, splice plates, and shear transfer devices can be added as required to deliver collector forces to the shear wall. Adding shear connectors from the diaphragm to the wall and/or to the collectors transfers shear. See FEMA 547 (2006) for additional guidance for various types of diaphragms.

A.5.2.2 Transfer to Steel Frames *Diaphragms are connected for transfer of loads to the steel frames for Collapse Prevention, and the connections are able to develop the lesser of the strength of the frames or the diaphragms for Immediate Occupancy.*

The floor and roof diaphragms must be adequately connected to the steel frames to provide a complete load path for shear transfer between the diaphragms and the frames. This connection may consist of shear studs or welds between the metal deck and steel framing. In older construction, steel framing may be encased in concrete. Direct force transfer between concrete and steel members by shear friction concepts should not be used unless the members are completely encased in concrete.

If the connection is nonexistent, mitigation with elements or connections needed to transfer diaphragm shear to the steel frames is necessary to achieve the selected performance level.

Adding collectors and connections to the diaphragm transfers forces to the frames. Connections can be provided along the collector length and at the collector-to-frame connection to withstand the calculated forces (see FEMA 547 2006).

A.5.2.3 Topping Slab to Walls or Frames *Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements for Collapse Prevention, and the dowels are able to develop the least of the shear strength of the walls, frames, or slabs for Immediate Occupancy.*

The topping slabs at each floor or roof must be connected to the shear walls or frame elements to provide a complete load path for the transfer of diaphragm shear forces to the vertical elements. Welded inserts between precast floor or roof elements are susceptible to weld fracture and spalling and are likely not adequate to transfer these forces alone.

If a direct topping slab connection is nonexistent, mitigation with elements or connections needed to transfer diaphragm shear to the vertical elements is necessary to achieve the selected performance level.

See Sections A.5.2.1 and A.5.2.2 for additional retrofit guidelines.

A.5.3 Vertical Components The following statements reflect a number of common concerns related to inadequate connections between elements. For example, members may be incapable of transferring seismic forces into the foundation or may be displaced where uplifted, resulting in reduced support for vertical loads. A potential deficiency common to all of the following statements would be a nonexistent connection.

A.5.3.1 Steel Columns *For Collapse Prevention and Immediate Occupancy, the columns in seismic-force-resisting frames are anchored to the building foundation with a minimum of two anchor rods and with the base plates bearing on concrete or a grout pad. For Immediate Occupancy, the anchor rods are capable of resisting the overturning force using the Quick Check procedure of Section 4.4.3.6.*

Steel columns that are part of the seismic-force-resisting system must be connected for the transfer of uplift and shear forces at the foundation (Figure A-36). The absence of a substantial connection between the columns and the foundation may allow the column to uplift or slide off of bearing supports, which may limit the ability of the columns to support vertical loads or resist seismic forces. If the base plate is supported only by leveling nuts or shims and not bearing on concrete or a grout pad, bending or axial load on the base plate may cause failure of the base plate or anchors. Although a minimum of four anchors for a base plate for steel columns is required by OSHA for new construction, this requirement is intended for construction safety and not a consideration for design strength. Therefore, this requirement is not applicable for existing structures. The requirement for a minimum of two anchors is judged to provide a nominal capacity to resist uplift and shear forces between the column base plate and foundation.

For the Immediate Occupancy Performance Level, the uplift of the connection is checked for the axial force due to overturning calculated using the Quick Check procedure without the resistance provided by dead load. Although the intent is to evaluate the uplift capacity of the base connection anchorage, the weak link in the load path between the superstructure and the supporting soil could be the uplift capacity of the pile, the connection between the pile and the cap, or the foundation dead load that can be activated by the column, the column tensile capacity, or the splice capacity. Other checklist statements check some of these weak links.

If the connection is nonexistent, mitigation with elements or connections needed to anchor the vertical elements to the foundation is necessary to achieve the selected performance level.

A.5.3.2 Concrete Columns *All concrete columns are doweled into the foundation with a minimum of four bars for Collapse Prevention, and the dowels are able to develop the tensile capacity of reinforcement in columns of the seismic-force-resisting system for Immediate Occupancy.*

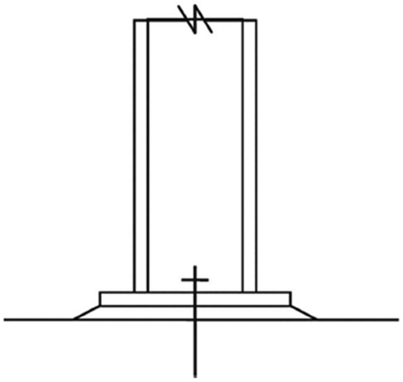


Figure A-36. Steel column connection.

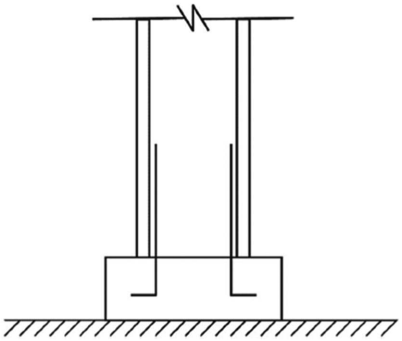


Figure A-37. Column doweled into foundation.

Concrete columns that are part of the seismic-force-resisting system must be connected for the transfer of uplift and shear forces to the foundation (Figure A-37). The absence of a substantial connection between the columns and the foundation may allow the column to uplift or slide off of bearing supports, which limits the ability of the columns to support vertical loads or resist seismic forces. Typically, at a minimum, the four corner bars of the column should be doweled into the foundation.

If the connection is nonexistent, mitigation with elements or connections needed to anchor the vertical elements to the foundation is necessary to achieve the selected performance level.

If concrete columns lack dowels, a concrete curb can be installed adjacent to the column by drilling dowels and installing anchors into the wall that lap with dowels installed in the slab or footing. However, this curb can cause significant architectural problems.

A.5.3.3 Wood or Cold-Formed Steel Posts *There is a positive connection of posts to the foundation.*

Typically, the bases of wood posts are connected to a wood block embedded in a concrete footing. The use of two or more toenails connecting the post to the block is considered to be the minimum positive connection.

The absence of a substantial connection between the posts and the foundation may allow the posts to slide off of bearing supports as the structure drifts in an earthquake.

Mitigation with elements or connections needed to anchor the posts to the foundation is necessary to achieve the selected performance level.

Wood posts can be anchored to concrete slabs or footings using expansion anchors and clip angles.

Cold-formed steel posts are typically supported with short sections of track (channel) or pairs of angles anchored to the foundation. Posts are attached to the track or angles with a minimum of two sheet metal screws on two sides of the post. Tracks or angles can be anchored to concrete slabs or footings using expansion or screw-type concrete anchors.

A.5.3.4 Wood Sills and Cold-Formed Steel Base Tracks *All wood sills and cold-formed steel base tracks are bolted to the foundation.*

The absence of a connection between the wood sills or cold-formed steel base tracks and the foundation is a gap in the load path that limits the ability of the shear walls to resist seismic forces. Structures may potentially slide off foundation supports.

Where some, but not all, of the sill plates or base tracks have been bolted or the sill or base track is attached by shot pins or other types of shear connections, an evaluation can be performed to check the adequacy of existing elements. The evaluation should consider only those elements located below shear-resisting elements of the seismic-force-resisting system.

Mitigation with elements or connections needed to anchor the sills or base tracks to the foundation is necessary to achieve the selected performance level. Expansion anchors or epoxy anchors can be installed by drilling through the wood sill or base track to the concrete foundation.

A.5.3.5 Foundation Dowels *Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical walls reinforcing immediately above the foundation for Collapse Prevention, and the dowels are able to develop the lesser of the strength of the walls or the uplift capacity of the foundation for Immediate Occupancy.*

The absence of an adequate connection between the shear walls and the foundation is a gap in the load path that limits the ability of the shear walls to resist seismic forces.

If the connection is nonexistent or if the size and spacing of the dowels is less than the vertical reinforcing in the walls, the capacity of the dowels to transfer the required forces should be evaluated, and mitigation with elements or connections needed to anchor the walls to the foundation may be necessary to achieve the selected performance level.

If the concrete or masonry walls lack dowels, a concrete curb can be installed adjacent to the wall or column by drilling dowels and installing anchors into the wall that lap with dowels installed in the slab or footing. However, this curb can cause significant architectural problems.

A.5.3.6 Precast Wall Panels *Precast wall panels are connected to the foundation for Collapse Prevention, and the connections are able to develop the strength of the walls for Immediate Occupancy.*

The absence of an adequate connection between the precast wall panels and the foundation is a gap in the load path that limits the ability of the panels to resist seismic forces.

If the connection is nonexistent, mitigation with elements or connections needed to anchor the precast walls to the foundation is necessary to achieve the selected performance level.

If precast walls lack adequate connections, a concrete curb can be installed adjacent to the wall by drilling dowels and installing anchors into the wall that lap with dowels installed in the slab or footing. However, this curb can cause significant architectural problems. Alternatively, steel angles may be used with drilled anchors.

A.5.3.7 Wood Sill and Cold-Formed Steel Base Track Bolts *Sill or base track bolts are spaced at 6 ft (1.8 m) or less for Collapse Prevention and 4 ft (1.2 m) or less for Immediate Occupancy, with acceptable edge and end distance provided for wood, steel, and concrete.*

The absence of an adequate connection between the wood sills or cold-formed steel base tracks and the foundation is a gap in the load path that limits the ability of the shear walls to resist seismic forces. Structures may slide off foundation supports.

Sill or base track bolt spacing has been limited in moderate and high seismic zones to limit the demand on individual bolts. Compliance can be demonstrated if the existing bolts are adequate to resist the demands in the building being evaluated.

To improve wood sill or cold-formed steel base track anchorage, expansion anchors or epoxy anchors can be installed by drilling through the wood sill or cold-formed steel base track to the concrete foundation.

A.5.3.8 Uplift at Pile Caps *Pile caps have top reinforcement, and piles are anchored to the pile caps for Collapse Prevention, and the pile cap reinforcement and pile anchorage are able to develop the tensile capacity of the piles for Immediate Occupancy.*

Pile foundations may have been designed considering downward gravity loads only. A potential problem is a lack of top reinforcement in the pile cap and a lack of a positive connection between the piles and the pile cap. The piles may be socketed into the cap without any connection to resist tension.

Seismic forces may induce uplift at the foundation that must be delivered into the piles for overturning stability. The absence of top reinforcement means that the pile cap cannot distribute the uplift forces to the piles. The absence of pile tension connections means that the forces cannot be transferred to the piles. Piles also should be checked for confinement and spacing of ties and spirals.

Typically, deficiencies in the load path at the pile caps are not a Life Safety concern. However, if the design professional has determined that there is a strong possibility of a Life Safety hazard because of this deficiency, piles and pile caps may be modified, supplemented, repaired, or in the most severe condition, replaced in their entirety. Alternatively, the building system may be retrofitted such that the pile caps are protected.

A.5.4 Interconnection of Elements

A.5.4.1 Girder–Column Connection *There is a positive connection using plates, connection hardware, or straps between the girder and the column support.*

The absence of a substantial connection between the girders and supporting columns may allow the girders to slide off bearing supports as the structure deforms in an earthquake.

Mitigation with elements or connections needed to connect the girders and columns is necessary to achieve the selected performance.

Bearing length conditions can be addressed by adding bearing extensions.

A.5.4.2 Girders *Girders supported by walls or pilasters have at least two ties securing the anchor bolts unless provided with independent stiff wall anchors with strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.*

Girders supported on wall pilasters may be required to resist wall out-of-plane forces. Without adequate confinement, anchor bolts may pull out of the pilaster (Figure A-38). The potential for

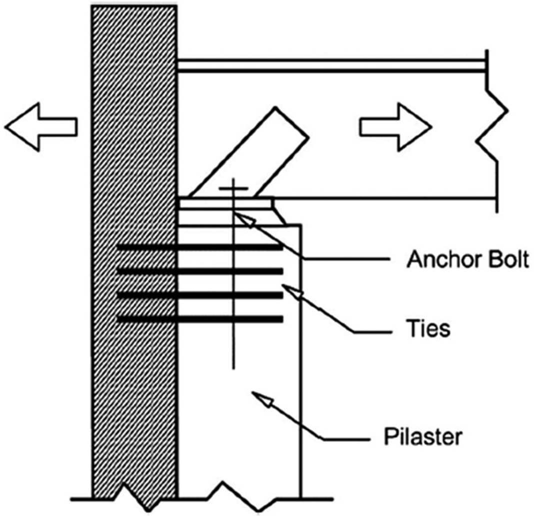


Figure A-38. Girder anchorage.

the pilaster to spall can lead to reduced bearing area or loss of bearing support for the girder.

Where there is concern about lack of pilaster ties, the existing reinforcing must be exposed and the connection must be modified as necessary. For out-of-plane forces, the number of column ties can be increased by jacketing the pilaster or, alternatively, by developing a second load path for the out-of-plane forces.

A.5.4.3 Corbel Bearing *If the frame girders bear on column corbels, the length of bearing is greater than 3 in. (76 mm).*

If drifts are sufficiently large, girders can slide off bearing supports without adequate length. At maximum drift, the bearing support may experience additional eccentricity not considered in the design. The support should be evaluated for strength at this extreme condition.

A.5.4.4 Corbel Connections *The frame girders are not connected to corbels with welded elements.*

Precast elements that are interconnected at the supports may develop unintended frame action and attract seismic forces. The concern is that the welded connections are unable to develop the strength of the members and are subject to sudden nonductile failure, possibly leading to partial collapse of the floor or roof.

Connections may be in compliance if failure of the connection does not jeopardize the vertical support of the girder.

A.5.4.5 Beam, Girder, and Truss Supports *Beams, girders, and trusses supported by unreinforced masonry walls or pilasters have independent secondary columns for support of vertical loads.*

Loss of masonry capacity caused by seismic forces also results in loss of vertical support without a secondary gravity system.

A.5.5 Panel Connections

A.5.5.1 Roof Panels *Where considered as diaphragm elements for lateral resistance, metal, plastic, or cementitious roof panels are positively attached to the roof framing to resist seismic forces.*

The absence of a positive connection between metal, fiberglass, or cementitious panels and the roof framing is a gap in the load path that limits the ability of the panels to act as a diaphragm.

Panels not intended to be a part of the diaphragm represent a potential falling hazard if not positively attached to the framing. In this case, the evaluation should be limited to the anchorage forces and connections of the panels. Consideration should be given to the ability of the connections to resist the deformations imposed by building movements.

If the connection is nonexistent, mitigation with elements or connections needed to attach the roof panels is necessary to achieve the selected performance level.

It may be possible to improve the connection between the roof and the framing. If architectural or occupancy conditions warrant, the roof diaphragm can be replaced with a new one. Alternatively, a new diaphragm may be added using rod braces or wood structural panels above or below the existing roof, which remains in place.

A.5.5.2 Wall Panels *Where considered as shear elements for lateral resistance, metal, fiberglass, or cementitious wall panels are positively attached to the framing to resist seismic forces.*

The absence of a positive connection between metal, fiberglass, or cementitious panels and the framing is a gap in the load path that limits the ability of the panels to resist seismic forces.

Panels not intended to be a part of the seismic-force-resisting system represent a potential falling hazard if not positively attached to the framing. In this case, the evaluation should be limited to the anchorage forces and connections of the panels. Consideration should be given to the ability of the connections to resist the deformations imposed by building movements.

If the connection is nonexistent, mitigation with elements or connections needed to attach the panels is necessary to achieve the selected performance level.

A.6 PROCEDURES FOR GEOLOGIC SITE HAZARDS AND FOUNDATIONS

This section provides guidelines for using the Tier 1 checklists and the Tier 2 deficiency-based evaluation and retrofit procedures that apply to foundations and supporting soils: geologic site hazards and the configuration of foundations.

A thorough seismic evaluation of an existing building should include an examination of the foundation, an assessment of the capability of the soil beneath the foundation to withstand the forces applied during an earthquake, and consideration of nearby geologic hazards that may affect the stability of the building during an earthquake.

To fully assess the potential hazard presented by local geologic site conditions, and to establish soil engineering parameters required for analysis of these hazards, it may be necessary to consult with a geotechnical design professional. The evaluating design professional is strongly urged to seek consultation with appropriate professionals wherever site conditions are beyond the experience or expertise of the design professional.

A.6.1 Geologic Site Hazards Certain geologic and local site conditions can lead to structural damage in the event of an earthquake. Large foundation movements attributable to any number of causes can severely damage an otherwise seismic-resistant building. Potential causes of significant foundation movement include settlement or lateral spreading caused by liquefaction, slope failure, or surface ruptures. An evaluation of the building should include consideration of these effects and the effect they might have on the superstructure.

Retrofit of structures subject to Life Safety hazards from ground failures is impractical unless site hazards can be mitigated to the point where acceptable performance can be achieved. Not all ground failures need necessarily be considered as Life Safety hazards. For example, in many cases liquefaction beneath a building does not pose a Life Safety hazard; however, related lateral spreading can result in collapse of buildings with inadequate foundation strength. For this reason, the liquefaction potential and the related consequences should be thoroughly investigated for sites that do not satisfy the requirements of this standard. Further information on the evaluation of site hazards is provided in Chapter 8.

A.6.1.1 Liquefaction *Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's foundation support and seismic performance do not exist in the foundation soils at depths within 50 ft (15.2 m) under the building.*

Soils susceptible to liquefaction may lose all vertical-load-bearing capacity during an earthquake. Loss of vertical support for the foundation causes large differential settlements and induces large forces in the building superstructure.

These forces are concurrent with all existing gravity loads and seismic forces during the earthquake.

A.6.1.2 Slope Failure *The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure.*

Steep slopes are susceptible to slides during an earthquake. Slope failures are possible in rock or on other nonliquefiable soils on slopes that normally exceed 6%. Slopes that exhibit signs of prior landslides require the most attention.

The concern for buildings on the uphill side of slopes is lateral spreading of the downhill footings. The concern for buildings on the downhill side is impact from sliding soil and debris.

A.6.1.3 Surface Fault Rupture *Surface fault rupture and surface displacement at the building site are not anticipated.*

In the near field of active faults, there is a potential for large fissures and differential movement to occur in the surface soils. Foundations of buildings located above these ruptures are subjected to large differential movements that induce large forces in the building superstructure.

These forces are concurrent with all existing gravity loads and seismic forces during the earthquake.

A.6.1.4 Tsunami *The building not located within a Tsunami Design Zone as defined by ASCE 7, Chapter 6 or is located in a Tsunami Design Zone where the inundation depth per ASCE 7, Chapter 6 is less than 3 ft (0.9 m).*

ASCE 7-22, Chapter 6 addresses design requirements for buildings located within Tsunami Design Zones, in particular for Risk Category IV structures, that pose a risk of being flooded or inundated. Because of the tsunami risk, buildings in these locations cannot be assumed to be able to achieve Immediate Occupancy seismic performance without an assessment of the tsunami hazard.

This standard does not contain any provisions for the evaluation of buildings subject to flooding or inundation resulting from a tsunami.

A.6.2 Foundation Configuration Building foundation elements normally have a capacity at least two times the gravity loads. If there are no signs of foundation distress caused by settlement, erosion, corrosion, or other reasons, the foundations are likely to have adequate vertical capacity if the total gravity and seismic overturning forces do not exceed the allowable static capacity by more than a factor of 2.0.

Foundations are considered to have adequate lateral capacity if the horizontal resistance of the foundation system exceeds the calculated seismic forces in Chapter 4 or 5 with horizontal resistance at the foundation treated as a force-controlled action.

Where the evaluation of foundation elements indicates significant problems, the evaluating design professional should consult with a qualified geotechnical design professional to establish rational criteria for foundation analysis and mitigation of unsatisfactory conditions.

The correction of seismic deficiencies in the foundations of existing buildings is expensive and may not be justified by more realistic analysis procedures. For this reason, the Tier 3 systematic retrofit procedure is recommended for these cases.

A.6.2.1 Overturning *The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than* $0.6S_a$.

Although the concentration of seismic overturning forces in foundation elements may exceed the capacity of the soil, the foundation structure, or both, experience suggests that foundation rocking has a very low risk of building collapse for building types eligible for the Tier 1 screening procedure. Foundation overturning could be a concern for systems containing relatively slender shear walls, moment frames, or braced frames, but not for buildings meeting the height limitations for Tier 1 eligibility. In addition, excessive foundation rotation could lead to collapse risk in the superstructure for even shorter buildings, but this risk is mainly in buildings with brittle elements (e.g., shear-critical concrete beams and columns). For these reasons, the foundation overturning statement has been removed from the CP checklists in this edition of the standard. However, excessive foundation deformation or rotation can lead to increased superstructure deformations and a risk of damage that may affect the building's ability to achieve IO performance, so the statement remains for some building types in the IO checklists. For other building types (e.g., wood- and cold-formed steel-framed buildings), foundation rotations are not expected to directly result in superstructure damage because the lateral elements tend to be well-distributed, and overturning is controlled by shear wall slenderness or presence or lack of adequate hold-downs. Therefore, for these structures, the foundation overturning statement has been removed for both CP and IO performance in this edition of the standard.

The effective horizontal dimension should be determined based on the ability of the seismic-force-resisting elements and foundations to act as a system. Therefore, the building dimension can be used if the elements are well connected, for example, by a full-width shear wall or basement wall or mat foundation. In other conditions, the building dimension should be taken as the overall width of the lateral elements. Refer to Figure A-39 for representative conditions. Therefore, multiple checks may be required for elements isolated on opposite sides of the building.

Existing foundations can be strengthened as needed to resist overturning forces. Spread footings may be enlarged, or additional piles, rock anchors, or piers may be added to deep foundations. It may also be possible to use grade beams or new wall elements to spread out overturning forces over a greater distance. Adding new seismic-force-resisting elements reduces overturning effects of existing elements.

A.6.2.2 Ties between Foundation Elements *For buildings supported on soils classified as Site Class D, DE, E, or F, the individual pile caps, piles, and piers are restrained by concrete beams or slabs adequate to resist seismic forces. For buildings supported on soils classified as Site Class E or F, individual spread footings are restrained by concrete beams or slabs adequate to resist seismic forces.*

Ties between discrete foundation elements, such as pile caps, piers, isolated footings, and pole footings, are required where the seismic ground motions are likely to cause significant lateral spreading of the foundations. Ties may consist of tie beams, grade beams, or slabs, including slabs on ground. A slab on ground not directly connected to the foundation elements may be considered to provide restraint if the slab surrounds the vertical element being supported by the foundation element and has adequate capacity to resist the imposed forces, and the effects of joints between the slab and foundation elements are taken into consideration. If the foundations are restrained laterally by competent soils or rock, ties are not required.

A.6.2.3 Deep Foundations *Piles that are required to transfer lateral and/or overturning forces between the structure and the soil shall have a positive connection between the piles and the pile cap, foundation mat, grade beam, or other element of the building foundation system. Cast-in-place and precast non-prestressed piles shall have a minimum longitudinal reinforcement ratio of 0.0025 and transverse reinforcing spaced at no more than 6 in. (152.4 mm) within a distance of three times the pile diameter from the bottom of the pile cap. Precast prestressesd piles shall have a minimum effective*

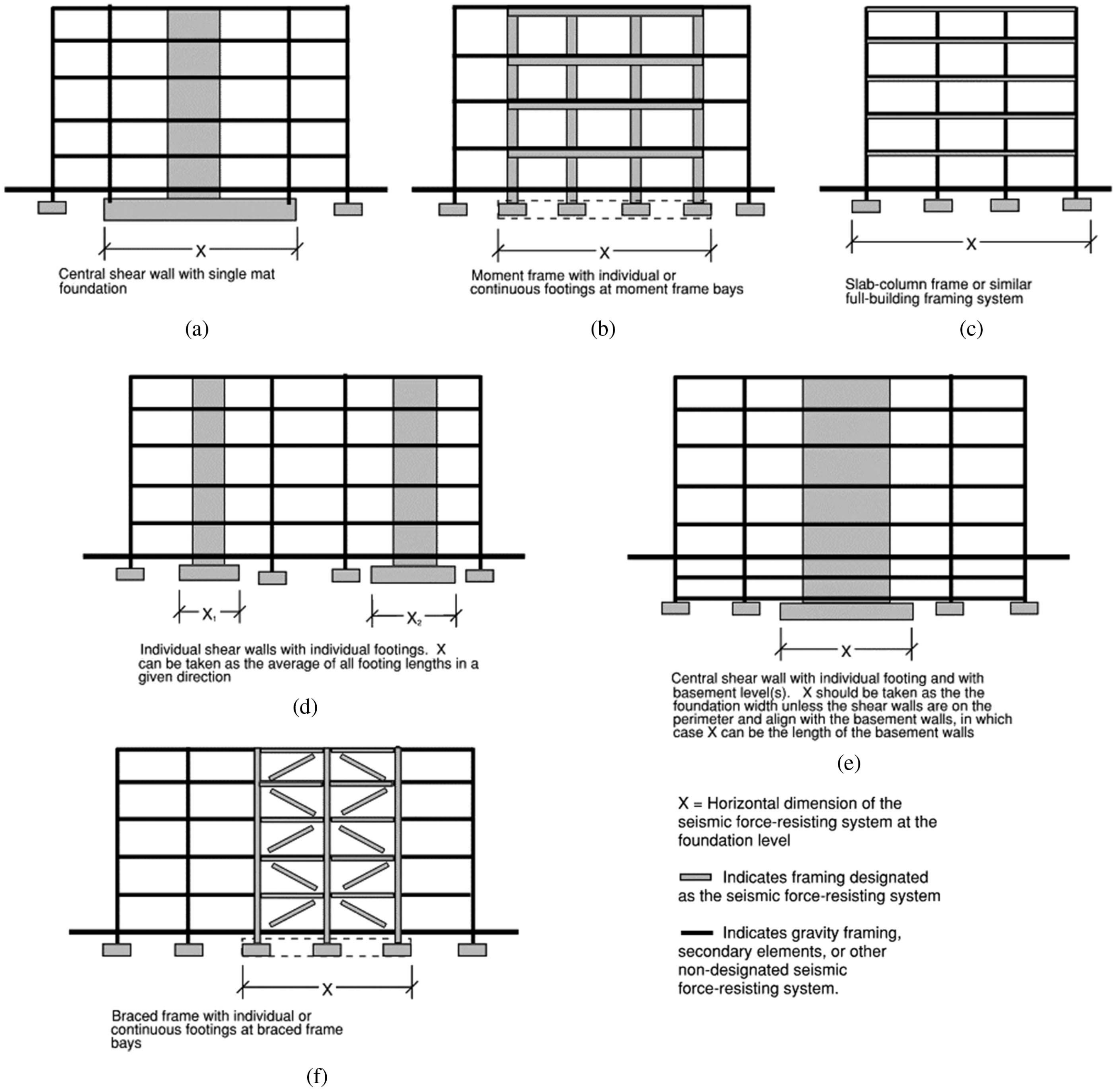


Figure A-39. Foundation dimensions.

prestress of 400 psi (2,758 kPa) and transverse reinforcing spaced at no more than 6 in. within a distance of 20 ft (6 m) from the top of the pile.

The most significant concern with respect to the performance of deep foundations is the connection of the pile to the pile cap or other foundation element. Another concern is the flexural strength and ductility in the upper portion of the pile.

For cast-in-place and precast concrete piles, a minimal amount of both longitudinal and transverse reinforcement in the upper portion of piles is important to avoid brittle failures. For other types of piles—steel and timber—there is generally less concern about lateral strength and ductility. Traditional practice for steel piles usually results in compact sections.

The design professional could check for unique conditions at deep foundations, in particular, older buildings, that may not provide a minimal amount of lateral capacity and ductility even where there is a nominal connection between the pile and the cap.

The correction of seismic deficiencies in the deep foundations of existing buildings is expensive and may not be required if the design engineer can take advantage of more rigorous analysis procedures. For this reason, the Tier 3 systematic evaluation or retrofit procedure is recommended for these cases.

A.6.2.4 Sloping Sites *The exterior grade difference from one side of the building to another does not exceed one story in height.*

The transfer of seismic force to the soil and the ability of the subgrade structure to resist seismic forces is more difficult where a permanent horizontal force is present as a result of unbalanced soil conditions at the building exterior.

The correction of seismic deficiencies in the foundations of existing buildings is expensive and may not be required if the design engineer can take advantage of more rigorous analysis procedures. For this reason, the Tier 3 systematic retrofit procedure is recommended for these cases.

A.7 PROCEDURES FOR NONSTRUCTURAL COMPONENTS

This section provides guidelines for using the Tier 1 checklist procedures that apply to nonstructural components.

Nonstructural components refer to architectural, mechanical, and electrical components. Additional guidance may be requested from another design professional with expertise in structural evaluation and design.

Investigation of nonstructural components can be very time-consuming because they usually are not well detailed on plans and because they often are concealed. It is essential, however, to investigate these items because their seismic support may have been given little attention in the past and they are potentially dangerous. Of particular importance in nonstructural component evaluation efforts are site visits to identify the present status of nonstructural items.

For nonstructural component evaluation in general, the key issue is generally whether the component or piece of equipment is braced or anchored. This issue is generally immediately visible and is part of the Tier 1 evaluation. If the component is braced or anchored, a Tier 3 evaluation per Chapter 13 may be necessary (based on the design professional's judgment) to establish the capacity of the components. Evaluation of cladding, exterior veneers, backup materials, and glazing requires more careful investigation because the critical components, such as connections and framing, often are concealed. In some cases, it is necessary to remove materials to conduct the evaluation. In addition, some calculations may be necessary to establish capacity to accommodate estimated seismic forces.

Several different types of deficiencies may be identified by the design professional in the Tier 1 evaluation. Some of these, such as the nonexistence of anchorage or bracing, are clearly in noncompliance, and any further evaluation is not necessary. In other cases, where some bracing or anchorage is provided or material is deteriorated or corroded, further evaluation and judgment are necessary to ascertain the extent of the deficiency and the consequences of the failure. Some simple calculations of weights, dimensional ratios, and forces are used in this tier of evaluation. A few critical components, such as heavy cladding, may justify a complete analysis (a Tier 3 evaluation) for ability to withstand forces and drifts and for achievement of the desired performance level.

Nonstructural elements can pose significant hazards to Life Safety under certain circumstances. In addition, certain types of building contents can pose hazards (e.g., toxic chemicals) and should be given attention during the evaluation. Special consideration also is warranted for nonstructural elements in essential facilities (e.g., hospitals and police and fire stations) and other facilities that must remain operational after an earthquake.

Any element with rigidity is a part of the seismic-force-resisting system until it fails. All walls have some rigidity, and they participate in resisting seismic forces in proportion to their relative rigidity. Walls of gypsum board or plaster have considerable rigidity. If connected at top and bottom, they can take a significant portion of the seismic force at low force levels; at some higher level, they crack and lose strength, and the main system then takes all of the seismic force.

A.7.1 Partitions

A.7.1.1 Unreinforced Masonry *Unreinforced masonry or hollow-clay tile partitions are braced at a spacing equal to or less than 10 ft (3.0 m) in low or moderate seismicity and 6 ft (1.8 m) in high seismicity.*

Hollow-clay tile units are brittle and subject to shattering. Unreinforced masonry units may have cracks, loose blocks, or weak mortar. Bracing is needed to prevent portions of the unreinforced masonry from dislodging because of out-of-plane seismic forces, especially at corridors, elevator shafts, and stairs. Door openings often create localized weaknesses because of inadequate support for the block masonry or clay tile at the head and at the sides of the opening.

If bracing is nonexistent, mitigation with elements or connections needed to brace the partitions is necessary to achieve the selected performance level.

A.7.1.2 Drift *Rigid cementitious partitions are detailed to accommodate the following drift ratios: in steel moment frame, concrete moment frame, and wood-frame buildings, 0.02; in other buildings, 0.005.*

Full-height partitions may fail because of lack of provision for building drift. Rigid cementitious partitions should be detailed to provide adequate space for the structure drift without racking the walls, while retaining out-of-plane support. In addition, if not separated from the structure at the top and sides, these walls may alter the response of the building.

A.7.1.3 Structural Separations *Partitions at structural separations have seismic or control joints.*

Seismic and control joints are necessary to permit differential structure movement at building separations without causing damage. However, if localized cracking of the partition does not lead to out-of-plane failure of the wall, the costs of a difficult retrofit process may not be justified.

A.7.1.4 Tops *The tops of framed or panelized partitions that extend only to the ceiling line have lateral bracing to the building structure at a spacing equal to or less than 6 ft (1.8 m).*

Partitions extending only to suspended ceilings may fall out of plane because of lack of bracing. Movement of the partition may damage the ceiling. Cross walls that may frame into the wall have a beneficial impact on preventing excessive out-of-plane movement and should be considered in the evaluation process.

If lateral bracing is nonexistent, mitigation with elements or connections needed to brace the partitions is necessary to achieve the selected performance level.

A.7.2 Ceiling Systems

A.7.2.1 Heavy or Light Partitions Supported by Ceilings *The tops of masonry, hollow-clay tile, or gypsum board partitions are not laterally supported by an integrated ceiling system.*

Heavy partitions, such as those of gypsum board, masonry, or hollow-clay tile, can be falling hazards if not properly restrained at their tops and bottoms. Integrated ceilings braced with diagonal wires generally do not have the strength and stiffness to adequately brace the tops of heavy partitions. Heavy partitions should be independently braced to the underside of the floor above.

A.7.2.2 Integrated Ceilings *Integrated suspended ceilings with continuous areas greater than 6 ft (1.8 m) and ceilings of smaller areas that are not surrounded by restraining partitions are laterally restrained at a spacing no greater than 12 ft (3.6 m) with members attached to the structure above. Each restraint* *location has a minimum of four diagonal wires and compression struts, or diagonal members capable of resisting compression.*

Without bracing, integrated ceiling systems are susceptible to vertical and lateral movement, which can damage fire sprinkler piping and other elements that penetrate the ceiling grid. Lightweight suspended ceilings may not pose a Life Safety hazard unless special conditions apply in the judgment of the design professional, such as a large area of ceiling, poor-quality construction, vulnerable occupancy, or egress route.

If bracing is inadequate or nonexistent, mitigation with elements or connections needed to brace the ceilings is necessary to achieve the selected performance level.

A.7.2.3 Suspended Lath and Plaster or Gypsum Board *Suspended lath and plaster or gypsum board ceilings have attachments that resist seismic forces for every 12 ft^2 (1.1 m^2) of area.*

Suspended plaster ceilings may behave like structural diaphragms and resist in-plane seismic forces. If the strength of the plaster is exceeded, cracking and spalling of portions of the ceiling are possible. Large areas of suspended plaster may separate from the suspension system and fall if not properly fastened. The interconnection of the plaster to the lath and of the lath to the support framing should also be specifically assessed.

If anchorage is nonexistent, mitigation with elements or connections needed to brace the ceilings is necessary to achieve the selected performance level.

A.7.2.4 Edge Clearance *The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 (13.4 m^2) have clearances from the enclosing wall or partition of at least the following: in moderate seismicity, 1/2 in. (13 mm); in high seismicity, 3/4 in. (19 mm).*

This provision relates especially to large suspended grid ceilings but also may apply to other forms of hung ceilings. The intent is to ensure that the ceiling is sufficiently detached from the surrounding structural walls, such that it can tolerate out-of-plane drift without suffering distortion and damage.

A.7.2.5 Continuity across Structure *The ceiling system does not extend continuously across any seismic joint and is not attached to multiple independent structures.*

Localized damage to ceilings is expected where seismic separations are not provided in the ceiling framing. Seismic or control joints should be provided based on a consideration of the consequences of local ceiling damage. If the damage is unlikely to create a falling hazard or prevent safe egress, the costs of a difficult retrofit process may not be justified.

A.7.2.6 Edge Support *The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 (13.4 m^2) are supported by closure angles or channels not less than 2 in. (51 mm) wide.*

This provision relates especially to large suspended grid ceilings but also may apply to other forms of hung ceilings. The intent is to ensure that the ceiling is supported by the surrounding structural or nonstructural walls, such that it can tolerate lateral movement but not fall.

A.7.2.7 Seismic Joints *Acoustical tile or lay-in panel ceilings have seismic separation joints such that each continuous portion of the ceiling is no more than 2,500 ft^2 (232.3 m^2) and has a ratio of long-to-short dimension no more than 4-to-1.*

This provision relates especially to large suspended grid ceilings. The intent is to ensure that the ceiling grid does not undergo excessive deformation because of its size or because of a very large aspect ratio such that it would collapse.

A.7.3 Light Fixtures

A.7.3.1 Emergency Lighting *Emergency and egress lighting equipment is anchored or braced.*

Emergency and egress lighting equipment and signs should be provided with positive anchorage and/or bracing to prevent falling hazards and to enhance the reliability of post-earthquake performance.

If bracing or anchorage is nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.3.2 Independent Support *Light fixtures that weigh more per square foot (square meter) than the ceiling they penetrate are supported independent of the grid ceiling suspension system by a minimum of two wires at diagonally opposite corners of each fixture.*

With lay-in fluorescent lighting systems, ceiling movement can cause fixtures to separate and fall from suspension systems. These fixtures perform satisfactorily when they are supported separately from the ceiling system or have backup support that is independent of the ceiling system. If the fixtures are independently supported by methods other than that described, the design professional should exercise judgment as to their adequacy.

If independent support is nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.3.3 Pendant Supports *Light fixtures on pendant supports are attached at a spacing equal to or less than 6 ft (1.8 m). Unbraced suspended fixtures are free to allow a 360 degree range of motion at an angle not less than 45 degrees from horizontal without contacting adjacent components. Alternatively, if fixtures are rigidly supported and/or braced, they are free to move with the structure to which they are attached without damaging adjoining components. Additionally, the connection to the structure is capable of accommodating the movement without failure.*

With stem-hung incandescent or fluorescent fixtures, the fixtures are usually suspended from stems or chains that allow them to sway. These components and connections are typically designed with limited ductility demand (an R_p of 2.5 or less per ASCE 7). Unfortunately, excessive movement and rotation may cause the light and/or fixture to break after encountering other building components. Another common failure observed is the connection to the structure, which often includes a series of connectors, fittings, and couplings between the fixture and the supporting structure. Lights supported on open S-hooks can “hop” out because of excessive movement and vertical acceleration. Lights attached to cold-formed steel strut can “pop” the spring clip if rotated too far. Lights supported with cables can fail at inadequate end connections. Long rows of fluorescent fixtures placed end to end have sometimes fallen because of poor connection ductility and/or flexibility, and their weight makes them hazardous. Long-stem fixtures, which may swing considerably, tend to suffer more damage than short-stem items.

If anchorage is inadequate or nonexistent, mitigation is necessary to achieve the selected performance level. Detailed field investigation is often required to evaluate the adequacy of an existing pendant fixture support system. Proof loading or testing under expected forces or displacements may be useful where there is insufficient information for evaluation or analysis. Consider limiting the ductility demand on the pendant fixture connections (unless demonstrated by testing or analysis).

A.7.3.4 Lens Covers *Lens covers on light fixtures are attached with safety devices.*

Devices or detailing to prevent lens covers from falling from the fixture are necessary to prevent damage to the lens and items below and may be a safety feature.

A.7.4 Cladding and Glazing

A.7.4.1 Cladding Anchors *Cladding components weighing more than 10 lb/ft^2 (0.48 kN/m^2) are mechanically anchored to the structure at a spacing equal to or less than the following: for Life Safety in moderate seismicity, 6 ft (1.8 m); for Life Safety in high seismicity and for Position Retention in any seismicity, 4 ft (1.2 m).*

Exterior cladding components, which are often heavy, can fail if their connections to the building frames have insufficient strength and/or ductility. The design professional should assess the consequences of failure, in particular the location of the panels in relation to building occupants and passersby. Adhesive anchorage of heavy exterior cladding components is unacceptable; such anchorages typically fail at lower drift ratios than are necessary to ensure Life Safety performance.

If anchorage is nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.4.2 Cladding Isolation *For steel or concrete moment-frame buildings, panel connections are detailed to accommodate a story drift ratio by the use of rods attached to framing with oversize holes or slotted holes of at least the following: for Life Safety in moderate seismicity, 0.01; for Life Safety in high seismicity and for Position Retention in any seismicity, 0.02, and the rods have a length-to-diameter ratio of 4.0 or less.*

High levels of drift and deformation may occur in moment frames. If cladding connections are not detailed to accommodate the drift, failure of connections can result and panels can become dislodged.

A.7.4.3 Multistory Panels *For multistory panels attached at more than one floor level, panel connections are detailed to accommodate a story drift ratio by the use of rods attached to framing with oversize holes or slotted holes of at least the following: for Life Safety in moderate seismicity, 0.01; for Life Safety in high seismicity and for Position Retention in any seismicity, 0.02, and the rods have a length-to-diameter ratio of 4.0 or less.*

The design professional should determine whether the panels themselves and/or their connections to the structure would deform to accommodate the story drift. If the connectors are expected to deform, they should be capable of doing so without loss of structural support for the panel. If the panels are expected to rack, they should be capable of deforming without becoming unstable and without loss of support for other interconnected systems, such as glazing.

A.7.4.4 Panel Connections *Cladding panels are anchored out of plane with a minimum number of connections for each wall panel, as follows: for Life Safety in moderate seismicity, 2 connections; for Life Safety in high seismicity and for Position Retention in any seismicity, 4 connections.*

A minimum of two connections, usually one at the top and bottom of the panel, are generally required for stability in resisting out-of-plane earthquake forces. Evaluation of connection adequacy should include consideration of all connection eccentricities.

If connections are nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.4.5 Bearing Connections *Where bearing connections are used, there is a minimum of two bearing connections for each cladding panel.*

A single bearing connection can result in a dangerous lack of redundancy. The adequacy of single-point bearing connections should be evaluated for resistance to in-plane overturning forces including all eccentricities. Small panels, such as some column covers, may have a single bearing connection and still provide adequate safety against failure.

If connections are nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.4.6 Inserts *Where concrete cladding components use inserts, the inserts have positive anchorage or are anchored to reinforcing steel.*

Out-of-plane panel connections that do not engage panel reinforcement are susceptible to pulling out when subjected to seismic forces.

A.7.4.7 Overhead Glazing *Glazing panes of any size in curtain walls and individual interior or exterior panes more than 16 ft^2 (1.5 m^2) in area are laminated annealed or laminated heat-strengthened glass and are detailed to remain in the frame when glass is cracked.*

Laminated glass remains in the frame after cracking or shattering, providing a temporary weather barrier and allowing for Immediate Occupancy after an earthquake.

A.7.4.8 Threaded Rods *Threaded rods for panel connections detailed to accommodate drift by bending of the rod have a length-to-diameter ratio greater than 0.06 times the story height in inches (millimeters) for Life Safety in moderate seismicity and 0.12 times the story height in inches (millimeters) for Life Safety in high seismicity and Position Retention in any seismicity.*

The limits on length-to-diameter ratios are needed to ensure proper connection performance. Longer rods in sliding connections will bind if there is significant bending and rotation in the rod, which may lead to a brittle failure. For rods that accommodate drift by flexure, longer rods reduce inelastic bending demands and provide better performance. Because anchor rods used in sliding and bending may undergo inelastic action, the use of mild steel improves ductility.

A.7.5 Masonry Veneer

A.7.5.1 Ties *Masonry veneer is connected to the backup with corrosion-resistant ties. There is a minimum of one tie for every 2-2/3 ft^2 (0.25 m^2), and the ties have spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 36 in. (914 mm); for Life Safety in High Seismicity and for Position Retention in any seismicity, 24 in. (610 mm).*

Inadequately fastened masonry veneer can pose a falling hazard if it peels away from its backing. Judgment may be needed to assess the adequacy of various attachments that may be used. For levels of lower seismicity, it may be easier to show compliance for a larger tie spacing and larger tie area.

Ordinary shop-galvanized wire ties are not very corrosion resistant and are likely to become heavily corroded within 15 years, if the environment is marine or causes continued wetting and drying cycles to the ties, such as at a windward or southern exposure. To be corrosion resistant, the ties should be stainless steel.

If anchorage is nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.5.2 Shelf Angles *Masonry veneer is supported by shelf angles or other elements at each floor above the ground floor.*

Inadequately fastened masonry veneer can pose a falling hazard if it peels away from its backing. Judgment may be needed to assess the adequacy of various attachments that may be used.

If anchorage is nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.5.3 Weakened Planes *Masonry veneer is anchored to the backup adjacent to weakened planes, such as at the locations of flashing.*

Inadequate attachment at locations of wall discontinuities is a potential source of weakness. Such discontinuities can be created by base flashing or architectural reveals. In areas of moderate and high seismicity, masonry veneer should be anchored to the backup system immediately above the weakened plane.

If anchorage is nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.5.4 Weep Holes *In veneer anchored to stud walls, the veneer has functioning weep holes and base flashing.*

Absence of weep holes and flashing indicates an inadequately detailed veneer. Water intrusion can lead to deterioration of the veneer and/or substrate. Destructive investigation may be needed to evaluate whether deterioration has taken place and mitigation is necessary.

If weep holes are noncompliant, mitigation is necessary to achieve the selected performance level.

A.7.6 Metal Stud Backup Systems

A.7.6.1 Stud Tracks *For veneer with metal stud backup, stud tracks are fastened to the structure at a spacing equal to or less than 24 in. (610 mm) on center.*

Without proper anchorage at top and bottom tracks, metal stud backup systems are susceptible to excessive movement during an earthquake.

A.7.6.2 Openings *For veneer with metal stud backup, steel studs frame window and door openings.*

This issue is primarily one of the general framing system of the building. Absence of adequate framing around openings indicates a possible out-of-plane weakness in the framing system.

A.7.7 Concrete Block and Masonry Backup Systems

A.7.7.1 Anchorage *For veneer with concrete block or masonry backup, the backup is positively anchored to the structure at a horizontal spacing equal to or less than 4 ft (1.2 m) along the floors and roof.*

Backup is the system that supports veneer for out-of-plane forces. Inadequate anchorage of the backup wall may affect the whole assembly's ability to withstand seismic motions and maintain attachment to backup.

A.7.7.2 Unreinforced Masonry Backup *There is no unreinforced masonry backup.*

Unreinforced masonry backup is common in early steel-framed buildings with cut stone exteriors. The design professional should use judgment in evaluating the condition and integrity of the backup and necessary remedial measures. Testing may be necessary to determine the strength of the URM backup.

Complete replacement of backup is extremely expensive; depending on the state of the installation and the facing materials, alternative methods may be possible.

To qualify as reinforced masonry, the area of reinforcing steel is greater than 0.002 times the gross area of the wall with a minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 in. (1,219 mm); and all vertical bars extend to the top of the backup walls.

Judgment by the design professional must be used to evaluate the adequacy of concrete block walls not classified as reinforced. Concrete block walls lacking the minimum reinforcement may be susceptible to in-plane cracking under seismic forces, and portions of the wall may become dislodged.

A.7.8 Parapets, Cornices, Ornamentation, and Appendages

A.7.8.1 Unreinforced Masonry Parapets or Cornices *Laterally unsupported unreinforced masonry parapets or cornices have height-to-thickness ratios no greater than the following: for Life Safety in low or moderate seismicity, 2.5; for Life Safety in areas of high seismicity and for Position Retention in any seismicity, 1.5.*

URM parapets present a major falling hazard and potential Life Safety threat. For sloped roofs, the highest anchorage level should not be taken at the ridge but should vary with roof slope when checking height-to-thickness ratios.

If anchorage is nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.8.2 Canopies *Canopies at building exits are anchored to the structure at a spacing no greater than the following: for Life Safety in low or moderate seismicity, 10 ft (3.0 m); for Life Safety in high seismicity and for Position Retention in any seismicity, 6 ft (1.8 m).*

Inadequately supported canopies present a Life Safety hazard. A common form of failure is pullout of shallow anchors from building walls.

If anchorage is nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.8.3 Concrete Parapets *Concrete parapets with height-to-thickness ratios greater than 2.5 have vertical reinforcement.*

Inadequately reinforced parapets can be severely damaged during an earthquake.

If anchorage is nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.8.4 Appendages *Cornices, parapets, signs, and other ornamentation or appendages that extend above the highest point of anchorage to the structure or cantilever from components are reinforced and anchored to the structural system at a spacing equal to or less than 6 ft (1.8 m). This checklist item does not apply to parapets or cornices covered by other evaluation statements.*

The aforementioned components may vary greatly in size, location, and attachment; the design professional should use judgment in making the assessment. If any of these items is of insufficient strength and/or is not securely attached to the structural elements, it may break off and fall onto storefronts, streets, sidewalks, or adjacent property and become a significant Life Safety hazard.

If anchorages are nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.8.5 Penthouses *Penthouses are constructed as an extension of the building's structural framing or shall have a lateral-force-resisting system in each direction consistent with structural systems listed in Table 15.4-1 of ASCE 7.*

Penthouse structures occupy a smaller footprint than the area of the roof and may be either constructed as extensions of the structural framing system of the building or small structures built on top of a roof surface. Penthouses may be used for housing mechanical or electrical equipment or may be used as habitable spaces for the building occupants. Where used as occupied spaces, the penthouse should be included in the structural evaluation of the building. One or more penthouses may exist on a roof; therefore, the aggregate area and weight of all of the penthouses should be included when considering whether the aggregate area of weight exceeds the limitations for consideration as a nonstructural component.

Penthouse structures that were not constructed as an extension of the structural framing of the building, particularly if added after construction, may not have a designated lateral-force-resisting system or may have a minimal connection of the penthouse framing to the framing of the structure below. For the penthouse framing to be considered an extension of the structural framing, the columns from the structural framing must be continuous up through the height of the penthouse so that they are considered fixed at the roof level. It is important to verify that the penthouse has a lateral-force-resisting system in each direction with a complete load path for forces from the penthouse to the structural framing. Rooftop structures, such as screens used to shield mechanical and electrical equipment need not be evaluated for seismic loads but may need to be evaluated for wind and snow loads.

A.7.8.6 Tile Roofs *For roofs with slopes greater than or equal to 3 vertical to 12 horizontal, heavy roof tiles weighing more than 4 lb/ft^2 (5.9 kg/m^2) are individually secured to the roof framing or substrate with wires, fasteners, or adhesive.*

Roof tiles, such has clay, concrete, or slate, are heavy and can be dislodged owing to earthquake shaking. If not individually secured, loose tiles can fall off the roof and cause a safety hazard to occupants exiting the building. Standard details using wire ties, nails, or adhesives can effectively hold individual tiles in place. Chapter 15 of the *International Building Code* (ICC 2018) provides requirements for the types of fasteners and the anchorage requirements. These requirements are based on wind loads but are considered to be adequate for anchorage of tiles for seismic forces. An occasional loose tile is not a substantial safety risk; however, where large sections of tiles are unanchored, a substantial Life Safety hazard could exist. Deterioration of the wire ties or nails is a concern, particularly for older structures, and the condition of the fasteners should be evaluated.

A.7.9 Masonry Chimneys

A.7.9.1 Unreinforced Masonry Chimneys *Unreinforced masonry chimneys extend above the roof surface no more than the following: for Life Safety in low or moderate seismicity, three times the least dimension of the chimney; for Life Safety in high seismicity and for Position Retention in any seismicity, two times the least dimension of the chimney.*

Unreinforced masonry chimneys are highly vulnerable to damage in earthquakes. Typically, chimneys extending above the roof more than twice the least dimension of the chimney crack just above the roof line and become dislodged. Chimneys may fall through the roof or onto a public or private walkway, creating a Life Safety hazard. Experience has shown that the costs of retrofitting masonry chimneys can sometimes exceed the costs of damage repair.

A.7.9.2 Anchorage *Masonry chimneys are anchored at each floor level, at the topmost ceiling level, and at the roof.*

Anchorage of chimneys has proven to be problematic at best and ineffective at worst in reducing chimney losses because anchorage alone does not typically account for incompatibility of deformations between the main structure and the chimney. Other retrofit strategies—such as the presence of plywood above the ceiling or on the roof to keep the falling masonry from penetrating or relocating occupant activities within a falling radius—may be more effective than anchoring chimneys.

A.7.10 Stairs

A.7.10.1 Stair Enclosures *Hollow-clay tile or unreinforced masonry walls around stair enclosures are restrained out of plane and have height-to-thickness ratios not greater than the following: for Life Safety in low or moderate seismicity, 15-to-1; for Life Safety in high seismicity and for Position Retention in any area, 12-to-1.*

Hollow-tile or unreinforced masonry walls may fail and block stairs and corridors. Post-earthquake evacuation efforts can be severely hampered as a result.

The procedures in Chapter 13 are recommended for analysis of the walls for both in-plane and out-of-plane forces. If bracing is nonexistent, mitigation may be necessary to achieve the selected performance level.

A.7.10.2 Stair Details *The connection between the stairs and the structure does not rely on post-installed anchors in concrete or masonry, and the stair details are capable of accommodating the drift calculated using the Quick Check procedure of Section 4.4.3.1, for moment-frame structures or 0.5 in. (13 mm) for all other structures without inducing any lateral stiffness contribution from the stairs.*

If stairs are not specially detailed to accommodate story drift, they can modify structural response by acting as struts attracting seismic force. Shallow anchors, such as expansion and sleeve anchors, rigidly connect the stairs to the structure. The connection of the stair to the structure must be capable of resisting the imposed forces without loss of gravity support for the stair.

A.7.11 Building Contents and Furnishing

A.7.11.1 Industrial Storage Racks *Industrial storage racks or pallet racks more than 12 ft (3.6 m) high meet the requirements of ANSI/RMI MH 16.1 (RMI 2008) as modified by ASCE 7, Chapter 15.*

Storage racks are usually constructed of metal. Storage racks are generally purchased as proprietary systems installed by a tenant and are often not under the direct control of the building owner. Thus, they are usually not part of the construction contract and often have no foundation or foundation attachment. However, they are often permanently installed, and their size and loaded weight make them an important hazard to life, property, or the surrounding structure.

A.7.11.2 Tall Narrow Contents *Contents more than 4 ft (1.2 m) high with a height-to-depth or height-to-width ratio greater than 3-to-1 are anchored to the floor slab or adjacent structural walls. A height-to-depth or height-to-width ratio of up to 4-to-1 is permitted when only the basic nonstructural component checklist is required by Table 3-2.*

Tall, narrow storage or file cabinets or racks can tip over if they are not anchored to resist overturning forces. Commercial kitchen equipment, such as freezer boxes, refrigerators, ovens, and storage racks, can be overturned if not properly fastened to adjacent structural walls and floors.

A.7.11.3 Fall-Prone Contents *Equipment, stored items, or other contents weighing more than 20 lb (9.1 kg) whose center of mass is more than 4 ft (1.2 m) above the adjacent floor level are braced or otherwise restrained.*

Contents heavier than 20 lb (9.1 kg) that are elevated more than 4 ft (1.2 m) above the floor level can fall from where they are located and be a potential Life Safety concern in earthquakes with strong ground shaking. That is why these types of contents should be braced or restrained, such as being placed in a cabinet with doors that latch in buildings located in a region of high seismicity.

A.7.11.4 Access Floors *Access floors more than 9 in. (229 mm) high are braced.*

Unbraced access floors can collapse onto the structural slab. Small areas of unbraced floors "captured" on all sides within full-height walls may be acceptable; however, the impact of ramps and/or other access openings should be considered in evaluating the adequacy of such unbraced access floors.

If bracing is nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.11.5 Equipment on Access Floors *Equipment and computers supported on access floor systems are anchored or braced to the structure independent of the access floor.*

Tall, narrow computers and communications equipment can overturn if not properly anchored. Where overturning is not a concern because of the aspect ratio of the equipment, and it is desirable to provide some isolation between the equipment and the structure, it may be acceptable to support the equipment on a raised floor without positive restraint. In this case, the consequences of equipment movement should be considered. Tethering or some other form of restraint may be appropriate for limiting the range of movement.

If anchorage is nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.11.6 Suspended Contents *Items suspended without lateral bracing are free to swing from or move with the structure from which they are suspended without damaging themselves or adjoining components.*

Suspended contents generally do not present a hazard unless they affect something else during seismic shaking.

A.7.12 Mechanical and Electrical Equipment

A.7.12.1 Emergency Power *Equipment used to power or control Life Safety systems is anchored or braced.*

Protection of the emergency power system is critical to post-earthquake recovery, and proper mounting of the components of the system is needed for reliable performance.

Nonemergency equipment located close to or above emergency equipment can be dislodged and fall onto, or cause piping to fail and flood out of, the emergency system.

If anchorage is nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.12.2 Hazardous Material Equipment *Equipment mounted on vibration isolators and containing hazardous material is equipped with restraints or snubbers.*

Heating, ventilating, and air conditioning (HVAC) or other equipment containing hazardous material on vibration isolation supports that are not restrained by snubbers may release their contents during an earthquake.

A.7.12.3 Equipment Support Deterioration *There is no evidence of deterioration, damage, or corrosion in any of the anchorage or supports of mechanical or electrical equipment.*

Damaged or corroded anchorage or supports of equipment may not have adequate capacity to resist seismic demands. Suspended or wall-mounted equipment is of more concern than floor- or roof-mounted equipment because failure of supports would create a falling hazard.

A.7.12.4 Fall-Prone Equipment *Equipment weighing more than 20 lb (9.1 kg) whose center of mass is more than 4 ft (1.2 m) above the adjacent floor level, and which is not in-line equipment, is braced.*

Equipment located more than 4 ft (1.2 m) above the floor poses a falling hazard unless it is properly anchored and braced. Suspended equipment is more susceptible to damage than floor-, roof-, or wall-mounted equipment. Unbraced suspended equipment can sway during an earthquake, causing damage on impact with other adjacent items.

If bracing is nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.12.5 In-Line Equipment *Equipment installed in line with a duct or piping system, with an operating weight more than 75 lb (34.0 kg), is supported and laterally braced independent of the duct or piping system.*

Pieces of equipment, such as large variable air volume (VAV) boxes, which are installed in line with distribution system components such as ducts or piping, can become falling hazards if they are not independently braced. It is common for these pieces of equipment to instead be supported by the piping or ducts with which they are in line and to which they are attached.

A.7.12.6 Tall Narrow Equipment *Equipment more than 6 ft (1.8 m) high with a height-to-depth or height-to-width ratio greater than 3-to-1 is anchored to the floor slab or adjacent structural walls.*

Tall, narrow equipment can tip over if not anchored to resist overturning forces.

A.7.12.7 Mechanical Doors *Mechanically operated doors are detailed to operate at a story drift ratio of 0.01.*

Doors that are stuck open or closed, such as fire house garage doors, can greatly affect essential services. Most large doors are not designed to accommodate earthquake-induced transient or permanent drifts in flexible buildings. Fire trucks and ambulances can be delayed in exiting. Critical minutes of emergency response time have been lost in past earthquakes when such doors have been rendered inoperable. Energy conservation measures and vandalism concerns have resulted in an evolution in modern door system designs. Most common door designs are drift intolerant and can result in egress difficulties in flexible buildings, requiring contingency planning and in many cases retrofits. Simple visual evaluations of drift incompatibility between doors that are critical to essential services, their frames, and supporting structures can quickly identify vulnerabilities.

A.7.12.8 Suspended Equipment *Equipment suspended without lateral bracing is free to swing from or move with the structure from which it is suspended without damaging itself or adjoining components.*

Suspended equipment generally does not present a hazard unless it impacts something else during seismic shaking.

A.7.12.9 Vibration Isolators *Equipment mounted on vibration isolators is equipped with horizontal restraints or snubbers and with vertical restraints to resist overturning.*

Many isolation devices for vibration-isolated equipment (e.g., fans or pumps) offer no restraint against lateral movement. As a result, earthquake forces can cause the equipment to fall off its isolators, usually damaging interconnected piping. Snubbers or other restraining devices are needed to prevent horizontal movement in all directions.

Seismic restraints or snubbers must have proper anchors to prevent pullout. The contact surfaces on the snubbers should be resilient to prevent impact amplification.

If restraints and snubbers are nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.12.10 Heavy Equipment *Floor-supported or platform-supported equipment weighing more than 400 lb (181.4 kg) is anchored to the structure.*

For rigidly mounted large equipment (e.g., boilers, chillers, tanks, or generators), inadequate anchorage can lead to horizontal movement. Unanchored equipment, particularly equipment with high aspect ratios such as all tanks, may overturn and/or move and damage utility connections. Performance generally is good when positive attachment to the structure is provided.

If bracing is nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.12.11 Electrical Equipment *Electrical equipment is laterally braced to the structure.*

Without proper connection to the structure, electrical equipment can move horizontally and/or overturn. The movement can damage the equipment and may create a hazardous condition. Equipment may be mounted to the primary structural system or on walls or ceilings that are capable of resisting the applied forces. Distribution lines that cross structural separations should be investigated. If relative movement of two adjacent buildings can be accommodated by slack in the distribution lines, the condition may be acceptable.

If attachment is nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.12.12 Conduit Couplings *Conduit greater than 2.5 in. (64 mm) trade size that is attached to panels, cabinets, or other equipment and is subject to relative seismic displacement has flexible couplings or connections.*

Conduit rigidly attached to electrical equipment can be damaged at the junction where it attaches to the equipment because of differential movement of the conduit and the equipment. Providing a flexible coupling or connection capable of accommodating the relative displacement mitigates this issue.

A.7.13 Piping

A.7.13.1 Fire Suppression Piping *Fire suppression piping is anchored and braced in accordance with NFPA 13.*

Fire sprinkler piping has performed poorly in past earthquakes, rendering systems unusable when most needed. Causes of fire sprinkler piping failure included inadequate lateral bracing of sprinkler mains and cross mains, inadequate flexibility and clearance around sprinkler piping, and impact between sprinkler pipes and other unbraced nonstructural elements. Proper pipe bracing is needed for reliable performance of the system. NFPA 13 is intended to provide Operational Nonstructural Performance.

If anchorage and bracing are nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.13.2 Flexible Couplings *Fluid, gas, and fire suppression piping have flexible couplings. For fire suppression piping, the couplings are in accordance with NFPA 13.*

Failures may occur in pipes that cross seismic joints because of differential movement of the two adjacent structures. Special detailing is required to accommodate the movement. Flexibility can be provided by a variety of means, including special couplings and pipe bends. Flexible couplings should be evaluated for their ability to accommodate expected seismic movements in all directions. NFPA 13 is intended to provide Operational Nonstructural Performance.

If flexible couplings are nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.13.3 Sprinkler Ceiling Clearance *Penetrations through panelized ceilings for fire suppression devices provide clearances in accordance with NFPA 13.*

A common failure of fire suppression piping is caused by the sprinkler heads impacting the ceiling where the sprinkler pokes down through. This problem can be mitigated by providing clearance around the sprinkler head or by providing flexible lines between the horizontal pipe and the sprinkler head.

A.7.13.4 Fluid and Gas Piping *Fluid and gas piping is anchored and braced to the structure to prevent or limit spills or leaks.*

Piping can fail at elbows, tees, and connections to supported equipment. The potential for failure is dependent on the rigidity, ductility, and expansion or movement capability of the piping system. Joints may separate and hangers may fail. Hanger failures can cause progressive failure of other hangers or supports. Smaller diameter pipes, which generally have greater flexibility, often perform better than larger-diameter pipes, but they are still subject to damage at the joints. Piping in vertical runs typically performs better than in horizontal runs if it is regularly connected to a vertical shaft.

When using flexible couplings, the following limitations should be considered:

- Metal flexible couplings can resist bending only;
- Ball joints can resist bending and torsion;
- Grooved couplings can resist only minimum bending and torsion; and
- Some building codes permit certain configurations and size of piping without bracing or anchorage. It may be possible to demonstrate compliance by showing that the piping meets current code requirements.

If anchorage and bracing are nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.13.5 C-Clamps *One-sided C-clamps that support piping greater than 2.5 in. (64 mm) in diameter are restrained.*

Unrestrained C-clamps (such as those connected to the bottom flange of structural steel beams) have proven to be unreliable during an earthquake. Pipe movement can cause the C-clamp to work itself off its support, causing local loss of gravity support for the pipe. The loss of a single C-clamp can lead to progressive collapse of other supports.

If C-clamps are noncompliant, mitigation is necessary to achieve the selected performance level.

A.7.13.6 Piping Crossing Seismic Joints *Piping that crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements.*

Because of the potential for portions of a building on either side of a seismic joint or isolation plane to move relative to each other, any piping that crosses the joint should have been detailed to accommodate whatever movement is anticipated across the joint. The same condition exists when the piping is supported by different structures that are independent of each other. If the piping does not have flexible couplings or other means to accommodate the movement, the pipe can be damaged such that it releases its contents.

A.7.14 Ducts

A.7.14.1 Stair and Smoke Ducts *Stair pressurization and smoke control ducts are braced and have flexible connections at seismic joints.*

Because these ducts are part of the fire protection system, they are more critical than normal air conditioning ducts. Depending on the duct layout and function of the building, however, the hazard may vary greatly and judgment should be exercised during the evaluation.

If bracing or flexible connections are nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.14.2 Duct Bracing *Rectangular ductwork larger than 6 ft^2 (0.56 m^2) in cross-sectional area and round ducts larger than*

28 in. (711 mm) in diameter are braced. The maximum spacing of transverse bracing does not exceed 30 ft (9.2 m). The maximum spacing of longitudinal bracing does not exceed 60 ft (18.3 m).

Large duct installations are heavy and can cause damage to other materials and may pose a hazard to occupants. Failures may occur in long runs because of large-amplitude swaying. Failure usually consists of leakage rather than collapse.

When evaluating the ductwork, the function of the duct system, proximity to occupants, and other materials likely to be damaged should be considered.

If bracing is nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.14.3 Duct Support *Ducts are not supported by piping or electrical conduit.*

Although generally undesirable, this condition is only serious when large ducts are supported by other elements that are poorly supported and braced.

A.7.14.4 Ducts Crossing Seismic Joints *Ducts that cross seismic joints or isolation planes or are connected to independent structures have couplings or other details to accommodate the relative seismic displacements.*

Because of the potential for portions of a building on either side of a seismic joint or isolation plane to move relative to each other, any ducts that cross the joint should have been detailed to accommodate whatever movement is anticipated across the joint. The same condition exists when the ducts are supported by different structures that are independent of each other. If the ducts do not have flexible couplings or other means to accommodate the movement, the ducts can be damaged to the point where they do not function.

A.7.15 Hazardous Materials

A.7.15.1 Hazardous Material Storage *Breakable containers that hold hazardous material, including gas cylinders, are restrained by latched doors, shelf lips, wires, or other methods.*

Unrestrained containers are susceptible to overturning and falling, resulting in release of materials. Storage conditions should be evaluated in relation to the proximity to occupants, the nature of the substances involved, and the possibility of a toxic condition.

If restraints are nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.15.2 Shutoff Valves *Piping containing hazardous materials has shutoff valves or other devices to prevent major spills or leaks.*

Post-earthquake recovery efforts are hampered if toxic releases cannot be promptly stopped. Shutoff valves should be accessible, and training should be provided to enhance the reliability of post-earthquake recovery efforts. The specifics of the materials and systems vary greatly. Federal, state, and local codes govern regarding the installation of shutoff devices.

Large spills of some nonhazardous materials, such as liquid soap or some food products, also can be environmentally damaging and can create a nuisance. Proper shutoff valves and containment structures can help to avert these problems.

If shutoff devices are nonexistent, mitigation is necessary to achieve the selected performance level. The need for and location of shutoff devices should be established in cooperation with local utility companies. Utility companies vary in their policies regarding the installation of shutoff devices.

A.7.15.3 Shutoff Valves *Piping containing hazardous material, including natural gas, has shutoff valves or other devices to limit spills or leaks.*

Post-earthquake recovery efforts have been severely hampered in cases where damaged utility lines could not be expediently isolated from main distribution systems. Shutoff valves are needed to allow for isolation of a building or portions of a building. The valves should be easily accessible, and training should be provided for reliable post-earthquake response.

Shutoff valves can be either manually operated or automatic. Automatic shutoff valves should conform to ASCE 25-97 (1999). Manually operated valves should conform to ASME B16.33 (2012) or ANSI Z21.15 (2019).

If shutoff devices are nonexistent, mitigation is necessary to achieve the selected performance level. The need for and location of shutoff devices should be established in cooperation with local utility companies. Utility companies vary in their policies regarding the installation of shutoff devices.

A.7.15.4 Flexible Couplings *Hazardous material ductwork and piping, including natural gas piping, have flexible couplings.*

Failures may occur in pipes that cross seismic joints because of differential movement of the two adjacent structures. Special detailing is required to accommodate the movement. Flexibility can be provided by a variety of means, including special couplings and pipe bends. Flexible couplings should be evaluated for their ability to accommodate expected seismic movements in all directions.

If flexible couplings are nonexistent, mitigation is necessary to achieve the selected performance level.

A.7.16 Elevators Elevator components are typically not dealt with by design professionals. If necessary, a design professional with experience in elevator design should be consulted.

A.7.16.1 Retainer Guards *Sheaves and drums have cable retainer guards.*

Strong earthquake motions cause the elevator hoistway cables to whip around and often misalign on the sheaves and drums. Retainer guards are effective at reducing the number of misalignments and improving the possibility that the elevator can continue in service after inspection.

A.7.16.2 Retainer Plate *A retainer plate is present at the top and bottom of both car and counterweight.*

Retainer plates are installed just above or below all roller guides and serve to prevent derailment. They are U-shaped, firmly attached to the roller guides, and run not more than 3/4 in. (19 mm) from the rail.

A.7.16.3 Elevator Equipment *Equipment, piping, and other components that are part of the elevator system are anchored.*

The successful performance of an elevator system requires that the various elements of the system remain in place, undamaged, and capable of operating after inspection. As a minimum, all equipment, including hoistway doors, brackets, controllers, and motors, must be anchored.

A.7.16.4 Seismic Switch *Elevators capable of operating at speeds of 150 ft/min (0.30 m/min) or faster are equipped with seismic switches that meet the requirements of ASME A17.1 or have trigger levels set to 20% of the acceleration of gravity at the base of the structure and 50% of the acceleration of gravity in other locations.*

Traction elevators, unless carefully designed and constructed, are highly vulnerable to damage during strong shaking. It is very common for the counterweights to swing out of their rails and collide with the car. Current industry practice and most elevator regulations ensure that the elevator occupants remain safe by installing seismic switches that sense when strong shaking has begun and automatically shut down the system. Seismic switches are generally located in the elevator machine room and are connected directly to the controller. The design professional should verify that the switch is operational, as they are often disabled because of malfunctioning.

A.7.16.5 ***Shaft Walls*** *Elevator shaft walls are anchored and reinforced to prevent toppling into the shaft during strong shaking.*

Elevator shaft walls are often unreinforced masonry construction using hollow-clay tile or concrete masonry block. In the event of strong shaking, these walls may experience significant damage caused by in-plane and out-of-plane forces and may fall into the shaft.

A.7.16.6 ***Counterweight Rails*** *All counterweight rails and divider beams are sized in accordance with ASME A17.1.*

The typically poor performance of counterweights is caused by the size of the rails and the spacing of the rail brackets. Eight-pound [8 lb (3.6 kg)] rails have routinely shown to be insufficient and are best replaced by 15 lb (6.8 kg) rails as a minimum.

A.7.16.7 ***Brackets*** *The brackets that tie the car rails and the counterweight rail to the structure are sized in accordance with ASME A17.1.*

The brackets that support the rails must be properly spaced and designed to be effective. It is common for brackets to be properly spaced but improperly designed. The design professional should be particularly aware of the eccentricities that often occur within the standard bracket systems most commonly used.

A.7.16.8 ***Spreader Bracket*** *Spreader brackets are not used to resist seismic forces.*

Spreader brackets are a useful element to maintain alignment of counterweight rails between supporting brackets. They have worked successfully under normal daily operating loads. However, they do not offer any protection to the rails under seismic loading because of the large eccentricities inherent in their shape.

A.7.16.9 ***Go-Slow Elevators*** *The building has a go-slow elevator system.*

The functionality of a building after an earthquake depends on the ability to move through it. However, elevators that are compliant with the code shut down after an earthquake. Therefore, even if the building has the ability to provide Immediate Occupancy after an earthquake, movement through the building is impeded until the elevators are reactivated. Go-slow elevators alleviate this problem by providing one elevator that functions at a lower speed after an earthquake.

APPENDIX B

APPLYING ASCE 41 IN BUILDING CODES, REGULATORY POLICIES, AND MITIGATION PROGRAMS

B.1 INTRODUCTION

This appendix discusses issues related to the ASCE 41 standard that are outside the scope of its technical provisions. The specification of a performance objective sets both the expected level of seismic performance and the seismic hazard in which it is to be achieved. There may be multiple performance objectives set for an analysis that are each to be satisfied. Different contexts lead to different conclusions for each issue's resolution. The standard can be applied for evaluation and/or mitigation programs for code-specified work or for voluntary efforts. The performance objectives can be the target for specific building types or occupancies. It is noted that in most of the country, mitigation is most commonly done either voluntarily or when triggered by the local building code or by other proposed actions. These variations call for different considerations when selecting a performance objective and applying the standard.

As described in Chapter 2, ASCE 41 accommodates a number of possible performance objectives. The performance objective, together with attributes of the site and the building, determines the applicable provisions for evaluation or retrofit. Thus, the first task for the decision maker applying the standard is to select a performance objective, and the second is to select the hazard level for which the performance is to be evaluated or retrofitted.

This standard does not specify a performance objective, but it provides the means to do so by selection of the intended structural and nonstructural performance levels and does not establish the Seismic Hazard Levels at which the performance level(s) are to be evaluated. The commentary provides some basis for understanding the differences. The purpose of this appendix is to describe how these objectives can be set, with reference to existing programs and precedents. This appendix references specific codes, jurisdictions, programs, and practices for illustration purposes only. No endorsement or critique is implied.The intent is to provide some general guidance in their selection to code developers, policy makers, building owners, and other stakeholders.

An evaluation and/or mitigation program can involve a single building, a portfolio or class of buildings, or an entire community of buildings and infrastructure. Seismic evaluation and retrofit of individual buildings, the subjects of this standard, is a key component of many programs, but a full program might also include other tasks, for example, financing, capital planning, legislation, or enforcement. These other tasks, though often essential to the success of a mitigation program, are within the scope for application of this standard and appendix.

Mitigation programs and regulations can vary in purpose, scope, duration, and in other ways. This appendix classifies them primarily by whether the mitigation is

- Mandatory, generally through a specific law or ordinance;
- Voluntary, at the discretion of one or more building stakeholders; or
- Triggered under certain conditions by a building code or by a regulation or policy of the Authority Having Jurisdiction.

The process and rationale for selecting a performance objective and applying the standard vary with the type of mitigation. Additional considerations—generally waivers or relaxed criteria—often apply to designated historic buildings, as noted briefly in the following sections. Commentary Section C1.1 discusses the application of the standard to historic buildings in more general terms.

The standard may be used for evaluations entirely separated from the enforcement of building codes or planning for structural modifications. These applications may include the following:

- Suitability for lease and/or occupancy providing a stated level of seismic performance, including for occupant safety and continuity of operations, or protection of key contents; and
- Financial decisions that are centered on understanding the expected seismic performance of the building and its sustainability of rents and revenues.

The latter applications may be triggered by ASTM E2557 or E2026 as evaluative methods for anticipating the seismic hazards and financial risks posed by the building.

B.2 MANDATORY MITIGATION

Mandatory mitigation is mitigation required by specific legislation regardless of the intentions of the building owner (or other stakeholders). Where mitigation is mandated, the ASCE 41 standard (or other engineering criteria) can be invoked by the legislation directly or by referenced regulations.

Mandatory mitigation has been used most often to target specific groups of buildings that are evaluated by the legislative body to unacceptable current extreme or urgent risks, especially where voluntary or triggered mitigation has been slow or ineffective from the perspective of public policy makers in reducing the community's seismic risk.

- In some cases, the urgency is related to safety and the likelihood of life-threatening structural collapse; the classic example is the case of unreinforced masonry buildings, or portions thereof, for example, parapets, in California. Other similarly hazardous conditions could, in some jurisdictions, pose risks that might warrant mandatory mitigation. These conditions might include certain concrete tilt-up structures, nonductile concrete structures, or even certain nonstructural components such as gas-fired equipment or brick chimneys. Examples include evaluation and mitigation of nonductile concrete moment-frame buildings in Los Angeles and the Orange County requirements for assessment and retrofit of some types of concrete tilt-up structures.

- In other cases, the urgency is related to essential post-earthquake services, regardless of structure type, such as those provided by hospitals, fire stations, and emergency operations centers.
- Legislation has also been proposed to target buildings that are neither historic collapse risks nor essential facilities, but which, as a group, are expected to be critical to a community's post-earthquake recovery. Programs addressing soft-story, multiunit residential buildings are examples.

Almost all communities have regulations charging the building official to mitigate hazardous buildings. Often, the determination of when a building is hazardous is not clearly stated, nor are definitive means given for verifying it is hazardous. Usually this designation is determined based on performance under gravity loads. Occasionally, a jurisdiction may want to allow voluntary structural modifications of the seismic performance of a building without invoking other code requirements. Usually, the notion is that as long as the seismic hazard is not increased from what it was before, the alterations are allowed on a voluntary basis. Thus, highly hazardous buildings can be modified as long as the seismic hazard has not been increased. ASCE 41 provides a method by which a jurisdiction could set a standard of seismic performance for a modified building to qualify for voluntary structural modifications, in which it becomes a mandatory use, not a voluntary provision. In other cases, the jurisdiction could prequalify use of ASCE 41 as acceptable, where it becomes permissive. One could be that the modified building could be determined to meet an S-5 performance level (Collapse Prevention) in a specified earthquake ground motion, say, the BSE-1E or other earthquake ground motion threat that has a risk level that the community evaluates as unacceptable. Use of ASCE 41 in this process would allow the building to be assessed easily as Compliant through successive application of the tiers until it is confirmed that the performance objectives are met, and if not, to provide a means of mitigating the hazard without invoking a full building performance evaluation. Such applications would probably be used only in High or Moderate seismic hazard locations (Table 2-5) and/or buildings not meeting the threshold ages of Table 4-7, and/or buildings well known to pose high life safety hazards in past earthquakes within the community, say URM load-bearing buildings and tilt-ups with deficient roof-to-wall connections and/or nonductile concrete-framed buildings.

B.2.1 Performance Objectives Because mandatory mitigation is driven by legislation, the stated purpose of the law or ordinance will usually suggest a suitable performance objective. Mandatory mitigation represents legislated public policy. As such, even though mitigation is performed through individual projects, building by building, the program's overall success is measured at the jurisdiction level. The appropriate performance objective is thus the one that, when applied to all subject buildings, results in the desired improvement for the jurisdiction as a whole. This perspective distinguishes mandatory mitigation from voluntary or triggered mitigation, which both deal primarily with individual buildings.

Where public safety is the primary concern, the standard's Life Safety Performance Level is often appropriate. The Life Safety structural and nonstructural provisions were developed to support programs focused on the safety of persons, as opposed to programs seeking to minimize repair cost or downtime. Additional considerations when selecting a safety-based performance objective include the following:

- Life Safety performance is traditionally paired with a hazard somewhat less than that required for new construction, such as the BSE-1E hazard. As discussed in Section C2.4.1, use of this lower hazard recognizes that achieving "code equivalent" performance with an obsolete structure type is often disproportionately expensive and disruptive; for mandated mitigation, this issue can affect the political viability of a proposed program. Nevertheless, if equivalence with new buildings is sought, a performance objective of Life Safety Structural Performance Level and Position Retention Nonstructural Performance Level in the BSE-1N earthquake might be more suitable (Section C2.4.4.)
- The standard's Basic Performance Objectives for Existing Buildings Tiers 1 and 2 have a single-level required assessment (Sections C2.4.1 and C2.4.4). Tier 3 has two levels of assessments, one of which considers performance at the BSE-2E or BSE-2N hazard level. Although use of the higher hazard level can distinguish robust performance from marginal performance at the lower BSE-1E or BSE-1N hazard level, it can also substantially increase the level of evaluation or design effort. Most mandatory mitigation programs have not used a two-part objective. This approach is consistent in principle with the standard, in which acceptable Tier 1 evaluation considering the BSE-2E hazard is deemed to comply with a corresponding performance under the BSE-1E hazard (Section C2.4.1). However, these mitigation programs may not have the same limitations as the Tier 1 procedure does; therefore, they may not provide the intended performance in the BSE-2E hazard without explicit consideration at that hazard level.
- Where the goal of the mandate is to remove the most egregious life-threatening conditions with the least expense and disruption, Collapse Prevention structural performance in the BSE-1E or BSE-1N earthquake might be appropriate. Note, however, that ASCE 41 does not provide Tier 1 evaluation criteria for Collapse Prevention performance. The standard's committee expects to develop such criteria in a future revision cycle. In the interim, Tier 1 Collapse Prevention evaluation criteria can be derived from the Life Safety criteria by extracting the checklist items and other relevant provisions that focus on the most egregious potential deficiencies.
- Where the legislation targets a specific structure type, nonstructural performance might be reasonably ignored. The standard's separate enumeration of structural and nonstructural performance levels supports such an approach. Similarly, where the targeted deficiency involves a specific nonstructural deficiency (such as an unbraced brick parapet or gas-fired equipment), an objective that ignores structural performance might be reasonable.

Where post-earthquake functionality is the primary concern, the standard's Immediate Occupancy Structural Performance Level and Operational Nonstructural Performance Level might be appropriate. These Performance Levels were developed to support programs focused on maintaining building services in the immediate post-earthquake period. Additional considerations are the following:

- As with safety-based mandates, functionality-based mandates often pair Immediate Occupancy performance with a reduced Seismic Hazard Level like BSE-1E (Section C2.4.1). For the most essential facilities, however, the deference to practicality represented by the use of a reduced hazard might not be warranted. A performance

objective involving the BSE-1N and/or the BSE-2N hazard might be more appropriate for mandating legislation that seeks equivalence with new buildings (Section 2.4.4).
- As described in Section C2.2.2.1, the standard does not provide a full set of evaluation or retrofit criteria for Operational Nonstructural performance, which relies in part on the performance of infrastructure and utilities external to the building. In some cases, or for some components or systems, the standard's Position Retention nonstructural criteria might be adequate. In Section 2.4.1, for example, the standard's basic performance objective for existing buildings (BPOE) calls for Position Retention nonstructural performance in the BSE-1E earthquake even for buildings assigned to Risk Category IV. In general, however, nonstructural performance is important for functionality-based objectives and should not be ignored.

Where the mandating legislation has other goals, appropriate performance objectives can be customized from the standard's defined performance and hazard levels.

- The Structural (S-1 to S-5) and Nonstructural (N-A to N-D) Performance Levels and the freedom to specify the evaluation Seismic Hazard Levels provide a broad range of opportunities to specify performance by triples of S-, N-, and seismic hazard. At times, these may include any number of triples. For example, the owner may want (S-1, N-A) performance in a magnitude 6 earthquake on the Hayward Fault, (S-3, N-B) performance in a magnitude 7 earthquake on the San Andreas Fault, and (S-4, N-C) performance in a magnitude 8 earthquake on the San Andreas Fault. ASCE 41 provides a way to systematically address such seismic performance objectives in ways that are not related to code enforcement.
- It should be noted that the standard can be used both as an acceptance standard or as a nonacceptance standard for actions outside the regulatory purview, for example, where a lease is anticipated and the occupants want to have a reasoned understanding that the seismic risks of occupancy are acceptable to them. Then an ASCE 41 evaluation that indicates a building does not achieve an S-5 (Collapse Prevention) or S-3 (Life Safety) in a prescribed seismic hazard gives clear guidance to the occupants of whether they are at risk or not in executing a lease for use of the property. The prescribed hazard could be the BSE-1R, the ground motion in a specific scenario earthquake, or a ground motion with a 10% probability of exceedance in terms of the lease. Similarly, a tenant may be interested in the possibility of not being able to use the property for its intended purposes during a lease and would want an S-2, NB in a ground motion with a 10% probability of exceedance in the terms of the lease. Such could be completed at the tenant's initiative or requested of the owner as a condition of considering leasing the building. The opportunities to use the ASCE 41 performance evaluation approach for other than capital investment or public standards enforcement are only limited by the need of the user in evaluating real estate for commercial, industrial, or personal goals.

Many owners developing a new building may want seismic performance requirements that are not well achieved by setting the ASCE 7 Importance Factor, I_e, higher. In such cases, the owner could require of the design team both meeting the minimum requirements of the applicable ASCE 7-based code and then evaluating the performance using ASCE 41 stated performance objectives and, if needed, requiring design modifications to meet these performance goals. This can be particularly useful for setting higher goals for nonstructural element performance and applying it to be more inclusive of elements not regulated by the code as mandatory. ASCE 41 is a convenient manner to achieve these objectives, because it is graded in its performance measures for both structural and nonstructural elements. This hybrid approach to new development evaluation has been used for the development of several buildings by the University of California, San Francisco.

B.2.2 Implementation Issues Because mandatory mitigation is based in legislation, the legislative language (or subsequent regulations) must account for the logistics of a whole program. Program development issues related to the use of ASCE 41 might include the following:

- Phasing: The standard's tiered methodology enables the phased approach often used in mandatory mitigation programs. The evaluation could start with a Tier 1 or Tier 2 assessment and progress through the tiers until it is found that the building performs acceptably or until a decision is made to retrofit. The standard also allows separate performance objectives for evaluation and retrofit.
- Quality assurance: Legislated mandates by their nature involve enforcement, reviews, and approvals by jurisdiction staff. This method can require the development of procedures, as well as the training of staff.

B.2.3 Historic Buildings Whereas designated historic buildings are often afforded waivers or special consideration by building codes, some of those variances might not be appropriate in the case of mandatory mitigation. Where a public safety risk or the need for an essential facility is urgent enough to justify a legislated mandate, that urgency might be prioritized over the objectives of historic preservation. Nevertheless, where ASCE 41 is applied to historic buildings, legislation (or its implementing regulations) might allow for certain exceptions to the normal mandated compliance.

B.2.4 Example Programs The following example programs represent the diversity of seismic mitigation mandates. They cover both private and public buildings, local and statewide scope, evaluation-only programs as well as mandated retrofit, and a variety of regulatory approaches.

- California unreinforced masonry buildings. In 1986, California required local jurisdictions in high-seismicity areas to compile inventories and adopt mitigation programs for unreinforced masonry buildings. In most of the jurisdictions, including Los Angeles and San Francisco, the resulting programs involved mandatory retrofit. The evaluation and retrofit criteria varied, but many used criteria similar to the special procedure now found in Section 16.2, of this standard. These programs were administered by the local building departments of individual jurisdictions.
- California hospitals. In 1994, California required certain hospital facilities to be replaced or retrofitted or to have acute care services relocated to other buildings. As of 2012, evaluation criteria were added to Chapter 6 of the *California Building Standards Administrative Code* reprint portions of the ASCE 31-03 Tier 1 checklists. Chapter 34A of the *California Building Code* references ASCE 41-06 and ties compliance to certain performance objectives, with an emphasis on post-earthquake functionality. This program is administered by the state's Office of Statewide Health Planning and Development, a state agency dedicated to specific health-care-related occupancies.

- California courthouses. In 2002, California required seismic evaluations of most of its courthouse facilities as part of an intended transfer of facility management responsibility from counties to the state. The evaluation criteria used a customized version of the ASCE 31-03 Life Safety criteria. This program was administered by the state's Administrative Office of the Courts, the agency that would become the owner or manager of the transferred buildings.
- Oregon schools and emergency facilities. The 2016 Oregon State Seismic Rehabilitation Grant Program requires ASCE 41-13, with revised ground motions, to be used for all applications of the program. Tier 1 or 2 must be used for evaluation.
- The Los Angeles Municipal Code (2015) in October established mandatory standards for earthquake hazard reduction in existing nonductile concrete buildings. It references ASCE 41-13 for application as an approved alternative to meet the requirements of Division 91 for nonductile concrete frame buildings.

B.3 VOLUNTARY MITIGATION

Voluntary mitigation is mitigation undertaken at the discretion of a building owner or other stakeholder. It is sometimes driven by an owner's intent to anticipate a future mandate or triggered work. Mandatory or triggered evaluation sometimes leads to voluntary retrofit. There are a few subcategories of voluntary mitigation, and they affect how ASCE 41 (or other engineering criteria) is invoked and applied:

- Some voluntary mitigation is entirely owner driven. Often, voluntary mitigation is done as a single project, as in the case of a homeowner retrofitting a house. In other cases, the mitigation is done to comply with an institutional policy covering multiple properties through a coordinated program, for example, by a university, corporation, or government agency. Although a driving policy implies a requirement of sorts, the mitigation is still said to be voluntary with respect to the local building department or Authority Having Jurisdiction. That is, if the mitigation is *not* done, no law or ordinance has been violated. (Policies for voluntary mitigation can make use of triggers, as discussed in Section B.4. The federal government, for example, triggers voluntary mitigation when a new space is purchased or leased.)
- Some voluntary mitigation is driven by industry standards or by contractual relationships between parties. For example, an owner might perform mitigation to secure a loan, to satisfy requirements of a potential tenant, or to qualify for an insurance discount. In these cases, acceptability is subject to the approval of a party other than the owner, but the mitigation is still voluntary with respect to the Authority Having Jurisdiction.
- Some jurisdictions offer incentives to encourage mitigation. The mitigation is voluntary in that no owner is required to pursue the incentive, but if an owner intends to qualify, the logistics of the mitigation become similar to those of mandatory work, involving specific criteria, approvals, and quality assurance by the authority administering the incentive program.

B.3.1 Performance Objectives The variety of defined performance and hazard levels in ASCE 41 makes it well suited to voluntary mitigation. By its nature, voluntary mitigation is about choice, so almost any pairing of performance and hazard can make sense as a performance objective. The appropriateness of the selected objective is measured only by the desires or preferences of the parties. Table C2-6, however, recommends against some combinations of structural and nonstructural performance levels; it does not make sense, for example, to seek exceptional nonstructural performance (to minimize downtime, perhaps) while allowing extensive structural damage (which would shut down the building anyway).

Where safety is an urgent concern, mitigation is often the subject of legislative mandates, as discussed earlier. Some jurisdictions, however, might determine that the safety risk does not justify a jurisdiction-wide mandate. In these cases, the standard's safety-based objectives (as discussed in Section B.2) might be appropriate for voluntary mitigation. Otherwise, objectives that focus on reducing property losses or downtime might be appropriate.

Voluntary mitigation is further distinguished from mandatory or triggered mitigation because its optional nature requires no strict compliance with any prescribed criteria. That is, assuming that all stakeholders agree, the owner is free to emphasize certain provisions and ignore or undercomply with others. This approach can make sense where strict compliance with certain provisions would be especially difficult or impractical. Voluntary mitigation is often scoped based on an available budget or by a desire to avoid disruption to tenants or building services. It can thus make sense, for example, to retrofit an exceptionally soft or weak first story using Life Safety structural criteria, while allowing marginal Life Safety deficiencies in occupied upper stories. On the nonstructural side, a voluntary project might seek Position Retention performance as a general rule but ignore components that are inaccessible or easier to repair than to retrofit.

The selective nature of voluntary mitigation is both common and explicitly allowed by building codes. (See, for example, Section 3404.5 of the 2012 *International Building Code* or Section 807.6 of the 2012 *International Existing Building Code*.) However, it is not allowed, and does not make sense, where voluntary work would create a structural irregularity or an unbalanced condition that would reduce performance of the building as a whole.

Where the mitigation is done to qualify for an incentive provided by a jurisdiction, the performance objective represents a public policy, much as it does in the case of a legislated mandate. The objective and the engineering criteria for achieving it are spelled out in the ordinance or regulation that offers the incentive. As with mandatory mitigation, the performance objective should match the driving policy issue, which could be rooted in safety, recovery planning, community stability, or other concerns. Guidance for selecting an objective using ASCE 41 is therefore similar to that given in Section B.2.

B.3.2 Implementation Issues Voluntary mitigation often lacks the criteria-setting and procedural documentation (the ordinances and codes) of mandatory or triggered mitigation. This fact, together with the generally flexible nature of voluntary work, raises some implementation issues related to the use of ASCE 41:

- Quality assurance. Where there is no requirement to do the mitigation, there is no basis for a building department or authority to check the work. For voluntary evaluation, the authority probably does not even see the report. For voluntary retrofit, building permits are generally needed, but reviews of seismic design calculations and drawings are often limited to a check that no harm is being caused. The burden of quality assurance thus falls to those who set the performance objective. (This concern applies less in the case of a jurisdictional incentive program, where the agency offering the incentive is motivated to confirm the quality of the voluntary work.)

- Certification. One benefit of using a document like ASCE 41 is that work can be said to meet (or not meet) a defined standard. As discussed earlier, however, voluntary mitigation, even if it references the standard, often makes exceptions for itself for practical reasons. Although rational, this situation can make it difficult for an owner or engineer to certify full compliance or to state with clarity exactly what performance has been sought.
- Records and disclosures. Different jurisdictions have different requirements regarding public records and disclosures of building information. This issue can affect how parties choose to apply ASCE 41 (or other engineering criteria) and report findings, especially where the work involves only voluntary evaluation.

B.3.3 Historic Buildings Special considerations often made for designated historic buildings are within the spirit of voluntary mitigation, which already allows for practical variances and exceptions even to standard criteria such as those in ASCE 41.

B.3.4 Example Programs Thousands of voluntary retrofits are completed every year throughout the country. Most are owner-initiated improvements of individual buildings, and they range in scope from simple nonstructural mitigation (for example, bracing bookshelves or water tanks) to full structural retrofits. The following examples represent the types of voluntary programs described previously:

- Federal government facilities. Federal agencies follow internal policies based on the recommended practice known as *RP 8*, "Standards of Seismic Safety for Existing Federally Owned and Leased Buildings" (NIST 2012b). *RP 8* relies on ASCE 31-03 and ASCE 41-06 for its technical criteria. For most buildings, it sets performance objectives based on safety. Some agencies use *RP 8* as a supplement to the applicable building code (see Section B.4.4); others cite it as a guideline and apply it voluntarily. For example, some agencies apply *RP 8* when leasing or buying private buildings otherwise regulated by the local building department. Because most local codes do not require seismic evaluation upon lease or purchase, the agency's application of *RP 8* in these cases is entirely voluntary from the perspective of the local code official. Many state and municipal agencies have similar policies.
- Private sector due diligence. Private sector lenders and equity investors often require seismic loss estimates as a precondition for financing, especially for commercial buildings. Loss estimates may be performed using ASTM E2026 and E2557 standards, both of which include optional criteria that reference ASCE 41.
- Portland, Oregon, schools. In 2009, Portland Public Schools engaged a consultant to conduct evaluations and prepare preliminary retrofit designs and project cost estimates for 12 campuses. The project used ASCE 31-03 and ASCE 41-06 with safety-based performance objectives. Many institutions, public and private, conduct similar assessments to inform their emergency response and capital improvement plans.
- San Francisco wood-frame residential buildings. In 2009, San Francisco implemented an incentive program to encourage voluntary retrofit of certain residential buildings with soft or weak stories, which have a history of poor performance. Owners who complete a voluntary retrofit have fees waived and are exempt from future mandates for 15 years. The retrofit criteria include ASCE 41-06 with a performance objective of Life Safety Structural Performance Level with the BSE-1 hazard.

B.4 TRIGGERED MITIGATION

Triggered mitigation is mitigation required by a standing regulation, typically the building code, when certain qualifying or "triggering" conditions are met. ASCE 41 is sometimes invoked as the criteria for triggered work and is sometimes allowed as an option.

In triggered cases, seismic mitigation is generally not part of the building owner's intended work. Rather, it is required as a condition of permitting the intended project. For example, the *International Building Code* requires seismic structural evaluation, and possibly retrofit, when an addition, change of occupancy, or extensive repair is made.

In concept, triggered mitigation is a combination of mandatory and voluntary work. To the extent that an owner avoids a triggering condition, the triggered mitigation is voluntary. Once the trigger is pulled, however, the work proceeds as if mandatory.

Triggers in current model building codes are based on conditions already regulated by the code, such as an increase in load, a decrease in capacity, an expectation of performance, or a change of occupancy. Some local codes use cost-based triggers as well, requiring seismic evaluation or retrofit when the cost of an intended alteration, for example, exceeds a specified amount.

B.4.1 Performance Objectives Because triggered mitigation involves compliance with a building code provision, the code sets the trigger, the scope of triggered work, and the criteria for that work. Where ASCE 41 is allowed or specified as a criterion, the triggering code provision specifies a performance objective.

Generally, the performance objectives for triggered work follow the building code's practice of setting criteria based on risk category, with essential or high-occupancy facilities subject to more aggressive requirements. ASCE 41's various performance and hazard levels can accommodate this approach.

In some cases, when a code triggers mitigation, it seeks performance, or compliance, similar to what it requires of new construction. ASCE 41's basic performance objective equivalent to new building standards (BPON) in Section 2.4.4, is suitable for this purpose. However, if the code only calls for structural mitigation or only requires compliance at a single hazard level, some parts of the BPON might not be triggered. For example, where code-level mitigation is triggered, the 2012 *International Existing Building Code* allows the use of ASCE 41-06 with a two-part structural objective, but it makes no nonstructural requirements. (In ASCE 41 terms, the triggered Nonstructural Performance Level would be N-D, Not Considered.)

In other triggered cases, the model codes explicitly allow lesser performance. The 2012 *International Existing Building Code*, for example, allows retrofits triggered by repair projects to use either ASCE 31-03 or ASCE 41-06 with just the BSE-1 hazard. This method is akin to using ASCE 41 with just the BSE-1E hazard level. Thus, in these cases, the code's performance objective, unlike the BPON, involves only one hazard level, and that hazard is lower than the one that would be used for the design of similar new buildings. Also, nonstructural performance is again ignored by the triggering provision.

Local amendments to the model codes sometimes apply different performance objectives for certain classes of buildings.

B.4.2 Implementation Issues Where mitigation is triggered, it is subject to a jurisdiction's normal code enforcement practices.

In this way, triggered mitigation presents many of the same implementation issues as mandatory mitigation, including the need for quality assurance, approvals, and inspections. Other implementation issues associated with triggered mitigation include the following:

- Enforcement of local amendments. Triggered mitigation generally starts with the building code. To the extent that model codes are modified locally, some of the triggers, triggered scope, and triggered criteria might vary between jurisdictions.
- Coordination with other code provisions. Triggered mitigation is often done as part of another intended project, such as a major alteration or repair. The other work is likely to be subject to building code provisions for new construction or unrelated to earthquake design at all, so coordination with the criteria of ASCE 41, including resolution of conflicting provisions, is often needed.

B.4.3 Historic Buildings Building codes typically waive the triggers or relax the triggered criteria for designated historic buildings. See, for examples, Section 3409 of the 2015 *International Building Code* or Chapter 12 of the 2015 *International Existing Building Code*.

B.4.4 Example Programs As discussed, triggered mitigation is typically initiated through building code provisions that regulate other intended work, such as additions, alterations, repairs, or changes of occupancy. As described in Section B.3, some organizations have internal policies that are also based on triggers, but in terms of public policy, those institutional programs are voluntary.

In the United States, the leading model building codes are the *International Building Code* and the *International Existing Building Code*, both of which regulate existing buildings through code triggers. Where seismic evaluation or retrofit is triggered, these codes consider primarily structural performance. The IEBC is not, however, as widely adopted as the IBC. The 2015 IEBC, under Alterations—Level 2 (i.e., paragraph 807.5) and Level 3 (i.e., paragraph 907.4), includes requirements for evaluating the effect of some alterations on the existing lateral system even if the intended alteration is not "structural" or does not affect the lateral force system. If the results of the evaluation are "unfavorable," then the existing lateral system might have to be improved or retrofitted (i.e., depending on the specific requirements associated with the evaluation or the IEBC provision improved as required to meet the specific requirements associated with the IEBC provisions).

Where seismic evaluation or retrofit is triggered, the 2015 IEBC explicitly references ASCE 41-13 as containing potential criteria. A full description of the differences between the two model codes is beyond the scope of this appendix. One difference worth noting, however, is that in regions of relatively high seismicity, the IEBC triggers upgrades for unreinforced masonry parapets and concrete or masonry wall anchorage when buildings with these historic deficiencies are altered.

Where seismic evaluation or retrofit is triggered, as may occur in alteration projects, the 2015 IEBC (which is referenced by the 2015 IBC) can trigger seismic mitigation depending on the outcome of an evaluation of the effect of the intended alteration on the existing lateral system. Depending on the outcome of the evaluation, a "nonstructural upgrade (i.e., architectural, mechanical, etc.)" could also trigger required improvements to the lateral system.

Local codes sometimes supplement the model code triggers, especially those based on building alterations. On alteration projects, the 2015 IEBC triggers seismic mitigation only when the intended alteration would make significant changes to the existing lateral system. Thus, a major architectural or mechanical upgrade that does not change the structure's seismic adequacy would not trigger any seismic improvements. Following are examples of local code provisions that amend the model codes to consider certain vulnerable structure types, the extent of a nonstructural alteration, or the cost of an alteration project.

- Federal government facilities. As noted in Section B.3.4, federal agencies use the recommended practice known as *RP 8* (NIST 2012b), which relies on ASCE 41-13 for its technical criteria. Some agencies that act as their own code officials, such as the Department of Defense, apply *RP 8* as a code, supplementing a model code's triggers with *RP 8*'s additional triggers. For example, *RP 8* includes cost-based triggers not found in the model codes.
- California state-owned buildings. Sections 3417–3422 of the 2017 *California Building Code* provide supplemental provisions for state-owned buildings. In addition to modifying CBC's typical triggers based on repair and structural alteration, these provisions also trigger seismic evaluation and potential retrofit whenever the cost of a renovation exceeds 25% of the building replacement cost, cumulative from 1995 to the date of application. The criteria apply ASCE 41-13, specifying structural and nonstructural performance levels at two Seismic Hazard Levels.
- Massachusetts unreinforced masonry buildings. In 2010, Massachusetts adopted the 2009 IEBC with amendments that address, among other things, the alteration and adaptive reuse of unreinforced masonry buildings. In Section 606.2, dealing with reroofing triggers, the state modified the IEBC provision to encompass a lower seismicity threshold than the model code. In Section 101.10, it modified the IEBC's change of occupancy trigger and added triggers for any project that significantly increases the occupant load or whose work area exceeds 50% of the building.
- Seattle substantial alterations. The 2009 *Seattle Building Code* amends Chapter 34 of the 2009 IEBC. In addition to modifying the IEBC's triggers based on repairs and change of occupancy, Section 3404.8 defines any project that substantially extends a building's useful life as a "substantial alteration" that triggers seismic evaluation and possibly retrofit. Identification of substantial alterations by the code official is based on case-specific considerations of the building size, the building condition, the scope and cost of the proposed alteration, and other factors. Unreinforced brick chimneys in buildings undergoing substantial alterations must be retrofitted. Triggered structural evaluations are permitted to use ASCE 31-03 or ASCE 41-06 with a specified one-part performance objective involving a BSE-1 hazard. Where deficiencies are found, the retrofit scope and objective are subject to negotiation.
- Portland alteration triggers. The city of Portland, Oregon, adopted the 2009 IEBC and amended its seismic provisions for existing buildings in Chapter 24.85 of the *City Code and Charter*. In addition to modifying the IEBC's triggers based on repairs and change of occupancy, Section 24.85.060 requires a seismic evaluation using ASCE 31-03 for most pre-1974 buildings (other than one- and two-family dwellings) where the cost of the intended alteration

exceeds a certain value. In addition, Section 24.85.065 addresses unreinforced masonry buildings, triggering parapet bracing and wall anchors upon reroofing and a full structural retrofit, again using ASCE 31-03 as a criterion, when the alteration cost exceeds a triggering value. This appendix refers to performance levels, Seismic Hazard Levels, and performance objectives defined in Chapter 2 of the standard.

REFERENCE

Los Angeles Municipal Code. 2015. "Los Angeles ordinance 183893." Accessed September 7, 2023. https://www.seismicordinances.com/wood-frame-soft-story-structures/los-angeles

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APPENDIX C
SUMMARY DATA SHEET

BUILDING DATA

Building Name: Date:
Building Address:
Latitude: Longitude: By:

Year Built: Year(s) Remodeled: Original Design Code:
Area [ft^2 (m^2)]: Length [ft (m)]: Width [ft (m)]:
No. of Stories: Story Height: Total Height:

USE ☐ Industrial ☐ Office ☐ Warehouse ☐ Hospital ☐ Residential ☐ Educational ☐ Other: ___________

CONSTRUCTION DATA

Gravity Load Structural System:
Exterior Transverse Walls: Openings?
Exterior Longitudinal Walls: Openings?
Roof Materials/Framing:
Intermediate Floors/Framing:
Ground Floor:
Columns: Foundation:
General Condition of Structure:
Levels Below Grade?
Special Features and Comments:

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:		
Vertical Elements:		
Diaphragms:		
Connections:		

EVALUATION DATA

BSE-1N Spectral Response Accelerations: S_{DS} = S_{D1} =
Soil Factors: Class = F_a = F_v=
BSE-_____ Spectral Response Accelerations: S_{XS} = S_{X1} =
Level of Seismicity: Performance Level:
Building Period: T =
Spectral Acceleration: S_a =
Modification Factor: $C_mC_1C_2$ = Building Weight: W =
Pseudolateral Force: $V = C_mC_1C_2S_aW$ =

BUILDING CLASSIFICATION:

REQUIRED TIER 1 CHECKLISTS

	Yes	No
Basic Configuration Checklist	☐	☐
Building Type ___ Structural Checklist	☐	☐
Nonstructural Component Checklist	☐	☐

FURTHER EVALUATION REQUIREMENT: ____________________

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CHAPTER C1
GENERAL REQUIREMENTS

C1.1 SCOPE

This standard consists of two parts: provisions, which contain the technical requirements, and commentary, intended to explain the provisions. Commentary for a given section is identified by the same section number preceded by the letter C, following the provision section. The standard is an update to ASCE 41-17.

Applicability of the Standard: This standard is intended to serve as a nationally applicable tool for design professionals, code officials, and building owners undertaking the seismic evaluation or retrofit of existing buildings. The evaluation and retrofit requirements are intended to be used for either mandatory requirement by an Authority Having Jurisdiction or for voluntary applications. This standard applies to the seismic evaluation and retrofit of the overall structural system of a building and its nonstructural components, including ceilings and partitions, as well as mechanical, electrical, and plumbing systems. All aspects of building performance are considered and defined in terms of structural, nonstructural, foundation, and geologic hazard issues. Lifelines such as lines for water, electricity, natural gas, and waste disposal beyond the perimeter of the building, which may be necessary for buildings to be occupied, are not considered in this document.

The procedures contained in this standard are specifically applicable to the evaluation and retrofit of existing buildings to ascertain compliance with specific Performance Objectives and, in general, are more appropriate for that purpose than are codes for new buildings. Codes for new construction are primarily intended to regulate the design and construction of new buildings; as such, they include many provisions that encourage or require the development of designs with features important for good seismic performance, including regular configuration, structural continuity, ductile detailing, and materials of appropriate quality. Many existing buildings were designed and constructed without these features and contain characteristics, such as unfavorable configuration and poor detailing, that preclude application of regulatory or building code provisions for their seismic evaluation or retrofit.

This standard is intended to be generally applicable to seismic evaluation and retrofit of all buildings regardless of importance, occupancy, historic status, or other classifications of use.

In addition to the direct effects of ground shaking, this standard also addresses, to a limited extent, other seismic hazards, such as liquefaction, slope failure, surface fault rupture, and effects of neighboring structures. Other earthquake-related phenomena, such as tsunami effects, are not considered.

Design of new buildings and evaluation of existing buildings and components for gravity and wind forces in the absence of earthquake demands are beyond the scope of this standard.

Provisions of this standard for seismic evaluation and retrofit are based on a performance-based design methodology that differs from seismic design procedures for the design of new buildings currently specified in national model building codes and standards. The framework in which these requirements are specified is purposefully broad so that Performance Objectives can accommodate buildings of different types, address a variety of performance levels, and reflect the variation of seismic hazards across the United States and US territories.

The 2013 edition of this standard merged ASCE 31-03 *Seismic Evaluation of Existing Buildings* with ASCE 41-06 *Seismic Rehabilitation of Existing Buildings* into a common document. The combination of these documents eliminated significant differences between the ASCE 31 seismic evaluation and ASCE 41 retrofit processes to form a common methodology and approach. The provisions and commentary of this standard are based primarily on ASCE 31 and ASCE 41 but have been significantly updated and reorganized.

ASCE 31 evolved from and replaced FEMA 310, *Handbook for the Seismic Evaluation of Buildings: A Prestandard* (FEMA 1998c). ASCE 31 was developed to reflect the evaluation experience of design professionals and lessons learned from past earthquakes.

The predecessor to ASCE 41 was FEMA 356 *Prestandard and Commentary for the Seismic Rehabilitation of Buildings* (FEMA 2000b). FEMA 356 was based on FEMA 273 (FEMA 1997c), which was developed by a large team of specialists in earthquake engineering and seismic evaluation and retrofit. The standard incorporates many advances made in the analysis and design evaluation of structures that are likely to have general or widespread application in the performance evaluation of existing structures and reflect known laboratory experience and field observations of earthquake damage. The acceptance criteria have been specified using actual laboratory test results, where available, supplemented by the engineering judgment of various development teams. Engineering judgment should be exercised in determining the applicability of various analysis techniques and material acceptance criteria in each situation.

With careful extrapolation, the procedures of this standard may also be applied to many nonbuilding structures, such as pipe racks, steel storage racks, structural towers for tanks and vessels, piers, wharves, and electrical power generating facilities. However, the applicability of these procedures has not been fully examined for every type of structure—particularly those that have generally been covered by specialized codes or standards, such as bridges and nuclear power plants.

Techniques for repair of earthquake-damaged buildings are not included in this standard but are referenced in Chapters C9 through C12 where such guidelines exist. Any combination of repaired components, undamaged existing components, and new components can be modeled using this standard, and each can be checked against performance level acceptance criteria. If the mechanical properties of repaired components are known, acceptance criteria for use with this standard can be either deduced by comparison with other similar components or derived.

Application to Historic Buildings: This standard is intended to be applicable to all buildings, including designated historic buildings. Although the engineering principles for evaluating and retrofitting historic structures are similar to those for other buildings, the protections afforded historic buildings can raise additional issues that limit some of the actions that could be taken to evaluate and retrofit other buildings. Certain evaluation or retrofit tasks or techniques suitable or even preferred for a typical project might not be acceptable from a historic preservation perspective. These techniques might include the following:

- Condition assessment or material testing that would disturb historic elements,
- Potential architectural damage that might otherwise be found acceptable by an evaluation with a safety-based Performance Objective,
- Retrofit measures that involve removal of architectural components to gain access to the structure, and
- Retrofit measures that alter the look or configuration of the building.

Although the expected performance of architectural elements and finishes must be considered for all types of buildings, the interaction of architectural and structural elements in historic buildings often plays a more important role in the overall seismic performance of the structural system. Disturbance of historic architectural elements and finishes to allow testing during evaluation and to implement the resulting retrofit measures may be unacceptable. It is often necessary to evaluate historic buildings on a case-by-case basis and using general performance, rather than prescriptive, criteria.

There are national and often state and municipal registers of historic places, buildings, and districts (neighborhoods). Additionally for some programs, "eligibility" for the register is sufficient cause for special treatment. All US states and territories have a designated state historic preservation officer, who should be consulted regarding these registers.

In addition, an appropriate level of performance for historic structures needs to be chosen that is acceptable to the Authority Having Jurisdiction. Some people feel that historic buildings should meet the safety levels of other buildings because these levels are a subset of the general seismic safety needs. Others feel that historic structures, because of their value to society, should meet a higher level of performance. In other cases, a reduced level of performance has been allowed to avoid damaging historic fabric during retrofit. In other cases, a higher Performance Objective has been used to enhance post-earthquake reparability of historic features.

Codes and policies regulating historic buildings have tried to balance a desire for improved seismic performance with a commitment to preservation. This standard's criteria, however, do not directly or explicitly address specific preservation objectives. Where historic preservation concerns would inform a project's seismic Performance Objective, this standard might therefore be inadequate if applied simply as written. In these cases, codes or policies that invoke this standard might prefer to use it as a guideline or to supplement it with criteria specific to historic buildings.

The following resources may be useful where evaluating historic structures:

- *Standards for the Treatment of Historic Properties* (Secretary of the Interior 1992);
- *Standards for the Treatment of Historic Properties with Guidelines for Preserving, Rehabilitating, Restoring, and Reconstructing Historic Buildings* (Secretary of the Interior 1995);
- *California Historical Building Code* (CBSC 2016b);
- *Disaster Management Programs for Historic Sites* (Secretary of the Interior 1998); and
- *Technical Preservation Services for Historic Buildings*, https://www.nps.gov/orgs/1739/tps-publications.htm.

Intent of This Standard: It is expected that most buildings shown to be in compliance or retrofitted in accordance with this standard would perform within the desired levels when subjected to the selected earthquake(s). However, compliance with this standard does not guarantee such performance; rather, it represents the current standard of practice in designing to attain this performance. The practice of earthquake engineering is rapidly evolving, and both the understanding of the behavior of buildings subjected to strong earthquakes and the ability to predict this behavior are advancing. In the future, new knowledge and technology will improve the reliability of accomplishing these goals.

Featured in this standard are descriptions of damage states in relation to specific performance levels. These descriptions are intended to aid the Authority Having Jurisdiction, design professionals, and owners in selecting appropriate performance levels for evaluation and retrofit design. They are not intended to be used for condition assessment of earthquake-damaged buildings. Although there may be similarities between these damage descriptions and those used for post-earthquake damage assessment, many factors enter into the processes of assessing seismic performance. No single parameter in this standard should be cited as defining either a performance level or the safety or usefulness of an earthquake-damaged building.

Guidance for Programs, Ordinances, and Laws: This standard does not explicitly address the determination of whether an evaluation or retrofit project should be undertaken for a particular building. Guidance on the use of this standard in voluntary, mandatory, or code-triggered risk-mitigation programs is provided in Appendix B. Determining where these provisions should be required is beyond the scope of this standard. Once the decision to evaluate or retrofit a building has been made, this standard can be referenced for detailed engineering guidance on how to conduct a seismic analysis and design.

Coordinating with Codes for New Construction and Ordinances: Application of these provisions should be coordinated with other requirements that may be in effect, such as ordinances governing historic structures or hospital construction. Because codes for new buildings have chapters that briefly address existing buildings, care must be taken in coordinating and referencing the adoption of this standard to avoid ambiguity and confusion with other ordinances and codes.

Overarching Philosophical Approach: This standard is based on both experience-based judgment and academic research and component testing.

Experience-based judgment is largely derived from the observations of unretrofitted building performance in past earthquakes and, to a much lesser extent, the observations of the performance of retrofitted buildings in earthquakes. In addition, experience from past evaluations and retrofits of existing buildings using ASCE 31-03, ASCE 41-06, and practice before these earlier editions were published has also helped inform changes to this standard. Earthquake observations that have significantly influenced this standard have been from the following earthquakes: 1971 Sylmar (San Fernando, California), 1985 Michoacan (Mexico

City), 1987 Whittier Narrows (Southern California), 1989 Loma Prieta (San Francisco), 1994 Northridge (Los Angeles), 1995 Hyogo-ken Nanbu (Japan), 2001 Nisqually (Washington state), 2003 San Simeon (central California), 2010 Chile, 2010 and 2011 Christchurch (New Zealand), 2011 Great East Japan earthquake and tsunami, and many other less significant earthquakes. More information about these observations can be obtained from reconnaissance reports, such as those produced by the Earthquake Engineering Research Institute, the Japan Association for Earthquake Engineering, and the New Zealand Society for Earthquake Engineering. Although each earthquake may help validate or revise the fundamental assumptions underlying the procedures presented in ASCE 31-03 and ASCE 41-06, each may also offer new insights into the potential weaknesses in certain systems that should be considered. This knowledge was incorporated into this updated standard. Users of this standard are strongly encouraged to learn from past observations and participate in future efforts to document and interpret the performance of buildings. Tier 1 screening procedures in Chapter 4, deficiency-based procedures in Chapter 5, and nonstructural provisions in Chapter 13 rely most heavily on experience-based information and judgment.

Research data from partial and full-scale structural and nonstructural component testing, using shaking tables, quasistatic component testing, materials testing, and computer modeling, and their adaptation to the practice of seismic evaluation and retrofit are the second major source of information for this standard. References to such tests are provided in the commentary, particularly in the Tier 3 analysis and materials chapters, C8 through C15.

Judgment by the Design Professional: Although this standard provides prescriptive direction for the evaluation and retrofit of existing buildings, it is not to be taken as the only direction. This standard provides direction for common details, deficiencies, and behavior observed in past earthquakes that are found in common building types. However, every structure is unique and may contain features and details that are not covered by this standard. It is important that the design professional use judgment when applying the provisions of this standard. The design professional should always look for uncommon details and behavior about the structure that may have the potential for damage or collapse or that may improve the performance of the building relative to buildings of the same building type.

The design professional may wish to review initial considerations with the Authority Having Jurisdiction to determine any restrictions that exist on the use of evaluation procedures. Initial considerations include structural characteristics of the building; seismic hazards, including geologic site hazards known to be present at the site; results of prior seismic evaluations; building use and occupancy requirements; historic status; economic considerations; societal issues; and local jurisdictional requirements.

C1.3 SEISMIC EVALUATION PROCESS

A major portion of the process is dedicated to instructing the evaluating design professional on how to determine if a building is adequately designed and constructed to resist seismic forces. The need for evaluation using this standard may have been caused by a client's concern for knowing the vulnerability of the building; by a regulation, building code, or policy trigger for analysis or modification of the building; by a requirement for a financial transaction; or from many other sources. When resources are limited, before using the evaluation methods of this standard, the design professional might consider using FEMA 154 (2015b), *Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook*, 3rd Edition.

Before conducting the seismic evaluation based on this standard, the design professional should understand the evaluation process and the basic requirements specified in this section. The evaluation process consists of the following three tiers, as shown in Figure C1-1: Tier 1 screening procedure, Tier 2 deficiency-based evaluation procedure, and Tier 3 systematic evaluation procedure.

As indicated in Figure C1-1, the design professional may choose to (1) report deficiencies and recommend mitigation, or (2) conduct further evaluation, after any tier of the evaluation process. The evaluation process can begin with the Tier 3 systematic evaluation and not incur the expense of the earlier tiers. This decision is appropriate when there is little professional doubt, either that the building has significant seismic deficiencies related to a selected Performance Objective or that the work to be done will trigger retrofit work. The advantage of doing the Tier 1 or 2 assessments as the starting point is that it may identify other deficiency-based alternatives for retrofitting the building.

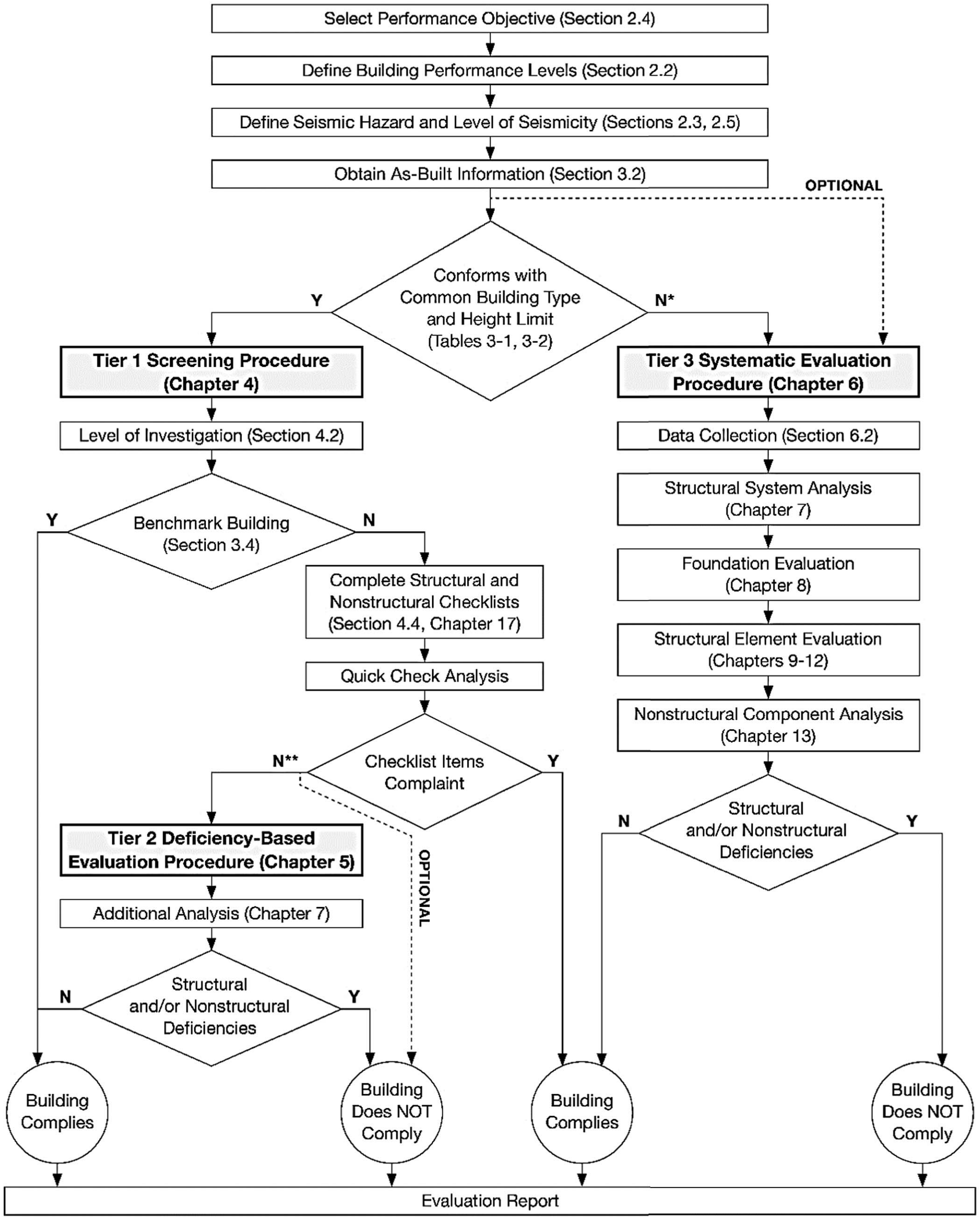
Some design professionals have based the seismic evaluation of buildings on the provisions for the design of new buildings. Although this route may seem appropriate, it must be done with full knowledge of the inherent assumptions. Codes for new buildings contain requirements that govern building configuration, strength, stiffness, detailing, and special inspection and testing. The strength and stiffness requirements are easily transferred to existing buildings; the other provisions are not. If the seismic-force-resisting elements of an existing building do not have details of construction similar to those required for new construction, the basic assumptions of ductility will not be met, and the results of the evaluation may not be valid. This procedure could lead to evaluating a building as unacceptable for a given Performance Objective when it is acceptable or to evaluating a building as acceptable when it is unacceptable for a given Performance Objective. Care must be taken in applying code provisions for new buildings to structures that have noncompliant elements; this subject is not addressed in this standard.

Potential seismic deficiencies in existing buildings may be identified using this standard. If the evaluation is voluntary, the owner may choose to accept the risk of damage from future earthquakes rather than upgrade or demolish the building. If the evaluation is required by a local ordinance for a hazard-reduction program or triggered by a regulation, building code, or policy, the owner may have to choose among retrofit, demolition, occupancy limitations, or other options.

C1.3.1 Assignment of Performance Objective This standard may be used on a voluntary basis or may be required by the Authority Having Jurisdiction. In jurisdictionally mandated seismic retrofit programs, the code official serves as the Authority Having Jurisdiction. In voluntary seismic retrofit programs, either the building owner or the owner's designated agents select Performance Objectives and decide at what stage to complete the evaluation. Appendix B presents discussion of topics that may be considered when determining the appropriate Performance Objective to assign.

Chapter 2 identifies five Structural Performance Levels (S1 through S5) plus S6 Not Considered, and four Nonstructural Performance Levels (NA through ND) plus NE Not Considered.

The concepts and terminology of performance-based design should be carefully studied and discussed with building owners before use. The terminology used for target building performance levels is intended to represent goals of design. The actual ground motion is seldom comparable to that specified in the Performance Objective, so in most events, designs targeted at various damage states may only determine relative performance. Even given a



* It may be beneficial for the engineer to perform a Tier 1 Screening Evaluation prior to a Tier 3 Systematic Evaluation even though it is not required.
** The evaluation process may proceed directly to the Tier 3 Systematic Evaluation as an option.

Figure C1-1. Evaluation Process.

ground motion similar to that specified in the Performance Objective and used in design, variations from stated Performance Objectives should be expected, and compliance with this standard should not be considered a guarantee of performance. Variations in actual performance could be associated with unknown geometry and member sizes in existing buildings, deterioration of materials, incomplete site data, variation of ground motion that can occur within a small area, and incomplete knowledge and simplifications related to modeling and analysis. Information on the expected reliability of achieving various target building performance levels when the requirements are followed can be found in Chapter 2 of FEMA 274 (FEMA 1997b).

C1.3.3 As-Built Information Collection of as-built information may be based on original design or construction documents, visual observations, a detailed condition assessment, material testing, or a combination. Chapters 3, 4, 5, and 6 have detailed requirements on the level of information that is required for different tiers of evaluation and retrofit. Appendix C contains a sample data sheet that can be used to summarize the as-built information of a building.

C1.3.4 Evaluation Procedures This standard contains three procedures for seismic evaluation. The Tier 1 screening and Tier 2 deficiency-based procedures are intended for buildings meeting the criteria for the common building types in Table 3-1 and limitations in Table 3-2. Where these two procedures are permitted and selected for use, the evaluation process must begin with a Tier 1 screening (Section 3.4.2), followed by Tier 2 (Section 3.4.3) as warranted.

Where the Tier 1 and Tier 2 procedures are not permitted based on Section 3.4 or by the Authority Having Jurisdiction or where the design professional chooses to conduct a more detailed evaluation, a Tier 3 evaluation shall be conducted in accordance with Section 3.4.4.

C1.4 SEISMIC RETROFIT PROCESS

The steps are presented in this section in the order in which they would typically be followed in the retrofit process. However, the criteria for performing these steps are presented in a somewhat different order to facilitate presentation of the concepts.

Figure C1-2 depicts the retrofit process specified in this standard and shows specific chapter references in parentheses at points where input from this standard is to be obtained. Although Figure C1-2 is written for voluntary retrofits, it can also be used as a guide for mandatory retrofits.

This standard requires the selection of a Performance Objective for a building that has been previously identified as needing seismic retrofit.

Before embarking on a retrofit program, an evaluation should be performed to determine whether the building, in its existing condition, has the desired seismic performance capability. This standard contains an evaluation methodology as summarized in Section 1.4 that may be used for this purpose. Evaluations can also be performed in accordance with other means that are acceptable to the owner and the Authority Having Jurisdiction. Such acceptable means could include qualitative review by a design professional of a building that is of a type that has performed poorly in past earthquakes. However, the determination of retrofit scope requires some process for identifying specific deficiencies to be mitigated for a selected Performance Objective.

The process of building retrofit will be simplified and made more efficient if information that significantly affects the retrofit

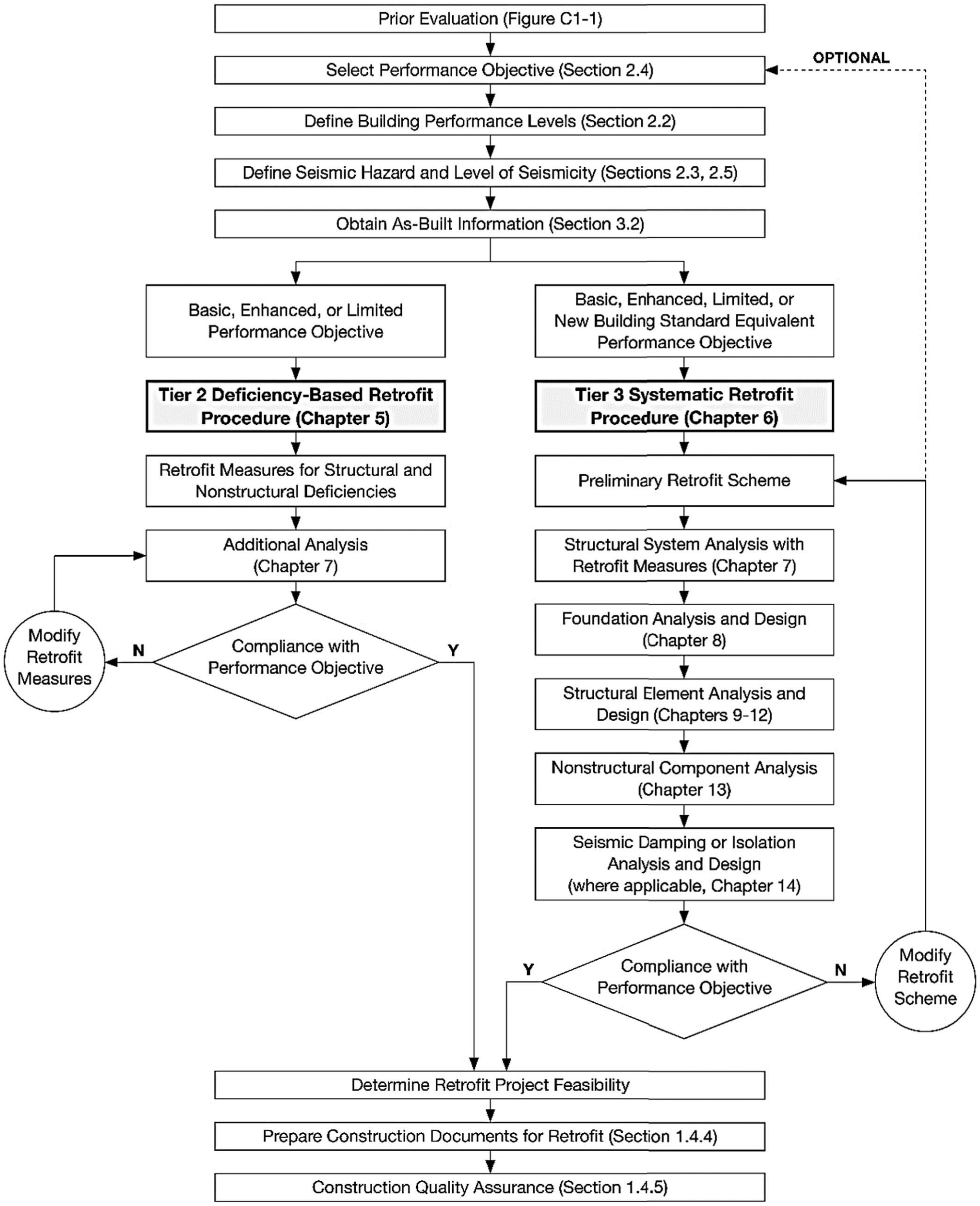


Figure C1-2. Retrofit process.

design is obtained and considered before beginning the process. Retrofit requirements mandated by the Authority Having Jurisdiction would be particularly important to determine in the initial stages of a project.

Unless already completed for a prior seismic evaluation (see Section C1.4), the design professional is encouraged to review the initial considerations with the Authority Having Jurisdiction to determine any restrictions that exist on the design of retrofit measures. Initial considerations include structural characteristics of the building; seismic hazards, including geologic site hazards known to be present at the site; results of prior seismic evaluations; building use and occupancy requirements; historic status; economic considerations; societal issues; and local jurisdictional requirements.

The building owner should be aware of the range of costs and impacts of retrofit, including both the variation associated with different Performance Objectives and the potential additional costs often associated with seismic retrofit, such as other Life Safety upgrades, hazardous material removal, work associated with the Americans with Disabilities Act, and nonseismic building remodeling. Also, to be considered are potential federal tax incentives for the retrofit of historic buildings and for some other older nonresidential buildings.

Seismic hazards other than ground shaking may exist at the building site. The risk and possible extent of damage from geologic site hazards identified in Section 8.2.2 should be considered before undertaking a retrofit aimed solely at reducing damage caused by shaking. In some cases, it may be feasible to mitigate the site hazard or retrofit the building and still meet the selected performance level. In other cases, the risk caused by site hazards may be so extreme and difficult to control that retrofit is neither cost-effective nor feasible.

The use of the building must be considered in weighing the significance of potential temporary or permanent disruptions associated with various risk-mitigation schemes. Other limitations on modifications to the building caused by historic or aesthetic features must also be understood. The historic status of every building at least 50 years old should be determined (see Appendix B, Section B.2.3, “Historic Buildings”). This determination should be made early as it could influence the choices of retrofit approaches and techniques.

There are many ways to reduce seismic risk, whether the risk is to property, Life Safety, or post-earthquake use of the building. The occupancy of vulnerable buildings can be reduced, redundant facilities can be provided, and nonhistoric buildings can be demolished and replaced. The risks posed by nonstructural components and contents can be reduced. Seismic site hazards other than shaking can be mitigated.

Most often, however, when all alternatives are considered, the options of modifying the building to reduce the risk of damage should be studied. Such corrective measures include stiffening or strengthening the structure, adding local components to eliminate irregularities or tie the structure together, reducing the demand on the structure through the use of seismic isolation or energy dissipation devices, and reducing the height or mass of the structure.

Although not specifically required by any of the strategies, it is beneficial for the retrofitted seismic-force-resisting system to have an appropriate level of redundancy so that any localized failure of a few components of the system will not result in local collapse or an instability. This should be considered when developing retrofit designs.

Local Modification of Components: Some existing buildings have substantial strength and stiffness, but some of their components may not have adequate strength, toughness, or deformation capacity to satisfy the Performance Objectives. An appropriate strategy for such structures may be to perform local modifications of components that are inadequate while retaining the basic configuration of the building’s seismic-force-resisting system. Local modifications that can be considered include improvement of component connectivity, component strength, component deformation capacity, or all three. This strategy tends to be the most economical retrofit approach where only a few of the building’s components are inadequate.

Local strengthening allows one or more understrength components or connections to resist the strength demands predicted by the analysis without affecting the overall response of the structure. This could include measures such as cover plating steel beams or columns or adding wood structural panel sheathing to an existing timber diaphragm. Such measures increase the strength of the component and allow it to resist more earthquake-induced force before the onset of damage.

Local corrective measures that improve the deformation capacity or ductility of a component allow it to resist large deformation levels with reduced amounts of damage without necessarily increasing the strength. One such measure is placement of a confinement jacket around a reinforced concrete column to improve its ability to deform without spalling or degrading reinforcement splices. Another measure is reduction of the cross section of selected structural components to increase their flexibility and response displacement capacity.

Removal or Reduction of Existing Irregularities: Removal or reduction of existing irregularities may be an effective retrofit strategy if a seismic evaluation shows that the irregularities result in the inability of the building to meet the selected Structural Performance Objective.

The results of analysis should be reviewed to detect existing irregularities. Stiffness, mass, and strength irregularities may be detected by reviewing the results of a linear analysis, by examining the distribution of structural displacements and demand-to-capacity ratios (DCRs), or by reviewing the results of a nonlinear analysis by examining the distribution of structural displacements and inelastic deformation demands. If the distribution of values of structural displacements, DCRs, or inelastic deformation demands predicted by the analysis is nonuniform with disproportionately high values within one story relative to the adjacent story, or at one side of a building relative to the other, then an irregularity exists.

Such irregularities are often, but not always, caused by the presence of a discontinuity in the structure, such as termination of a perimeter shear wall above the first story. Simple removal of the irregularity may be sufficient to reduce demands predicted by the analysis to acceptable levels. However, removal of discontinuities may be inappropriate in the case of historic buildings, and the effect of such alterations on important historic features should be considered carefully.

Effective corrective measures for removal or reduction of irregularities, such as soft or weak stories, include the addition of braced frames or shear walls within the soft or weak story. Torsional irregularities can be corrected by the addition of moment frames, braced frames, or shear walls to balance the distribution of stiffness and mass within a story. Discontinuous components such as columns or walls can be extended through the zone of discontinuity.

Partial demolition can also be an effective corrective measure for irregularities, although this method obviously has a significant effect on the appearance and utility of the building, and this may not be an appropriate alternative for historic structures. Portions of the structure that create the irregularity, such as setback towers or side wings, can be removed. Expansion joints can be created to transform a single irregular building into multiple regular structures; however, care must be taken to avoid the potential problems associated with pounding.

Global Structural Stiffening: Global stiffening of the structure may be an effective retrofit strategy if the results of a seismic evaluation show deficiencies attributable to excessive lateral deflection of the building and critical components do not have adequate ductility to resist the resulting deformations. Construction of new braced frames or shear walls within an existing structure are effective measures for adding stiffness.

Global Structural Strengthening: Global strengthening of the structure may be an effective retrofit strategy if the results of a seismic evaluation show unacceptable performance attributable to a global deficiency in structural strength. Such deficiency can be identified where the onset of global inelastic behavior occurs at levels of ground shaking that are substantially less than the selected level of ground shaking, or large DCRs (or inelastic deformation demands) are present throughout the structure. By providing supplemental strength to such a seismic-force-resisting system, it is possible to raise the threshold of ground motion at which the onset of damage occurs. Shear walls and braced frames are effective elements for this purpose, but they may be significantly stiffer than the structure to which they are added, requiring them to provide nearly all of the structure's lateral resistance. Moment-resisting frames, being more flexible, may be more compatible with existing elements in some structures; however, such flexible elements may not become effective in the building's response until existing brittle elements have already been damaged.

Mass Reduction: Mass reduction may be an effective retrofit strategy if the results of a seismic evaluation show deficiencies attributable to excessive building mass, global structural flexibility, or global structural weakness. Mass and stiffness control the amount of force and deformation induced in a structure by ground motion. Reductions in mass can result in direct reductions in both the amount of force and the deformation demand produced by earthquakes and, therefore, can be used in lieu of structural strengthening and stiffening. Mass can be reduced through demolition of upper stories, replacement of heavy cladding and interior partitions, or removal of heavy storage and equipment loads.

Seismic Isolation: Seismic isolation may be an effective retrofit strategy if the results of a seismic evaluation show deficiencies attributable to excessive seismic forces or deformation demands, or if it is desired to protect important contents and nonstructural components from damage. Where a structure is seismically isolated, compliant bearings are inserted between the superstructure and its foundations. This insertion produces a system (structure and isolation bearings) with a nearly rigid body translation of the structure above the bearings. Most of the deformation induced in the isolated system by the ground motion occurs within the compliant bearings, which are specifically designed to resist these concentrated displacements. Most bearings also have excellent energy dissipation characteristics (damping). Together, this results in greatly reduced demands on the existing structural and nonstructural components of the building and its contents. For this reason, seismic isolation is often an appropriate strategy to achieve Enhanced Performance Objectives that include the protection of historic fabric, valuable contents, and equipment, or for buildings that contain important operations and functions. This technique is most effective for relatively stiff buildings with low profiles and large mass. It is less effective for light, flexible structures.

Supplemental Energy Dissipation: Installation of supplemental energy dissipation devices may be an effective retrofit strategy if the results of a seismic evaluation show deficiencies attributable to excessive deformations caused by global structural flexibility in a building. Many available technologies allow the energy imparted to a structure by ground motion to be dissipated in a controlled manner through the action of special devices—fluid viscoelastic dampers (hydraulic cylinders), yielding plates, or friction pads—resulting in an overall reduction in the displacements of the structure. The most commonly used devices dissipate energy through frictional, hysteretic, or viscoelastic processes. To dissipate substantial energy, dissipation devices typically must undergo significant deformation (or stroke), which requires that the structure experience substantial lateral displacements. Therefore, these systems are most effective in structures that are relatively flexible and have some inelastic deformation capacity. Energy dissipaters are most commonly installed in structures as components of braced frames. Depending on the characteristics of the device, either static or dynamic stiffness is added to the structure as well as energy dissipation capacity (damping). In some cases, although the structural displacements are reduced, the forces delivered to the structure can actually be increased.

C1.4 SEISMIC RETROFIT PROCESS

C1.4.1 Assignment of Performance Objective The determination of the Performance Objective differs depending on whether the retrofit is mandated or voluntary. For a voluntary building retrofit, the building owner shall select a seismic retrofit Performance Objective for the building as specified in Section 2.4. In a mandated retrofit project, the minimum retrofit objective is either stipulated directly by local code or ordinance, or the code official is provided with guidelines for negotiating the retrofit objective.

Because almost all structural seismic retrofit work requires a building permit, the code official will become an important part of the process. For voluntary retrofit efforts, the building owner and the code official need to come to agreement about the intended retrofit objective. The code official will verify that the proposed voluntary upgrade does not violate any other regulatory, building code, or policy requirements or trigger additional code-stipulated work. For jurisdictionally required retrofit efforts, whether caused by passive or active programs (see Appendix B), the code official will verify that the required objective is met. Because the approaches and technology of this standard are not yet in the mainstream of design and construction practices of the United States, it is imperative that the code official either develop the expertise in this methodology or utilize a peer review type of process to verify the appropriate application of this standard. A jurisdiction must also remain flexible and open to other approaches to evaluation and retrofit, which may provide a reasonable assurance of meeting the appropriate Performance Objective.

C1.4.4 Verification of Retrofit Design Retrofit procedures include the Tier 2 deficiency-based retrofit procedures or the Tier 3 systematic retrofit procedures. These procedures are defined in Section 3.3 and further explained in the associated commentary of that section.

This standard is arranged such that there are four analysis procedures that can be used, including the linear static procedure, linear dynamic procedure, nonlinear static procedure, and nonlinear dynamic procedure. The linear analysis procedures are intended to provide a conservative estimate of building response and performance in an earthquake. Because the actual response of buildings to earthquakes is not typically linear, the nonlinear analysis procedures should provide a more accurate representation of building response and performance. In recognition of the improved representation of building behavior when nonlinear analysis is conducted, the nonlinear procedures have less conservative limits on permissible building response than do linear procedures. Buildings that are found to be seismically deficient based on linear analysis may comply with this standard if a nonlinear analysis is performed. Therefore, performing a nonlinear analysis can minimize or eliminate unnecessary seismic retrofit and potentially lower construction costs.

Nonlinear analysis procedures are more complicated, take more time to implement, and require a considerable amount of expertise to properly implement. The requirements for nonlinear analysis application to a specific structural system may involve subtle and exacting modeling assumptions that should be reviewed in context to ensure that they are consistent with current knowledge and understanding. The owner or reviewing officials should take care to institute qualified, independent technical review procedures and actions where the consequences of the analysis overturn earlier assessments of unacceptable performance. Indeed, it may be prudent to institute independent technical peer review for most such analysis-based designs. Often, it is advisable to institute independent peer review at the beginning of the analysis rather than at the end; this method avoids disputes when the budget has been spent and technical issues are not satisfactorily resolved.

An analysis of the building with all proposed retrofit measures included should demonstrate that all elements meet the acceptance criteria for the Performance Objective being targeted. When an element does not meet the performance requirements specified in the standard, the element or the structure as a whole can be modified, or one can show that the element's behavior does not affect the performance of the building. If the element's failure does not have a deleterious effect on other elements, compromise the support of gravity load, and compromise the achievement of the total structure meeting the Performance Objective, then the element need not be modified. Analysis must be performed to justify those conclusions, which includes an analysis of performance of the building without reliance on any gravity and lateral load resistance characteristics of the element under consideration before its failure.

C1.4.5 Quality Assurance and Structural Observation It is important to verify that the seismic retrofit is constructed in accordance with the approved construction documents. Building codes require special inspection, testing, and structural observation for new construction, but existing building model codes such as the *International Existing Building Code*, are not clear on such requirements for alterations to existing buildings. This section invokes the special inspection, testing, and structural observation requirements for new constructions for new elements added as part of the seismic retrofit, alterations of existing elements of the seismic retrofit, and connections between new elements and the existing structure as part of the seismic retrofit. The section treats retrofit components in a similar manner as codes for new construction treat special seismic-force-resisting systems, which generally require a significant amount of additional special inspection and testing compared with other structural components of the same construction material. The design professional responsible for the seismic retrofit of a specific building may find it appropriate to specify more stringent or more detailed requirements. Such additional requirements may be particularly appropriate for those buildings targeting Damage Control or Immediate Occupancy Structural Performance Levels or Operational Nonstructural Performance Level.

C1.4.5.1 Special Inspections and Testing The intent of the provisions is that alterations to existing elements and new elements added as part of a retrofit and their connection to existing elements to have the same level of special inspection as the seismic-force-resisting system in a new building.

C1.4.5.2 Structural Observation Structural observation should be provided for all components added as part of the seismic retrofit, alteration of existing components as part of the seismic retrofit, and connection of new components to existing components as part of the seismic retrofit. Many details are likely repetitive, and a representative percentage can be observed. However, it is important that the registered design professional observe a representative sample of the retrofit construction for general conformance to the retrofit design, especially the connection of new elements to existing elements. Failure of these connections can significantly compromise the intent of the seismic retrofit.

CHAPTER C2

PERFORMANCE OBJECTIVES AND SEISMIC HAZARDS

C2.2 PERFORMANCE LEVELS

Building performance is a combination of the performance of both structural and nonstructural components. Table C2-1 describes the approximate limiting levels of structural and nonstructural damage that might be expected of buildings evaluated or retrofitted to the levels defined in this standard. On average, the expected damage would be less. For comparative purposes, the estimated performance of a typical new building subjected to the BSE-1N level of shaking is indicated. Performance descriptions in Table C2-1 are estimates rather than precise predictions, and variation among buildings of the same target Building Performance Level must be expected.

Building performance in this standard is expressed in terms of target Building Performance Levels. These target Building Performance Levels are discrete damage states selected from among the infinite spectrum of possible damage states that buildings could experience during an earthquake. The particular damage states identified as target Building Performance Levels in this standard have been selected because they have readily identifiable consequences associated with the post-earthquake disposition of the building that are meaningful to the building community. These consequences include the ability to resume normal functions within the building, the advisability of post-earthquake occupancy, and the risk to life safety.

Because of inherent uncertainties in prediction of ground motion and analytical prediction of building performance, some variation in actual performance should be expected. Compliance with this standard should not be considered a guarantee of performance. Information on the reliability of achieving various performance levels can be found in Chapter 2 of FEMA 274 (1997b).

Table C2-2 describes damage patterns commonly associated with structural elements for Structural Performance Levels when the assessed seismic hazard has occurred. The damage states described in the table might occur in some elements at the Structural Performance Level, but it is unlikely that all the damage states described will occur in all elements of a building at that Structural Performance Level. The descriptions of damage states do not replace or supplement the quantitative definitions of performance provided elsewhere in this standard and are not intended for use in post-earthquake evaluation of damage or for judging the safety of, or required level of repair to, a structure after an earthquake. They are presented to assist engineers using this standard to understand the relative degrees of damage at each defined performance level.

Damage patterns in structural elements depend on the modes of behavior of those elements. More complete descriptions of damage patterns and levels of damage associated with damage levels can be found in other documents, such as FEMA 306 (1998a) for concrete and masonry wall buildings and FEMA 352 (2000c) for steel moment-frame buildings.

In Table C2-2, the difference between damage associated with Collapse Prevention and Life Safety Performance Levels is a matter of degree or certainty. For a given structure, the damage patterns and the locations of initial damage are similar for both performance levels, but damage at the Life Safety Performance Level is somewhat less extensive and, because of differences in quantitative acceptance criteria, less likely to give rise to collapse.

C2.2.1 Structural Performance Levels and Ranges Different structural performance requirements might be desired by individual building owners for specific buildings and time periods of concern. The first five Structural Performance Levels defined in this standard have been selected to correlate with the most commonly specified structural performance requirements.

Table C2-2 relates these Structural Performance Levels to the limiting damage states for common vertical and horizontal elements of lateral-force-resisting systems. Later sections of this standard specify design parameters (such as *m*-factors, component capacities, and inelastic deformation capacities) specified as limiting values for attaining these Structural Performance Levels for a selected earthquake demand.

The post-earthquake state of the buildings described in these tables is for illustrative purposes to convey conceptually what earthquake damage correlates with the different performance levels. This table is not intended for and should not be used in the post-earthquake safety evaluation process or as an expectation of post-earthquake performance of a building evaluated or retrofit to this standard.

Immediate Occupancy Structural Performance Level (S-1). Structural Performance Level S-1, Immediate Occupancy, refers to the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical- and lateral-force-resisting systems of the building retain almost all their pre-earthquake strength and stiffness. The risk of life-threatening injury as a result of structural damage is very low, and although some minor structural repairs might be appropriate, these repairs would generally not be required before reoccupancy. Continued use of the building is not limited by its structural condition but might be limited by damage or disruption to nonstructural elements of the building, furnishings, or equipment and availability of external utility services.

Damage Control Structural Performance Level (S-2). The Damage Control Structural Performance Level is set forth as a midway point between Life Safety and Immediate Occupancy. It is intended to provide a structure with a greater reliability of resisting collapse and being less damaged than a typical structure but not to the extent required of a structure designed to meet the Immediate Occupancy Performance Level.

Although this level is a numerically intermediate level between Life Safety and Immediate Occupancy, the two

Table C2-1. Damage Control and Building Performance Levels.

Overall damage	Collapse Prevention Level (5-D) Severe	Life Safety Level (3-C) Moderate	Immediate Occupancy Level (1-B) Light	Operational Level (1-A) Very light
	Target Building Performance Levels			
Structural components	Little residual stiffness and strength to resist lateral loads, but gravity load-bearing columns and walls function. Large permanent drifts. Some exits blocked. Building is near collapse in aftershocks and should not continue to be occupied.	Some residual strength and stiffness left in all stories. Gravity-load-bearing elements function. No out-of-plane failure of walls. Some permanent drift. Damage to partitions. Continued occupancy might not be likely before repair. Building might not be economical to repair.	No permanent drift. Structure substantially retains original strength and stiffness. Continued occupancy likely.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. All systems important to normal operation are functional. Continued occupancy and use highly likely.
Nonstructural components	Extensive damage. Infills and unbraced parapets have failed or are at incipient failure.	Falling hazards, such as parapets, mitigated, but many architectural, mechanical, and electrical systems are damaged.	Equipment and contents are generally secure but might not operate due to mechanical failure or lack of utilities. Some cracking of facades, partitions, and ceilings as well as structural elements. Elevators can be restarted. Fire protection operable.	Negligible damage occurs. GPower and other utilities are available, possibly from standby sources.
Comparison with performance intended for typical buildings designed to codes or standards for new buildings, for the design earthquake	Significantly more damage and greater life-safety risk.	Somewhat more damage and slightly higher life-safety risk.	Less damage and low life-safety risk.	Much less damage and very low life-safety risk.

performance objectives are essentially different from each other. The primary consideration for Immediate Occupancy is that the damage is limited in such a manner as to permit reoccupation of the building, with limited repair work occurring while the building is occupied. The primary consideration for Life Safety is that a margin of safety against collapse be maintained and that consideration for occupants to return to the building is a secondary impact to the Life Safety objective being achieved. The Damage Control Performance Level provides for a greater margin of safety against collapse than would the Life Safety Performance Level. It might control damage in such a manner as to permit return to function more quickly than the Life Safety Performance Level but not as does quickly as the Immediate Occupancy Performance Level.

Life Safety Structural Performance Level (S-3). Structural Performance Level S-3, Life Safety, refers to the post-earthquake damage state in which significant damage to the structure has occurred but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this damage has not resulted in large falling debris hazards, either inside or outside the building. Injuries might occur during the earthquake; however, the overall risk of life-threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure; however, for economic reasons, this repair might not be practical. Although the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing before reoccupancy.

Limited Safety Structural Performance Level (S-4). The Limited Safety Structural Performance Level is set forth as a midway point between Life Safety and Collapse Prevention. It is intended to provide a structure with a greater reliability of resisting collapse than a structure that only meets the Collapse Prevention Performance Level but not to the full level of safety that the Life Safety Performance Level would imply.

Collapse Prevention Structural Performance Level (S-5). Structural Performance Level S-5, Collapse Prevention, refers to the post-earthquake damage state in which the building is on

Table C2-2. Structural Performance Levels and Illustrative Damage.

Seismic-Force-Resisting System	Type	Structural Performance Levels		
		Collapse Prevention (S-5)	**Life Safety (S-3)**	**Immediate Occupancy (S-1)**
Concrete frames	Primary elements	Extensive cracking and hinge formation in ductile elements. Limited cracking or splice failure in some nonductile columns. Severe damage in short columns.	Extensive damage to beams. Spalling of cover and shear cracking in ductile columns. Minor spalling in nonductile columns. Joint cracks.	Minor cracking. Limited yielding possible at a few locations. Minor spalling of concrete cover.
	Secondary elements	Extensive spalling in columns and beams. Limited column shortening. Severe joint damage. Some reinforcing buckled.	Major cracking and hinge formation in ductile elements. Limited cracking or splice failure in some nonductile columns. Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints.
	Drift	Transient drift sufficient to cause extensive nonstructural damage. Extensive permanent drift.	Transient drift sufficient to cause nonstructural damage. Noticeable permanent drift.	Transient drift that causes minor or no nonstructural damage. Negligible permanent drift.
Steel moment frames	Primary elements	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact. A few elements might experience partial fracture.	Hinges form. Local buckling of some beam elements. Severe joint distortion; isolated moment connection fractures, but shear connections remain intact.	Minor local yielding at a few places. No fractures. Minor buckling or observable permanent distortion of members.
	Secondary elements	Same as for primary elements.	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Same as for primary elements.
	Drift	Transient drift sufficient to cause extensive nonstructural damage. Extensive permanent drift.	Transient drift sufficient to cause nonstructural damage. Noticeable permanent drift.	Transient drift that causes minor or no nonstructural damage. Negligible permanent drift.
Braced steel frames	Primary and secondary elements	Extensive yielding and buckling of braces. Many braces and their connections might fail.	Many braces yield or buckle but do not totally fail. Many connections might fail.	Minor yielding or buckling of braces.
	Drift	Transient drift sufficient to cause extensive nonstructural damage. Extensive permanent drift.	Transient drift sufficient to cause nonstructural damage. Noticeable permanent drift.	Transient drift that causes minor or no nonstructural damage. Negligible permanent drift.
Concrete walls	Primary elements	Major flexural or shear cracks and voids. Sliding at joints. Extensive crushing and buckling of reinforcement. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Some boundary element cracking and spalling and limited buckling of reinforcement. Some sliding at joints. Damage around openings. Some crushing and flexural cracking. Coupling beams: extensive shear and flexural cracks; some crushing but concrete generally remains in place.	Minor diagonal cracking of walls. Coupling beams experience diagonal cracking.

continues

Table C2-2 *(Continued)*. Structural Performance Levels and Illustrative Damage.

Seismic-Force-Resisting System	Type	Structural Performance Levels		
		Collapse Prevention (S-5)	Life Safety (S-3)	Immediate Occupancy (S-1)
	Secondary elements	Panels shattered and virtually disintegrated.	Major flexural and shear cracks. Sliding at construction joints. Extensive crushing. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Minor cracking of walls. Some evidence of sliding at construction joints. Coupling beams experience x-cracks. Minor spalling.
	Drift	Transient drift sufficient to cause extensive nonstructural damage. Extensive permanent drift.	Transient drift sufficient to cause nonstructural damage. Noticeable permanent drift.	Transient drift that causes minor or no nonstructural damage. Negligible permanent drift.
Unreinforced masonry infill walls*	Primary and secondary	Extensive cracking and crushing; portions of outer wythe shed; some infill walls on the verge of falling out.	Extensive cracking and some crushing but wall remains in place. No falling units. Extensive crushing and spalling of veneers at corners of openings and configuration changes.	Minor cracking of masonry infills and veneers. Minor spalling in veneers at a few corner openings.
	Drift	Transient drift sufficient to cause extensive nonstructural damage. Extensive permanent drift.	Transient drift sufficient to cause nonstructural damage. Noticeable permanent drift.	Transient drift that causes minor or no nonstructural damage. Negligible permanent drift.
Unreinforced masonry (noninfill) walls	Primary elements	Extensive cracking; face course and veneer might peel off. Noticeable in-plane and out-of-plane offsets.	Major cracking. Noticeable in-plane offsets of masonry and minor out-of-plane offsets.	Minor cracking of veneers. Minor spalling in veneers at a few corner openings. No observable out-of-plane offsets.
	Secondary elements	Nonbearing panels dislodge.	Same as for primary elements.	Same as for primary elements.
	Drift	Transient drift sufficient to cause extensive nonstructural damage. Extensive permanent drift.	Transient drift sufficient to cause nonstructural damage. Noticeable permanent drift.	Transient drift that causes minor or no nonstructural damage. Negligible permanent drift.
Reinforced masonry walls	Primary elements	Crushing; extensive cracking. Damage around openings and at corners. Some fallen units.	Major cracking distributed throughout wall. Some isolated crushing.	Minor cracking. No out-of-plane offsets.
	Secondary elements	Panels shattered and virtually disintegrated.	Crushing; extensive cracking; damage around openings and at corners; some fallen units.	Same as for primary elements.
	Drift	Transient drift sufficient to cause extensive nonstructural damage. Extensive permanent drift.	Transient drift sufficient to cause nonstructural damage. Noticeable permanent drift.	Transient drift that causes minor or no nonstructural damage. Negligible permanent drift.
Wood stud walls	Primary elements	Connections loose. Nails partially withdrawn. Some splitting of members and panels. Sheathing pulled away from studs.	Moderate loosening of connections and minor splitting of members.	Distributed minor hairline cracking of gypsum and plaster veneers, primarily at door and window openings.
	Secondary elements	Sheathing sheared off. Let-in braces fractured and buckled. Framing split and fractured.	Connections loose. Nails partially withdrawn. Some splitting of members and panels.	Same as for primary elements.

continues

Table C2-2 *(Continued)*. Structural Performance Levels and Illustrative Damage.

Seismic-Force-Resisting System	Type	Structural Performance Levels		
		Collapse Prevention (S-5)	Life Safety (S-3)	Immediate Occupancy (S-1)
	Drift	Transient drift sufficient to cause extensive nonstructural damage. Extensive permanent drift.	Transient drift sufficient to cause nonstructural damage. Noticeable permanent drift.	Transient drift that causes minor or no nonstructural damage. Negligible permanent drift.
Cold-formed steel light-frame construction with wood structural panel shear walls	Primary elements	Connections loose. Screw hole deformation at panels and members. Some screws withdrawn. Significant yielding and distortion of members. Significant damage to panels and/or anchors. Loose connections of hold-downs to studs.	Moderate loosening of connections and minor yielding of members. Some damage to panels.	Distributed minor hairline cracking of gypsum and plaster veneers applied to shear walls, primarily at door and window openings.
	Secondary elements	Sheathing sheared off. Members yielded with significant distortion. Many broken windows, major sheetrock cracks, inoperable doors.	Connections loose. Screws partially withdrawn. Some yielding of members and damage to panels. Moderate cracking of sheetrock, several broken windows.	Similar to primary elements.
	Drift	Transient drift sufficient to cause extensive nonstructural damage. Significant permanent drift.	Transient drift sufficient to cause nonstructural damage. Noticeable permanent drift.	Transient drift that causes minor or no nonstructural damage. Negligible permanent drift.
Cold-formed steel light-frame construction with steel sheet sheathing shear walls	Primary elements	Connections loose. Screw hole deformation at panels and members. Some screws withdrawn. Some yielding of members and panels. Some out-of-plane deformation (buckling) of the steel sheet sheathing panels. Possible damage to anchors. Loose connections of hold-downs to studs.	Moderate loosening of connections and minor yielding of members and panels.	Distributed minor hairline cracking of gypsum and plaster veneers, primarily at door and window openings.
	Secondary elements	Sheathing sheared off. Members yielded with significant distortion. Many broken windows, major sheetrock cracks, inoperable doors.	Connections loose. Screws partially withdrawn. Some yielding of members. Moderate cracking of sheetrock panel, several broken windows.	Similar to primary elements.
	Drift	Transient drift sufficient to cause extensive nonstructural damage. Significant permanent drift.	Transient drift sufficient to cause nonstructural damage. Noticeable permanent drift.	Transient drift that causes minor or no nonstructural damage. Negligible permanent drift.
Cold-formed steel light-frame construction with strap-braced walls	Primary elements	Extensive yielding of straps. Some straps and connections might fail. Some yielding or buckling of boundary elements. Possible damage to anchors.	Many straps yield but do not fracture. A limited number of connections might fail. Minor yielding or buckling of boundary elements.	Minor yielding of straps. No damage to connections, boundary elements or anchors. Minor elongation of screw holes at strap connections.

continues

Table C2-2 *(Continued)*. Structural Performance Levels and Illustrative Damage.

Seismic-Force-Resisting System	Type	Structural Performance Levels		
		Collapse Prevention (S-5)	Life Safety (S-3)	Immediate Occupancy (S-1)
	Secondary elements	Sheathing sheared off. Members yielded with significant distortion. Many broken windows, major sheetrock cracks, inoperable doors.	Connections loose. Screws partially withdrawn. Some yielding of members and straps. Moderate cracking of sheetrock panels, several broken windows.	Similar to primary elements.
	Drift	Transient drift sufficient to cause extensive nonstructural damage. Significant permanent drift.	Transient drift sufficient to cause nonstructural damage. Noticeable permanent drift.	Transient drift that causes minor or no nonstructural damage. Negligible permanent drift.
Precast concrete walls	Primary elements	Some wall connection failures, but no wall elements dislodged.	Local crushing and spalling at wall connections, but no gross failure of connections.	Minor working and cracking at connections.
	Secondary elements	Same as for primary elements.	Some connection failures, but no elements dislodged.	Same as for primary elements.
	Drift	Transient drift sufficient to cause extensive nonstructural damage. Extensive permanent drift.	Transient drift sufficient to cause nonstructural damage. Noticeable permanent drift.	Transient drift that causes minor or no nonstructural damage. Negligible permanent drift.
Foundations	General	Significant settlement and tilting of buildings with shallow foundations or buildings on liquefiable soils.	Localized settlement of buildings with shallow foundations.	Minor settlement and negligible tilting.
Diaphragms	Metal deck	Large distortion with buckling of some units and tearing of many welds and seam attachments. Withdrawal or shearing of many fasteners.	Some localized failure of welded or mechanical connections of deck to framing and between panels. Minor local buckling of deck.	Connections between deck units and framing intact. Minor distortions.
	Wood	Large permanent distortion with partial withdrawal of nails and extensive splitting of elements.	Some splitting at connections. Loosening of sheathing. Observable withdrawal of fasteners. Splitting of framing and sheathing.	No observable loosening or withdrawal of fasteners. No splitting of sheathing or framing.
	Wood structural panel on cold-formed steel light-frame construction	Large permanent distortion with partial withdrawal of screws and extensive splitting of wood sheathing or yielding of cold-formed steel framing.	Some splitting at connections. Loosening of wood sheathing. Observable withdrawal of fasteners. Splitting of wood sheathing. Yielding of cold-formed steel framing.	No observable loosening or withdrawal of fasteners. No splitting of wood sheathing. No yielding of cold-formed steel framing.
	Cast-in-place concrete	Extensive crushing and observable offset across many cracks.	Extensive cracking. Local crushing and spalling.	Distributed cracking. Some minor cracks of larger size.
	Precast concrete	Connections between units fail. Units shift relative to each other. Crushing and spalling at joints.	Extensive cracking. Local crushing and spalling.	Some minor cracking along joints.

*For limiting damage to frame elements of infill frames, refer to the rows for concrete or steel frames.

the verge of partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral-force-resisting system, large permanent lateral deformation of the structure, and—to a more limited extent—degradation in vertical-load-carrying capacity. However, all significant components of the gravity-load-resisting system must continue to carry their gravity loads. Significant risk of injury caused by falling hazards from

Table C2-3. Nonstructural Performance Levels and Illustrative Damage: Architectural Components.

	Nonstructural Performance Levels		
Component Group	**Life Safety (N-C)**	**Position Retention (N-B)**	**Operational (N-A)**
Cladding panels	Distortion in connections and damage to cladding components, including loss of weather-tightness and security. Overhead panels do not fall.	Distortion in connections and damage to cladding components, including loss of weather-tightness and security. Overhead panels do not fall.	Negligible damage to panels and connections. No loss of function or weather-tightness.
Glazing	Some cracked panes; none broken. Limited loss of weather-tightness.	Some cracked panes; none broken. Limited loss of weather-tightness.	No cracked or broken panes. No loss of function or weather-tightness.
Heavy partitions (masonry and hollow clay tile or stud walls with tile or masonry veneer)	Distributed damage; cracking, crushing, and dislodging of veneer or parge coat in some areas. Damage to adjacent ceiling, but no wall failure.	Distributed damage; cracking, crushing, and dislodging of veneer or parge coat in some areas.	Minor crushing and cracking at corners. Limited dislodging of veneer or parge coat.
Light partitions (plaster and gypsum)	Distributed damage; some severe cracking of sheathing and racking in some areas.	Cracking at openings. Minor cracking of sheathing.	Minor cracking.
Ceilings	Extensive damage to suspended acoustical ceilings and grids. Plaster ceilings cracked and spalled but do not drop as a unit. Tiles in grid ceilings dislodged and falling; grids distorted and pulled apart. Plaster and gypsum board ceilings cracked and spalled but did not drop as a unit.	Limited damage. Plaster ceilings cracked and spalled but did not drop as a unit. Suspended ceiling grids largely undamaged, although individual tiles have fallen.	Generally negligible damage with no impact on reoccupancy or functionality.
Parapets and ornamentation	Minor damage; some falling of unreinforced elements in unoccupied areas.	Minor damage.	Negligible damage.
Canopies and marquees	Some damage to the elements, but essentially in place.	Some damage to the elements, but essentially in place.	Minor damage to the elements.
Chimneys and stacks	Minor damage. No collapse.	Minor damage. No collapse.	Negligible damage.
Stairs and fire escapes	Minor damage. Usable.	Minor damage. Usable.	Negligible damage.

Notes: This table describes damage patterns commonly associated with Nonstructural components for Nonstructural Performance Levels. The anticipated performance of components for Hazards Reduced Performance Level are intended to be the same as for Life Safety Performance Level only for those components evaluated or retrofitted to that Performance Level. The damage states described in the table might occur in some elements at the Nonstructural Performance Level, but it is unlikely that all the damage states described will occur in all components at that Nonstructural Performance Level. The descriptions of damage states do not replace or supplement the quantitative definitions of performance provided elsewhere in this standard and are not intended for use in post-earthquake evaluation of damage or for judging the safety of, or required level of repair to, a structure after an earthquake. They are presented to assist engineers using this standard to understand the relative degrees of damage at each defined performance level.

Damage patterns in nonstructural elements depend on the modes of behavior of those elements. More complete descriptions of damage patterns and levels of damage associated with damage levels can be found in other documents, such as FEMA E-74 (2011).

structural debris might exist. The structure might not be technically practical to repair and is not safe for reoccupancy because aftershock activity could induce collapse.

Structural Performance Not Considered (S-6). Some owners might desire to address certain nonstructural vulnerabilities in an evaluation or retrofit program—for example, bracing parapets or anchoring hazardous material storage containers—without addressing the performance of the structure itself. Such retrofit programs are sometimes attractive because they can permit a significant reduction in seismic risk at relatively low cost.

C2.2.2 Nonstructural Performance Levels

Nonstructural Performance Levels other than Hazards Reduced (N-D) and Not Considered (N-E) are summarized in Tables C2-3, C2-4, and C2-5. The Hazards Reduced Nonstructural Performance Level is not included in the table because it is simply a subset of the Life Safety Nonstructural Performance Level, which limits the items that are considered. For items that are considered in both Life Safety and Hazards Reduced, the anticipated performance is the same. Between the discrete Nonstructural Performance Levels, there are ranges of performance that can result from a partial set of nonstructural components meeting a discrete performance level and the remainder of the nonstructural components meeting a lower performance level. The Not Considered (N-E) Performance Level is intended to denote the performance level for which nonstructural components have not been evaluated, installed, or retrofitted, with specific attention paid to seismic design, or a situation in which only selected components have been retrofitted but not enough to fully conform to the Life Safety

Table C2-4. Nonstructural Performance Levels and Illustrative Damage: Mechanical, Electrical, and Plumbing Systems and Components.

System or Component Group	Nonstructural Performance Levels		
	Life Safety (N-C)	**Position Retention (N-B)**	**Operational (N-A)**
Elevators	Elevators out of service; cab and counterweights may be damaged but do not dislodge.	Elevators out of service until safety switches reset and power restored; cab and counterweight do not dislodge.	Elevators operate once safety switches are reset.
HVAC equipment	Units shifted on supports, rupturing attached ducting, piping, and conduit, but did not fall. Units might not operate.	Units are secure and possibly operate if power and other required utilities are available.	Units are secure and operate if emergency power and other utilities provided.
Manufacturing equipment	Units secure but potentially not operable.	Units secure but potentially not operable.	Units secure and operable if power and utilities available.
Ducts	Ducts broken loose from equipment and louvers; limited sections of ductwork dislodge.	Minor damage but ducts remain serviceable.	Negligible damage.
Piping	Some lines rupture at joints. Some supports damaged, but systems remain suspended.	Minor leaks develop at a few joints. Some supports damaged but systems remain suspended.	Negligible damage.
Fire suppression piping	Some sprinkler heads damaged by swaying ceilings. Minor leakage at a few heads or pipe joints. System remains operable.	Minor leakage at a few heads or pipe joints. System remains operable.	Negligible damage. System remains operable.
Fire alarm systems	Ceiling-mounted sensors damaged. Might not function.	System is functional.	System is functional.
Emergency lighting	Some lights fall. Power might be available from emergency generator or battery.	Some lights fall. Power might be available from emergency generator or battery.	System is functional.
Electrical distribution equipment	Units shift on supports and might not operate. Generators provided for emergency power start; utility service lost.	Units are secure and generally operable. Emergency generators start but might not be adequate to service all power requirements.	Units are functional. Emergency power is provided, as needed.
Light fixtures	Minor damage. Some pendant lights damaged.	Minor damage. Some pendant lights damaged.	Negligible damage.
Plumbing	Some fixtures broken, lines broken, but systems remain suspended.	Fixtures and lines may be damaged but serviceable; however, utility service might not be available.	System is functional if on-site water supply provided.

Notes: This table describes damage patterns commonly associated with Nonstructural components for Nonstructural Performance Levels. The anticipated performance of components for Hazards Reduced Performance Level are intended to be the same as for Life Safety Performance Level only for those components evaluated or retrofitted to that performance level. The damage states described in the table might occur in some elements at the Nonstructural Performance Level, but it is unlikely that all the damage states described will occur in a component at that Nonstructural Performance Level. The descriptions of damage states do not replace or supplement the quantitative definitions of performance provided elsewhere in this standard and are not intended for use in post-earthquake evaluation of damage or for judging the safety of, or required level of repair to, a structure after an earthquake. They are presented to assist engineers using this standard to understand the relative degrees of damage at each defined performance level.

Damage patterns in nonstructural elements depend on the modes of behavior of those elements. More complete descriptions of damage patterns and levels of damage associated with damage levels can be found in other documents, such as FEMA E-74 (2011).

Nonstructural Performance Level. For some nonstructural components at the Not Considered Performance Level (N-E), the typical installation or attachment details for the nonstructural component might provide some nominal capacity to resist seismic forces, including resistance by the use of friction.

For simplicity and ease of use, this standard treats Nonstructural Performance Levels N-A through N-C as cumulative. That is, any provision required to achieve N-B performance is also required to achieve N-A performance, and any provision required to achieve N-C performance is also required to achieve N-A and N-B performance. Although this is rational in most cases, there are cases in which a safety-related N-C provision might have little actual relevance to a cost- or downtime-based objective. For example, an unessential piece of overhead equipment or an unreinforced masonry partition might legitimately threaten safety during the shaking, but if the damage is easily contained and the component is easily removed, repaired, or replaced, the effect on functional recovery is likely to be small. Nevertheless, for purposes of creating a usable and enforceable standard, these cases are not formally recognized as exceptions. Negotiation of scope exceptions among stakeholders on a given project or mitigation program is outside the scope of this standard.

Table C2-5. Nonstructural Performance Levels and Illustrative Damage: Contents.

	Nonstructural Performance Levels		
Contents	**Life Safety (N-C)**	**Position Retention (N-B)**	**Operational (N-A)**
Storage Racks	Localized damage to rack system. Spilled contents.	Unrestrained contents toppled.	Restrained contents remain on shelves.
Bookshelves	Spilled contents.	Unrestrained contents toppled.	Most contents remain on shelves.
Hazardous Materials	Negligible damage; materials contained.	Negligible damage; materials contained.	Negligible damage; materials contained.

Notes: This table describes damage patterns commonly associated with nonstructural components for Nonstructural Performance Levels. The anticipated performance of components for Hazards Reduced Performance Level are intended to be the same as for Life Safety Performance Level only for those components evaluated or retrofitted to that performance level. The damage states described in the table might occur in some elements at the Nonstructural Performance Level, but it is unlikely that all the damage states described will occur in a component at that Nonstructural Performance Level. The descriptions of damage states do not replace or supplement the quantitative definitions of performance provided elsewhere in this standard and are not intended for use in post-earthquake evaluation of damage or for judging the safety of, or required level of repair to, a structure after an earthquake. They are presented to assist engineers using this standard to understand the relative degrees of damage at each defined performance level.

Damage patterns in nonstructural elements depend on the modes of behavior of those elements. More complete descriptions of damage patterns and levels of damage associated with damage levels can be found in other documents, such as FEMA E-74 (2011).

By necessity, this standard is generic with respect to building uses. Although certain Nonstructural Performance Levels might be more or less appropriate for certain large classes of buildings (e.g., buildings assigned to different Risk Categories as defined by the applicable regulations, building codes, policy standards, or ASCE 7), the standard does not distinguish between actual uses within a class. For example, a rational safety-based objective for an assisted living facility or daycare center might consider certain vulnerabilities that would be reasonably ignored in an office building. Similarly, a downtime-based objective for an apartment building might reasonably require less attention to certain items than a downtime-based objective for a restaurant or department store that provides a public accommodation or for a manufacturing facility sensitive to dust and debris. Customized scopes that borrow from the N-A, N-B, and N-C provisions thus make sense for special occupancies. Nevertheless, this standard provides only generic provisions expected to apply to most buildings similarly situated. Again, negotiation of scope exceptions among stakeholders on a given project or mitigation program is outside the scope of this standard.

Operational Nonstructural Performance Level (N-A). At this performance level, most nonstructural systems required for normal use of the building are functional, although minor cleanup and repair of some items might be required. Achieving the Operational Nonstructural Performance Level requires considerations of many elements beyond those that are normally within the sole province of the structural engineer's responsibilities. For N-A performance, in addition to ensuring that nonstructural components are properly mounted and braced within the structure, it is often necessary to provide emergency standby equipment to provide utility services from external sources that might be disrupted. It might also be necessary to perform qualification testing to ensure that all necessary equipment will function during or after strong shaking.

Specific design procedures and acceptance criteria for this Nonstructural Performance Level are included in this standard. One of the major requirements for Operational Nonstructural Performance is equipment certification for function following the design Seismic Hazard Level event. The following documents, although they do not comprise a complete set of references, might be useful for qualifying equipment for Operational Nonstructural Performance:

1. AC156. *Acceptance Criteria for Seismic Certification by Shake-Table Testing of Nonstructural Components* (ICC-ES 2010).
2. DOE/EH-545. *Seismic Evaluation Procedure for Equipment in U.S. Department of Energy Facilities* (US Department of Energy 1997).
3. IEEE 693. *IEEE Recommended Practice for Seismic Design of Substations* (IEEE 1997).
4. CERL Technical Report 97/58. *The CERL Equipment Fragility and Protection Procedure (CEFAPP): Experimental Definition of Equipment Vulnerability to Transient Support Motions* (Wilcoski et al. 1997).
5. ASCE 7-22. *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 2022).

Requirements and criteria for seismic qualification testing are outside the scope of this standard. Nevertheless, where such testing is performed, the general philosophy of this standard suggests that the testing protocols and documentation should be independently peer-reviewed for adequacy by a qualified structural engineer. Design review procedures similar to those in Sections 14.7 and 15.7 might be appropriate.

The Operational Nonstructural Performance Level essentially mirrors the requirements of ASCE 7 nonstructural seismic provisions for cases where I_p is taken as 1.5. Chapter 13 of ASCE 7 and its associated commentary provide additional detail.

Position Retention Nonstructural Performance Level (N-B). This level of performance is more restrictive than the Life Safety level because it involves bracing and anchorage of certain components that, based on their past performance, are not expected to pose significant risks to Life Safety.

Presuming that the building is structurally safe, occupants of a building or space performing at the N-B level are able to occupy the building safely, although normal use might be impaired, some cleanup might be needed, and some inspection might be warranted. In general, building equipment is secured in place and might be able to function if the necessary utility service is available. However, some components might experience misalignments or internal damage and be inoperable. Power, water, natural gas, communications lines, and other utilities required for normal building use might not be available. Cladding, glazing, ceilings, and partitions might be damaged but would not present safety hazards or unoccupiable conditions. The risk of life-threatening injury caused by nonstructural damage is very low.

The Position Retention Performance Level essentially mirrors the requirements of ASCE 7 nonstructural seismic provisions for cases where I_p is taken as 1.0. Chapter 13 of ASCE 7 and its associated commentary provide additional detail.

Life Safety Nonstructural Performance Level (N-C). In a building performing at the N-C level, nonstructural components might have sustained significant and costly damage, but they would not become dislodged and fall in a manner that could cause death or serious injury, either to occupants or to people in immediately adjacent areas. Egress routes within the building are not extensively blocked but might be impaired by lightweight structural, architectural, mechanical, or furnishings debris, but Life Safety systems (including fire suppression systems) and hazardous materials storage and distribution should be functional.

Hazards Reduced Nonstructural Performance Level (N-D). Hazards Reduced Nonstructural Performance (N-D) represents a post-earthquake damage state in which extensive damage has occurred to nonstructural components, but large or heavy items that pose a high risk of falling hazard to a large number of people—such as parapets, cladding panels, heavy walls or ceilings, or storage racks—are prevented from falling. The hazards associated with exterior components along portions of the exterior of the building that are available for public occupancy have been reduced. Although isolated serious injury could occur from falling debris, failures that could injure large numbers of persons—either inside or outside the structure—should be avoided. The philosophy is to provide a nonstructural performance level that has the same life safety consequences as a partial or total collapse of a building, injuring or killing many people as opposed to one or two, which is what the Life Safety Nonstructural Performance Level addresses. Chapter 13 in ASCE 7 allows for judgment-based determination of whether the hazard poses a threat to many people, as opposed to one or two people. The decision was made to keep this somewhat judgment-based because there was no means by which a specific number of people could be affected. Nonstructural components that are small, lightweight, or close to the ground may fall but should not cause serious injury. Larger nonstructural components in areas that are less likely to be populated may also fall.

The intent of the Hazards Reduced Performance Level is to address significant nonstructural hazards that pose a threat to multiple people without needing to rehabilitate all of the nonstructural components in a building. Chapter 13 in ASCE 7 provides language that permits a component to be exempt from the Hazards Reduced Nonstructural Performance Level if it can be demonstrated that the failure or falling hazard of the component will not pose a risk of serious injury to multiple people. This is done to permit falling hazards in unoccupied areas to be ignored and focus the user on areas where significant risk is greatest, such as egress areas and public assembly areas. When using this performance level, it is generally appropriate to consider Hazards Reduced Performance as equivalent to Life Safety Performance for the most hazardous, highest-risk subset of the nonstructural components in the building.

Nonstructural Performance Not Considered (N-E). In some cases, the decision to rehabilitate the structure might be made without addressing the vulnerabilities of nonstructural components. In practice, this decision is often made where nonstructural mitigation would disrupt normal uses of the building. Because many more earthquake-related deaths result from structural collapse than from nonstructural hazards, mitigation programs focused on reducing casualties might reasonably require only structural evaluation and retrofit. Another possibility is to address structural issues and only those nonstructural hazards where very heavy elements can fall on occupants or hazards around the perimeter of the building. The crushing injuries caused by falling hazards have a higher likelihood of life loss than other types of earthquake-caused injuries. For example, parapet bracing ordinances were one of the first seismic building safety requirements because these nonstructural elements were observed to fail at earthquake ground motions much lower than those that damaged most buildings.

Mitigation of any select subset of high-hazard nonstructural elements, where the subset is less than the complete set required for Hazards Reduced Nonstructural Performance (N-D), would fall under this performance level solely because all nonstructural hazards not included in that performance level would not have been addressed in a manner sufficient to qualify for Hazards Reduced Nonstructural Performance (N-D).

Designation of Building Performance Levels Several common target Building Performance Levels described in this section are shown in Figure C2-1. Many combinations are possible because structural performance can be selected at any level in the two Structural Performance Ranges. Table C2-6 indicates some of the possible combinations of target Building Performance Levels and provides names for those most likely to be selected as the basis for design.

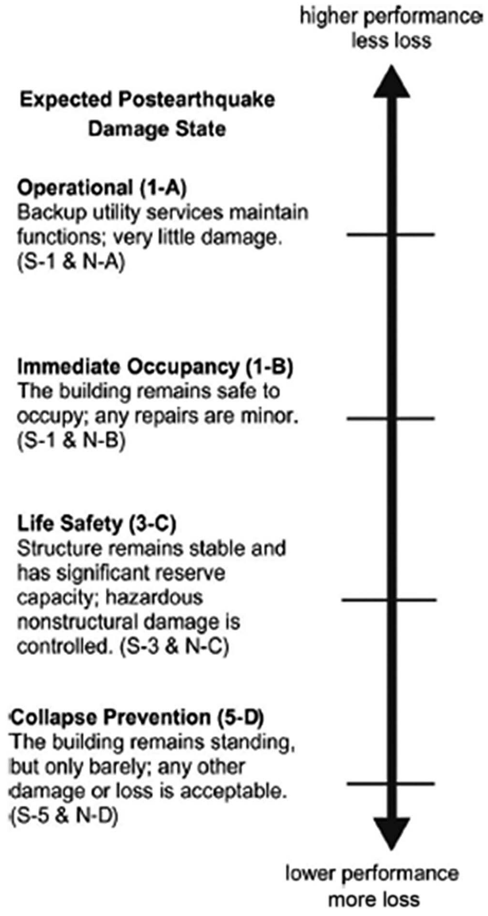


Figure C2-1. Target building performance levels and ranges.

Table C2-6. Target Building Performance Levels.

Nonstructural Performance Levels	Structural Performance Levels: Immediate Occupancy (S-1)	Damage Control (S-2)	Life Safety (S-3)	Limited Safety (S-4)	Collapse Prevention (S-5)	Not Considered (S-6)
Operational (N-A)	Operational 1-A	2-A	NR	NR	NR	NR
Position Retention (N-B)	Immediate Occupancy 1-B	2-B	3-B	4-B	NR	NR
Life Safety (N-C)	1-C	2-C	Life Safety 3-C	4-C	5-C	6-C
Hazards Reduced (N-D)	NR	NR	3-D	4-D	Collapse Prevention 5-D	6-D
Not Considered (N-E)	NR	NR	3-E	4-E	5-E	No evaluation or retrofit

Notes: NR = Not recommended. Combining low Structural Performance Level with high Nonstructural Performance Level, or the converse, is not recommended for several reasons. For example, having a low Structural Performance Level may lead to damage that prohibits actually achieving the desired Nonstructural Performance Level regardless of whether the nonstructural elements were retrofit to meet that Performance Level. In addition, addressing nonstructural hazards when a higher Structural Performance Level retrofit is undertaken may lead to an unbalanced design, where life-safety hazards caused by nonstructural items are still present.

Operational Building Performance Level (1-A). Buildings meeting this target Building Performance Level are expected to sustain minimal or no damage to their structural and nonstructural components. The building is suitable for its normal occupancy and use, although possibly in a slightly impaired mode, with power, water, and other required utilities provided from emergency sources, and possibly with some nonessential systems not functioning. Buildings meeting this target Building Performance Level pose an extremely low Life Safety risk.

Under very low levels of earthquake ground motion, most buildings should be able to meet or exceed this target Building Performance Level. Typically, it is not economically practical to modify existing buildings to meet this target Building Performance Level for severe ground shaking, except for buildings that house essential services.

Immediate Occupancy Building Performance Level (1-B). Buildings meeting this target Building Performance Level are expected to sustain minimal or no damage to their structural elements and only minor damage to their nonstructural components. Although it would be safe to reoccupy a building meeting this target Building Performance Level immediately after a major earthquake, nonstructural systems might not function, either because of the lack of electrical power or internal damage to equipment. Therefore, although immediate reoccupancy of the building is possible, it might be necessary to perform some cleanup and repair and await the restoration of utility service before the building can function in a normal mode. The risk to Life Safety at this target Building Performance Level is very low.

Many building owners might wish to achieve this level of performance when the building is subjected to moderate earthquake ground motion. In addition, some owners might desire such performance for very important buildings under severe earthquake ground shaking. This level provides most of the protection obtained under the Operational Building Performance Level without the cost of providing standby utilities and performing rigorous seismic qualification of equipment performance.

Life Safety Building Performance Level (3-C). For purposes of this document, the term *Life Safety* as a seismic performance descriptor is used in a specific way. A building conforming to a Life Safety description does not mean that there will be no injuries to occupants or persons in the immediate vicinity of the building in an earthquake of the Seismic Hazard Level assessed, but few, if any, of the occupant injuries are expected to be serious enough to require skilled medical attention for the injured person to survive. An injury to a person that occurs because of the earthquake performance of a building evaluated as not *life safe* is one that requires skilled medical attention within 24 hours of the injury for the person to survive. It is recognized that many injuries, indeed most, that could occur to occupants of a building are not likely to be evaluated as posing a survival threat because the injury could be treated by first aid treatment. An injury might be evaluated as consistent with this Life Safety descriptor even though the person has been injured.

Buildings meeting this level may experience extensive damage to structural and nonstructural components. Repairs may be required before reoccupancy of the building occurs, and repair may be deemed economically impractical. The risk to Life Safety in buildings meeting this target Building Performance Level is low.

This target Building Performance Level may entail more damage than anticipated for new buildings that have been properly designed and constructed for seismic resistance when subjected to their design earthquakes. Building owners may desire to meet this target Building Performance Level for severe ground shaking.

Collapse Prevention Building Performance Level (5-D). For purposes of this document, the term *collapse prevention* as a seismic performance descriptor is used in a specific way. Buildings meeting this level may experience extensive damage to structural and nonstructural components. Nonstructural falling hazards that cause serious injury or death may occur. Total or partial building collapse is not anticipated to occur at this level. Major nonstructural falling hazards that can cause serious injury or death to large numbers of people are not likely to occur. Extensive repairs may be required before reoccupancy of the building occurs, and repair may be deemed economically impractical. There is a risk to safety in buildings meeting this target Building Performance Level.

This target Building Performance Level will likely entail more damage than anticipated for new buildings that have been properly designed and constructed for seismic resistance when subjected to their design earthquakes. Building owners may desire to meet this target Building Performance Level for the most severe ground shaking.

C2.3 SEISMIC HAZARD

Although the performance objective options featured in this standard allow consideration of any Seismic Hazard Level that might be of interest. It is intended that the seismic hazards used in this standard are derived using a similar theoretical basis and methodology as the hazards developed for the design of new buildings in ASCE 7, regardless of whether the hazards are intended to be the same, based on different probabilities of exceedance, other risk targets, or specific deterministic events. Therefore, the standard directly references ASCE 7 for much of the determination of seismic hazard information and defers to ASCE 7 for requirements like the need for site specific procedures to determine the seismic hazard information. The user is referred to the corresponding commentary in Chapters 11, 20, 21, and 22 of ASCE 7 for further discussion on seismic hazard topics.

C2.3.1 Seismic Hazard This standard explicitly specifies four Seismic Hazard Levels for use with specific performance objectives in Section 2.4. Two of the specified hazards are the same hazards specified in ASCE 7, the risk-targeted maximum considered earthquake ($\mathrm{MCE_R}$) hazard and the design earthquake that is defined as two-thirds the $\mathrm{MCE_R}$ hazard, identified as the BSE-2N and BSE-1N, respectively. In the 2013 edition of the standard, when ASCE 31-03 was combined with ASCE 41-06, the standard's committee decided to retain the historic practice of using reduced hazard intensities to evaluate existing buildings. The other two hazards, the BSE-2E and the BSE-1E, are the correlated reduced hazards to the ASCE 7 new building hazards, BSE-2N and BSE-1N. The standard also permits the use of any other type of seismic hazard, provided the hazard used and information on how it was developed is explicitly provided in the evaluation report or the retrofit construction documents.

The general seismic hazard representations used in this standard use seismic hazard data prepared by the US Geologic Survey (USGS) National Seismic Hazard Modeling Project based on its 2018 update (Petersen et al. 2020). The 2018 USGS Hazard Model incorporates updated ground motion models in the central and eastern United States based on the NGA-East project and modifications to the ground motion models in deep portions of basins in the western United States, among many other changes with respect to their previous editions. Refer to the commentary of Chapter 21 of ASCE 7 for a more detailed discussion of the 2018 USGS Hazard Model.

It is important to note that the USGS hazard data incorporate adjustments from "geomean" ground motions (the product of hazard assessment using modern ground motion attenuation functions) to "maximum-direction" ground motions, for reasons explained in the Part 1 commentary of FEMA P-750 (2009b). The adjustment to get "maximum direction values" from "geomean values" is per Section 21.2 of ASCE 7. Although the USGS Seismic Design Geodatabase provides a ready source for this type of information, this standard may be used with approved seismic hazard data from any source, as long as the basis for the hazard data is clearly defined and specific information related to the development of the response spectrum or ground motion models used to represent the hazard are identified. Site-specific procedures can be used where available seismic hazard maps do not adequately characterize the local hazard or if ground motion models that better capture the specific seismic hazard are available. Such conditions might exist at some locations near active seismic faults. Such site-specific hazard values can be determined by a knowledgeable professional expert on such studies.

This standard requires that "maximum direction" values be used. However, at the discretion of the designer or regulator, the "geomean" values may be used for Reduced Performance Objective evaluations or retrofits by dividing the" maximum direction" response spectra by the adjustment factors in Section 21.2 of ASCE 7 or by site-specific procedures.

C2.3.1.1 BSE-2N Seismic Hazard Level The BSE-2N Seismic Hazard Level is consistent with the $\mathrm{MCE_R}$ ground motions in ASCE 7. The $\mathrm{MCE_R}$ ground motion was chosen for use with the new design code equivalent performance objectives so that consistent ground motion parameters are used between ASCE 7 and this standard. In most areas of the United States, the BSE-2N Seismic Hazard Level can be thought of as the acceleration parameters for a seismic hazard with a 2% probability of exceedance in 50 years (2% in 50 years) multiplied by a risk coefficient at each period. The resulting $\mathrm{MCE_R}$ ground motion, which can be larger or smaller than the 2% in 50-year values, is such that new buildings designed by the IBC (ICC 2021) for that ground motion have a 1% probability of collapse in 50 years (approximately). At sites close to known faults with significant slip rates, the $\mathrm{MCE_R}$ ground motion is limited by a deterministic estimate of ground motion based on the 84th-percentile shaking likely to be experienced in a scenario event. Ground-shaking levels determined in this manner typically correspond to risks of collapse greater than 1% in 50 years. The design professional is referred to FEMA P-1050 (2015) and Luco et al. (2007) for further discussion of $\mathrm{MCE_R}$ ground motions and risk targeting, respectively.

C2.3.1.2 BSE-1N Seismic Hazard Level The BSE-1N parameters are intended to match the design earthquake ground motions in ASCE 7 for use in the Basic Performance Objective Equivalent to New Building standards (BPON).

In building design provisions before the 1997 NEHRP (FEMA 1997e, d), the seismic hazard was generally based on an earthquake with a 10% probability of exceedance in 50 years. That hazard was retained in ASCE 41-06 as one of two options for the BSE-1, along with two-thirds of the MCE. Starting with the 1997 NEHRP provisions, and subsequently the 2000 IBC (ICC 2000), the 10% in 50-year Seismic Hazard Level is no longer explicitly referenced in new building design standards and is no longer explicitly referenced in this standard. This lack of inclusion in the standard's predefined Seismic Hazard Levels, however, does not prohibit the use of the 10% in 50-year ground motion as the Seismic Hazard Level for any performance objective other than the explicitly defined BPOE or BPON Performance Objectives.

C2.3.1.3 BSE-2E Seismic Hazard Level For the BSE-2E Seismic Hazard Level, the 5% in 50-year probability of exceedance was chosen initially because it represented ground motions approximately 75% as large as those prescribed for new buildings in California, where the 75% approach originated and has been most widely used (see Section C2.4.1). This definition has also been used in the California State Building Code for state buildings since the mid-1990s. Furthermore, when examining the anticipated risk of collapse using the same idealized fragility curves used in developing the Risk-Targeted Maximum Considered Earthquake ($\mathrm{MCE_R}$) hazard parameters in ASCE 7, one finds that on average the risk of collapse for structures designed using the 5% in 50-year Seismic Hazard Level is more uniform than would be achieved with a constant 75% demand adjustment factor.

Because of the deterministic caps placed on some of the probabilistic ground motions for new building designs, some of the 5% in 50-year hazard parameters are greater than their MCE_R counterparts. Given that the philosophy is to provide for lesser design parameters than for new buildings (as discussed in Section C2.4.1), it is not consistent to have the BSE-2E ground motions be greater than the BSE-2N values, notwithstanding the different bases of analysis of the two standards. It is for this reason that the 5% in 50-year hazard parameters are capped at the BSE-2N values. Furthermore, this limit means that in locations where the MCE_R demand is capped, the BSE-2E demand is the same as the BSE-2N demand (or more than 75% of it), eliminating some or all of the intended, traditional effect of the BPOE, as discussed in Section C2.4.1.

C2.3.1.4 BSE-1E Seismic Hazard Level The BSE-1E Seismic Hazard Level is the analogous reduction to BSE-1N as the BSE-2E is to the BSE-2N.

C2.3.1.5 Seismic Hazard Levels for Other Probabilities of Exceedance, Risk Targets, or Deterministic Hazards Response acceleration parameters other than those specifically defined in this standard can be used for any performance objectives other than the BPOE or BPON. Seismic hazard parameters based on the 2018 USGS Hazard Model for other probabilities of exceedance than the BSE-1E and BSE-2E are available from the USGS: (https://doi.org/10.5066/F7NK3C76). Other ways to represent Seismic Hazard Level, such as explicit deterministic hazards, may be used. While the USGS may not directly provide information for such hazards, site-specific procedures can be used.

Regardless of how hazards other than the four specified in Sections 2.3.1.1 through 2.3.1.4 are developed, if they are used as the basis for an evaluation or retrofit, they should clearly be documented. It is important to document how the hazard is defined—probabilistic, risk-targeted, or deterministic and if deterministic—whether it is the mean or some percentile or standard deviation from the mean. If the USGS Seismic Design Geodatabase was not used to develop the hazard parameters, the ground motion models and site-specific studies used to develop the response spectrum for the hazard should be summarized. Whether the hazard is based on a maximum direction, geomean or other representation should be clearly stated. If the spectrum is something other than a 5% damped spectrum, the damping ratio should be indicated.

C2.3.2 General Response Spectrum The 2018 USGS Hazard Model now provides data to construct a multiperiod response spectrum using 22 periods as opposed to two for a given site class. In developing these multiperiod response spectra, it became clear that the resolution between site classes could be too broad; therefore, three additional intermediate site classes—BC, CD, and DE— were incorporated into the provisions. The commentary to Chapters 11, 20, 21, and 22 of ASCE 7 provide a more detailed discussion on 2018 USGS Hazard Model and how it was adopted to produce the multi-period response spectra data. Use of the multiperiod response spectra is given precedence over the two-point spectra where such information is available. The USGS had multi period data for the United States and its territories).

C2.3.2.1 Multiperiod General Horizontal Response Spectrum The multiperiod design response spectrum from the USGS Seismic Design Geodatabase is intended to be the basis for the seismic hazard parameters in this standard, when used within the United States and its territories. The database provides unique spectrum ordinates for 22 periods between 0.0 s and 10 s. Instead of applying a factor to adjust for site class, a spectral ordinate for each site class exists in the database.

When spectral ordinates beyond 10 s are needed, the provisions assume that the spectrum's shape varies with either 1/T or $1/T^2$, depending on whether the period is less than or greater than the long-period transition factor, T_L. T_L is provided in the USGS Seismic Design Geodatabase for the BSE-1E and BSE-2E, in addition to the MCE_R.

The spectral ordinates and periods provided in the USGS Seismic Design Geodatabase are for a 5% damping ratio. The method to adjust for damping ratios other than 5% is the same as provided in this standard, where values beyond T_0 are divided by the parameter B_1 in Equation (2-1). For values between 0.0 and T_0, the factor by which to divide the spectral ordinate varies linearly from 1.0 at 0.0 s to B_1 at T_0.

Even when a multi-period response spectrum is used, this standard contains provisions that require the use of one or both of the two-point spectrum parameters S_{XS} and S_{X1}. Those parameters can be obtained from the multiperiod response spectrum using the provisions in Section 21.4 of ASCE 7. While that section is part of the site-specific ground motion procedures chapter of ASCE 7, it is not the intent that one needs to perform a site-specific analysis to determine the two-point spectrum parameters, it is simply the place in ASCE 7 where the rules to establish those parameters are located.

C2.3.2.2 Two-Period General Horizontal Response Spectrum Although the standard preferences the multi-period spectra, the committee chose to retain the procedure for constructing a response spectrum from two periods. This was done in recognition that the standard is used outside of the United States, where the USGS provides multi-period data, and seismic hazard information based on a short-period and a long-period may be the only information available. When that is the case, these provisions, which are essentially unchanged from the 2017 edition of the standard may be used. However, a mechanism to adjust periods for site effects is not provided in this standard or in the current edition of ASCE 7. One potential option to adjust for site effects is the provisions of ASCE 7-16, including Supplements 1 and 3.

C2.3.2.3 General Vertical Response Spectrum In previous editions of the standard, the vertical response spectra are taken as two-thirds of the horizontal spectrum developed for the site. Although this method produces a reasonable approximation for most sites, vertical response spectra at sites located within a few kilometers of the zone of fault rupture can have stronger vertical response spectra than those determined by this approximation. Since the 2016 edition, ASCE 7 has included provisions to develop a vertical response spectrum that is a function of the horizontal spectrum. These provisions are primarily for sites in the western United States, delineated as west of -105 longitude. For those sites, there are specific correlations based on the period range and the horizontal shaking intensity. An important change was made in the 2022 edition of ASCE 7 that adjusts the horizontal spectrum from the maximum direction to the geomean before applying the correlation factors. This aligns with the research that led to the development of the correlation factors, which used geomean spectra. For sites east of -105 longitude, ASCE 7 still uses the method of multiplying the horizontal spectrum by two-thirds to approximate the vertical spectrum.

C2.3.3 Site-Specific Procedure for Hazards Caused by Ground Shaking This section points to ASCE 7 for the procedures to develop a site-specific response spectrum. Where a probabilistically defined spectrum, such as the BSE-1E or BSE-2E, is desired, the procedure should be followed, except that there is no deterministic cap, but rather a cap of the BSE-1N or BSE-2N, respectively. The site-specific spectrum shall be anchored to the general response spectrum with a lower limit based on a percentage of the general response spectra from the USGS Seismic Design Geodatabase as directed in Chapter 21 of ASCE 7.

C2.3.4 Ground Motion Acceleration Histories Linear and nonlinear response history analyses require ground motion acceleration histories that are representative of the seismic hazard at the site. There is considerable variability in the manner in which the ground shaking occurs at a site, for example, because of earthquakes occurring on different faults near the site or by earthquakes of different magnitudes. Because of that variability, several different ground motion acceleration histories should be used when performing response history analysis. Also, because each specific ground motion acceleration history causes the structure to respond differently, there is dispersion in the response parameters. ASCE 7-16 requires the use of 11 records as the minimum number of ground motion acceleration histories.

Recognizing that actual earthquakes do not affect the structure in one direction only, pairs of horizontal records are required to be used when performing a three-dimensional analysis. Vertical records should be included when the provisions require the consideration of vertical seismic effects, per Section 7.2.6.2.

The general response spectra in Section 2.3.2 are uniform hazard response spectra, which aggregate seismic hazard from all known earthquake sources at the given site. When a single suite of ground motions is selected, the individual ground motion acceleration histories are chosen to match the uniform hazard spectrum over a relatively wide period range. Another option (Method 2) is to develop two or more target response spectra that together represent the hazard at the site and select two or more suites of ground motions that are each targeted to one of the target response spectra. An example of this approach is the conditional mean spectrum (CMS). With the second approach, lengthening of the elastic period of the model should be considered during period selection but is not required when the linear dynamic procedure is used. To address the difference between the selected ground motions and the target spectrum, the maximum-direction spectra from each pair are constructed for each point on the spectrum. After that, the spectra from the maximum direction from each pair are then averaged together. That average spectrum is then compared with the design response spectrum, and the records are scaled if that spectrum does not exceed the general response spectrum. Refer to ASCE 7, Chapter 16 commentary, for additional discussion.

The material in this section is based on changes made in the 2016 edition of ASCE 7. The majority of the requirements are the same. One of three major changes from the ASCE 7 material was the decision to retain the upper-bound period of the scaling range of 1.5 $T_{\max}$ instead of the increased value of 2.0 $T_{\max}$. This was increased in ASCE 7-16 because of concern that ductile structures could experience significant period lengthening due to inelastic response. However, it was felt that existing buildings generally do not possess the same level of ductility and therefore will not experience as significant lengthening of the structural response period. The second change includes placing a lower limit of 1 s on the upper-bound scaling range period, which was done to prevent underestimation of the period of stiff, short-period buildings where significant period elongation due to structural softening and soil–structure interaction may significantly increase the effective period of response.

The last exception limits the use of Method 2 (e.g., conditional mean spectrum) with spectral matching unless a realistic record-to-record dispersion is preserved in the suite after spectral matching. If spectral matching is used with conditional mean spectra (Method 2) without preserving dispersion, then none of the ground motion records will reach the target spectrum at any period except at the conditioning period. This can lead to an underestimation of seismic demands, especially on force-controlled components.

C2.4 PERFORMANCE OBJECTIVES

Performance objectives may be selected as basic, enhanced, or limited, as defined in Sections 2.4.1 through 2.4.3, or an objective intended to be equivalent with the provisions for new buildings, as defined in Section 2.4.4. Recommendations regarding the selection of a performance objective for any building are beyond the scope of this standard. FEMA 274 (1997b) discusses issues to consider when combining various Performance and Seismic Hazard Levels. Not all combinations constitute reasonable or cost-effective performance objectives.

This standard accommodates a myriad of performance objectives, including specific objectives that are intended to be equivalent to the performance objectives of buildings designed to new building standards and specific objectives that are intended to mimic the performance historically accepted for what is deemed “reduced code performance” in documents such as the *International Existing Building Code* (ICC 2021). These performance objectives provide Structural and Nonstructural Performance Levels at specifically defined Seismic Hazard Levels for buildings based on the different risk categories a building could be classified in based on the *International Building Code* (ICC 2021) or ASCE 7. Determination of which risk category a building should be classified in is outside the scope of this document.

Building performance can be described qualitatively in terms of the safety afforded to building occupants during and after the event; the cost and feasibility of restoring the building to its pre-earthquake condition; the length of time the building is removed from service to effect repairs; and economic, architectural, or historic effects on the larger community. These performance characteristics are directly related to the extent of damage that would be sustained by the building and its systems in the seismic event.

In this standard, the performance of a building’s structure and nonstructural components together in a specified earthquake ground motion is defined as a Building Performance Level.

This standard uses several probabilistic seismic hazard levels to describe earthquake ground motions for which performance evaluations are made, except in certain areas near active faults, where deterministic caps are imposed on the probabilistic hazard parameters. Such ground motions are often referred to either as a probability of exceedance in a specified period, say 20% probability of exceedance in 50 years, or as a return period for exceedance of the specified ground motion, such as 225 years. Table C2-7 shows the ground motion probabilities of exceedance and corresponding return period used in this standard.

Table C2-7. Probability of Exceedance and Mean Return Period.

Probability of Exceedance	Mean Return Period (years)
50%/30 years	43
50%/50 years	72
20%/50 years	225
10%/50 years	475
5%/50 years	975
2%/50 years	2,475

Table C2-8. Performance Objectives.

Seismic Hazard Level	Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-D)
	Target Building Performance Levels			
50%/50 years	a	b	c	d
BSE-1E (20%/50 years)	e	f	g	h
BSE-2E (5%/50 years)	i	j	k	l
BSE-2 N (ASCE 7 MCE_R)	m	n	o	p

Note: Each cell in this above matrix represents a discrete performance objective.

This standard defines four commonly used seismic hazard levels in Section 2.3.

The performance objective selected as a basis for design determines, to a great extent, the cost and feasibility of any project and the benefit to be obtained in terms of improved safety, reduction in property damage, and interruption of use in the event of future earthquakes. Table C2-8 indicates the range of performance objectives that might be considered in the use of this standard for a typical building, such as one classified under Risk Category II, based on the performance levels described in Section 2.2 and the Seismic Hazard Levels set forth in Section 2.3 for both structural and nonstructural system expected performance.

The performance objectives in Table C2-8 can be used to represent three types of performance objectives, as discussed next, that might be selected for a building that is assigned to Risk Category I or II, as shown in Table C2-9.

C2.4.1 Basic Performance Objective for Existing Buildings (BPOE) The Basic Performance Objective for Existing Buildings (BPOE) is one specific, named performance objective. This standard does not mandate specific performance objectives. It only defines them for use. The notation (S-N) in Tables 2-1 and 2-2 is used where S and N are the respective Structural Performance Levels and Nonstructural Performance Levels, as defined in Sections 2.2.1 and 2.2.2.

The BPOE varies by risk category. This standard does not specify how to assign a building to a risk category. Risk categories are used here to facilitate the coordination with regulations, building codes, and policies, such as the *International Building Code* (ICC 2021a) and the *International Existing Building Code* (ICC 2021b), which do use them. The intention is that regulations, building codes, and policies need to cover all risk categories but might prefer to cite this standard in a simple way. Defining the BPOE allows a regulation, building code, or policy to find a consistent set of objectives covering all risk categories in one place within this standard.

Table C2-9. Enhanced or Limited Performance Objectives.

Basic Performance Objective for Existing Buildings (BPOE)	g and l
Enhanced objectives	g and either i, j, m, n, o, or p
	l and either e or f
	g and l plus either a or b
	k, m, n, or o alone
Limited objectives	g alone
	l alone
	c, d, e, or f

The BPOE, or objectives close to it, has been used for characterizing seismic performance in other standards and regulations and has been implemented in many individual projects and mitigation programs. The BPOE also approximates the regulatory policy traditionally applied to existing buildings in many seismically active areas of the United States. The BPOE accepts a lower level of safety and a higher risk of collapse than what is provided by similar standards for new buildings. Buildings meeting the BPOE are expected to experience little damage from relatively frequent, moderate earthquakes but significantly more damage and potential economic loss from the most severe and infrequent earthquakes that could affect them. The level of damage and potential economic loss experienced by buildings rehabilitated to the BPOE likely will be greater than that expected in similar, properly designed and constructed new buildings or existing buildings evaluated and retrofit to the BPON, defined in Section 2.4.4.

There are three overarching historical reasons for accepting a somewhat greater risk in existing buildings:

- Accepting performance less than “full code” ensures that recent buildings are not immediately rendered deficient whenever the code changes in such a manner as to become more conservative.
- The increase in risk is tempered by the recognition that an existing building often has a shorter remaining life than a new building. That is, if the traditional code-based demand for new buildings presumes a 50-year life, then an existing building with, say, a 30-year life has a smaller chance of experiencing the code-level event over its remaining years (or an equivalent chance of experiencing a somewhat smaller maximum event). This rationale is less applicable when the retrofit is part of a change of occupancy to a higher Risk Category, or where the retrofit is part of a major renovation that “renews” the building or is intended to substantially extend its useful life.
- The BPOE recognizes that the cost of achieving the higher level of certainty in performance that comes with “new building equivalence” is often disproportionate to the incremental benefit. For new construction, building code provisions ensure a high probability of safety in the design earthquake (as well as a reasonable expectation of reparability). Because of more complete design flexibility and construction quality control, the new building code can achieve higher confidence for new buildings at marginal additional cost.

The constraints of existing buildings, however, often make the same level of performance reliability as a new building much more expensive. Therefore, whereas the BPOE seeks safety with reasonable confidence, it rationally reduces the incremental certainty of performance that comes cheaply with new construction but is costly for retrofit.

The traditional reasons for the lower performance objective might not apply in all cases. Nevertheless, the BPOE and similar objectives have been deemed appropriate for many mitigation programs and remain valuable for the precedent they provide. Where the desired (or required) performance is similar to that required of new buildings assigned to Risk Category III or IV, the BPOE has not traditionally been used and might not be appropriate. For those buildings, the evaluation or retrofit performance objective has sometimes been to a level consistent with a new building assigned to that risk category. As noted in Section C2. 4, however, the selection of what performance objective one should use is beyond the scope of this standard.

Past codes and guidelines allowed a higher risk similar to the BPOE by applying a reduction factor to the code-level force demand used to design the building. FEMA 178 (1992a), for example, modified the demand by factors of 0.67 or 0.85. This approach was retained in national model codes, such as the *International Building Code* and the *International Existing Building Code* (ICC 2021a, b), which allow a 0.75 factor on earthquake loads for certain triggered evaluations or retrofit. ASCE 31-03 achieved approximately the same effect by increasing component capacities, *m*-factors, in its Tier 2 procedure from the commensurate *m*-factors in ASCE 41-06 and by applying a 0.75 factor to code-based demands in its Tier 3 procedure.

Many jurisdictions have adopted such reductions in their building regulations for a long time. The cities of Long Beach, Los Angeles, Oakland, and San Francisco are among many communities that have used the 0.75 reduction for many decades. The *California Building Code* (CBSC 2010a) has, since the 1998 edition, permitted the use of a lower probabilistic hazard for retrofit of state-owned buildings of 20% in 50 years, where the traditional 10% in 50-year hazard was used for new building design. In some cases, there have also been hazardous building ordinances that required owners to undertake seismic safety evaluations and seismic retrofit using seismic hazards less than those for new building design for these actions. Thus, there is a precedent both in standards formulation and enforced building regulations for using a reduced hazard for the evaluation and retrofit of existing buildings.

Simply reducing the ground motion demand by a factor of 0.75 does not result in a spatially uniform hazard because of differences in the seismic hazard curves for different locations. For example, reducing 2% in a 50-year ground motion parameter in San Francisco by 25% results in a ground motion parameter with approximately a 5% in 50-year probability of exceedance, whereas the same 25% reduction in the 2% in 50-year ground motion for Memphis results in an approximately 3% in 50-year hazard.

Therefore, the seismic hazard used in the BPOE does not apply a single factor to the code-level demand. Instead, it specifies a different demand with a higher probability of exceedance. For new buildings, probabilities of exceedance of 2% in 50 years and 10% in 50 years have commonly been used (before the adoption of Risk-Targeted Maximum Considered Earthquake ground motions in ASCE 7). For the BPOE, the Seismic Hazard Levels are based on 5% in 50-year and 20% in 50-year probabilities of exceedance.

The three-tiered evaluation procedure requires a successively more complete engineering assessment of the expected seismic performance of the building, with successively more effort to determine compliance. Tier 1 screening requirements tend to be general and conservative in nature, Tier 2 procedures are more detailed, and Tier 3 procedures are specific and involved.

When these tiers were formulated, it was expected that a Tier 1 screening would identify more buildings as potentially unsafe than would a Tier 2 procedure because it used more exacting standards and significantly more work. Similarly, it was expected that a full-building, systematic Tier 3 assessment would find some buildings that did not pass a Tier 2 assessment to be acceptable. In essence, these tiers have been formulated so that the likelihood of an error in assessing a building as acceptable in a lower tier is less than in a higher tier.

Where the BPOE is selected as the performance objective, Table 2-4 shows that when Tier 1 and Tier 2 procedures are used, the Structural Performance Levels need to be checked only at the BSE-2E Seismic Hazard Level for buildings assigned to Risk Categories I through III. Where Tier 3 is used, checks of Structural Performance Levels at the BSE-1E and BSE-2E Seismic Hazard Levels are required. For example, considering a Risk Category II building, checking BPOE with Tier 1 and Tier 2 does not include an evaluation of the Structural Life Safety Performance Level, whereas checking with Tier 3 does. For Tier 1 or Tier 2, Life Safety with the BSE-1E hazard is implied by meeting the criteria for Collapse Prevention Structural Performance Level with the BSE-2E hazard and the requirements in Chapter 3 that permit the use of Tier 1 and Tier 2 deficiency-based procedures. In other words, although Tier 1 and Tier 2 procedures do not explicitly address Life Safety, they are deemed to comply with the standard for the full BPOE based on demonstrated compliance with requirements for the Collapse Prevention portion. This is an allowance that the standard makes by judgment for the BPOE only.

For Tier 1 and Tier 2, buildings assigned to Risk Category IV require a two-level check. This is because of the difference in structural performance between Immediate Occupancy and Life Safety. In this standard, Structural Life Safety is a margin against collapse, whereas Immediate Occupancy implies that there is a limitation of damage to the structural system such that the building would likely be able to be occupied following BSE-2E seismic hazard shaking intensity. While the difference between the BSE-1E and BSE-2E hazard intensity levels in many areas of the country is significant, a building satisfying Life Safety in the BSE-2E may not have sufficient strength or ductility in the structure to provide Immediate Occupancy Structural Performance in the BSE-1E hazard intensity. Since the declaration of meeting the Immediate Occupancy Structural Performance Level is a significant statement about a building, an explicit check for it in the BSE-1E was deemed important enough to warrant a two-level check.

The 2017 edition of this standard introduces consideration of nonstructural hazards at the BSE-2E Seismic Hazard Level for the BPOE and BPON. This is a change from previous editions of the standard, which did not consider nonstructural performance at the BSE-2E hazard level. The reason for this change was initially the concern that, like structural performance, if the BSE-1E hazard was significantly less than the BSE-2E hazard, sufficient safety might not be provided. The committee discussion regarding the issue of significant intensity variation between the BSE-1E and BSE-2E led to the committee's position that some nonstructural hazards can have as great an effect on occupant safety as a local collapse of a structure, which then led to the reintroduction of the Hazards Reduced Nonstructural Performance Level and its consideration in the BSE-2E. Those nonstructural hazards are identified in Chapter 13 and the Chapter 17 Nonstructural Tier 1 Checklist. Since the Tier 1 screening is purely qualitative, there is no increased level of effort to consider nonstructural performance at two Seismic Hazard Levels. For the Tier 2 evaluation or retrofit, no additional level of effort is required, other than determining a different force level for the subset of items considered in the Hazards Reduced Nonstructural Performance Level and the Life Safety Nonstructural Performance Level.

Keeping with the desire for this standard to not require anything that would be above and beyond what would be required for a new building, the table footnote limits the requirements for the Performance Objective of Hazards Reduced Nonstructural Performance Level in the BSE-2E Seismic Hazard Level to be no greater than what would be required per ASCE 7. Therefore, if the force level calculated using the BSE-2E Seismic Hazard Level exceeds the force level calculated using ASCE 7's Design Earthquake (DE), which is equivalent to the BSE-1N level, then the force is capped at the BSE-1N level. Furthermore, if ASCE 7 does not require consideration of a specific component because of the Seismic Design Category of the building, then it need not be considered in the BSE-2E Hazards Reduced Nonstructural Performance Level.

It is important to recognize that the inventory of damaged buildings used to infer the deficiency-based procedure was mostly of moderate size and height. The standard's committee felt that a similar limitation was needed to designate when the deficiency-only procedures could be used. A number of criteria regarding the building's size, structural system, and configuration were developed; these criteria must be met to be able to use the deficiency-based provisions.

The Tier 3 procedure was intended as a systematic procedure for all buildings, regardless of configuration size or structural system. This range includes complex buildings that could not be classified into one of the common building types from which the experience base for Tiers 1 and 2 was derived. For such buildings, where there are not sufficient observations of their performance from past earthquakes, a rigorous, full-building assessment should be conducted to ensure sufficient robustness and margin of safety beyond the design-level earthquake.

C2.4.2 Enhanced Performance Objectives Enhanced performance objectives can be obtained by using higher target Building Performance Levels, higher Seismic Hazard Levels, a higher risk category, or any combination thereof. By definition then, the BPON defined in Section 2.4.4 is also an Enhanced Performance Objective.

C2.4.3 Limited Performance Objectives Life Safety Building Performance at the BSE-1E hazard level is a commonly used performance objective. Although it matches part of the BPOE, it might be considered a reduced objective for buildings that do not meet the limitations when Tier 1 and Tier 2 procedures can be used and a Tier 3 procedure is used because it ignores the other part of the BPOE, Collapse Prevention Building Performance at the BSE-2E level.

C2.4.4 Basic Performance Objective Equivalent to New Building Standards (BPON) The BPON is intended to provide performance equivalent to that which is intended for new buildings designed to ASCE 7. Table 2-5 relates the risk categories to ASCE 41 Performance Objectives using Seismic Hazard Levels defined to match those in ASCE 7. The BPON is classified as a special case of an Enhanced Performance Objective because it seeks the same structural and higher nonstructural performance levels as the BPOE with higher Seismic Hazard Levels. The BPON is provided as guidance to the engineer, owner, or building official wishing to evaluate or retrofit to an equivalent performance objective as a new code-designed building.

The 2017 edition of the standard adds the Hazards Reduced check to the BSE-2N for the BPON, even though ASCE 7 does not have an explicit nonstructural check at the MCE_R level. Although this may seem like an increase in the performance requirements, it is not. The table footnotes limit the requirements for the Performance Objective of Hazards Reduced Nonstructural Performance Level in the BSE-2N Seismic Hazard Level to be no greater than what would be required per ASCE 7. Also, in ASCE 7, the Seismic Hazard Level at the Design Earthquake is a uniform reduction of the MCE_R, so there is protection from major falling hazards at the MCE_R level by that tethering of the DE to the MCE_R, which is why the footnote effectively deems ASCE 7 requirements to comply with the Performance Objective of Hazards Reduced Nonstructural Performance Level in the BSE-2N Seismic Hazard Level. Also, because ASCE 41 does allow for the use of performance-based procedures beyond what is found explicitly in the standard, it was believed that if a performance-based retrofit chooses to evaluate or retrofit nonstructural performance using methods other than Chapter 13 or ASCE 7, an explicit requirement is that major falling hazards should be prevented at the BSE-2N/MCE_R hazard intensity because they can have similar impact as a local building collapse in injuring or killing multiple people.

The relationships in Table C2-1 provide guidance for relating new building performance using seismic performance terminology of this standard. Although the BPON attempts to provide equivalent performance with new building design standards, the gravity-load-resisting and original lateral systems of an existing building, even after retrofit, are generally not as robust as those of a new building. This is the result of prescriptive requirements contained within the new building standards that might not have been present either in the original design standard to which the building was constructed or in the requirements of this standard. Use of this standard does not preclude the use of prescriptive detailing provisions required in current building design standards.

Therefore, compared with a similarly configured new building, there is a higher degree of uncertainty in obtaining the targeted performance objective for the existing building retrofitted according to the provisions of this standard than would be expected for a new building. The uncertainty is generally biased toward the new design standard producing a building that will perform better than the intended performance of the code. However, that degree of improved performance is variable and difficult to quantify. Conversely, the provisions of this standard can provide a more reliable and predictable assessment of the building's performance to design-level earthquake shaking.

The acceptance criteria for structural components given in this standard have not been directly calibrated to the expected performance of new building components designed to new building codes and standards.

C2.4.5 Partial Retrofit A partial retrofit should be designed and constructed assuming future completion of a performance objective intended to improve the performance of the entire structure. Care must be taken so that the partial retrofit does not decrease the performance of the entire building.

The goal of retrofit is to improve the earthquake performance of the building. A reduction in performance of individual components should not necessarily be a measure of the overall building performance. A partial retrofit could increase forces on some noncritical components while improving the overall performance of the building.

C2.4.6 System-Specific Performance Procedures System-specific performance procedures have traditionally been used to achieve a Reduced Performance or Partial Retrofit Objective where performance is less than the BPOE. Each procedure defines its performance objective at the beginning of each section in Chapter 16.

C2.5 LEVEL OF SEISMICITY

The Levels of Seismicity in this standard have been adjusted to match the Seismic Design Categories in ASCE 7 as follows:

SDC A: Very low
SDC B: Low
SDC C: Moderate
SDC D–F: High

Therefore, the parameters S_{DS} and S_{D1} correspond to the S_{XS} and S_{X1} parameters at the BSE-1N Seismic Hazard Level.

CHAPTER C3

EVALUATION AND RETROFIT REQUIREMENTS

C3.2 AS-BUILT INFORMATION

Existing building characteristics pertinent to seismic performance should be obtained from the following sources, as appropriate for original construction and all structural modifications of the building completed:

1. Field observation of exposed conditions and configuration made accessible by the owner;
2. Construction documents, engineering analyses, reports, soil borings and test logs, maintenance histories, and manufacturers' literature and test data, which may be available from the designers owner, or the code official;
3. Reference standards and codes from the period of construction, as cited in the design documents or Chapters 9 through 12;
4. Destructive and nondestructive examination and testing of selected building materials and components as specified in Section 6.2; and
5. Interviews with building owners, tenants, managers, the original architect and engineer, contractor(s), and the local building official.

The information required for an existing building may also be available from a previously conducted seismic evaluation of the building. Where seismic retrofit has been mandated according to building construction classification, familiarity with the building type and typical seismic deficiencies is recommended. Such information is available from several sources, including Chapters 4 and 5 of this standard. Such information may be sufficient for the Tier 1 screening and Tier 2 Deficiency-Based Procedures, but additional as-built information may be needed for the Tier 3 Systematic Procedures.

Where a destructive and nondestructive testing program is necessary to obtain as-built information, it is prudent to perform preliminary calculations on select key locations or parameters before establishing a detailed testing program. These calculations are meant to provide knowledge at a reasonable cost and with as little disruption as practical of construction features and material properties at concealed locations.

If the building is a historic structure, it is also important to identify the locations of historically significant features and fabric, which should be investigated and determined by the client's historic preservation expert. Care should be taken in the design and investigation process to minimize the effect of work on these features. Refer to the *Standards for the Treatment of Historic Properties with Guidelines for Preserving, Rehabilitating, Restoring, & Reconstructing Historic Buildings* (Secretary of the Interior 1995).

C3.2.1 Building Type Refer to Section C3.3 for commentary.

C3.2.2 Building Configuration The as-built information on building configuration should identify the load-resisting components. Load-resisting components may include structural and nonstructural components that participate in resisting seismic loads, whether or not they were intended to do so by the original designers. This information should identify potential seismic deficiencies in load-resisting components, which may include discontinuities in the load path, weak links, irregularities, and inadequate strength and deformation capacities.

C3.2.3 Component Properties Meaningful structural analysis of a building's probable seismic behavior and reliable design of retrofit measures require good understanding of the existing components (such as beams, columns, and diaphragms), their interconnection, and their material properties (mainly the mechanical properties, such as strength, deformability, and toughness). The strength and deformation capacity of existing components should be computed, as specified in Chapters 8 through 13, based on derived material properties and detailed component knowledge. Existing component action strengths must be determined for two basic purposes: to allow calculation of their ability to deliver load to other components, and to allow determination of their capacity to resist forces and deformations.

C3.2.4 Site and Foundation Information Sources of applicable existing site and foundation information include original design information, foundation capacity information included on the drawings, and previous geotechnical reports for the site or for other sites in the immediate vicinity.

Adjacent building development or grading activities that impose loads on or reduce the lateral support of the structure can affect building performance in a future earthquake. Evidence of poor foundation performance includes settlement of building floor slabs and foundations, differential movement visible at adjacent exterior sidewalks, and other miscellaneous site construction.

C3.2.5 Adjacent Buildings

C3.2.5.1 Building Pounding Building pounding can alter the basic response of the building to ground motion and impart additional inertial loads and energy to the building from the adjacent structure. Of particular concern is the potential for extreme local damage to structural elements at the zones of impact, particularly where the floor and roof levels of adjacent buildings do not align in height.

C3.2.5.2 Shared Element Condition Buildings sharing common elements, such as party walls, have several potential problems. If the buildings attempt to move independently, one building may pull the shared element away from the other, resulting in a partial collapse. If the buildings behave as an integral unit, the additional mass and inertial loads of one structure may result in extreme demands on the seismic-force-resisting system of the other. All instances of shared elements should be reported to the building owner, and the owner should be encouraged to inform adjacent building owners of identified hazards.

C3.2.5.3 Hazards from Adjacent Buildings Hazards from adjacent buildings, such as falling debris, rooftop equipment and tanks, cladding panels, aggressive chemical leakage, fire, or explosion that may affect building performance or the operation of the building after an earthquake should be considered and discussed with the building owner. Consideration should be given to hardening those portions of the building that may be impacted by debris or other hazards from adjacent structures. Where Immediate Occupancy Performance is desired and ingress to the building may be impaired by such hazards, consideration should be given to providing suitably resistant access to the building. Sufficient information should be collected on adjacent structures to allow preliminary evaluation of the likelihood and nature of hazards, such as potential falling debris, fire, and blast pressures. Evaluations similar to those in FEMA 154 (FEMA 2015b) and FEMA 155 (FEMA 2015c) may be adequate for this purpose.

C3.3 COMMON BUILDING TYPES

Fundamental to the Tier 1 and Tier 2 procedures is the grouping of buildings into sets that have similar behavioral characteristics. The classification of building type is required to determine whether the Tier 1 or Tier 2 procedures are permitted for evaluation or retrofit, as indicated in Section 3.5.

By their nature, these designations require judgment based on the building's characteristics to make assignments. The universe of existing buildings is very large with many distinctions of materials and structural and construction types. The classification of building type intends to group buildings that share similarities in their seismic response by the observed and/or expected seismic performance of the structures.

These groups of building types were first defined in ATC-14 (1987) and have since been used in the FEMA guideline documents and previous editions of ASCE 31 and 41. Descriptions of the cold-formed steel light-frame construction were not part of the original source documents and were introduced in the 2017 edition of ASCE 41.

The Common Building Types are defined in Table 3-1, with additional description and background information contained in Table C3.1. Because most structures are unique in some fashion, judgment should be used where selecting the building type, with the focus on the seismic-force-resisting system and elements.

Except where explicitly defined in Table 3-1, the building type classification is independent of foundation system, because the specific foundation system is not a significant factor in the types of seismic behavior assumed for the various common building types. Therefore, the foundations for the defined Common Building Types can consist of a variety of systems, including shallow spread or strip footings, mat foundations, and deep foundation systems. Whereas the foundation type does not generally impact the classification of a common building type, evaluation of the foundation system is included in the Tier 1 screening process.

In the specific case of building Type S4: Dual Frame System with Backup Steel Moment Frames and Stiff Diaphragms, the building may be reclassified as S2: Steel Braced Frames with Stiff Diaphragms or C2: Concrete Shear Walls with Stiff Diaphragms if the secondary moment frame is not strong enough or stiff enough to behave as a true dual system.

It is expected that not all buildings will be assessable within these classes. Application of Tier 1 and Tier 2 assessments requires that the building be classifiable in one of the Table 3-1 designations. Tier 3 procedures are required for building types not listed.

C3.4 BENCHMARK BUILDINGS

The methodology in this standard is substantially compatible with what are considered "modern" building code provisions for specific building types; however, the nature of the methodology is such that complete compatibility may not be achievable. From observed earthquake damage, it can be inferred that certain building types designed and constructed to recent building codes can be expected to provide a level of structural seismic performance consistent with the Basic Performance Objective for Existing Buildings (BPOE). Similarly, building types evaluated or retrofitted in accordance with the precursors to this standard can be expected to achieve the BPOE. However, without Benchmark Building provisions, even those recently designed, evaluated, or retrofitted structures would need to be evaluated to demonstrate compliance with the BPOE. Although many buildings pass the Tier 1 screening as compliant, the conservative nature of the standard is such that some adequate buildings would be found noncompliant. The intent of this section is to resolve this incompatibility by recognizing structure types and code editions that have provided a level of seismic performance consistent with the BPOE in past earthquakes. Benchmarking is aligned with the BPOE and by risk category because this generally represents the minimum level of performance for existing buildings that is required when an evaluation or retrofit is triggered by the building code or other adopted provisions, and therefore benchmarking provides a means for demonstrating compliance with these requirements when triggered. Benchmarking is not permitted for higher levels of seismic performance, including the Basic Performance Objective for New Buildings (BPON), because the provisions for new buildings continue to evolve over time, and compliance with older versions of codes and standards may not result in expected seismic performance that is consistent with the BPON.

Although Benchmark Buildings need not proceed with further structural evaluation, it should be noted that they are not simply exempt from the criteria of this standard. The design professional should document that a complete benchmark procedure was followed by completing the checklist in Table 3-5.

Where the provision refers to retrofits that meet an approved standard, its intent is to exclude partial retrofits as described in Section 2.4.5. Many retrofits, whether voluntary, mandatory, or code-triggered, are partial retrofits in the sense that they improve or eliminate specific well-known deficiencies but do not evaluate or retrofit the rest of the structure. For example, the Appendix A chapters of the IEBC, as well as other guidelines such as FEMA P-807 and ICC-1300 take this approach. Although these retrofits are common and cost-effective, benchmarking is intended to represent compliance with a given structural performance objective for the entire structure. Therefore, only retrofits that completely meet the standards set forth in Tables 3-2, 3-3, and 3-4 are eligible for benchmarking. Partial retrofits are not eligible for benchmarking.

Because nonstructural components have been found routinely to have been designed, installed, or modified without enforcement of applicable building code provisions (Masek and Ridge 2009), benchmark provisions do not apply to evaluation of nonstructural components.

The Benchmark Building provisions are optional. A design professional may choose to perform a structural Tier 1 screening

Table C3-1. Common Building Types.

Wood Light Frames, Small Residential W1	This building type aligns with what is known as conventional construction under the IBC or IRC and is based on the code limits for one- and two-family dwellings and townhouses. Building loads are light, and the framing spans are short. Chimneys, where present, consist of solid brick masonry, masonry veneer, or wood frame with internal metal flues. Floor and roof diaphragms usually consist of straight or diagonal lumber sheathing, tongue-and-groove planks, oriented strand board, plywood, or other materials. Shear walls usually consist of straight or diagonal lumber sheathing, plank siding, oriented strand board, plywood, stucco, gypsum board, particleboard, fiberboard, or similarly performing materials. Interior partitions are sheathed from floor to floor with plaster or gypsum board. This type of construction may have open-front garages at the lowest story, and/or split-level floors.
Wood Frames, Large Residential, Commercial, Industrial, and Institutional W2	This building type generally consists of (a) large commercial, industrial, warehouse, retail, or agricultural type buildings, often which have been converted to office buildings or other uses; (b) multiunit residential buildings such as apartments, townhomes, condominiums or (c) large one- and two-family dwellings with heights exceeding three stories, plan areas over 3,000 sf (280 m^2) per floor or over 6,000 sf (1,520 m^2) total. Multiunit residential buildings can be up to five stories in height and typically have interior shear walls that are stacked vertically. Building loads are relatively light, and framing consists of large span trusses or girders with limited interior columns. Gravity framing can consist of wood or steel elements; however, the lateral systems consist of wood elements comprised of flexible diaphragms and exterior walls consisting of straight or lumber sheathing, plank siding, oriented strand board, plywood, stucco, gypsum board, particleboard, fiberboard, or similarly performing materials. The lateral system may also consist of cantilevered or knee-braced wood columns; however, the Tier 1 checklists do not provide Quick Checks for or address these elements. Bracing with other materials such as steel braced frames should be evaluated as a mixed system. In some cases, these buildings may be located over a concrete, steel framed, or wood framed podium level and can be evaluated as a mixed system using W2 and C2, RM2, or S2. Discontinuous shear walls at the lower common areas or parking levels are often present.
Steel Moment Frames S1 (with Stiff Diaphragms)	These buildings consist of a frame assembly of steel beams and steel columns in which seismic forces are resisted by steel moment frames that develop their stiffness through fully restrained or partially restrained beam–column connections. Where all connections are moment-resisting connections, the entire frame participates in seismic force resistance. Where only selected connections are moment-resisting connections, resistance is provided along discrete frame lines. Columns are oriented so that each principal direction of the building has columns resisting forces in strong axis bending. Diaphragms consist of rigid construction that is stiff relative to the frames. The exterior of the structure is permitted to be concealed; the environmental closure walls consist of any type, including both ductile, flexible systems, and rigid, nonductile systems (e.g., unreinforced masonry either interior or exterior to the frame line). Where the interior of the structure is finished, frames are concealed by ceilings, partition walls, and architectural column furring. The foundation system could consist of a variety of elements but generally include grade beams beneath the moment frame locations. Steel moment frame beam–column connections have developed over time as design and construction methods have changed and improved. Older, historic connections consist of riveted or bolted connections using plates, angles, and tees to join the main members and are considered partially restrained. Many of these historic frames are encased in concrete for fire protection, however the concrete cover changes the stiffness of the system. “Pre-Northridge” connections consist of full penetration welded beam and column connections, often with beam flange continuity plates and column panel zone doubler plates and are considered fully restrained connections. The term pre-Northridge connections typically specifies standard design practices that resulted in damage to steel moment frames noted following the 1994 Northridge Earthquake, including potential deficiencies in welding, strong-beam/weak-column conditions, and yielding at undesired joint locations. Following the Northridge Earthquake, significant research and code changes were implemented for steel moment frame design to provide for better and more reliable performance of these systems.
S1a (with Flexible Diaphragms)	These buildings are similar to S1 buildings, except that diaphragms are flexible relative to the frames, generally resulting in tributary area distribution to the lateral frame elements. Gravity systems can consist of a variety of elements but generally are comprised of wood sheathing or steel deck supported by wood framing, openweb steel joists, steel joists and beams, or a hybrid combination of wood and steel.

continues

Table C3-1 ***(Continued).*** **Common Building Types.**

Steel Braced Frames S2 (with Stiff Diaphragms)	The lateral force resisting system for these buildings utilize frames comprised of steel columns, beams, and braces. Braced frames develop resistance to seismic forces by the bracing action of the diagonal members. The braces induce forces in the associated beams and columns such that all elements work together in a manner similar to a truss; all element stresses are primarily axial. Diaphragms transfer seismic loads to braced frames. The diaphragms consist of concrete or steel deck with structural reinforced concrete fill and are stiff relative to the frames. The foundation system could consist of a variety of elements including spread footings and pile foundations, although grade beams are often provided at frames with high overturning or tension loads. Steel braced frame design has developed over time as design, and construction methods have changed and improved. Older braced frames may consist of flat straps, angles, or double angle braces, which act as tension-only members. These slender members may yield or buckle in compression, resulting in reduced capacity during cyclic loading. In addition, connection strength for older braced frames was often based on design loads and may be weaker than the member strength. Before the 1988 Uniform Building Code (UBC), there were no special code provisions for the design of braced frames intended to resist forces induced by earthquake motions. The 1988 UBC addressed member slenderness, built-up members, compression elements, bracing connection forces, and brace configurations (which penalized Chevron and prohibited K-bracing configurations). The 1988 UBC also introduced provisions for eccentric braced frames (EBF). More recent developments also include seismic damping braced systems, and buckling resistant braced frames (BRBF).
S2a (with Flexible Diaphragms)	These buildings are similar to S2 buildings, except that diaphragms consist of wood or cold-formed steel framing with wood sheathing; horizontal rod bracing; untopped bare steel deck; or steel deck fill other than reinforced structural concrete, and they are flexible relative to the frames. These buildings are typically one-story in height and are relatively low in mass.
Metal Building Frames S3	These buildings use transverse steel moment frames, or bents, typically comprised of tapered beams and columns that are generally engineered by the manufacturer and assembled on site. The buildings are very light-weight and relatively flexible. Lateral loads in the longitudinal direction generally consist of steel tension rod bracing or steel shear panels, which are connected to the column portion of the transverse frames. Similar tension rod bracing or shear panels may also be provided at the end walls in the transverse direction in lieu of moment frames. The ground floors typically consist of concrete slabs-on-grade, and the buildings are typically supported by shallow concrete footings.
Dual Frame Systems with Backup Steel Moment Frames and Stiff Diaphragms S4	The gravity load system for these buildings utilizes frames consisting of steel joists, beams, open web joists, and/or trusses and steel columns. The floor and roof diaphragms consist of cast-in-place concrete slabs or steel deck with reinforced structural concrete fill, although the roof levels often do not contain reinforced structural concrete fill. The primary seismic-force-resisting system for the buildings consists of either steel braced frames or constructed-in-place concrete shear walls in combination with backup secondary steel moment frames. The concrete shear walls are typically bearing walls where the steel frame does not provide a complete vertical support system. In modern dual systems, the steel moment frames are designed to work together with the steel braced frames or concrete shear walls in proportion to their relative rigidity. The steel moment frames provide a secondary seismic-force-resisting system based on the stiffness of the frame and the moment capacity of the beam–column connections. Such moment frames are typically designed to be capable of resisting 25% of the building's seismic forces. In older dual systems, the moment frames may not have been designed for a specific percentage of the seismic forces, but instead will resist forces in proportion to relative rigidity. In all cases, the Tier 1 evaluation for this system must consider the behavior of both systems.
Steel Frames with Infill Masonry Shear Walls S5 (with Stiff Diaphragms)	This is an older type of building construction that utilizes frame assemblies of steel beams and steel columns. The floor and roof diaphragms consist of cast-in-place concrete slabs or steel deck with reinforced structural concrete fill and are stiff relative to the walls. Framing consists of steel beams, open web joists, or trusses. Walls consist of infill panels constructed of solid clay brick, concrete block, or hollow clay tile masonry, which are in-plane with and infill the steel beam-column frames; however, window or other openings may be provided within the infill wall panel. Infill walls are permitted to completely encase the frame members and present a smooth masonry, exterior with no indication of the frame. The seismic performance of this type of construction depends on the interaction between the frame and infill panels. The combined behavior is more like a shear wall structure than a frame structure. Solidly infilled masonry panels form diagonal compression struts between the intersections of the frame members. If the walls are offset from the frame and do not fully engage the frame members, diagonal compression struts do not develop. The strength of the infill panel is limited by the shear capacity of the masonry bed joint or the compression capacity of the strut. The post-cracking strength is determined by an analysis of a moment frame that is partially restrained by the cracked infill.
S5a (with Flexible Diaphragms)	These buildings are similar to S5 buildings, except that diaphragms consist of wood straight or diagonal sheathing; horizontal rod bracing; bare steel deck; or steel deck with fill other than reinforced structural concrete, and they are flexible relative to the frames.

continues

Table C3-1 *(Continued)*. Common Building Types.

Steel Plate Shear Walls S6	This building type is generally considered a more recently developed lateral system, although some steel plate shear walls can be found in 1960s and older vintage buildings. These buildings have a frame of steel columns, beams, and shear walls. Shear walls are constructed with steel plates with horizontal and vertical boundary elements adjacent to the webs. The boundary elements are typically designed to remain essentially elastic under maximum forces that can be generated by the fully yielded webs. The diaphragms consist of concrete or steel deck with reinforced structural concrete fill and are stiff relative to the shear walls.
Cold-Formed Steel Light-Frame Construction CFS1 (Shear Wall System)	Lateral loads for these buildings are carried by wood sheathed or steel sheet sheathed shear walls placed over cold-formed steel light-frame studs. Floor and roof framing consists of cold-formed steel joists or rafters on cold-formed steel studs spaced no more than 24 in. (61 cm) apart, wood or cold-formed steel trusses, structural steel or cold-formed steel beams, and structural steel or cold-formed steel columns. The first-floor framing is supported directly on the foundation system or is raised up on cripple studs and post-and-beam supports. Chimneys, where present, consist of solid brick masonry, masonry veneer, or cold-formed steel frame with internal metal flues. Seismic forces are resisted by wood sheathed or bare steel deck diaphragms that transfer loads to either wood structural panel sheathed shear walls, steel sheet sheathed shear walls, or steel sheet backed gypsum board panels. Interior surfaces are typically sheathed with plaster or gypsum board. Buildings of this type that have precast concrete plank diaphragms are excluded from this Common Building Type because the seismic behavior characteristics of the system are less well understood and not compatible with the Tier 1 screening procedure due to the heavy floor mass and relative diaphragm rigidity.
Cold-Formed Steel Light-Frame Construction CFS2 (Strap-Braced Wall System)	Lateral loads for these buildings are carried by cold-formed steel light-frame stud walls with flat strap bracing, similar to let-in bracing for a wood stud wall system, where the bracing stiffens the wall segment. Floor and roof framing consists of cold-formed steel joists or rafters on cold-formed steel studs spaced no more than 24 in. (61 cm) apart, wood or cold-formed steel trusses, structural steel or cold-formed steel beams, and structural steel or cold-formed steel columns. The first-floor framing is supported directly on the foundation system or is raised up on cripple studs and post-and-beam supports. Interior surfaces are sheathed with plaster or gypsum board. Chimneys, where present, consist of solid brick masonry, masonry veneer, or cold-formed steel frame with internal metal flues. Buildings of this type that have precast concrete plank diaphragms are excluded from this Common Building Type because the seismic behavior characteristics of the system are less well understood and not compatible with the Tier 1 screening procedure due to the heavy floor mass and relative diaphragm rigidity.
Concrete Moment Frames C1	These buildings consist of a frame assembly of cast-in-place reinforced concrete beams and columns. Floor and roof framing consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Seismic forces are resisted by concrete moment frames that develop their stiffness through monolithic beam–column connections. In older construction, or in levels of low seismicity, the moment frames are permitted to consist of the column strips of two-way flat slab systems. Modern frames in levels of high seismicity have joint reinforcing, closely spaced ties, and special detailing to provide ductile performance. This detailing is usually not present in older construction.
Concrete Shear Walls C2 (with Stiff Diaphragms)	These buildings have rigid floor and roof diaphragms that consist of cast-in-place concrete slabs integral with concrete beams, one-way joists, two-way waffle joists, or concrete flat slabs. Roof and floor framing may also consist of steel deck with reinforced structural concrete fill diaphragms supported by steel beams, steel columns, or cold-formed steel light-frame construction. Roof and floors are supported on concrete or steel columns or bearing walls. Seismic forces are resisted by cast-in-place concrete shear walls. In older construction, shear walls are lightly reinforced but often extend throughout the building. In more recent construction, shear walls occur in isolated locations, are more heavily reinforced, and often have heavily reinforced boundary elements for overturning.
C2a (with Flexible Diaphragms)	These buildings are similar to C2 buildings, except that diaphragms consist of wood straight, diagonal, or panelized sheathing, bare steel deck, or steel deck with fill other than reinforced structural concrete, or horizontal rod bracing.
Concrete Frames with Infill Masonry Shear Walls C3 (with Stiff Diaphragms)	This is an older type of building construction that utilizes frame assemblies of cast-in-place concrete beams and columns. The floor and roof diaphragms consist of cast-in-place concrete slabs and are stiff relative to the walls. Walls consist of infill panels constructed of solid clay brick, concrete block, or hollow clay tile masonry that are in-plane with and infill the concrete beam/column frames; however, window or other openings may be provided within the infill wall panel. The seismic performance of this type of construction depends on the interaction between the frame and the infill panels. The combined behavior is more like a shear wall structure than a frame structure. Solidly infilled masonry panels form diagonal compression struts between the intersections of the frame members. If the walls are offset from the frame and do not fully engage the frame members, the diagonal compression struts do not develop. The strength of the infill panel is limited by the shear capacity of the masonry bed joint or the compression capacity of the strut. The post-cracking strength is determined by an analysis of a moment frame that is partially restrained by the cracked infill. The shear strength of the concrete columns, after racking of the infill, is permitted to be limited by the semiductile behavior of the system.

continues

Table C3-1 ***(Continued).*** **Common Building Types.**

C3a (with Flexible Diaphragms)	These buildings are similar to C3 buildings, except that diaphragms consist of wood straight or diagonal sheathing; bare steel deck; steel deck with fill other than reinforced structural concrete; or horizontal rod bracing.
Precast or Tilt-Up Concrete Shear Walls PC1 (with Flexible Diaphragms)	These buildings have precast concrete perimeter wall panels, and often interior walls, that are typically cast on site and tilted into place. The panels are interconnected by weldments, cast-in-place concrete pilasters, or collector elements. Floor and roof framing consists of wood sheathing supported by wood subpurlins; wood or open-web wood or steel joists; and wood, glulam, or steel beams. Horizontal rod bracing may also be present in older building or structures in lower seismic hazard regions. Framing is supported on interior steel or wood columns and perimeter concrete bearing walls. Seismic forces are resisted by the flexible wood sheathed or bare steel deck diaphragms, which transfer loads to precast concrete perimeter wall panels through wood or steel ledgers or top plates. Critical load path connections consist of out-of-plane wall anchorage between the walls and roof or floor diaphragms and generally consist of embedded bolts or post-installed mechanical, epoxy, or thru-bolt anchors. Wall panels are permitted to be solid or have large window and door openings that cause the panels to behave more as frames than as shear walls. The roof framing is permitted to have tension-capable connections between elements.
PC1a (with Stiff Diaphragms)	These buildings are similar to PC1 buildings, except that diaphragms consist of precast elements, cast-in-place concrete, or steel deck with reinforced structural concrete fill, and they are stiff relative to the walls.
Precast Concrete Frames PC2 (with Shear Walls)	These buildings consist of a frame assembly of precast concrete beams, girders, and columns with the presence of concrete shear walls. Floor and roof framing consists of cast-in-place slabs, precast concrete planks, tees, or double-tees supported on precast concrete girders and columns, some or all of which are permitted to be pre- or post-tensioned. The precast elements are generally fabricated off site, then assembled using welded connections, cast-in-place closure strips, or reinforced concrete topping slabs over reinforcing provided in the precast elements. Seismic forces are resisted by precast or cast-in-place concrete shear walls, which are permitted to also bear gravity loads.
PC2a (without Shear Walls)	These buildings are similar to PC2 buildings, except that concrete shear walls are not present. Seismic forces are resisted by precast concrete moment frames that develop their stiffness through beam–column joints rigidly connected by welded inserts or cast-in-place concrete closures. Diaphragms consist of precast elements interconnected with welded inserts, cast-in-place closure strips, or reinforced concrete slabs or topping slabs.
Reinforced Masonry Bearing Walls RM1(with Flexible Diaphragms)	These buildings have bearing walls that consist of reinforced brick or concrete block masonry. The floor and roof framing consists of a variety of wood or steel framing with straight or diagonal wood sheathing, plywood, or bare steel deck diaphragms that are flexible relative to the walls. Gravity load members consist of the reinforced masonry walls, wood or cold-formed steel light-frame construction, or by wood, steel, masonry or concrete columns. Seismic forces are resisted by the flexible wood or steel deck diaphragms, which transfer loads to reinforced brick or concrete block masonry shear walls through wood or steel ledgers and top plates. Critical load path connections consist of out-of-plane wall anchorage between the walls and roof or floor diaphragms and collector beam connections to the walls.
Reinforced Masonry Bearing Walls RM2 (with Stiff Diaphragms)	These building are similar to RM1 buildings, except that the diaphragms consist of steel deck with reinforced structural concrete fill, precast concrete planks, tees or double-tees, with or without a cast-in-place concrete topping slab, and are stiff relative to the walls. The floor and roof framing is supported on interior steel or concrete frames or interior reinforced masonry walls.
Unreinforced Masonry Bearing Walls URM (with Flexible Diaphragms)	These buildings have perimeter and sometimes interior bearing walls that consist of unreinforced clay brick, stone, or concrete masonry. In older construction, floor and roof framing consists of straight or diagonal lumber sheathing supported by wood joists, which are supported on timber or steel beams and wood, steel, or cast-iron columns. In more recent construction, floors consist of structural panel or plywood sheathing rather than lumber sheathing. The diaphragms are flexible relative to the walls. Where they exist, original out-of-plane wall anchor ties between the walls and diaphragms consist of bent steel plates embedded in the mortar joints and attached to framing. Seismically retrofitted anchors consist of thru-bolts with exterior rosettes, bolts grouted into the walls, and, more recently, epoxy or adhesive grouted bolts that are attached to the roof and floor framing with bolted plates or manufactured hardware.
URMa (with Stiff Diaphragms)	In older construction or large, multistory buildings, roof and floor systems may consist of cast-in-place concrete supported by concrete or concrete encased steel beams and columns. Arched or flat brick or tile floors with or without concrete topping slabs and laminated wood decks are also common to some regions. In levels of low seismicity, more recent construction consists of steel deck with reinforced structural concrete fill supported on steel framing. The unreinforced masonry walls resisting seismic forces are also gravity load bearing walls.

or more detailed evaluation, even if the building meets the requirements of Section 3.4.

In some areas, the design seismicity may have changed since the building was originally designed. Slight changes in the seismicity are not expected to result in a change in the Level of Seismicity for the purposes of Tier 1 screening. Because the building code adoption dates for most of the benchmark codes are no older than 1997. There are some exceptions, such as light wood-frame buildings. These buildings generally present a low risk of collapse, so changes in the Level of Seismicity should not affect the ability to benchmark the building to the BPOE. Conversely, if the design of the building is known to have neglected or inadequately addressed the applicable seismic design provisions, the building should not be benchmarked.

Tables 3-2, 3-3, and 3-4 identify the first year of publication of provisions whose seismic criteria are acceptable for certain building types, so that further structural evaluation is not required. The intention of Tables 3-2, 3-3, and 3-4 is that buildings designed to the specific code edition (or more recent editions) would be benchmarked, not buildings that were designed to an earlier edition but can be shown to meet the provisions of the benchmark code.

If the design or retrofit code or standard for the subject building is not listed in the relevant table or if the edition of design or retrofit code or standard is prior to the benchmark edition year listed in the table, then the building cannot be benchmarked for that particular performance objective or risk category. However, if the building was designed or retrofitted in comformance to a higher performance objective than the benehmark code, the benchmarking can be allowed. For example, a building designed in accordance with a code listed in Table 3-4 for Risk Category IV can be benchmarked for Risk Category III even if that specific code is not listed in Table 3-3.

In this edition of the standard, benchmarking has been revised from a specific seismic performance level to achieving the BPOE. This allows for a more direct connection between benchmarking and the seismic performance objective generally required for existing buildings where an evaluation or retrofit is triggered and more directly ties the benchmarking with risk category consistent with the BPOE. To facilitate this direct linkage, a new table has been added for Risk Category III and the two previous tables have been renamed to apply specifically to Risk Categories I, II, and IV, respectively.

In previous versions of the standard, Risk Category III buildings were not eligible for benchmarking based on the performance objectives listed in the current text, since these buildings require Damage Control structural performance and none of the benchmark codes or standards listed in Table 3-2 are deemed to comply with this performance level. To allow benchmarking for Risk Category III, a new table is provided that includes only the codes and standards that have provisions for Risk Category III that are consistent with current building codes and standards. Building codes prior to the IBC are not considered benchmark codes since the Importance Factor for Risk Category III was typically 1.0. FEMA 310/ASCE 31 are not considered benchmark codes since they only considered Life Safety and Immediate Occupancy performance.

There were several updates to the benchmark codes and standards for this edition of the standard. Specifically, FEMA 178 (1992a) has been removed from Table 3-2 because it was judged to be too old to be considered a "modern standard" for the limited number of buildings for which it was previously permitted. Also, with the exception of W1 buildings, the benchmark year for the UBC has been updated to 1997. This relates to updates to the seismic hazard, the inclusion of the post-Northridge Earthquake emergency provisions for steel moment frames, and the inclusion of consideration of gravity framing and punching shear for concrete buildings.

For unreinforced masonry buildings, the codes and guidelines traditionally used for evaluation and retrofit are not necessarily considered consistent with the intent of Collapse Prevention structural performance in the BSE-2E level earthquake as defined for the BPOE for Risk Categories I and II buildings in this standard. Therefore, these codes and guidelines—*Guidelines for the Seismic Retrofit of Existing Buildings* (ICBO 2001), *Uniform Code for Building Conservation* (ICBO 1997), FEMA 178 (1992a), and Appendix A1 of the *International Existing Building Code* (ICC 2003)—all of which were based on the ABK (1981) methodology were removed from the benchmark provisions in the 2017 edition of the standard. This is consistent with the Special Procedure in Section 16.2 of this standard, which is similar to those other provisions and is defined as achieving Collapse Prevention performance for the BSE-1E Seismic Hazard Level. The intent of removing these traditional provisions is not to suggest that the previous retrofits are deficient, but that a Tier 1 evaluation is required to demonstrate compliance with the Performance Objectives of this standard.

C3.4.1 Benchmark Procedure Checklist Section 3.4.1 and Table 3-5 are new in this edition of the standard, but their substance is not. The content in Table 3-5 is essentially identical to provisions in previous editions of this standard. The design professional is to document the benchmarking procedure by completing the checklist. Each of those items is discussed in the subsequent commentary. As with many other provisions in the standard, these procedural requirements rely on and allow for ample engineering judgment. The final two items in Table 3-5 regarding Level of Seismicity are addressed in Sections C3.4.2.1 and C3.4.2.2

Existing Documents

The evaluating design professional should determine that the building is in general compliance with the benchmark provisions of Tables 3-2, 3-3, or 3-4. The reason for this requirement is that sometimes the building is not properly detailed to meet the provisions of the benchmark code or standard. The intent is to require the evaluating engineer to consider the actual design of the structure, not just the code that was said to have applied. Even with this requirement, the expectation is that most buildings that qualify for benchmarking will not require any detailed review of original calculations or old code provisions.

Knowledge that a code was in effect at the time of construction is not necessarily enough information. A statement on the drawings that the building was designed to the provisions of the benchmark code or standard is not necessarily enough information. Rather, the cited drawings should provide evidence that relevant provisions regarding the detailing of primary elements were applied. At a minimum, there must be evidence of an intended lateral load path on the drawings. Although a general reference to the applicable code is not necessarily enough information, it may be sufficient if the cited code or standard is consistent with the specific notes or references regarding the lateral system type (such as specification of the response modification factor R_W or R value used for the design), soil profile type, and other detailing provisions shown on the drawings. Similarly, the degree of detailing can indicate a conscientious design. The use of generic typical details for varied and complex conditions or notes calling for detailing by others can indicate an incomplete design that does not qualify for benchmarking. Some judgment by the evaluating design professional is often needed.

For example, for concrete tilt-up wall buildings, the most critical elements are the out-of-plane connections between the diaphragm and the tilt-up panels. Provisions dealing with the specific detailing of these elements were not prevalent until the 1997 UBC. Therefore, if an engineer examines the construction documents and notices that the out-of-plane connections can induce cross-grain ledger bending, then they can make a decision that the building does not meet the detailing provision of the benchmark code.

Field Verification

The evaluating design professional should confirm the record drawings with a site visit. The reason for this requirement is that sometimes the existing building has been built to plans different from available design drawings or has been altered since original construction. Also, poor construction quality sometimes compromises the original design. As in Section 3.4.1, the intent is to require the evaluating engineer to consider the actual construction, not just the plans. Even with this requirement, the expectation is that most buildings that qualify for benchmarking do not require any comprehensive or destructive investigation.

The field verification intended by this provision should not require confirmation of every important detail. Rather, the purpose is to rule out with confidence those errors and activities that might cause the structure to perform significantly worse than the confirmed existing documents would suggest. Some judgment by the evaluating engineer is often needed.

Consider as another example using a concrete tilt-up wall building, the engineer has discovered that two new wall openings have significantly reduced the lateral-force-resisting capacity of the system, and no documentation of this structural modification can be found. Without further evaluation, the engineer cannot conclude that the building meets the benchmark code provisions.

Condition Assessment

Significant deterioration can compromise structural performance. Although the requirement is not as detailed as the condition assessment requirements of Section 4.2.1 the engineer should still determine whether any deterioration discovered will affect the behavior of the lateral-force-resisting system suggested by the confirmed drawings and construction. The Tier 1 condition assessment provisions of Section 4.2.1 may be used as a guide to the scope and nature of the effort needed to satisfy this requirement.

If deterioration is discovered in the building, then it is possible to repair the damage such that the repaired building demonstrates compliance with this item, so that the benchmarking provisions can be utilized.

In the example building described earlier, the engineer has verified that the out-of-plane anchors are detailed correctly. However, if while visiting the building, the engineer notices that a chronic roof drainage problem has corroded half the out-of-plane anchors on one side of the building. Because the force transfer mechanism is now partially compromised, the engineer now concludes that without further evaluation, the building no longer meets the benchmark code provisions.

Geologic Site Hazards

Even if an existing building was properly designed and constructed to the benchmark code, site conditions not explicitly addressed by the benchmark code can compromise performance. Large foundation movements caused by any number of site hazards can severely damage an otherwise seismic-resistant building. Potential causes of significant foundation movement include settlement or lateral spreading caused by liquefaction, slope failure, or surface fault ruptures.

If such a geologic site hazard exists, the design of the lateral-force-resisting system of the building should consider this hazard, such as the use of a deep foundation system for an area of liquefaction potential.

C3.4.2.1 Level of Seismicity For a building site, if the original Level of Seismicity as defined by Table 3-6 is less than the Level of Seismicity as defined by this standard, the building design and detailing requirements would most likely be less stringent than if the building were to be designed today and would thus not meet the intent of the benchmark requirements of this standard. Therefore, this benchmarking statement applies to buildings where the Level of Seismicity is Low, Moderate, or High as defined by this standard and where the equivalent Level of Seismicity in the original design code or standard was lower.

The Level of Seismicity generally determines the seismic design provisions that the building characteristics are required to meet. For Very Low and Low Levels of Seismicity, buildings will generally just have a load path to meet wind design requirements, which would be the governing lateral force case. For Moderate Levels of Seismicity, buildings generally meet configuration and strength requirements. For High Levels of Seismicity, buildings generally meet configuration, strength, and ductility requirements.

Although the term *Level of Seismicity* has generally only been defined in ASCE 41, ASCE 31, and their predecessor documents, it can be derived from previous codes and standards using other parameters, such as seismic design category and seismic zone. Table 3-6 provides correlation between previous codes and standards, and Levels of Seismicity as defined by this standard.

It is recognized that the "equivalent" Level of Seismicity determined in Table 3-6 is an imperfect comparison between various legacy codes and standards. This comparison may not account for changes in seismic design basis, changes in seismic performance goals, how soil effects are accounted for, and other possible changes between legacy codes and the current building code. Nonetheless, regardless of how these items were treated by the legacy codes, if the current seismic design provisions are significantly different from the design provisions of the legacy codes, that is reason enough to negate the benchmark code for a particular building.

C3.4.2.2 Seismic Force Provisions Characterization of seismic hazard, particularly in parts of the United States with a history of damaging earthquakes, has become increasingly nuanced since the advent of modern seismic requirements for buildings. Near-field effects, probabilistic models, site-specific parameters, and enhanced performance expectations have generally resulted in seismic-force demand increases with each new cycle of applicable standards. Structures designed and erected under previous assumptions of seismic demand and expected performance may possess inherent toughness and ductility sufficient to offset these increases; however, where the difference between the lateral force and displacement requirements used for the design of a given building differ significantly from those that would be required of the same structure under current code, re-evaluation of that building for seismic hazard is appropriate.

The seismic response parameter used for the comparison corresponds to the design short-period spectral response acceleration parameter associated with peak value of the response spectrum curve. Table 3-7 provides correlation between previous codes and standards, and the seismic response parameters defined by this standard.

For a building site, if the seismic response parameter as defined by this standard exceeds the original seismic response parameter as defined by Table 3-7 by a factor of 1.5 or more, there is a reasonable risk that even with modern ductile detailing, a building would experience more damage and pose a greater risk of unacceptable seismic performance than intended by the benchmark provisions of this standard. Therefore, consistent with the benchmark provisions of this standard, at a minimum, a Tier 1 screening should be performed to demonstrate compliance with the selected performance objective.

The original seismic parameter in Table 3-7 is based on a Life-Safety performance level for a design of a new building. This means that for ASCE 41-13 and ASCE 41-17, the parameter associated with this is that from BSE-1N. Note that this parameter is for comparison purposes only and not intended to imply that the building meets the criteria associated with BPON.

It is recognized that the comparable seismic response parameter determined in Table 3-7 is an imperfect comparison between various legacy codes and standards. This comparison may not account for changes in seismic design basis, changes in seismic performance goals, how soil effects are accounted for, and other possible changes between legacy codes and the current building code. Nonetheless, regardless of how these items were treated by the legacy codes, if the current seismic design force levels are significantly different from the seismic force levels of the legacy codes, that is reason enough to negate the benchmark code for a particular building.

C3.5 EVALUATION AND RETROFIT PROCEDURES

C3.5.1 Limitations on the Use of Tier 1 and Tier 2 Evaluation and Retrofit Procedures The intent of the Tier 1 screening and Tier 2 Deficiency-Based Procedures is to evaluate and, where warranted, reduce seismic risk efficiently, using simplified procedures targeted to specific building types. The Tier 1 and Tier 2 procedures are less complicated and less thorough than the Tier 3 Systematic Procedures, so they are only appropriate for certain straightforward cases. By default, the Tier 3 procedure is to be used where Tier 1 and Tier 2 procedures are not permitted. Unlike the Tier 1 and Tier 2 procedures, the Tier 3 systematic procedure may be used to demonstrate compliance with any performance objective and any building.

The first part of Section 3.5.1 limits the Tier 1 and Tier 2 procedures to certain Performance Objectives. This limitation is consistent with the predecessor provisions in ASCE/SEI 31-03, which presumed hazard levels, like BSE-1E and BSE-2E, lower than those used for the design of new buildings. The two conditions ensure that Tier 1 and Tier 2 are available for the BPOE objectives. The conditions also effectively prohibit the use of Tier 1 and Tier 2 procedures for demonstrating equivalence to new buildings. That level of performance, represented by the BPON objectives, requires consideration of the BSE-2N hazard, which is outside the scope of either of the two conditions, except for the rare cases in which the BSE-2N and the BSE-2E are the same. Because the second condition allows Tier 1 and Tier 2 for an objective of Life Safety (S3-NC) with any hazard up to the BSE-2E, it admits an objective of Life Safety (S3-NC) with the BSE-1N hazard for cases where the BSE-1N parameters are less than or equal to those of the BSE-2E.

The final part of Section 3.5.1 sets limits on the building types for which the Tier 1 and Tier 2 procedures are appropriate. The purpose of Table 3-8 is to identify buildings where the Tier 1 and Tier 2 procedures might not reach a correct conclusion and a more rigorous procedure is required. If the number of stories exceeds the limits in Table 3-8, the more detailed Tier 3 systematic procedures are required to adequately evaluate or retrofit the building.

In many cases, deficiency-based retrofit represents a cost-effective improvement in seismic performance, and it often requires less detailed evaluation or partial analysis to qualify for a specific performance level. Partial Retrofit Objective measures, which target high-risk building deficiencies such as parapets and other exterior falling hazards, are included as deficiency-based techniques. Partial Retrofit Objective measures need not be limited to buildings that conform to the limitations of Table 3-8. Acceptance of the specific partial retrofit method for regulatory purposes depends on the Authority Having Jurisdiction.

Regardless of whether it is permitted for use, the Tier 1 screening in Chapter 4 is a good starting point for the identification of potential deficiencies for any building type covered here and being evaluated using this standard.

C3.5.1.2 Buildings Composed of More than One of the Common Building Types Although the Tier 1 and Tier 2 procedures are based on experience with buildings conforming to one of the Common Building Types in Table 3-1, there are conditions where the Tier 1 and Tier 2 procedures are valid indicators of performance in a building with more than one type of seismic-force-resisting system. Examples of such combinations are noted in the following sections.

C3.5.1.2.1 Combinations of Systems in Different Directions Where a building consists of different systems in each of the two principal directions, the systems can be evaluated and retrofitted somewhat independently using the Tier 1 and Tier 2 procedures. An example is a concrete building with shear walls (C2) in one direction and moment frames (C1) in the orthogonal direction.

C3.5.1.2.2 Combinations of Systems in the Same Direction Under certain conditions, the Tier 1 and Tier 2 procedures are considered valid indicators of performance for mixed systems. Sections 3.5.1.2.2.1 through 3.5.1.2.2.3 provide three specific cases where the checklists and deficiency-based procedures can be used because the mixed systems can be evaluated individually with sufficient certainty and reliability.

In addition, where no irregularities exist, multiple checklists can be used for evaluating combinations of systems without the additional restrictions in Sections 3.5.1.2.2.1 through 3.5.1.2.2.3. In this condition, design professionals must use appropriate judgment in completing some of the Quick Check procedures in Section 4.4 because of the potential complexity of determining average stress levels across different seismic-force-resisting systems. If any statements in the Basic Configuration Checklist are found to be "Noncompliant" or "Unknown," then because of the presence of an irregularity, the combination of systems is judged to be too different from the assumptions inherent in the Common Building Types that serve as the basis for the Tier 1 and Tier 2 procedures. Tier 3 is required for that condition unless the building, even with irregularities, meets the requirements of Sections 3.5.1.2.2.1 through 3.5.1.2.2.3.

C3.5.1.2.2.1 Horizontal Combinations An example of a building meeting the requirements of this section is a Precast or Tilt-Up Concrete Shear Wall building (PC1) with a wood structural panel diaphragm and a line of steel braced frames (S2) in the interior.

C3.5.1.2.2.2 Vertical Combinations An example of a building meeting the requirements of this section is a multistory, multi-unit, residential, wood light-frame structure (W1A) over a 1-story concrete shear wall structure (C2) at the base.

C3.5.1.2.2.3 Combinations of Stiff and Flexible Diaphragms An example of a building meeting the requirements of this section is a reinforced masonry bearing wall building with concrete diaphragms at the floor levels (RM2) and a bare steel deck diaphragm at the roof (RM1).

C3.5.2 Tier 1 Screening Procedure The Tier 1 screening procedure uses sets of checklists that allow a rapid evaluation of the structural, nonstructural, foundation, and geologic hazard elements of the building and site. The purpose of a Tier 1 procedure is to screen out buildings that are reliably expected to comply with this standard or to quickly identify potential deficiencies. Tier 1 analysis, using Quick Checks, involves a minimal level of effort. Benchmark Building criteria in Section 3.4 may also be used to further reduce the level of effort. If the Tier 1 procedures identify potential deficiencies, the design professional may stop the evaluation or may conduct a more detailed evaluation using the Tier 2 deficiency-based procedure.

The Tier 1 procedure includes acceptance criteria for the Immediate Occupancy and Collapse Prevention Structural Performance Levels (S1 and S5) and for the Position Retention and Life Safety Nonstructural Performance Levels (NB and NC). The Tier 1 procedure may be used to evaluate other structural performance levels with modifications to the M_s factors as specified in Table 2-4.

C3.5.3 Tier 2 Deficiency-Based Evaluation and Retrofit Procedures The Tier 2 deficiency-based procedure reflects a level of analysis and design that is appropriate for buildings in which the structural system is uncomplicated, the deficiencies are relatively well understood, and the mitigation techniques are generally straightforward. The procedure is limited to specific sets of defined Performance Objectives in accordance with Section 3.5.1.

The Tier 2 procedure may yield a more conservative result than the Tier 3 procedure because of a variety of simplifying assumptions.

The Tier 2 procedure includes acceptance criteria for the Immediate Occupancy and Collapse Prevention Structural Performance Levels (S1 and S5) and for the Position Retention and Life Safety Nonstructural Performance Levels (NB and NC). The Tier 2 procedure may be used to evaluate other structural performance levels by using the acceptance criteria specified in Chapters 8 through 12.

C3.5.3.1 Evaluation Requirements For the Tier 2 procedure, an analysis of the building that addresses all the potential deficiencies identified in Tier 1 screening shall be performed. Analysis in Tier 2 is limited to simplified linear analysis methods. As in Tier 1, evaluation in Tier 2 is intended to identify buildings not requiring retrofit. If the potential deficiencies identified in the Tier 1 screening are confirmed during the Tier 2 evaluation, the design professional may choose to either conclude the evaluation and report the deficiencies or proceed to Tier 3 and conduct a more comprehensive, systematic seismic evaluation.

For checklist statements identified as “Unknown” in the Tier 1 checklists, the design professional may, upon determining the information necessary for assessing the specific element, either use the Tier 1 or Tier 2 procedures for determining compliance.

C3.5.3.2 Retrofit Requirements For relatively simple buildings with specific deficiencies, it is possible and advisable to prioritize the retrofit measures. This prioritization is often done where the construction has limited funding or must take place while the building is occupied. In both cases, it is preferable to correct the worst deficiency first. Refer to Section C5.8 for additional commentary on the prioritization of seismic deficiencies.

If only a Partial Retrofit Objective is intended, deficiencies should be corrected in priority order and in a way that will facilitate fulfillment of the requirements of a higher objective at a later date. Care must be taken to ensure that a Partial Retrofit Objective effort does not make the building’s overall performance worse by unintentionally channeling failure to a more critical component.

C3.5.4 Tier 3 Systematic Evaluation and Retrofit Procedures

C3.5.4.1 Evaluation Requirements Recent research has shown that certain types of complex structures can be shown to be adequate using nonlinear analysis procedures, even though other common procedures do not. Although these procedures are complex and expensive to carry out, they often result in construction savings equal to many times their cost.

Tier 3 systematic evaluation may be used at any time or may be used to further study potential deficiencies identified in Tier 1 or Tier 2 evaluations.

C3.5.4.2 Retrofit Requirements Tier 3 systematic retrofit may be applied to any building and involves thorough checking of each existing structural component, the design of new ones, and verification of acceptable overall performance represented by expected displacements and internal forces. The Tier 3 procedure focuses on the nonlinear behavior of structural response and uses advanced analysis techniques.

CHAPTER C4

TIER 1 SCREENING

C4.1 SCOPE

The purpose of the Tier 1 screening phase of the evaluation process is to quickly identify buildings that comply with the provisions of this standard. A Tier 1 screening also familiarizes the design professional with the building, its potential deficiencies, and its potential behavior.

Regardless of the requirements of this standard, a Tier 1 screening is recommended for all buildings so that potential deficiencies may be quickly identified. Further evaluation using a Tier 2 or Tier 3 evaluation then focuses, at a minimum, on the potential deficiencies identified in Tier 1. Alternatively, the design professional may choose to end the investigation and report the deficiencies in accordance with Chapter 1 or, after consultation with the owner, may choose to proceed to a retrofit design without performing a Tier 2 or Tier 3 evaluation.

C4.1.1 Performance Level As described in Section C2.2.1 the structural portion of the BPOE was revised from the definition in ASCE 41-13, and this revision has required related changes to the performance level for the Tier 1 screening. For Risk Categories I and II buildings, this results in a change to Collapse Prevention instead of Life Safety in previous editions of the standard. For Risk Categories III and IV buildings, the performance level is still essentially scaled from Risk Categories I and II buildings as before, but in this edition of the standard, the scaling is from Collapse Prevention. Refer to Section C2.2.1 for additional information.

C4.2 SCOPE OF INVESTIGATION REQUIRED

C4.2.1 On-Site Investigation and Condition Assessment Deteriorated structural materials may jeopardize the capacity of the vertical- and lateral-force-resisting systems. The most common type of deterioration is caused by the intrusion of water. Stains may be a clue to water-caused deterioration where the structure is visible on the exterior, but the deterioration may be hidden where the structure is concealed by finishes. In the latter case, the design professional may have to find a way into attics, plenums, and crawl spaces to assess the structural systems and their condition.

The design professional should be careful when dealing with a building that appears to be in good condition and is known to have been subjected to earthquakes in the past. One is tempted to say that the building has ‘withstood the test of time’; however, the earthquakes the building was subjected to may not have been significant, or the good appearance may only be a good cosmetic repair that hides damage that was not repaired. Examples of problems include cracked concrete walls and frames, torn steel connections, bent fasteners or torn plywood in diaphragms and walls, and loose anchors in masonry. Evaluations should include consideration of long-term effects, especially if deterioration is currently minor and repair to the source of deterioration is not completed in a timely manner. Table C4-1 provides additional descriptions of evaluation of defects and deterioration.

C4.2.3 Default Material Values This standard does not permit the use of default material properties for Tier 2 and Tier 3 evaluations without the application of the knowledge factor, κ. Although the default material properties herein are reproduced from Chapters 9 through 12, application of κ is not required because, as explained in more detail subsequently, these properties are conservative versions of those presented in Chapters 9 through 12. The default values for concrete compressive strength, f'_c in Table 4-2 are taken from ACI 369 as referenced in Chapter 10 with some simplications made as appropriate for the approximate nature of the Tier 1 screening procedure.

No default values for E for concrete are provided because there are not any Quick Checks that require a value for E. If for some reason a value of E for concrete is needed in performing a Tier 1 screening, it is recommended that it be derived using the equation applicable to normal-weight concrete in ACI 318 (2019):

$$E_c = 57{,}000\sqrt{f'_c}$$

The default values for reinforcing steel, f_y, in Table 4-3 are taken from ACI 369 as referenced in Chapter 10, with some simplifications made as appropriate for the approximate nature of the Tier 1 screening procedure.

The default values for structural steel yield strength, F_y, in Tables 4-4 and 4-5 are taken from AISC 342, Section A5. Refer to that standard for additional commentary on the background on the default steel properties.

The default value for F_{pe} is based on a ½ in.-diameter strand of ASTM A416 (2002b) material (i.e., breaking strength = 270 kip/in.^{2}, and effective prestress = 0.6 × breaking strength).

C4.3 SELECTION AND USE OF CHECKLISTS

The evaluation statements provided in the checklists form the core of the Tier 1 screening methodology. These evaluation statements are based on observed earthquake structural damage during actual earthquakes. The checklists do not necessarily identify the response of the structure to ground motion; rather, the design professional obtains a general sense of the structure’s deficiencies and potential behavior during an earthquake.

C4.4 TIER 1 ANALYSIS

C4.4.2.1 Pseudo Seismic Force The seismic forces for the Tier 1 screening procedure are based on the linear static analysis

Table C4-1. Patterns of Defects and Deterioration.

Foundation	The integrity and strength of foundation elements may be reduced by cracking, yielding, tipping, or buckling of the foundation. Such weakening may be critical in the event of an earthquake. Lower-level walls, partitions, grade beams, visible footings, pile caps, and similar elements should be visually examined for cracking, yielding, buckling, and out-of-level conditions. Any such signs should be identified and further evaluated.
Foundation Elements	Deterioration can cause weakening of the foundation elements, limiting their ability to support the building. Historical records of foundation performance in the local area may help assess the possibility of deterioration in the foundation of the building being evaluated.
Wood	The condition of the wood in a structure has a direct relationship to its performance in a seismic event. Wood that is split, rotten, or has insect damage may have a very low capacity to resist forces imposed by earthquakes. Structures with wood elements depend to a large extent on the connections between members. If the wood at a bolted connection is split, the connection possesses only a fraction of the capacity of a similar connection in undamaged wood. Limited intrusive investigation may be required to determine the cause and relative magnitude of the damage.
Wood structural panel shear wall fasteners	Fasteners connecting structural panels to the framing are supposed to be driven flush with but should not penetrate the surface of the sheathing. Overdriven fasteners effectively reduce the shear capacity of the fastener and increase the potential for the fastener to fail by pulling through the sheathing. For structures built before the wide use of nailing guns (pre-1970), the problem is generally not present. More recent projects are often constructed with alternate fasteners, such as staples, T-nails, clipped head nails, or cooler nails, which, where installed with pneumatic nail guns, are often overdriven, completely penetrating one or more panel plies. For cold-formed steel light-frame construction, fasteners are commonly screws. Screw heads should be driven flush to the surface or slightly recessed, but not penetrating through any panel plies. Other issues regarding fasteners that could reduce the capacity of shear wall include omitted blocking, excessive fastening spacing, and inadequate edge distance.
Steel ≥ 1/8 in. thick	Environmental effects over prolonged periods of time may lead to deterioration of steel elements. Significant rusting or corrosion can substantially reduce the member cross sections, with a corresponding reduction in capacity.
Steel <1/8 in. thick	Often steel elements have surface corrosion that looks worse than it is and is likely not a concern. Where corrosion is present, care should be taken to determine the actual loss in cross section. Such deterioration must be considered in the evaluation where it occurs at critical locations in the lateral-force-resisting system.
Concrete	Deteriorated concrete and reinforcing steel can significantly reduce the strength of concrete elements. This statement is concerned with deterioration such as spalled concrete associated with rebar corrosion and water intrusion. Cracks in concrete are covered elsewhere in this standard. Spalled concrete over reinforcing bars reduces the available surface for bond between the concrete and steel. Bar corrosion may significantly reduce the cross section of the bar. Deterioration is a concern where the concrete cover has begun to spall and there is evidence of rusting at critical locations.
Concrete walls	Cracks in concrete elements have little effect on the strength of well-reinforced wall elements. A significant reduction in strength is usually the result of large displacements or crushing of concrete. Only where the cracks are large enough to prevent aggregate interlock or to allow for the potential for buckling of the reinforcing steel does the adequacy of the concrete capacity become a concern. Cracks in unusual patterns, such as concentrated on one floor or at one end of the wall, usually indicate a specific cause. The cause of observed cracking needs to be identified to determine whether future cracking will affect the capacity of the wall. Crack width is commonly used as a convenient indicator of damage to a wall. However, it should be noted that some studies, such as FEMA 306 and 307 (1998a, b), list other factors, such as location, orientation, number, distribution, and pattern of the cracks, to be equally important in measuring the extent of damage present in the shear walls. All these factors should be considered when evaluating the reduced capacity of a cracked element.

continues

Table C4-1 *(Continued)*. Patterns of Defects and Deterioration.

Concrete columns encasing masonry infill	Small cracks in concrete elements have little effect on strength. A significant reduction in strength is usually the result of large displacements or crushing of concrete. Only where the cracks are large enough to prevent aggregate interlock or to allow for the potential for buckling of the reinforcing steel does the adequacy of the concrete element capacity become a concern. Columns are required to resist diagonal compression strut forces that develop in infill wall panels. Vertical components induce axial forces in the columns. The eccentricity between horizontal components and the beams is resisted by the columns. Extensive cracking in the columns may indicate locations of possible weakness. Such columns may not be able to function in conjunction with the infill panel as expected.
Unreinforced masonry units	Deteriorated or poor-quality unreinforced masonry elements can result in significant reductions in the strength of structural elements. Damaged or deteriorated masonry may not be readily observable.
Unreinforced masonry joints	Older buildings constructed with lime mortar may have surface repointing but still have deteriorated mortar in the main part of the joint. One test is to tap a small hole with a nail in the repointing and, if it breaks through, powdery lime mortar shows on the nail. If it does not break through after moderate-to-hard blows, the wall probably is repointed full depth. Deteriorated mortar can also be seen by looking behind exterior trim or wall fixtures where the new repointing never reached. Mortar that is severely eroded or can be easily scraped away has been found to have low shear strength, which results in low wall strength. Destructive or in-plane shear tests, such as those referenced in Chapter 11, are required to measure strength of the bond between the brick and mortar to determine the shear capacity of the walls.
Unreinforced masonry walls	Diagonal wall cracks, especially along the masonry joints, may affect the interaction of the masonry units, leading to a reduction of strength and stiffness. The cracks may indicate distress in the wall from past seismic events, foundation settlement, or other causes. Crack width is commonly used as a convenient indicator of damage to a wall, but it should be noted that studies, such as FEMA 306 and 307 (1998a, b), list other factors, such as location, orientation, number, distribution, and pattern of the cracks, to be equally important in measuring the extent of damage present in the shear walls. All these factors should be considered where evaluating the reduced capacity of a cracked element.
Infill masonry walls	Diagonal wall cracks, especially along the masonry joints, may affect the interaction of the masonry units, leading to a reduction of strength and stiffness. The cracks may indicate distress in the wall from past seismic events, foundation settlement, or other causes. Offsets in the bed joint along the masonry joints may affect the interaction of the masonry units in resisting out-of-plane forces. The offsets may indicate distress in the wall from past seismic events or just poor construction. Crack width is commonly used as a convenient indicator of damage to a wall, but it should be noted that some studies (FEMA 306 1998a, FEMA 307 1998b) list other factors, such as location, orientation, number, distribution, and pattern of the cracks, to be equally important in measuring the extent of damage present in the shear walls. All these factors should be considered when evaluating the reduced capacity of a cracked element.
Post-tensioning anchors	Corrosion in posttensioning anchors can lead to failure of the gravity load system if ground motion causes a release or slip of prestressing strands. Coil anchors (Figure C4-1), with or without corrosion, have performed poorly under cyclic forces and are no longer allowed by current standards. The deficiency is the ability of the coil anchor to maintain its grip under cyclic loading. There is no Tier 2 procedure for coil anchors.
Precast concrete walls	Precast concrete elements are sometimes only nominally interconnected and may be subject to shrinkage, creep, or temperature stresses that were not adequately considered in design. Distress caused by these factors could directly affect the lateral strength of the building. The most common damage is cracking and spalling at embedded connections between panels. This damage includes both the nominal connections along the vertical edges and the chord connections at the level of the diaphragm. The performance of precast concrete wall systems is completely dependent on the condition of the connections.
Reinforced masonry walls	Diagonal wall cracks, especially along the masonry joints, may affect the interaction of the masonry units, leading to a reduction of strength and stiffness. The cracks may indicate distress in the wall from past seismic events, foundation settlement, or other causes. Cracks in unusual patterns, such as concentrated on one floor or at one end of the wall, usually indicate a specific cause. The cause of observed cracking needs to be identified to determine whether future cracking will affect the capacity of the wall. Crack width is commonly used as a convenient indicator of damage to a wall. However, it should be noted that some studies (FEMA 306 1998a, FEMA 307 1998b) list other factors, such as location, orientation, number, distribution, and pattern of the cracks, to be equally important in measuring the extent of damage present in the shear walls. All these factors should be considered where evaluating the reduced capacity of a cracked element.

continues

Table C4-1 *(Continued)*. Patterns of Defects and Deterioration.

Masonry veneer	Corrosion can reduce the strength of connections and lead to deterioration of the adjoining materials. The extent of corrosion and its impact on the wall cladding and structure should be considered in the evaluation.
	Water leakage into and through exterior walls is a common building problem. Damage caused by corrosion, rotting, freezing, or erosion can be concealed in wall spaces. Substantial deterioration can lead to loss of cladding elements or panels.
	Exterior walls should be checked for deterioration. Wall spaces should be probed if necessary, and signs of water leakage should be sought at vulnerable locations (e.g., at windows and at floor areas). Particular attention should be paid to elements that tie cladding to the backup structure and that tie the backup structure to the floor and roof slabs.
	Extremes of temperature can cause substantial structural damage to exterior walls. The resulting weakness may be brought out in a seismic event. Exterior walls should be checked for cracking caused by thermal movements.
	Inadequate mortar affects the veneer's ability to withstand seismic motions and maintain attachment to the backup system.
	If mortar is noncompliant, mitigation is necessary to achieve the selected Performance Level.
	Cracking in the masonry units, depending on the material, may be caused by weathering or by stresses imposed by movement of the structure or connection system. Severely cracked masonry units probably require replacement.
	Veins in the stone can create weak points and potential for future cracking and deterioration.
Cladding	Corrosion can reduce the strength of connections and lead to deterioration of the adjoining materials. The extent of corrosion and its impact on the wall cladding and structure should be considered in the evaluation.
	Water leakage into and through exterior walls is a common building problem. Damage caused by corrosion, rotting, freezing, or erosion can be concealed in wall spaces. Substantial deterioration can lead to loss of cladding elements or panels.
	Exterior walls should be checked for deterioration. Wall spaces should be probed if necessary, and signs of water leakage should be sought at vulnerable locations (e.g., at windows and at floor areas). Particular attention should be paid to elements that tie cladding to the backup structure and that tie the backup structure to the floor and roof slabs.
	Extremes of temperature can cause substantial structural damage to exterior walls. The resulting weakness may be brought out in a seismic event. Exterior walls should be checked for cracking caused by thermal movements.

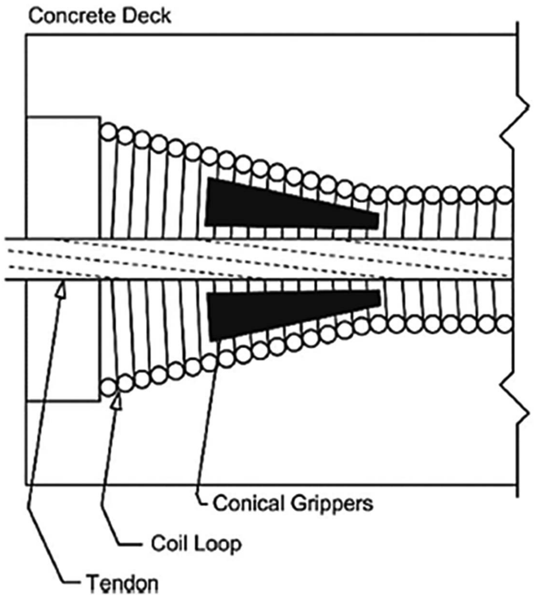


Figure C4-1. Coil anchor.

procedure in Section 7.4.1. Pseudo static seismic forces are applied to the structure to obtain actual displacements during a design earthquake. The pseudo seismic force of Equation (4-1) represents the force required, in a linear static analysis, to impose the expected deformation of the structure in its yielded state where subjected to the design earthquake motions. The modification factor C in Equation (4-1) is intended to replace the product of modification factors C_1, C_2, and C_m in Section 7.4.1. The factor C increases the pseudo seismic force where the period of the structure is low. The effect of the period of the structure is replaced by the number of stories in Table 4-7. Furthermore, the factor C is larger where a higher level of ductility in the building is relied on. Thus, unreinforced masonry buildings have a lower factor compared with concrete shear wall or moment-frame structures. In assigning values for coefficient C, representative average values (instead of using most conservative values) for coefficients C_1, C_2, and C_m were considered.

The pseudo seismic force does not represent an actual seismic force that the building must resist in traditional design codes. In summary, this procedure is based on equivalent displacements and pseudo seismic forces. For additional commentary regarding this linear static analysis approach, please refer to the commentary of Chapter C7.

C4.4.2.4 Period The values of C_t given in this standard are intended to be reasonable lower-bound (not mean) values for structures, including the contribution of nonstructural elements. The value of T used in the evaluation should be as close as possible to, but less than, the true period of the structure.

C4.4.3.1 Story Drift for Moment Frames Equation (4-6) assumes that all the moment frames generally have similar stiffness. This includes similar stiffness of columns, beams, and bay spacing at a given floor in a given direction. If this is not the case, then the approximate results from Equation (4-6) may not be a valid predictor of drift. For configurations that include significant variations in bay spacing, column and beam stiffness, or involve column transfers, an alternate approach to estimating story drift must be used. If story drift cannot be reliably estimated using the approximate methods of Tier 1, then the Tier 1 procedure is not applicable, and a Tier 2 evaluation must be used to demonstrate compliance.

C4.4.3.2 Shear Stress in Concrete Frame Columns Equation (4-7) assumes that all of the columns in the frame have similar stiffness.

The inclusion of the term $[n_c/(n_c - n_f)]$ in Equation (4-7) is based on the assumption that the end column carries half the load of a typical interior column. This equation is not theoretically correct for a one-bay frame and yields shear forces that are twice the correct force; however, because of the lack of redundancy in one-bay frames, this level of conservatism is considered appropriate.

C4.4.3.5 Precast Connections The term $[1/(n_c - n_f)]$ in Equation (4-10) is based on the assumption that the end column carries half the load of a typical interior column.

C4.4.3.6 Column Axial Stress Caused by Overturning The 2/3 factor in Equation (4-11) assumes a triangular force distribution with the resultant applied at 2/3 the height of the building.

C4.4.3.7 Flexible Diaphragm Connection Forces Consistent with the general approach to the quick check procedures, this section provides a simplified method for determining the wall anchorage forces for walls that are braced by flexible diaphragms. However, due to the simplified nature of the quick check, the forces used for evaluation of existing buildings using the Tier 1 screening procedure can exceed the forces used to design new buildings in accordance with ASCE 7. Therefore, this section includes a cap on the forces based on the wall anchorage forces in Chapter 7. The forces determined using Section 7.2.13.1 requires more analytical effort but may result in lower demands in some conditions.

C4.4.3.8 Prestressed Elements The average prestress is simply calculated as the effective force of a prestressed strand times the number of strands divided by the gross concrete area. In many cases, half-inch strands are used, which correspond to an effective force of 25 kips (111 kN) per strand.

C4.4.3.9 Flexural Stress in Columns and Beams of Steel Moment Frames Equation (4-14) assumes that all of the columns in the frame have similar stiffness.

The inclusion of the term $[n_c/(n_c - n_f)]$ in Equation (4-14) is based on the assumption that the end column carries half the load of a typical interior column. This equation is not theoretically correct for a one-bay frame and yields forces that are twice the correct force. However, because of the lack of redundancy in the one-bay frame, this level of conservatism is considered appropriate. The equation may also be conservative when checking the top level of a frame.

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CHAPTER C5
TIER 2 DEFICIENCY-BASED EVALUATION AND RETROFIT

C5.1 SCOPE

Tier 2 deficiency-based evaluation is new nomenclature for what ASCE 31-03 refers to as Deficiency-Only Tier 2 Evaluation. Tier 2 deficiency-based evaluation limits the scope of the evaluation to examining all potential deficiencies associated with Tier 1 noncompliant statements. The deficiency-based retrofit is new nomenclature for what ASCE 41-06 refers to as the Simplified Rehabilitation Method.

Because of the nature of mitigating seismic hazards of nonstructural components, the individual components should be evaluated and/or retrofitted in a systematic manner in accordance with the provisions of Chapter 13. Therefore, whereas ASCE 31-03 had Tier 2 provisions for the evaluation of nonstructural components, these provisions have been replaced by the nonstructural provisions of ASCE 41-06 as updated in this standard, because there is no practical difference between the Tier 2 and Tier 3 procedures for nonstructural components.

C5.2 GENERAL REQUIREMENTS

The deficiency-based methods (for evaluation and retrofit) only apply to buildings that sufficiently fit into one of the common building types in Table 3-1 and conform to the limitations of Section 3.5 and Table 3-8.

C5.2.1 Performance Level and Seismic Hazard Level Deficiency-based procedures are intended for further evaluation or mitigation of the deficiencies identified in a Tier 1 screening, and therefore it is only appropriate to use the same Performance Level and Seismic Hazard Level for Tier 2 as are addressed in the Tier 1 screening procedure. These methods reflect a level of analysis and design that is appropriate for relatively small buildings with well-understood, straightforward seismic-force-resisting systems that are consistent with the Tier 1 Performance Levels.

For those buildings that satisfy the limitations of Section 3.5 and Table 3-8 and for which Life Safety Performance Level or Immediate Occupancy Performance Level can be achieved by passing a Tier 1 evaluation, it is logical that such buildings can achieve the same performance levels by either (1) demonstrating by further evaluation that the building is adequate for all potential deficiencies identified by noncompliant statements or (2) modifying the building in an appropriate way such that it complies with a Tier 1 screening and Tier 2 evaluation.

C5.2.2 As-Built Information As-built information beyond that required for Tier 1 may be required to perform Tier 2 evaluations and retrofits, including destructive examination and testing. The design professional must ensure sufficient understanding of actual conditions to properly evaluate if buildings are adequate with respect to all the potential deficiencies found in the Tier 1 screening procedure. Default material property values from the material chapters may be used, or the design professional can assume the values to be as indicated in available design drawings; however, capacities of elements must include the knowledge factor, as specified in Section 6.2.3.1. Material testing is required to achieve a knowledge factor of 1.0.

C5.2.3 Condition Assessment The design professional should identify the cause and the extent of the damage. Determining the cause is useful to ensure that the condition is well understood and the extent of the damage defined. If the damage does not reduce system, element, or connection capacity, an explicit evaluation of adequacy is not required. If the capacity is reduced, the reduced capacity must be evaluated for demands. The design professional may choose to conservatively evaluate the damaged component as a force-controlled element or proceed to a Tier 3 full-building analysis. The applicable building code may have design provisions for repair of damage to the seismic-force-resisting system, including acceptance criteria, which are not covered in these provisions.

C5.2.4 Tier 2 Analysis Methods The Tier 2 analysis requirements point to those for the linear static procedure (LSP) and linear dynamic procedure (LDP) in Chapter 7. Tier 2 linear procedures are not limited by Section 7.3.1.1 because Tier 2 procedures only apply to buildings that sufficiently fit into one of the common building types in Table 3-1 and conform to the limitations of Section 3.5 and Table 3-8. However, the use of the LSP is limited by Section 7.3.1.2, which identifies when LDP procedures are required. In addition, certain Tier 2 evaluation procedures require the use of the LDP.

Analysis procedures for Tier 2 deficiency-based evaluation and retrofit are limited to the linear procedures. Tier 2 procedures limit the scope of members, components, and details that require evaluation based on expert judgment informed by past performance of similar common buildings types. Buildings that pass deficiency-based procedures may not pass all the numerical evaluations of a Tier 3 evaluation. However, buildings that meet all the appropriate requirements of the Tier 2 deficiency-based evaluation or retrofit are deemed to comply with the Basic Performance Objective for Existing Buildings (BPOE). Nonlinear analysis procedures require detailed consideration of all primary and secondary elements to determine which elements remain elastic and which are required to be modeled as nonlinear elements, and the behavior of the structure as a whole depends on the accurate consideration of all nonlinearity. Thus, nonlinear analysis implies an evaluation of all elements and connections to ensure that the results of the nonlinear analysis are accurate, which is inconsistent with the Tier 2 deficiency-based procedures and implies a level of certainty beyond Tier 2 and more consistent with Tier 3.

Experienced users of nonlinear analysis may choose to use nonlinear procedures to evaluate potential deficiencies identified by the Tier 1 evaluation when using this standard on a voluntary basis, just as users may use Tier 3 procedures for evaluations and partial upgrades.

Although the Tier 2 deficiency-based evaluation limits the scope of the evaluation to specific systems, elements, connections, and details associated with a potential deficiency identified in Tier 1, the design professional often needs to perform a full analysis of the entire building's structure to obtain the necessary actions (e.g., deformations or forces) to evaluate the structure's adequacy for the potential deficiency. The general requirements of Section 7.2 provide procedures for demands on diaphragm elements and on walls from out-of-plane response. The Tier 2 analysis requirements are not meant to preclude the design professional from demonstrating adequacy of the structure for the potential deficiency by using upper-bound demands, such as using a limit state analysis or force-controlled methods, in lieu of complete analysis. Such limit state analyses may require approval of the Authority Having Jurisdiction.

C5.2.5 Tier 2 Acceptance Criteria Tier 2 acceptance criteria are the same as for Tier 3 procedures and are prescribed in Chapter 7, which references material-specific requirements in Chapters 8 through 12.

C5.2.6 Knowledge Factor Because of uncertainties in the material properties in existing buildings, the potential exists for there to be significant variation from what is specified in the construction documents or from the default material properties. To account for this potential variability, material testing is required or the values are reduced by the knowledge factor, κ. See Section C6.2.3.1 for additional discussion.

C5.3 TIER 2 DEFICIENCY-BASED EVALUATION REQUIREMENTS

The design professional is to determine through further analysis and evaluation if a potential deficiency identified in Tier 1 screening is indeed a deficiency or if all structural systems, elements, connections, and details associated with the potential deficiency are adequate. Chapter 4 of ASCE 31-03 included each evaluation statement followed by Tier 2 evaluation procedures and commentary, most of which was commentary on the potential deficiency associated with the statement. Whereas in ASCE 31-03 the Tier 2 procedures were organized by Tier 1 statements, in this standard, the Tier 2 procedures are organized in a manner that allows elimination of repetitive Tier 2 requirements and clarification of the scope of Tier 2 deficiency-based evaluations. Sections 5.4 to 5.8 provide consolidated Tier 2 evaluation procedures and commentary on the Tier 2 procedures. Commentary from ASCE 31-03 on the statement's potential deficiency is now in Appendix A, organized by statements, along with commentary on deficiency-based rehabilitation strategies for certain statements.

Where the provisions in Sections 5.4 through 5.7 indicate that there is no Tier 2 procedure for a particular Tier 1 checklist statement, the design professional may either terminate the evaluation or consider a retrofit measure for that deficiency. The items without Tier 2 procedures generally involve lack of structural load path or interconnection such that there is no system to analyze or evaluate.

C5.4 PROCEDURES FOR BASIC CONFIGURATION OF BUILDING SYSTEMS

C5.4.1 General

C5.4.1.1 Load Path A complete load path is a basic requirement for all buildings evaluated using this standard. If the design professional does not identify a complete load path, a Tier 2 deficiency-based evaluation is not sufficient. The absence of a complete, well-defined load path does not mean that there is no seismic force load path. Alternate load paths through the secondary elements may be present. In that case, the building requires a Tier 3 systematic evaluation to assess the adequacy of any alternative load paths. The design professional should use judgment to decide if the alternate load path is so egregiously deficient that the Tier 3 evaluation would provide little added value and the evaluation should be concluded.

C5.4.1.2 Adjacent Buildings The design professional needs to analyze the structure to determine story drifts of the building, or alternatively, to develop a conservative upper bound for the drift magnitude. Similarly, the design professional has to develop an estimate of the drift for the adjacent building. The standard recognizes that available information for the adjacent building may be limited and an estimate may need to be developed using approximate methods appropriate for the information available. The estimate should be conservative if not based on analysis conforming to Chapter 7 requirements. Observations from past earthquakes support the notion that if buildings have similar structural systems, and thus similar stiffness, and the floors align, then the prescribed separation is not necessary to achieve Life Safety Performance Level. However, the response of a stiff building adjacent to a flexible building may be significantly amplified by pounding from the flexible building and vice versa where there exists insufficient separation, even with matching floor levels and heights.

C5.4.1.3 Mezzanines The design professional needs to perform sufficient analysis and evaluation to determine if there is an adequate load path to transfer forces associated with the mass of the mezzanine to the main seismic-force-resisting system. The evaluation should include connections to the elements of the main structure and their adequacy to accommodate the mezzanine forces. Particular attention should be paid to transverse forces on columns, out-of-plane forces on walls, and weak axis bending of unbraced beams.

C5.4.2 Building Configuration

C5.4.2.1 Weak Story Irregularity An analysis of the entire structure is required to determine the seismic demands at locations of strength discontinuities. However, the demand from a linear analysis does not include the potential concentrated postelastic drift demands at the story if there is a story mechanism caused by the weak story. Modifying the m-value as indicated conservatively accounts for the potential concentrated drift demands of a story mechanism. A Tier 3 nonlinear analysis will more accurately predict inelastic drift demands.

The elements of the story's seismic-force-resisting system include those elements in the seismic-force-resisting system that are in the floor or roof directly above and below the story, in addition to the columns, walls, or braces in the story.

C5.4.2.2 Soft Story Irregularity A dynamic analysis of the entire structure is required to determine the seismic demands at locations of stiffness discontinuities. The elements of the seismic-force-resisting system are required to meet the Tier 2 acceptance criteria. In addition, all elements need to be evaluated for the drift of the soft story. The evaluation is only required at noncompliant stories. The elements of the story's seismic-force-resisting system include those elements in the seismic-force-resisting system that are in the floor or roof directly above and below the story, in addition to the columns, walls, or braces in the story.

C5.4.2.3 Vertical Irregularities Calculation of the demand-capacity ratio (DCR) for elements is used to determine if linear procedures are applicable given the irregularity. Systems, elements, and connections that transfer seismic forces at the discontinuity are to be considered force-controlled elements to ensure that yielding does not occur in these elements.

C5.4.2.4 Geometric Irregularity Geometric irregularities affect the dynamic response of the structure and may lead to unexpected higher mode effects or concentrations of demand. A dynamic analysis is required to calculate the distribution of seismic forces more accurately.

C5.4.2.5 Mass Irregularity Mass irregularities affect the dynamic response of the structure and may lead to unexpected higher mode effects or concentrations of demand. A dynamic analysis is required to calculate the distribution of seismic forces more accurately.

C5.4.2.6 Torsion Irregularity A three-dimensional LDP analysis of the entire structure is required to capture the additional demands from torsion response.

C5.4.3 Geologic Site Hazards and Foundation Components

C5.4.3.1 Geologic Site Hazards The potential for liquefaction, slope failure, or surface fault rupture at a site requires a level of evaluation beyond the Tier 2 procedures. The provisions in Chapter 8 are more appropriate for the analysis of these conditions.

C5.4.3.3 Overturning For shallow foundations, the shear and moment capacity of the foundation elements should be evaluated for adequacy to resist calculated seismic forces. The vertical bearing pressure of the soil under seismic loading conditions caused by the total gravity and overturning loads should be calculated. For deep foundations, the vertical capacity of the pile or pier under seismic loads should be determined. The foundation capacity, determined in accordance with Chapter 8, shall then be compared with the demands caused by gravity loads plus overturning.

C5.5 PROCEDURES FOR SEISMIC-FORCE-RESISTING SYSTEMS

C5.5.1 General

C5.5.1.1 Redundancy Tier 1 Quick Checks are not sufficient if there is a lack of redundancy. When stories do not meet the redundancy requirements, the design professional must perform analysis to determine demands and evaluate the adequacy of the systems, elements, and connections of the seismic-force-resisting system.

C5.5.2 Procedures for Moment Frames

C5.5.2.1 General Procedures for Moment Frames

C5.5.2.1.1 Interfering Walls A moment-frame system that has interfering walls requires evaluation as an infill frame. Interfering walls should be checked for forces induced by the frame, particularly where damage to these walls can lead to falling hazards near means of egress. The frames should be checked for forces induced by contact with the walls, particularly if the walls are not full height or do not completely fill the bay.

C5.5.2.1.5 Strong Column–Weak Beam When weak column–strong beam joints exist in a moment frame, there are two potential issues: (1) whether the column can accept a hinge and still carry the gravity load, and (2) whether enough hinges can form in the columns in a given story to create a potential for a story mechanism and potential collapse. If there is a sufficient number of strong column joints in a given frame in a given story, then a story mechanism can be disregarded and the columns with weak column joints can be checked using appropriate m-factors from the material chapters. However, if a large fraction of the joints in a given frame in a given story are weak column joints, checking the columns using the material m-values does not ensure that there will not be a story mechanism because the linear analysis to generate the demands does not include the potential concentrated postelastic drift demands at the story if there is a mechanism. Modifying the m-factor as indicated conservatively accounts for the potential concentrated drift demands of a story mechanism. A Tier 3 nonlinear analysis more accurately predicts inelastic drift demands.

C5.5.2.2 Procedures for Steel Moment Frames

C5.5.2.2.3 Panel Zones Where panel zones cannot develop the strength of the beams, compliance can be demonstrated by checking the panel zones for actual shear demands.

C5.5.2.2.5 Compact Members The adequacy of the frame elements should be demonstrated using the appropriate m-factors in consideration of reduced ductility for noncompact sections.

C5.5.2.2.7 Girder Flange Continuity Plates Without continuity plates, the column flanges must be able to transfer the beam flange forces to the column panel zone. In addition, the lack of continuity plates affects the ductility of the beam-to-column connection and therefore requires evaluation of any beam-to-column connection where there are no continuity plates using appropriate acceptance criteria.

C5.5.2.3 Procedures for Concrete Moment Frames

C5.5.2.3.4 No Shear Failures Members that cannot develop the flexural capacity in shear should be checked for adequacy against calculated shear demands. Note that, for columns, the shear capacity is affected by the axial loads and should be based on the most critical combination of axial load and shear.

C5.5.2.3.5 Continuous Beam Bars Because noncompliant beams are vulnerable to collapse, the beams are required to resist demands at an elastic level. Continuous slab reinforcement adjacent to the beam may be considered as continuous top reinforcement.

C5.5.2.3.6 Column and Beam Bar Splices Beams and columns with noncompliant lap splices are checked using smaller m-factors to account for this potential lack of ductility. If the members have sufficient capacity, the demands are less likely to cause degradation and loss of bond between the concrete and the reinforcing steel.

C5.5.2.3.7 Column-Tie Spacing and Beam Stirrup Spacing Elements with noncompliant confinement are checked using smaller m-factors to account for this potential lack of ductility.

C5.5.2.3.9 Joint Eccentricity The demand associated with the smallest column plan dimension should be calculated for the column at each joint under consideration.

C5.5.2.3.10 Stirrup and Tie Hooks Elements with noncompliant confinement are checked using smaller m-factors to account for potential lack of ductility for Life Safety and Immediate Occupancy Performance Levels.

C5.5.3 Procedures for Shear Walls

C5.5.3.1 General Procedures for Shear Walls

C5.5.3.1.1 Shear Stress Check When a story fails the Tier 1 Quick Check for stress, a full-building analysis is required to get the proper distribution of forces to individual shear walls. The shear walls are then checked against the acceptance criteria for shear and flexure. The check is required for the highest nonconforming story and all stories below it.

*C5.5.3.3 **Procedures for Precast Concrete Shear Walls***

C5.5.3.3.1 Wall Openings Walls are compliant if an adequate load path for shear transfer, collector forces, and overturning resistance can be demonstrated.

C5.5.4 Procedures for Braced Frames

C5.5.4.1 Axial Stress Check The axial stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building.

C5.7 PROCEDURES FOR CONNECTIONS

C5.7.4 Interconnection of Elements

C5.7.4.4 Beam, Girder, and Truss Supported on Unreinforced Masonry (URM) Walls or URM Pilasters Retrofit measures include adding secondary columns that support vertical loads of roof and floor members of beams, girders, or trusses supported on URM walls or pilasters.

C5.8 TIER 2 DEFICIENCY-BASED RETROFIT REQUIREMENTS

The Tier 2 retrofit procedure is used to mitigate the seismic deficiencies, as identified in Tier 1 screening and confirmed with Tier 2 evaluation, by means of a seismic retrofit based on the Tier 2 acceptance criteria. Consistent with the methodology of the deficiency-based procedures, only the identified deficiencies that are addressed in the Tier 2 retrofit, based on the assumption that features of a common building type eligible for these procedures that have been previously identified as compliant, are not likely to be a cause of unacceptable seismic performance. The degree of certainty that a building undergoing a Tier 2 retrofit will achieve a given seismic performance objective is comparable to the degree of certainty associated with a Tier 2 evaluation—less than a Tier 3 evaluation or retrofit, but acceptable in the experience-based context of the standard.

A Tier 2 retrofit involves making changes to the seismic-force-resisting system of an existing building. To maintain an acceptable degree of certainty that the retrofitted building will achieve the selected performance objective, there are certain requirements for a Tier 2 retrofit—both in terms of not accidently making the expected seismic performance worse and by ensuring that the retrofit is effective.

C5.8.1 Compliance with Deficiency-Based Evaluation The limited scope of the Tier 2 retrofit still relies on the experience-based understanding of these relatively common buildings. The modifications to the building for the retrofit should not result in a building that does not sufficiently conform to the features of the common building type and that does not meet all the limitations for a Tier 1 and Tier 2 eligibility.

C5.8.2 Additional Evaluation of the Resulting Building

C5.8.2.1 Building Configuration The modifications to building for the seismic retrofit should not cause inadvertent harm to the building with respect to seismic performance. In particular, the standard requires evaluating any irregularities the retrofit may introduce. Introducing irregularities should be avoided but it is not uncommon to find practical retrofit strategies that may introduce an irregularity such as adding shear walls on three sides of a small building. If irregularities are introduced, they must pass Tier 2 evaluation procedures and all pertinent acceptance criteria must be met. The irregularities listed in this section are taken from the Tier 1 Building Configuration checklist since that is consistent with the deficiency-based procedures. The modified building must still conform with all the limitations for Tier 1 and Tier 2 retrofit procedures set forth in Section 3.5.1.

The requirements of this section are different from some model building code requirements for voluntary seismic retrofits (e.g., the IEBC), which prohibit the introduction of any new structural irregularity. Because the Tier 2 procedure is robust enough to adequately assess the seismic performance of buildings with irregularities, it is not necessary to prohibit these irregularities, but only to require that the building meets the selected performance objective(s) with the irregularities present. However, the requirements of an adopted building code or local amendments may still govern the assessment of irregularities in a Tier 2 retrofit.

C5.8.2.2 Increased Gravity Demands to Existing Elements The requirement to check existing elements for increased gravity loads is consistent with model building code requirements for buildings undergoing any type of structural alteration and is a necessary check on inadvertently causing an overloaded condition in an existing element not otherwise assessed in a deficiency-based evaluation.

C5.8.2.3 Increased Seismic Demands to Existing Elements Because a retrofit may increase seismic demands on existing elements that may not have been addressed by a Tier 1 screening, it is necessary to make sure such elements in the lateral load path remain within compliance with the selected performance objective(s) and the retrofit provides the intended strengthening. The requirement to check elements subjected to the 10% increase in seismic demands is limited to that portion of the increase that is due to seismic mass or change in the seismic load path to avoid this trigger being applied to the entire structure when the base shear is increased due to adding stiffness and reducing the period, which may increase the spectral response. For example, if adding shear walls or braced frames to an existing structure, the intent is to only check those elements or connections where the 10% change in seismic load is due to change in load path or seismic mass. An example of a load path change is adding a shear wall or braced frame to an existing building that could change the force distribution in the floor diaphragm, which would then trigger evaluation of the diaphragm to ensure compliance with the selected performance objective(s).

C5.8.3 Evaluation of New and Modified Structural Elements and Connections Passing only Tier 1 quick stress checks is not sufficient for the retrofit design. The resulting retrofitted building must pass Tier 2 analysis and evaluation procedures for all new structural elements and connections and all existing elements and connections that are modified.

C5.8.4.2 Design and Detailing Requirements A retrofit is likely to introduce new structural elements into an existing building with elements that do not meet the detailing requirements for new buildings; therefore, the Tier 2 retrofit procedure does not require all new elements to meet detailing requirements for new buildings (Item 1 in the list in this section). The important consideration is that the detailing of the new structural elements is consistent with the level of ductility assumed for the acceptance criteria (m-values) of the Tier 2 retrofit analysis and design. The model building codes have traditionally required all new elements to meet the detailing requirements of the building code for new construction, in large part because the building code contains no provisions for evaluating the performance of new elements that do not meet these detailing requirements. Because a fundamental purpose of this standard is to evaluate the performance of noncode complying elements, there is no technical or philosophical reason for new elements to meet new code detailing as long as they achieve the selected performance objective. Therefore, this provision of the standard is intentionally different from the model building code. Because the new elements are proportioned, analyzed, and designed to achieve compliance with the selected performance objective(s), compliance with all specific system requirements for a specific seismic design category is not deemed necessary. However, in some jurisdictions, the Authority Having Jurisdiction may require all new elements to meet the detailing requirements for new buildings.

Item 2 in the list complements Item 1 by ensuring that a minimum amount of ductility is prescribed for each new element or system by requiring that the added elements meet or exceed the ductility requirements consistent with an m-factor of 2.0 for Collapse Prevention performance. This will effectively prohibit highly nonductile elements being introduced into a Tier 2 retrofit.

Item 3 in the list is effectively a minimum strength requirement for new elements. New elements are to be sized for strength based on an m-factor not greater than two times the lowest m-factor of all primary elements of the seismic-force-resisting system for the selected performance objective(s) to assure compatibility with existing elements given the Tier 2 retrofit procedures using only linear analysis. New elements may be detailed for more ductility consistent with higher m-values; however, when proportioning the element and checking acceptance, the m-value is limited to confirm that the use of the strength-based linear procedures is valid. By setting a maximum m-value, this provides a reasonable level of confidence that the new elements will not yield too far before any lower ductility elements, which would potentially overload these elements.

Item 4 states that all connections are to be evaluated as force-controlled elements so that the new elements are adequately engaged into the existing structure and that the interconnection is not the weak link. Besides connections, other elements of the seismic retrofit system may be required to be evaluated as force-controlled, including collectors and columns supporting discontinuous lateral elements. These requirements are addressed elsewhere in Chapters 5 and 7.

C5.8.4.3 Scope of Evaluation Requirements for Existing Components The scope of the analysis and evaluation of retrofitted structure needs to be sufficient to assure the added elements will be mobilized and perform as intended. Therefore, this section outlines the minimum scope of evaluation for existing elements in a structure subject to the Tier 2 retrofit, even if these elements were not addressed by the Tier 1 screening.

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CHAPTER C6
TIER 3 SYSTEMATIC EVALUATION AND RETROFIT

C6.2 DATA COLLECTION REQUIREMENTS

C6.2.2 Condition Assessment The condition assessment provides an opportunity to review other conditions that may influence the elements, systems, and overall building performance. Of particular importance is the identification of other elements and components that can contribute to or impair the performance of the primary and secondary components and connections, as well as the potential effects of adjacent buildings and nonstructural components. Degradation caused by deterioration or damage should also be considered if the degradation is anticipated to reduce the structural capacity of the primary and secondary components and connections along the load paths.

Although it would be best to observe every component in a building, that is often not practical to do so. Often a subset of building comments that can be easily accessed is all that can be observed. The standard defines two different levels of condition assessment, visual and comprehensive, with the latter requiring significantly more components be observed. The material chapters and the nonstructural chapter contain detailed provisions for the minimum number of components that should be observed during a visual or comprehensive condition assessment.

The condition assessment should include all visible portions of the primary and secondary components, including foundation elements, where accessible. Selected removal of architectural finishes may be necessary to allow for visual observations of some components, particularly where visible damage or deterioration of nonstructural components, such as architectural finishes, are observed.

Where construction documents exist for the building, measurements of the existing dimensions of the structural components should be used to verify that the existing building construction corresponds with the available construction documents.

C6.2.3 Material Properties Effective use of this standard for evaluating or retrofitting existing buildings depends on whether the material properties of the construction materials are well and reliably characterized or not. The standard uses a knowledge factor to modify component capacities in recognition of uncertainties in their actual material properties. The basis for assignment of the knowledge factor depends on whether there is determinative information on material properties. The hierarchy of reliability of such information is considered to be, from lowest to highest:

- Default values typical of the specific construction in the era of construction for the region and type of construction; the material chapters, Chapters 9 through 12, provide typical values;
- Values specified in the available construction documents, which may include drawings or specifications;
- Values provided in as-built documents and contemporary testing reports for the specific materials used; and
- Values determined by destructive and nondestructive testing completed as part of the current project.

The usual testing provisions in Chapters 9 through 12 do not require a sufficient number of samples to develop a statistically significant representation of the material properties. Therefore, the standard treats usual testing as a means to validate the material properties specified in the design drawings or default values when no values are specified on the design drawings. The standard does not permit the use of test values obtained from usual testing in the evaluation or retrofit design. When the lowest usual testing value is less than 85% of the drawing value or default value, then the registered design professional must perform comprehensive testing. The comprehensive testing is only intended for the components that required testing under usual testing, it is not meant to require additional testing. For example, if a steel building's steel properties are unknown, but the bolts or rivets are specified on the plans, comprehensive testing would not be required of the bolts or rivets, because they were not tested as part of the usual testing.

It should be recognized that the material properties may vary throughout the building and that there may have been modifications to portions of the building inconsistent with the recorded values. It is expected in all cases that the appropriateness of the material values so determined will be moderated by observation of the condition of the materials, workmanship, and care in their placement. For instance, if voids are observed in a reinforced concrete wall, then use of specified values may not be appropriate. Similarly, default values are considered to represent acceptable materials quality and placement practice at the time of placement; if there are obviously poor materials or craft, then lower values should be considered. In all cases, the engineer is urged to use judgment and sound reasoning.

Statistical tests provided in ASTM E178 (2016b) can be used to determine whether an extreme test value should be rejected as an outlier. The additional testing should be further broken up by element type (e.g., walls, beams, columns, or slabs) and by floor level if a coefficient of variation is not achieved in the initial groupings.

C6.2.3.1 Knowledge Factor for Linear Procedures The knowledge factor, κ, is used to express the confidence with which the properties of the building components are known when calculating component capacities using the provisions in Chapters 9 through 12. The knowledge factor reduces the component capacity from the value obtained using the material properties specified on the design or construction documents or the default values in Chapters 9 through 12 to account for uncertainty in the accuracy of the specified or assumed default material property. Less confidence or higher likelihood of variability in the material properties results in a lower knowledge factor. The value of the factor is established from the knowledge obtained based on access to original construction documents or condition assessments, including destructive or nondestructive testing of representative components. The values of the factor have been established, indicating whether the level of material testing is Usual or Comprehensive. In some instances, usual testing still requires a considerable amount of disruption. A user may elect to propose a testing scope less than Usual to their Authority Having Jurisdiction or peer reviewer, which could be used to validate the properties on the construction documents.

It is important to note that use of $\kappa = 1.0$ should be restricted to those conditions where it can be technically supported. The user should be careful to note that in the material chapters, Chapters 9 through 12, the default values therein not only reflect the era and conditions of placement but also their current physical condition and reliability. A $\kappa = 1.0$ may still be appropriate for degraded materials if a testing program has validated its use. The user may elect to perform an analysis before material testing occurs using an assumed value of κ, provided that the value of κ is substantiated by the material testing in accordance with the requirements of this section before completion of the evaluation of implementation of retrofit strategies. If the assumed value of κ is not supported by subsequent material testing, the analysis should be revised to include a value of κ consistent with the data collected.

C6.2.3.2 Property Bounding for Nonlinear Procedures In some cases, variability of the material properties can greatly affect the building's nonlinear response. However, material testing can be very intrusive, and the cost to perform appropriate material testing may become prohibitive. As an alternate, this standard permits the user to perform multiple analyses to bound the structural performance based on a range of material properties that could be present in the building. Specifically, the material strength is adjusted to accomplish this bounding. Where stiffness is a function of the material strength, like concrete or reinforced masonry, the stiffness should be bounded based on the bounding of the material strength. The nonlinear modeling parameters beyond the strength and stiffness do not need to be bounded, unless they are a function of the strength or stiffness that is affected by the bounding.

The standard requires one model to be based on lower-bound assumptions and another to be based on upper-bound assumptions. Where the standard already requires bounding analyses, such as seismic isolation and supplemental energy dissipation device properties, the material bounding properties shall be aligned with the other bounding properties. For example, lower-bound material properties should be used in conjunction with lower-bound isolator properties and vice versa. In addition, certain configurations, such as a discontinuous wall, may necessitate a combination of upper-bound properties on one component (the wall) and lower-bound properties (the column or beam supporting the wall), for example.

Bounding analyses are not permitted as an alternative to material testing for nonlinear analysis of unreinforced masonry (URM) buildings. URM buildings should always be subject to testing owing to the large amount of test variation in values of as-built material properties and deviation of as-built conditions from the default values in some instances. (The tabulated values are not a reliable source of information for completing a retrofit design when Usual, much less Comprehensive, testing is otherwise required.)

Determining the appropriate lower- and upper-bound properties will require some knowledge of the likely material properties. If information is available on the drawings, it is reasonable to begin with nominal material properties for the material and the coefficients of variations of those nominal properties based on the era of construction. In some cases, the material properties may not be known but can be postulated based on the era of construction. The bounding analysis should encompass the range of material properties considering both the different possible nominal properties and the coefficient of variation of the nominal properties.

There are instances in Chapters 9 through 12 where there is sufficient information on the design drawings that material testing is not required. For those cases, the material properties and component actions derived from those properties do not need to be bounded as part of the bounding analysis. For example, Chapter 9 usual testing provisions allow steel yield and ultimate strengths to be taken from material manufacturing standards if the designation of the steel ASTM standard and year of construction of the building are known. The bounding analysis is only intended to capture the influence of material variability for those parameters that Chapters 9 through 12 require material testing for.

The standard permits the use of drawing or default values when usual testing is performed without bounding analysis. However, there are cases, particularly in nonlinear analysis, where assuming lower strengths may lead to unconservative results, such as the prediction of demands on discontinuous elements. Therefore, the user may wish to consider bounding analysis using the higher tested values as the upper-bound values or to perform additional tests to satisfy comprehensive testing if the results of usual testing indicate that the material strength is significantly higher than the values specified on the drawings or the default values.

The component action force–displacement behavior has other sources of variability beyond the strength of the construction materials. How the construction material properties translate to the component action capacity often have some variability. The shape of the action's backbone curve and the specific points along the backbone curve all have variability too. NIST GCR 17-917-45 (2017) provides mean parameters and coefficients of variation of the component action force–deformation relationships for many construction materials.

Default values have been provided in Table 6-3 for property bounding based on the coefficients of variation identified in NIST GCR 17-917-45 for all construction material components presented in that report. The coefficient of variation for yield strength provided in NIST GCR 17-917-45 typically ranged between 0.1 and 0.15. The yield strength coefficient of variation assumes that the material strength is known and only represents the variation between the actual strength and the strength predicted by the equations specified in this standard or the reference material standards. The coefficient of variation for yield strength was increased to account for the variability between the in situ material properties and the material properties specified on the construction documents. The values presented in Table 6-3 are based on a coefficient of variation of 0.25 that includes the deviation of material properties from those specified on the design document.

The coefficient of variation from the material testing program may be significant. The user can perform more tests than the minimum specified extent in an effort to develop a more accurate coefficient of variation. The additional testing may be further broken up by element type (e.g., walls, beams, columns, or slabs) and by floor level as a means to reduce the coefficient of variation.

In some cases, an outlier result may be obtained from testing. The exclusion of a test result should be undertaken with care, as such an outlier could be indicative of a lack of quality control or quality assurance in the original construction, or an indication that multiple grades of materials may have been used in the original construction.

C6.3 TIER 3 EVALUATION REQUIREMENTS

The Tier 3 systematic evaluation may be used as a follow-up to a deficiency-based evaluation (Tier 1 or 2) or as an initial evaluation where deficiency-based procedures are not permitted by this standard or the Authority Having Jurisdiction or not desired to be used by the registered design professional. The Tier 3 procedure contains an evaluation and analysis of all of the components of the structure to determine compliance with the selected Performance Objective. The structural systems to be analyzed, as well as the procedures for analyzing the structural components, are specified in Section 7.2.

Refer to Section C6.4 for additional information about the Tier 3 procedure.

C6.4 TIER 3 RETROFIT REQUIREMENTS

The Tier 3 systematic retrofit procedure is intended to be complete and contains all requirements to reach any specified performance level. Systematic retrofit is an iterative process, similar to the design of new buildings, in which modifications of the existing structure are assumed for the purposes of a preliminary design and analysis, and the results of the analysis are verified as acceptable on a component basis. If either new or existing components still prove to be inadequate, the modifications are adjusted, and, if necessary, a new analysis and verification cycle is performed. A preliminary design is needed to define the extent and configuration of corrective measures in sufficient detail to estimate the interaction of the stiffness, strength, and post-yield behavior of all new, modified, or existing components to be used for seismic force resistance. The designer is encouraged to include all components with significant lateral stiffness in a mathematical model to ensure deformation capability under realistic seismic drifts. However, just as in the design of new buildings, it may be determined that certain components will not be considered part of the seismic-force-resisting system, as long as deformation compatibility checks are made on these components to ensure their adequacy.

A mathematical model, developed for the preliminary design, must be constructed in connection with one of the analysis procedures defined in Chapter 7. These procedures are the linear procedures (linear static and linear dynamic) and the nonlinear procedures (nonlinear static and nonlinear dynamic). With the exception of the nonlinear dynamic procedure, this standard defines the analysis and retrofit design procedures sufficiently that compliance can be checked by an Authority Having Jurisdiction in a manner similar to design reviews for new buildings. Modeling assumptions to be used in various situations are given in Chapters 8 through 12, and in Chapter 13 for nonstructural components. Requirements for seismic demand are given in Chapter 2. Requirements are specified for use of the nonlinear dynamic procedure; however, considerable judgment is required in its application. Criteria for applying ground motion for various analysis procedures is given, but definitive rules for developing ground motion input are not included in this standard.

This standard specifies acceptance criteria for stiffness, strength, and ductility characteristics of structural components for three discrete structural Performance Levels in Chapters 9 through 12 for use in the Tier 3 systematic retrofit procedure, and acceptance criteria for the performance of nonstructural components in Chapter 13.

Inherent in the concept of performance levels and ranges is the assumption that performance can be measured using analytical results such as story drift ratios or strength and ductility demands on individual components. To enable structural verification at the selected performance level, stiffness, strength, and ductility characteristics of many common components have been derived from laboratory tests and analytical studies and are presented in a standard format in Chapters 9 through 12 of this standard.

This standard specifies two alternate technologies in Chapters 14 and 15—seismic isolation and supplemental energy dissipation—for use in seismic retrofit of buildings using the Tier 3 systematic retrofit procedure.

It is expected that testing of existing materials and components will continue and that additional corrective measures and products will be developed. It is also expected that systems and products intended to modify structural response beneficially will be advanced. The format of the analysis techniques and acceptance criteria of this standard allows rapid incorporation of such technology. Chapter 7, Section 7.6 gives specific requirements in this regard. It is expected that this standard will have a significant effect on testing and documentation of existing materials and systems and on new products.

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CHAPTER C7

ANALYSIS PROCEDURES AND ACCEPTANCE CRITERIA

C7.1 SCOPE

This chapter covers analysis for both the evaluation of an existing building and the design of retrofit measures. It describes the loading requirements, mathematical model, and detailed analytical procedures required to estimate seismic force and deformation demands on components of a building. General analysis requirements are specified in Section 7.2 for gravity loads, primary and secondary components, damping, foundation modeling, multidirectional excitation, vertical seismic effects, P-Δ effects, overturning, diaphragms, continuity of the framing system, walls, buildings sharing common components, and building separations.

The relationship of the analysis procedures described in this chapter with provisions in other chapters is as follows:

- Information on Performance Objectives, including Seismic Hazard Levels and target Building Performance Levels, is provided in Chapter 2.
- For Tier 3 systematic procedures in Chapter 6, the analysis must include the entire structural system in accordance with Section 7.2.
- Information on the calculation of appropriate stiffness and strength characteristics for components is provided in Chapters 8 through 12, 14, and 15.
- Component force and deformation demands obtained from analysis using procedures described in this chapter, based on component acceptance criteria outlined in this chapter, are compared with permissible values provided in Chapters 8 through 12, 14, and 15 for the desired performance level.
- Evaluation and retrofit methods for nonstructural components (including mechanical and electrical equipment) are presented in Chapter 13.

C7.2 GENERAL ANALYSIS REQUIREMENTS

C7.2.2 Effective Seismic Weight The effective seismic weight of the building is needed to determine the seismic forces. It includes the dead load plus any permanent live load present and a realistic estimate of the snow load that might be present on the roof when an earthquake occurs. For the sake of computing effective seismic weight, where ASCE 7 requires an allowance for partitions whose location is not fixed, the seismic weight should include the value to represent partitions that exist or could be added.

C7.2.3 Component Gravity Loads and Load Combinations Gravity load combinations in this section are intended to be used with the seismic forces computed in this standard. They are not to be used to evaluate components for gravity-load-supporting ability. Evaluation of components for gravity loads and wind forces, in the absence of earthquake forces, is beyond the scope of this document.

C7.2.3.1 Dead Load ASCE 7 provides provisions to determine what permanent loads should be considered dead load versus live load. In addition, ASCE 7 contains specific provisions for determining dead loads associated with vegetative and landscaped roofs and rooftop solar panels. Instead of restating all those requirements in this standard, the user is directed to ASCE 7 for pertinent information.

C7.2.3.2 Live Load ASCE 7 contains provisions for live load and for a specific category referred to as roof live load. Live loads are due to the occupancy or use of specific portions of the building. ASCE 7 defines a separate category of live load specific to roofs called "roof live load" that is intended to account for maintenance of the roof and incidental occupancy. Roof live load does not need to be included with other live loads when used in conjunction with seismic forces. This is consistent with the approach ASCE 7 takes. However, ASCE 7 recognizes that roofs can be used for occupancy-related reasons and then specifies that such live load should be treated as regular live load as opposed to roof live load and included in load combinations with seismic forces.

The live loads in ASCE 7 often represent a worst-case loading, which is why they are subject to reductions over large areas. The 25% factor stipulated in this section is intended to reduce the unreduced live loads in ASCE 7 to better approximate the actual live load that would be present in the building during an earthquake. The provisions stipulate that this reduced live load should not be less than the actual live load. Specific regions where this caveat is most applicable are areas where live loads are intended to represent storage. The user should specially consider if a 75% reduction from the ASCE 7 live load is appropriate wherever ASCE 7 does not permit live loads to be reduced and adjust the reduction to better approximate the actual live load present.

C7.2.3.3 Snow Load Snow load exceedances and reductions are taken directly from ASCE 7, refer to ASCE 7 Section C12.7.2.

Snow loads in ASCE 7 are based on risk categories, which are not completely compatible with the building performance levels of ASCE 41. Reference is made to the definitions of Section 1.2 for selection of the appropriate snow load geodatabase in ASCE 7. In some cases, the governing building code or regulation may not specify a risk category for a particular building type. In those cases, it is permitted to base snow loads on Risk Category II. Finally, because of the lack of strict compatibility, snow loads may be based on the performance-based procedures of ASCE 7 Section 1.3.1.3.

C7.2.4 Mathematical Modeling

C7.2.4.1 Basic Assumptions For two-dimensional models, the three-dimensional nature of components and elements should be recognized in calculating their stiffness and strength properties. For example, shear walls and other bracing systems may have "L" or "T" or other three-dimensional cross sections where contributions of both the flanges and webs should be accounted for in calculating stiffness and strength properties.

In this standard, component stiffness is generally taken as the effective stiffness based on the secant stiffness to yield level forces. Specific direction on calculating effective stiffness is provided in each material chapter for each type of structural system.

Examples of where connection flexibility may be important to model include the panel zone of steel moment-resisting frames and the "joint" region of perforated masonry or concrete walls.

C7.2.4.2 Torsion Historical observation and numerical studies have shown that torsion is a result of many factors, including torsional ground motion input to the structure, soil–structure interaction effects, variation in mass distribution, and changes in component and system stiffness and strength.

To properly account for the effects of torsion, the gravity loads must be properly distributed over the building plan. As the building plan rotates as a result of inherent and accidental torsion, there can be additional rotation because of P-Δ effects. This additional rotation is referred to as P-Theta and can only be captured in a 3D analysis where geometric stiffness is included and where the gravity loads are accurately distributed in plan at each floor and roof level (Flores et al. 2018).

C7.2.4.2.1 Total Torsional Moment The actual torsional moments determined from the building mathematical model capture eccentricity only between the centers of mass and stiffness as represented in the mathematical model. The mathematical model is an idealization of the structure. Accidental torsion is required in the analysis to account for the additional contributing factors to torsion response that are not represented in the mathematical model, such as differences between the computed and actual stiffness and mass, contributions from nonstructural components or secondary components not modeled, and torsional response at the structure's base resuling from nonuniform ground shaking along the base creating a torsional input motion. The accidental torsional moment in a building is estimated using an additional mass eccentricity.

C7.2.4.2.2 Consideration of Torsional Effects for Linear Procedures Torsional response caused by actual and accidental torsion is required in linear procedures unless the building is torsionally regular in both strength and stiffness. Section 7.3.1.1.4 prohibits the use of linear procedures if there is a torsional strength irregularity because the linear procedures cannot properly capture the response of the building if one side experiences significant nonlinear response before the other side. Recognizing that accidental torsion can have limited influence on buildings (De la Llera and Chopra 1995), the standard allows accidental torsion to be ignored in linear analysis if the accidental torsional moment is significantly less than the actual torsional moment or the building has sufficient torsional stiffness that accidental torsion is unlikely to affect the results. Sufficient torsional rigidity is represented by limiting the ratio of displacement multiplier based on actual plus accidental torsion to 1.1 times the value based solely on actual torsion. The limit of 1.1 for accidental torsion is based on judgment.

C7.2.4.2.3 Consideration of Torsional Effects for Nonlinear Procedures Inclusion of accidental torsion in nonlinear analysis can be burdensome because it can quadruple the number of analysis cases and provide little difference in the evaluation. The standard permits consideration of accidental torsion in one direction when nonlinear analysis with a three dimension mathematical model is used. Linear analysis is used to determine the most significant direction to offset the center of mass an additional 5%. If the building has a torsional strength irregularity, that is considered the more significant direction. If both directions have torsional strength irregularities, then two different accidental torsion offsets must be considered, or the accidental torsion offset should occur in both directions at the same time. If the building does not have a torsional strength irregularity in either direction, the direction that is more torsionally flexible, measured by the η parameter, is considered the most significant direction. Because accidental torsion should not reduce demands, the provisions require a mathematical model without accidental torsion. In most cases, this will result in two analysis cases as opposed to four—one without any accidental torsion, and one with a worst-case estimate of accidental torsion.

Accidental torsion may be excluded from the nonlinear analysis when the building does not possess a torsional strength irregularity and accidental torsion does not influence the building's behavior (DeBock et al. 2013, De la Llera and Chopra 1995). The first means to demonstrate accidental torsion does not influence the building's behavior is to show that moment caused by accidental torsion is a fraction of the total torsional moment, meaning the building's torsion response will likely be dominated by the actual torsion response. The second means is based on an η parameter less than 1.2 on all floors in all directions when actual and accidental torsion are considered. This limit implies that the building has significant torsional stiffness and is not sensitive to torsional response.

For nonlinear analysis procedures, three-dimensional models tend to better capture some of the aforementioned torsional contributions, and so accidental torsion need not always be explicitly included in the assessment. The provisions permit accidental torsion to be omitted in certain cases for lower seismic hazard levels where multiple hazard levels are being considered. For example, in the BPOE and BPON for a building assigned to Risk Category II, the higher hazard's structural performance level is Collapse Prevention and the lower hazard's level is Life Safety. In this case, torsional effects may be omitted for the lower hazard level because the highest hazard is more sensitive to torsion, in which case there may be significant changes in building response and evaluation outcome because of the impact of accidental torsion. This example assumes that there is at least a 1.5 times difference between hazard levels' spectral acceleration parameters, which is the ratio of the BSE-2N to BSE-1N. Because the standard permits performance-based designs using any combination of hazard levels, it requires that the highest hazard's spectral acceleration parameters are at least 150% of all the spectral acceleration parameters in the lower hazard levels. In addition, if the structural performance level for any of the lower hazard levels is Damage Control or Immediate Occupancy, accidental torsion must be included in the lower hazard levels' analyses. The acceptance criteria between Damage Control and Immediate Occupancy are different enough from Collapse Prevention that capturing the effects of accidental torsion in the higher hazard does not necessarily confirm that accidental torsion effects will not affect the lower hazard conformance with the Damage Control and Immediate Occupancy Performance Levels. Last, if the highest hazard predicts an unacceptable response, it may be an indicator that accidental torsion played a significant role in the building's response and should be investigated at the lower hazard levels to confirm it does or does not affect conformance to the lower hazard's structural performance level.

Chapters 14 and 15 have specific accidental torsion requirements for buildings that include seismic isolation and supplemental energy dissipation devices. The user is referred to those chapters for the requirements instead of the requirements in this section.

C7.2.4.3 Primary and Secondary Components The standard classifies components of a building as either primary or secondary. Primary components are elements that resist seismic forces and whose failure or degradation could compromise the lateral stability of the building. Secondary components are the other structural elements in the building, whose primary purpose is to support gravity loads. The standard requires that both primary and secondary components be evaluated to confirm that the seismic forces and deformations do not cause the components to behave in a manner inconsistent with the selected structural performance level(s). In all cases, the engineer verifies that gravity loads are sustained by the structural system, regardless of the designation of primary and secondary components.

The secondary designation may be used where a structural component does not contribute significantly to resist earthquake effects. A slab–column interior frame is an element whose structural components might be designated as secondary in a building braced by much stiffer and stronger shear walls. If the shear walls exist only in one direction, the components of the slab–column interior frame may be designated as secondary for that direction only. The secondary designation may be used where a component, intended in the original design of the building to be primary, is deformed beyond the point where it can be relied on to resist earthquake effects. For example, it is conceivable that coupling beams connecting wall piers might exhaust their deformation capacity before the entire structural system capacity is reached. In such cases, the engineer may designate these beams as secondary, allowing them to be deformed beyond their useful limits, provided that damage to these secondary components does not result in loss of gravity load capacity.

The standard sets a limit on the total stiffness of all elements designated as secondary components of 20% of the stiffness of the primary components. Stiffness rather than element type was chosen as the metric because elements that were never intended to be considered primary components may actually resist enough seismic force to affect the buildings response and should be treated as primary components. This is a major point where this standard deviates from the ASCE 7 standard for new building design. In ASCE 7 the user identifies the seismic-force-resisting system and designs its components using explicit provisions. All other components need only be assessed for deformation compatibility. This standard also requires that secondary components be assessed for deformation compatibility, but recognizes that some elements that would be excluded in a new building's lateral-force-resisting system may have a major impact on the overall building's behavior. For example, consider a reinforced concrete moment frame building that has interior flat-plate slabs supporting gravity loads. In a new building, the moment frames would be designated as the seismic-force-resisting system, regardless of how stiff the flat-plate slab–column frame was relative to the moment frames. In this standard, the flat-plate slab–column frame would need to be considered primary components if its stiffness was more than 20% of the moment frames.

Secondary components can have an effect on the torsional response of a structure. The standard requires some secondary components be reclassified as primary components if the secondary components shift the center of rigidity of the system at a story or cause an increase in displacements at the extreme ends of the structure.

There may be instances where the secondary components are not stiff enough relative to the primary components to be reclassified as primary or stiff enough to change the torsional response of the floor, but the secondary components may cause an increase in forces or deformations in one or more primary components. An example of this is when concrete flat-plate frames that are integral with structural walls create an outrigger effect on the wall, leading to an increase in the shear force in the wall while not affecting the moment as significantly. For a case like this, the slab–column frames creating the outrigger with the structural wall should be included as primary components.

The contribution of secondary components can be checked by temporarily including them in the analysis model and examining the change in response. Alternatively, the structural system may be such that it is clear what are primary and what are secondary components based on judgment of the user or simplified calculations. An example of this is the partially restrained moment frames created by steel gravity framing beams connecting to steel columns in a building with braced frames.

In some cases, nonstructural components are attached between two or more floors of a building. Three common examples of this are cladding, partitions, and stairs. At a minimum, these components should be treated as secondary components to assess if they would affect the seismic response of the building because of their relative stiffness compared to the primary components, their ability to shift the center of rigidity, or their locally increasing demands on a primary component. In older construction, precast cladding was attached to the building without the ability to accommodate story drift. In moment frame buildings, the precast cladding may be stiff enough relative to the moment frame to classified as primary components. If that is the case, then the cladding should be included in the analysis model and evaluated based on the provisions for primary structural components. On the other hand, a gypsum partition is a nonstructural component that might be designated secondary in a building because it does not provide significant stiffness or strength in any direction.

C7.2.4.3.1 Linear Procedures Because of limitations inherent in each analysis method, the manner in which primary and secondary components are handled differs for linear and nonlinear procedures.

For linear procedures, the standard limits the amount of inelastic deformation that a primary component can undergo. Primary component acceptance criteria for the Collapse Prevention Structural Performance Level are limited to the deformation at which the component action begins to degrade (Point C in Figure 7-9). This is done because of the inability of the linear procedures to accurately predict the change in response of a building if some of the primary components' strength and stiffness degrade past Point C. This limitation minimizes the potential for sudden loss of seismic-force-resisting component strength to produce irregular structural response that is difficult to evaluate reliably. Secondary components, on the other hand, are permitted to deform past Point C in linear procedures because their overall contribution to the lateral strength and stiffness of the structure is not significant. In that case, the Collapse Prevention limit for secondary components is the point at which the component action fails to resist all force or support gravity loads. For the Immediate Occupancy Performance Level, the acceptance criteria is the same for primary and secondary components because the performance level is explicitly attempting to limit damage irrespective of whether the component is a primary or secondary component.

The provisions permit using a representative subset of the secondary components to evaluate all the secondary components. An example of when this might be appropriate is to build a subassemblage model of secondary components at one frame at the point of largest displacement in the primary model, provided that the secondary components are similar enough that evaluating one frame for the largest deformations can be a surrogate for the behavior of all the secondary components.

C7.2.4.3.2 Nonlinear Procedures In nonlinear procedures, strength and stiffness degradation can be modeled. Because degradation of the overall system can increase displacement demands, inclusion of both primary and secondary components provides a more accurate assessment in nonlinear analyses. The standard permits excluding secondary components from the nonlinear analysis model provided it can be justified that excluding those elements will not adversely affect the response of the primary component model. If the demands in the primary components are larger with the secondary components included in the model, they cannot be excluded. However, if exclusion of the secondary components results in an increase in forces or deformations on the primary components, that is acceptable, provided the other rules are adhered to. If the secondary components are excluded, they must be evaluated for deformation compatibility using either a linear static or nonlinear static procedure with imposed displacements corresponding to the mean deformations from the analysis of the primary component model at the location of the secondary components. For example, excluding partially restrained moment frames created by connections between gravity framing and columns in steel buildings usually does not have a negative impact on the performance of the structure. In many cases, excluding those secondary components produces an increase in demands on the primary components, which is conservative, meaning the exclusion of the secondary components from the primary nonlinear model is acceptable.

When the nonlinear dynamic procedure (NDP) is used for the primary components, models of the secondary components can be checked using the mean response parameters of a nonlinear dynamic analysis similar to how those components would be evaluated if they were included in the nonlinear model with the primary components. In addition, if the nonlinear static procedure is used to evaluate the secondary components, the nonlinear model of the secondary components should be checked for unacceptable responses at the maximum deformation of the primary components in the same manner as those components would be evaluated for unacceptable response if they were included in the nonlinear dynamic model with the primary components.

C7.2.4.5 Foundation Modeling Methods for modeling foundations, including flexibility and estimation of ground movements caused by seismic geologic site hazards, are referenced in Chapter 8 and may require the expertise of a geotechnical engineer or a geologist.

The person who decides to model foundation flexibility must consider impacts on the behavior of structural components in the building. Rigid base models for concrete shear walls on independent spread footings may maximize deformation demands on the walls themselves but could underestimate the demands on other secondary components in the building, such as beams and columns in moment frames, which may be sensitive to additional building movement.

C7.2.4.6 Damping Nonstructural components, such as cladding and partitions, typically affect the structure's response with additional energy dissipation. The general requirements of 5% damping for linear static, linear dynamic, and nonlinear static procedures and 3% damping for the nonlinear dynamic procedure apply for most building structures in which nonstructural components, including partitions and elements of the exterior building envelope, are expected to dissipate energy not captured explicitly in the building computer model. Bare structures, such as some canopies, nonbuilding structures, and parking garages, are common examples of structures without exterior cladding or interior partitions that may be expected to have relatively low effective damping, as reflected in the provisions.

The damping provisions differentiate between the NDP and the LSP, LDP, and NSP. The lower damping limits associated with the NDP in Section 7.4.4.4 relative to the linear and nonlinear static procedures account for the explicit modeling of hysteretic damping in the analysis.

C7.2.5 Configuration One objective of seismic retrofit should be the improvement of the regularity of a building through the judicious placement of new framing elements.

Adding seismic framing elements at certain locations improves the regularity of the building and should be considered as a means to improve seismic performance of the building.

Secondary components can lose significant strength and stiffness after initial earthquake shaking and may no longer be effective. Therefore, regularity of the building should be determined both with and without the contribution of secondary components.

C7.2.6 Multidirectional Seismic Effects

C7.2.6.1 Concurrent Seismic Effects The hazard information is consistent with ASCE 7 for depicting the maximum direction of response. This depiction permits alternate means of addressing bidirectional loading than have historically been the case. For consistency, the traditional 100% plus 30% combinations are included in Items 1 and 2. For Item 2, the NSP, an additional technique is permitted that may be simpler to implement than the traditional 100% plus 30% combinations.

The alternate technique is simply to apply the pushover load vector in the critical direction, the direction of maximum response, for the component being evaluated. For components of typical orthogonal frame buildings, this technique amounts to pushing to 100% of the target displacement applied separately along each frame axis. For nonorthogonal frames, additional pushover cases would be applied with the load vector aligned along the direction of each frame.

For bidirectional components, for example, columns or foundations loaded by orthogonal frames, a vector direction must be estimated that is the critical direction of loading. For the simple example of the corner columns in a square, doubly symmetric perimeter frame building, the appropriate additional load vector directions would be at 45 degrees to both frames. If the frames were nonorthogonal, then the appropriate load vector might be one that bisects the two frames. If the frames are of substantially different stiffness or strength, then this difference may need to be reflected in the direction of application of the pushover load vector. Unless the difference is significant, the results are unlikely to be sensitive to the vector direction of the pushover load vector; this difference should be verified by parameter study.

If the site is in the near field, then there may be different spectra in the fault-normal and fault-parallel directions. If target displacements are calculated in different vector directions, then technically the appropriate spectrum should be computed based on the pushover application angle relative to the local fault-normal and fault-parallel axes. The same situation also exists for the 100% plus 30% combinations. If the fault-normal to fault-parallel ratio is close to unity, then it may be simpler to calculate everything conservatively using the larger fault-normal spectrum.

A suggested method for determining the appropriate value of η for different component response parameters in different parts of the building is suggested in Section 7.2.4.2.2.

The requirement for a "random" orientation in the far field is meant to achieve approximately equal input spectra along each orthogonal building axis. This result can be achieved in several ways, for example, by randomizing the input angles or by arbitrarily orienting one half of either the fault-normal or the stronger components in one direction and one half in the orthogonal direction. The components should be randomized even if spectral matching techniques are used.

Appropriate record application in the analysis model is more complex in the near field. The components already have been rotated to fault-normal and fault-parallel relative to their governing fault as part of the selection and scaling process. For the amplitude scaling technique, this technique usually results in the fault-normal components being higher than the fault-parallel components, although the ratio varies significantly with period. If spectral matching techniques have been used and different fault-normal and fault-parallel spectra were developed, then the average spectrum of each set of components closely matches the target.

The records should be applied to the model with fault-normal components aligned appropriately relative to the nearby fault that dominates the hazard. Additional considerations and measures may be required if there are multiple nearby faults that contribute significantly to the site hazard, especially if these faults are not relatively parallel to one another.

C7.2.7 P-delta Effects Static P-Δ effects are caused by gravity loads acting through the deformed configuration of a building and result in an increase in lateral displacements.

Dynamic P-Δ effects are caused by a negative post-yield stiffness that increases story drift and the target displacement. The degree by which dynamic P-Δ effects increase displacements depends on the following:

1. The ratio of the negative post-yield stiffness to the effective elastic stiffness;
2. The fundamental period of the building;
3. The strength ratio, μ_{strength};
4. The hysteretic load-deformation relations for each story;
5. The frequency characteristics of the ground motion; and
6. The duration of the strong ground motion.

Because of the number of parameters involved, it is difficult to capture dynamic P-Δ effects in linear and nonlinear static analysis procedures. For the NSP, dynamic instability is measured by the strength ratio, μ_{strength}. For the NDP, dynamic P-Δ effects are captured explicitly in the analysis.

C7.2.8 Soil–Structure Interaction Interaction between the structure and the supporting soil consists of the following:

1. Foundation flexibility: introduction of flexibility and strength at the foundation–soil interface;
2. Kinematic effects: filtering of the ground motions transmitted to the structure based on the geometry and properties of the foundation; and
3. Foundation damping effects: dissipation of energy through radiation and hysteretic soil damping.

Consideration of soil–structure interaction (SSI) effects caused by kinematic interaction or foundation damping, which serve to reduce the shaking input to the structure relative to the free-field motion, is covered in Section 8.5.

SSI may modify the seismic demands on a building. It can reduce or increase spectral accelerations and seismic forces, but it can also increase lateral displacements and secondary forces caused by P-Δ effects. Changes in seismic demand caused by explicit modeling of foundation flexibility, foundation damping, or kinematic effects can be significant and should be used where applicable. Where SSI effects are not required to be evaluated, use of all three effects alone or in combination is permitted.

For those cases, such as near-field and soft-soil sites or buildings with short fundamental periods on the ascending branch of the general response spectrum or a site-specific response spectrum, in which the increase in period caused by SSI increases spectral accelerations, the effects of SSI, specifically the inclusion of foundation flexibility, on building response must be evaluated. Further discussion of SSI effects can be found in FEMA 440 (2005) and NIST GCR 12-917-21 (NIST 2012a).

C7.2.9 Overturning Response to earthquake ground motion results in a tendency for buildings and individual vertical elements of buildings to overturn about their bases. Although actual overturning failure is rare, overturning effects can result in significant stresses, as demonstrated in some local and global failures. In new building design, earthquake effects, including overturning, are evaluated for seismic forces that are significantly reduced (by an *R*-factor) from those that may actually develop.

For elements with positive attachment between levels that behave as single units, such as reinforced concrete walls, the overturning effects are resolved into component forces (e.g., flexure and shear at each level and at the bases of the walls). For linear procedures, the element is then proportioned with adequate strength using *m*-factors, where appropriate, to resist overturning effects resulting from these force levels.

Some elements, such as wood shear walls, may not have positive attachments between levels. An overturning stability check is typically performed for such elements when buildings are designed using codes for new buildings. If the element has sufficient dead load to remain stable under the overturning effects of the design seismic forces and has sufficient shear connection to the level below, then the design is deemed adequate. However, if dead load is inadequate to provide stability, then tie-downs or other types of uplift anchors are provided to resist the residual overturning caused by the design forces.

In the linear and nonlinear procedures of this standard, seismic forces are not reduced by an *R*-factor, as they are for new buildings, so computed overturning effects are larger than those typically calculated for new buildings. Although the procedure used for new buildings is not completely rational, it has resulted in successful performance. Therefore, it may not be appropriate to require that structures and elements of structures remain stable for the pseudo seismic forces used in the linear procedures in this standard. Instead, the analysis must determine if positive direct attachment is used to resist overturning effects or if dead loads are used. If positive direct attachment is used, then the overturning effect at this attachment is treated just as any other component action.

However, if dead loads alone are used to resist overturning, then overturning is treated as a force-controlled behavior. The expected overturning demands can be estimated by considering the overall limiting strengths of the components.

There is no simple rational method available shown to be consistent with observed behavior to evaluate or retrofit elements for overturning effects. The method described in this standard is rational but inconsistent with procedures used for new buildings. To ensure Damage Control, the full seismic forces used in the linear procedures of this standard are required for checking acceptability for performance levels higher than Life Safety.

C7.2.9.1 Overturning Effects for Linear Procedures For evaluating whether dead loads provide stability against overturning, the alternative procedure of Section 7.2.9.1 is intended to provide a method that is consistent with prevailing practice specified in current codes for new buildings.

C7.2.10 Sliding at the Soil–Structure Interface If, for a structure supported on shallow foundations, the total resistance capacity at the soil–structure interface is less than the base shear capacity of the superstructure, the likely consequence is that the structure will slide with respect to the moving ground. This need not be considered detrimental in many cases. If the sliding resistance along a line of frames or shear walls is not commensurate to the proportion of seismic shear assumed to be carried along those lines, then differential sliding may occur within the building. This is addressed by the requirement of Section 7.2.10.1 to tie foundation elements together. It is intended that Section 7.2.10.1 be applied if the sliding resistance along any frame or shear wall line is less than the effective yield capacity of the frames or shear walls along that line. The engineer may choose to comply with Section 7.2.10.1 without carrying out the line-by-line sliding check specified in Section 7.2.10.

In addition to ground-level displacements, sliding may interact with rocking and result in larger displacements in the superstructure. For buildings with relatively tall and narrow lines of lateral resistance, rocking may preempt sliding; a rocking-sliding interaction is more likely to occur at lower aspect ratios.

Sliding at the soil–structure interface should be considered distinct from slope failure (landsliding) and lateral movements caused by liquefaction. Such hazards should be addressed as indicated in Chapter 8. Also, this section is intended to apply only to buildings with shallow foundations; where sliding resistance is provided by deep foundations, the provisions of Chapter 8 should be followed instead.

Movements at the soil–structure interface may activate passive soil pressure against basement retaining walls as well as shear and bending in deep foundation elements, and friction beneath gravity column footings may also impose bending and shear on those elements. Such actions should be considered where associated structural impacts are relevant to the structural performance level. Such elements are to be evaluated as other similar structural elements, and may be considered primary or secondary, or force controlled or deformation controlled, according to the provisions of the relevant material chapter of this standard.

The displacements associated with sliding may damage utilities connecting into the building, both at underground connections and where utilities cross between adjacent buildings in the superstructure; hence, sliding may be a nonstructural concern if the selected nonstructural performance level requires that specific components remain operational. The provisions of Chapter 13 should be used for evaluating such components. Where sliding is expected, utilities that cross the interface between the structure and the soil may be treated as crossing a seismic joint. A rational method should be used for estimating the magnitude of displacement to be accommodated. Such movement may be accommodated by intentional joints placed in utility lines or by the flexibility of the utility lines themselves.

C7.2.10.1 Foundation Interconnection A fixed-base superstructure analysis may significantly overestimate the lateral stiffness of lines of lateral resistance if such lines do not correspond to points of soil resistance, and such analyses may thus miss potentially significant failure mechanisms in the superstructure. For example, a concrete shear wall supported on a shallow foundation may be represented as a stiff element in a fixed-base superstructure analysis, yet the wall may not carry much gravity load. Much of the lateral soil resistance in a shallow foundation may be provided by friction underneath gravity column foundations. If the gravity columns in such a structure are not tied to the shear walls by a stiff diaphragm, then the superstructure analysis may significantly underestimate the shear which will be resisted by gravity columns. Although the redistribution of lateral loads at the base diaphragm might be considered an out-of-plane discontinuity irregularity, this standard does not intend for this feature to limit the use of the linear static procedure. Instead, this provision is intended to ensure that the diaphragm is sufficiently stiff and robust. Structures having a mat foundation are likely to satisfy this provision without need for substantial justification.

The first method provided for checking the diaphragm or horizontal bracing assumes that these elements and locations of soil resistance are not explicitly modeled. This method entails solving for static equilibrium between forces delivered from the superstructure and the probable distribution of forces at points of contact with the soil. The magnitude of force to be redistributed may be limited by the seismic hazard, by the capacity of the superstructure, or by the capacity of the soil. In this analysis, this total force may be applied at frame or shear wall lines in proportion to their relative stiffness, and resisted at points where soil friction or passive pressure may act in proportion to the capacities of these mechanisms. To ensure that the base diaphragm is not underdesigned, the limiting forces should be based on expected values for the superstructure capacity and/or upper-bound values for the soil capacity. Given that upper-bound demands are applied, limited ductility should be permitted in the diaphragm, as is permitted in diaphragms of the superstructure. This evaluation must also include the force transfer of the diaphragm or bracing to the foundation elements to be interconnected.

This method does not require that the stiffness of the diaphragm or horizontal bracing be explicitly checked, nor that deformation compatibility be enforced between these elements and the soil resistance mechanisms. It is assumed that if the diaphragm has sufficient strength to redistribute these forces (even allowing for some minimum ductility), then its stiffness will be large compared to the bending/shear stiffness of the gravity columns, which this provision is primarily intended to protect.

The second method allows for the base diaphragm and soil stiffness to be incorporated into the mathematical model of the superstructure to explicitly account for this redistribution.

C7.2.11 Diaphragms, Chords, Collectors, and Ties Diaphragms transfer inertial forces from the floors to the vertical elements of the seismic-force-resisting system. Diaphragms consist of the floor slab or sheathing and chord, collector, and tie elements. It is important to consider all portions of the diaphragm in an evaluation and the connection of the diaphragm to the vertical elements of the seismic-force-resisting system. The concept of a diaphragm chord, consisting of an edge member provided to resist diaphragm flexural stresses through direct axial tension or compression, is not familiar to many engineers. Buildings with solid structural walls on all sides often do not require diaphragm chords. However, buildings with highly perforated perimeter walls do require these components for proper diaphragm behavior. This section of this standard requires that these components be provided where appropriate.

A common problem in buildings that nominally have robust seismic-force-resisting systems is a lack of adequate attachment between the diaphragms and the vertical elements of the seismic-force-resisting system to effect shear transfer. This lack of shear transfer is particularly a problem in buildings that have discrete walls or frames as their vertical seismic-force-resisting elements and do not have collector elements to deliver diaphragm forces to the vertical elements. This section provides a reminder that it is necessary to detail a formal system of force delivery from the diaphragm to the walls and frames.

Diaphragms that support heavy perimeter walls have occasionally failed because of tension induced by out-of-plane forces generated in the walls. This section is intended to ensure that sufficient tensile ties are provided across diaphragms to prevent such failures. The force for these tensile ties, taken as $0.4S_{XS}$ times the weight, is an extension of provisions contained in the 1994 *Uniform Building Code* (ICBO 1994). In that code, parts and portions of structures are designed for a force calculated as C_p/Z times the weight of the component, where typical values of C_p are 0.75, and Z is the effective peak ground acceleration for which the building is designed. The 1994 *Uniform Building Code* provisions (ICBO 1994) use an allowable stress basis. This standard uses a strength basis. Therefore, a factor of 1.4 was applied to the C_p value, and a factor of 1/(2.5) was applied to adjust the Z value to an equivalent S_{XS} value, resulting in a coefficient of 0.4.

For flexible diaphragms, evaluation of diaphragm demands should be based on the likely distribution of horizontal inertial forces. For flexible diaphragms, such a distribution may be given by Equation (C7-1) and is illustrated in Figure C7-1.

$$f_d = \frac{1.5F_d}{L_d}\left[1 - \left(\frac{2x}{L_d}\right)^2\right] \qquad \text{(C7-1)}$$

where

f_d = Inertial load per foot;
F_d = Total inertial force on a flexible diaphragm;
x = Distance from the centerline of the flexible diaphragm, in feet; and
L_d = Distance between lateral support points for the diaphragm, in feet.

C7.2.12 Continuity A continuous structural system with adequately interconnected elements is one of the most important prerequisites for acceptable seismic performance. The requirements of this section are similar to parallel provisions contained in ASCE 7 and FEMA P-750 (2009b).

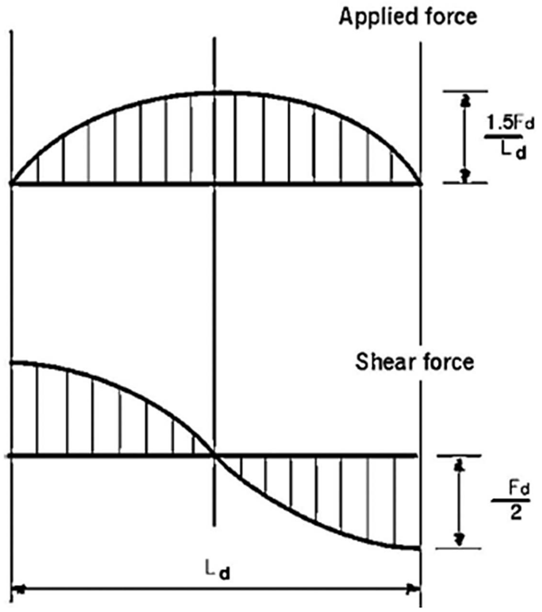


Figure C7-1. Plausible force distribution in a flexible diaphragm.

C7.2.13 Structural Walls and Their Anchorage

C7.2.13.2 Out-of-Plane Strength of Walls Application of these requirements for unreinforced masonry walls and infills is further defined in Chapter 11.

C7.2.15 Building Separation

C7.2.15.2 Separation Exceptions This standard permits retrofitted buildings to experience pounding as long as the effects are adequately considered by analysis methods that account for the transfer of momentum and energy between the structures as they impact.

Approximate methods of accounting for these effects can be obtained by performing nonlinear response history analyses of both structures (Johnson et al. 1992). Approximate elastic methods for evaluating these effects have also been developed and are presented in the literature (Kasai et al. 1990).

Buildings that are likely to experience significant pounding should not be considered capable of meeting Enhanced Performance Objectives. This limit is so because significant local crushing of components is likely to occur at points of impact. Furthermore, the nature of the impact is such that high-frequency shocks can be transmitted through the structures and can potentially be damaging to architectural components and mechanical and electrical systems. Such damage is not consistent with the performance expected of buildings evaluated or retrofitted to meet Enhanced Performance Objectives.

C7.2.16 Verification of Analysis Assumptions It is important that assumptions about locations of potential inelastic activity in the structure be verified. In linear procedures, the potential for inelastic flexural action is restricted to the beam ends because flexural yielding along the span length can lead to unconservative results. In nonlinear procedures, potential inelastic activity should occur only where specifically modeled. Where demands caused by gravity load combinations of Section 7.2.3 exceed 50% of the capacity of the component at any location along its length, the potential for inelastic activity exists and should be investigated. Sample procedures for verifying analysis assumptions are contained in Section C3.2.9 of FEMA 274 (1997b).

C7.3 ANALYSIS PROCEDURE SELECTION

Static procedures are appropriate where higher mode effects are not significant. This is generally true for short, regular buildings. Dynamic procedures are required for tall buildings and for buildings with torsional irregularities or nonorthogonal systems.

The NSP is acceptable for most buildings but should be used in conjunction with the LDP if mass participation in the first mode is low.

The term *linear* in linear analysis procedures implies "linearly elastic." The analysis procedure, however, may include geometric nonlinearity of gravity loads acting through lateral displacements and implicit material nonlinearity of concrete and masonry components using properties of cracked sections. The term *nonlinear* in nonlinear analysis procedures implies explicit material nonlinearity or inelastic material response, but geometric nonlinearity may also be included.

The linear procedures maintain the traditional use of a linear stress–strain relationship but incorporate adjustments to overall building deformations and material acceptance criteria to permit better consideration of the probable nonlinear characteristics of seismic response. The nonlinear static procedure (NSP), often called "pushover analysis," uses simplified nonlinear techniques to estimate seismic structural deformations. The nonlinear dynamic procedure (NDP), also known as nonlinear response history analysis, requires considerable judgment and experience to perform, as described in Section C7.3.2.2.

C7.3.1 Linear Procedures The results of the linear procedures can be very inaccurate when applied to buildings with highly irregular structural systems, unless the building is capable of responding to the selected Seismic Hazard Level in a nearly elastic manner. The procedures of Section 7.3.1.1 are intended to evaluate whether the building is capable of nearly elastic response.

C7.3.1.1 Method to Determine Limitations on Use of Linear Procedures The magnitude and distribution of inelastic demands are indicated by demand–capacity ratios (DCRs). These DCRs represent local ductility demands on component actions. A provision to limit the use of linear procedures appeared first in FEMA 273 (FEMA 1997c) with commentary in FEMA 274 (FEMA 1997b). FEMA 274 noted that

> *Linear procedures, while easy to apply to most structures, are most applicable to buildings that actually have sufficient strength to remain nearly elastic when subjected to design earthquake demands, and buildings with regular geometries and distributions of stiffness and mass.... Buildings that have relatively limited inelastic demands under a design earthquake may be evaluated with sufficient accuracy by linear procedures, regardless of their configuration. If the largest component DCR calculated for a structure does not exceed 2.0, the structure may be deemed to fall into this category.*

Linear procedures were not permitted when the DCR for a component action exceeded 2.0 and any of the following irregularities were present: in-plane discontinuity (unless it was checked as force controlled with $J = 1.0$), out-of-plane discontinuity (unless it was checked as force controlled with $J = 1.0$), severe weak story irregularity, or a severe torsional strength irregularity.

Revisions were made in FEMA 356 (FEMA 2000) and then in ASCE/SEI 41-06, Supplement 1 (ASCE 2007), and again in ASCE/SEI 41-13; no additional changes were made in ASCE/SEI 41-17 (FEMA 2018b). The revisions made the limitation provision less restrictive in some cases as the triggering irregularities were reduced to only the weak story irregularity and torsional strength irregularity, and the DCR was revised to trigger when the DCR for a component action exceeded the lesser of 3.0 and the m-factor for the component action. In a building with a weak story irregularity or a torsional strength irregularity, the deflected shape of the structure when analyzed using linear procedures may not represent the actual deflected shape of the structure accurately, and as a result the local demands used to check acceptance may not capture actual demands.

The linear procedure limitation provision was placed in the standard based on engineering judgment, but a recent study documented in FEMA validates the provision. The study included case study evaluations of a set of alternative options to the linear procedure limitation provision and did not find a clear basis for modifying the provision for the buildings studied. However, the study did recommend (1) exempting wood frame, cold-formed steel light-frame, and URM buildings meeting the requirements of the Chapter 16 special procedure; (2) clarifying that a nonlinear analysis is not required when a linear evaluation has shown the structure to be inadequate; and (3) making revisions to the commentary. The broad findings of the study, which apply to linear procedures in general, included the following:

- Consistency between the results of linear procedures and nonlinear procedures varies because of many reasons in addition to ductility demands and irregularities, such as component modeling and acceptance criteria, boundary conditions at the foundation, the Seismic Hazard Level, and the characteristics of ground motions used in nonlinear response history modeling.
- When the analysis does not meet the limitation provisions, and linear procedures show the building does not meet the performance objective, there is no need to require a further nonlinear analysis to prove the point. The engineer may perform a nonlinear analysis if the building was close to meeting the performance objective (as the linear procedures are often conservative), but it is not required. If a retrofit is considered, the distribution of demands obtained from the linear procedures cannot be relied on, and a nonlinear analysis may be required if a more accurate assessment of the location and extent of elements with inadequate capacities is desired, unless the irregularities are corrected.
- There are often relatively narrow bounds between triggering the linear procedure limitation provision and not failing the linear procedure Acceptance Ratios (as defined in Section 7.5.2.2). When the portion of the limitation provision requiring the DCR to be greater than the lesser of 3.0 and the m-factor of the component action is met, the limitation is triggered, and linear procedures are not permitted, then the building will often have already failed the linear evaluation because of Acceptance Ratios that exceed 1.0. Thus, even though the linear procedure is not permitted theoretically, it does not matter from a practical point of view. The engineer could reasonably conclude that a building does not meet the performance objective with a linear procedure evaluation. A nonlinear procedure evaluation could show the building does or does not meet the performance objective should the engineer choose to pursue such a more detailed evaluation.
- High DCRs in linear procedure results for buildings that meet the limitation requirements for linear analysis procedures still highlight areas of high ductility demands and areas where further investigation and scrutiny could be desirable.
- The definition of "weak story" in Section 7.3.1.1.3 of the standard is when the ratio of the average shear DCR for elements in any story to the average DCR of an adjacent story exceeds 1.25. In comparison, in Standard ASCE/SEI 7-16 (ASCE 2017), the weak story definition is where the "story lateral strength is less than 80% of the story above," and the extreme weak story is where "the story lateral strength is less than 65% of the story above." Thus, the standard compares DCRs, but ASCE/SEI 7-16 compares only strength. Studies show that the definition used by the standard is more conservative but is often more appropriate for existing buildings that may not have sufficient strength and leads to results that are more consistent with nonlinear analysis findings.
- Wood frame, cold-formed steel light-frame, and URM buildings consistent with the Chapter 16 special procedure were exempted from the linear procedure limitation provision in the revisions for this edition of the standard. Both new and existing buildings of these types have traditionally been analyzed using linear procedures. Based on engineering judgment, it is believed that linear procedures are adequate to evaluate these building types, even those with weak story irregularities, and high ductility demands, and it is considered unnecessary to require nonlinear analysis for these building types.

For URM buildings, incidental concrete walls are allowed where the structure consists primarily of URM, but there may be concrete walls at the basement level, in limited lengths as part of a structural retrofit or as small infills in the original URM. The building behavior should be dominated by the flexible diaphragms and the URM walls.

When calculating DCRs for checking the linear procedure limitations for the weak story and torsional irregularities, the diaphragm DCRs are excluded from the calculations because these equations were developed based on behavior of the vertical seismic-force-resisting system, not the behavior of the diaphragms. If the linear procedure is permitted, the diaphragms are still checked according to the linear procedure.

C7.3.1.2 Limitations on Use of the Linear Static Procedure For buildings that have long periods, major setbacks, torsional or vertical stiffness irregularities, or nonorthogonal seismic-force-resisting systems, the distribution of demands predicted by an LDP analysis are more accurate than those predicted by the LSP. Either the response spectrum method or response history method may be used for evaluation of such structures.

C7.3.2 Nonlinear Procedures

C7.3.2.1 Nonlinear Static Procedure The NSP is generally a more reliable approach to characterizing the performance of a structure than are linear procedures. However, it is not exact and cannot accurately account for changes in dynamic response as the structure degrades in stiffness; nor can it account for higher mode effects in multiple-degree-of-freedom (MDOF) systems. Where the NSP is used on a structure that has significant higher mode response, the LDP is also used to verify the adequacy of the evaluation or retrofit. Where this approach is taken, less-restrictive criteria are permitted for the LDP because it is recognized that improved knowledge is obtained by performing both analysis procedures.

The strength ratio, μ_{strength}, is a measure of the extent of nonlinearity, and $\mu_{\max}$ is a measure of the system degradation. Structures that experience nonlinear demands exceeding $\mu_{\max}$ have significant degradation, and an NDP is required to confirm the dynamic stability of the building.t

C7.3.2.2 Nonlinear Dynamic Procedure The nonlinear dynamic procedure (NDP) consists of nonlinear response history analysis, a sophisticated approach to examining the inelastic demands produced on a structure by a specific suite of ground motion acceleration histories. As with the NSP, the results of the NDP can be directly compared with test data on the behavior of representative structural components to identify the structure's probable performance when subjected to a specific ground motion. Potentially, the NDP can be more accurate than the NSP in that it avoids some of the approximations made in the more simplified analysis. Response history analysis automatically accounts for higher mode effects and shifts in inertial load patterns as structural softening occurs. In addition, for a given earthquake record, this approach directly solves for the maximum global displacement demand produced by the earthquake on the structure, eliminating the need to estimate this demand based on general relationships.

Despite these advantages, the NDP requires considerable judgment and experience to perform. These analyses can be highly sensitive to small changes in assumptions with regard to either the character of the ground motion record used in the analysis or the nonlinear stiffness behavior of the elements. As an example, two ground motion records enveloped by the same response spectrum can produce radically different results with regard to the distribution and amount of inelasticity predicted in the structure. To apply this approach reliably to evaluation or retrofit, it is necessary to perform a number of such analyses, using varied assumptions. The sensitivity of the analysis results to the assumptions incorporated is the principal reason why this method should be used only on projects where the engineer is thoroughly familiar with nonlinear dynamic analysis techniques and limitations.

C7.4 ANALYSIS PROCEDURES

C7.4.1 Linear Static Procedure

C7.4.1.1 Basis of the Procedure The magnitude of the pseudo seismic force has been selected with the intention that, when applied to the linearly elastic model of the building, it results in displacement amplitudes approximating maximum displacements expected during the selected Seismic Hazard Level. The procedure is keyed to the displacement response of the building because displacements are a better indicator of damage in the nonlinear range of building response than are forces. In this range, relatively small changes in force demand correspond to large changes in displacement demand. If the building responds essentially elastically to the selected Seismic Hazard Level, the calculated internal forces are reasonable approximations of those expected during the selected Seismic Hazard Level. If the building responds inelastically to the selected Seismic Hazard Level, as is commonly the case, the actual internal forces that would develop in the building are less than the internal forces calculated using a pseudo seismic force.

Calculated internal forces typically exceed those that the building can develop because of anticipated inelastic response of components. These forces are evaluated through the acceptance criteria of Section 7.5.2, which include modification factors and alternative analysis procedures to account for anticipated inelastic response demands and capacities.

C7.4.1.2 Period Determination for Linear Static Procedure

C7.4.1.2.1 Method 1: Analytical For many buildings, including multistory buildings with well-defined framing systems, the preferred approach to obtaining the period for analysis is Method 1. By this method, the building is modeled using the modeling procedures of Chapters 8 through 13, and the period is obtained by eigenvalue analysis. The effective stiffnesses, not gross section properties, of components should be used for period determination. Flexible diaphragms may be modeled as a series of lumped masses and diaphragm finite elements.

Contrary to procedures in codes for new buildings, there is no maximum limit on period calculated using Method 1. This omission is intended to encourage the use of more advanced analyses. It is felt that sufficient controls on analyses and acceptance criteria are present within this standard to provide appropriately conservative results using calculated periods.

C7.4.1.2.2 Method 2: Empirical Empirical equations for period, such as that used in Method 2, intentionally underestimate the actual period and generally result in conservative estimates of pseudo seismic force. Studies have shown that, depending on actual mass or stiffness distributions in a building, the results of Method 2 may differ significantly from those of Method 1.

C7.4.1.2.3 Method 3: Approximate Rayleigh's method for approximating the fundamental period of vibration of a building is presented in Equation (C7-2), which uses the shape function

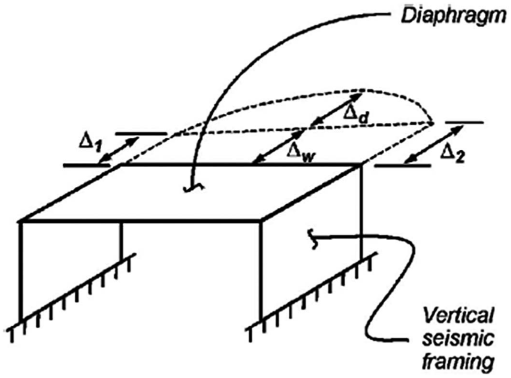


Figure C7-2. Diaphragm and wall displacement terminology.

given by the static deflections of each floor caused by the applied lateral forces:

$$T = 2\pi \left[\frac{\sum_{i=1}^{n} w_i \delta_i^2}{g \sum_{i=1}^{n} F_i \delta_i} \right]^{1/2} \qquad \text{(C7-2)}$$

where

w_i = Portion of the effective seismic weight located on or assigned to level i,
δ_i = Displacement at floor i caused by lateral force F_i,
F_i = Lateral force applied at level i, and
n = Total number of stories in the vertical seismic framing above the base.

Equations (7-19) and (7-20) of Method 3 are appropriate for systems with rigid vertical elements and flexible diaphragms in which the dynamic response of the system is concentrated in the diaphragm. Use of Method 2 on these systems to calculate the period based on the stiffness of the vertical elements substantially underestimates the period of actual dynamic response and overestimates the pseudo seismic force.

Equation (7-20) is a special case developed specifically for unreinforced masonry (URM) buildings. In this method, wall deformations are assumed to be negligible compared with diaphragm deflections. Wilson et al. (2013) used analytical methods to validate Equation (7-20) for flexible wood diaphragms with straight sheathing that can be considered for period determination of URM buildings. Such diaphragms are assumed to be idealized as shear dominated for flexibility and subject to a parabolic inertial load distribution by Equation (C7-1) and Figure C7-1. Flexible diaphragms with stiffer shear and/or flexural properties have shorter period estimates using Equation (7-20) and therefore result in more conservative pseudo seismic forces per Section 7.4.1.3. Equation (7-19) is a variation of Equation (7-20) specifically for one-story buildings.

For illustration of wall and diaphragm displacements, see Figure C7-2. Where calculating diaphragm displacements for the purpose of estimating period using Equation (7-19) or (7-20), the diaphragm is considered to remain elastic under the prescribed lateral forces.

C7.4.1.3 Determination of Forces and Deformations for Linear Static Procedure

C7.4.1.3.1 Pseudo Seismic Force for Linear Static Procedure

Coefficient C_1. This modification factor is used to account for the difference in maximum elastic and inelastic displacement amplitudes in structures with relatively stable and full hysteretic loops. The values of the coefficient are based on analytical and experimental investigations of the earthquake response of yielding structures. The quantity μ_{strength} is the ratio of the required elastic strength to the yield strength of the structure.

The alternative expression for μ_{strength} is obtained by substituting Equation (7-17) into Equation (7-32) and assuming that the elastic base shear capacity (fully yielded strength, V_y) is mobilized at a shear that is 1.5 times the shear at first yield (as indicated by the largest primary component DCR). The latter assumption is based on representative values for system overstrength. As is indicated in Figure C12.1-1 of FEMA P-750 (2009b), the factor relating force level to fully yielded strength is Ω_0. Sources of overstrength are design ϕ factors, expected material properties in excess of nominal material properties, and global system response. Because this standard prescribes use of $\phi = 1$ and expected material properties, the only additional source of overstrength is global system response. Using representative values for these contributions to overstrength ($\Omega_0 = 2.5$, $\phi = 0.75$, and a ratio of expected to nominal of 1.25), the factor relating shear at first yield to elastic base shear capacity is 1.5. Additional commentary regarding this coefficient is provided in C7.4.3.3.2.

C_1 and C_2 were derived in the FEMA 440 (2005) research for the nonlinear static procedure. That procedure uses the effective fundamental period rather than the elastic fundamental period, as shown in Equation 7-28 and Figure 7-3. The coefficient C_k adjusts the period from the elastic fundamental period to the effective fundamental period and is influenced by building type, number of stories, and overall behavior. The value of 1.1 is considered to be conservative based on engineering judgment from past nonlinear static procedure projects.

Coefficient C_2. This coefficient adjusts pseudolateral force values based on component hysteresis characteristics, cyclic stiffness degradation, and strength deterioration. For buildings with systems that do not exhibit degradation of stiffness and/or strength, the C_2 coefficient can be assumed to be 1.0. This situation would include buildings with modern concrete or steel special moment-resisting frames, steel eccentrically braced frames, and buckling-restrained braced frames as either the original system or the system added during seismic retrofit. See Section C7.4.3.3.2, FEMA 274 (1997b), and FEMA 440 (2005) for additional discussion.

Simplified C_1C_2 Table. As an alternative to the iterative process of calculating DCR, C_1, and C_2, Table 7-3 is provided. The table is based on the equations for C_1 and C_2, assuming Site Class D. The intent of the table is to provide a simplified way to select an appropriate C_1C_2 based on the building's period and the expected ductility demand based on the maximum m-factor that is permitted for all the primary seismic-force-resisting system elements.

Coefficient C_m. The effective mass factor was developed to reduce the conservatism of the LSP for buildings where higher mode mass participation reduces seismic forces up to 20% depending on building type. See FEMA 357, Appendix E (2000a), for more information on the development of C_m.

Response spectrum acceleration S_a. With the change from a two-period spectrum to the multipoint spectrum (see Section 2.4 and Section C2.4), the definition of S_a has been revised. This revision matches the updated equivalent lateral force procedure of ASCE 7. The seismic response coefficient is computed from the design spectral acceleration S_a for the period of the structure, except at very short periods, 90% of the maximum S_a is used (Figure C7-3). This limit at short periods is out of concern that a small change in period on the ascending side of the spectrum

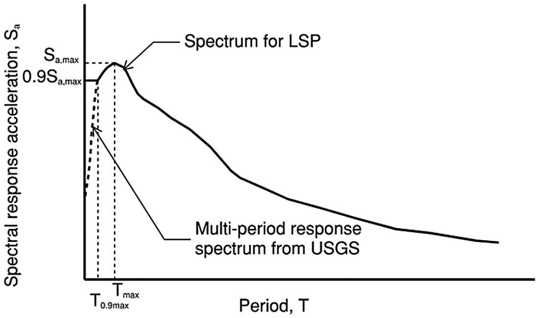


Figure C7-3. LSP horizontal response spectrum.

could be unconservative if the actual period is slightly larger, for instance if there is a difference between the estimated building mass and the actual mass, soil–structure interaction, or if damage softens the building's response. Maintaining a plateau at lower periods is also consistent with the 2017 and earlier standard editions that did not allow the ascending branch from $T = 0$ to $T = T_0$ for the linear static procedure. The 90% limit, rather than 100% of $S_{a,\max}$, recognizes that there is inelastic behavior that effective damping would increase, which would reduce the spectral acceleration.

The two-period spectrum of the exception in Section 2.4 is included only if the multipoint spectrum is not available at the site location per the reference in Section 2.4.1 and a site-specific response spectrum has not been developed.

C7.4.1.3.4 Diaphragms for Linear Static Procedure Diaphragms need to be evaluated for the forces from the analysis. The diaphragm chords, collectors, ties, and connections to the vertical seismic-force-resisting elements are considered part of the diaphragm and should be evaluated for the analysis forces in addition to the forces in Section 7.2.11. Section 7.2.11 specified design forces that are independent of analysis procedure used. The diaphragms still need to be evaluated for the forces from the evaluation.

The diaphragm inertial forces determined by Equation (7-27) will often be larger than the forces determined by Equation (7-25) to proportion the vertical elements of the seismic-force-resisting system. This is because Equation (7-27) is intended to estimate the inertial forces in the diaphragm owing to the maximum acceleration at a given floor. It is unlikely that every floor in the building will experience its maximum acceleration at the same time in an earthquake, which is why forces from Equation (7-27) need only be applied to the floor whose diaphragm is being evaluated. Diaphragm forces from Equation (7-27) should be distributed over the diaphragm based on the diaphragm's relative stiffness to the vertical force-resisting elements when the diaphragm is modeled as stiff or rigid. Further information on force distribution in flexible diaphragms is given in Section C7.2.11.

Diaphragm collectors and connections between the diaphragm and the vertical elements of the seismic-force-resisting system should be evaluated for the larger of the diaphragm inertial forces from Equation (7-27) plus any pseudo seismic forces from discontinuous elements and the difference in forces in the vertical elements above and below the diaphragm based on the pseudo seismic forces. Equation (7-27) estimates the inertial forces in the diaphragm based on the internal forces in the diaphragm caused by the floor acceleration. Diaphragm inertial forces from Equation (7-27) do not account for additional load transferred through the diaphragm owing to discontinuous elements or changes in stiffness of the vertical elements, so this must be explicitly added to the diaphragm inertial forces in the diaphragm from Equation (7-27). Alternatively, the forces that the diaphragm should deliver to the vertical elements, whether through direct connection to the diaphragm, collector elements, or a combination of the two, can be determined by taking the difference in the pseudo seismic forces in the vertical elements, such as a wall or moment frame, below the diaphragm and above the diaphragm due to the LSP.

Chapters 9 through 12 provide requirements to determine whether diaphragm components should be treated as force or deformation controlled. Diaphragms transferring forces from discontinuous systems or requiring transfer of forces from continuous vertical elements to vertical elements that only exist below the diaphragm are treated as force controlled, regardless of whether Chapters 9 through 12 would permit them to be treated as deformation controlled. This is done to recognize that linear analysis is unlikely to capture changes in forces to the vertical elements of the seismic-force-resisting system and the transfer diaphragms themselves if the transfer diaphragms yield.

C7.4.1.3.5 Distribution of Seismic Forces for Unreinforced Masonry Buildings with Flexible Diaphragms for Linear Static Procedure These provisions are based on Chapter A1 of the 2012 International Existing Building Code (ICC 2012). See FEMA 357 (2000h), Appendix D, for more information.

C7.4.2 Linear Dynamic Procedure

C7.4.2.1 Basis of the Procedure Modal spectral analysis is carried out using linearly elastic response spectra that are not modified to account for anticipated nonlinear response. As with the LSP, it is expected that the LDP will produce displacements that approximate maximum displacements expected during the selected seismic hazard level but will produce internal forces that exceed those that would be obtained in a yielding building.

Calculated internal forces typically exceed those that the building can sustain because of anticipated inelastic response of components. These forces are evaluated through the acceptance criteria of Section 7.5.2, which include modification factors and alternative analysis procedures to account for anticipated inelastic response demands and capacities.

C7.4.2.2 Modeling and Analysis Considerations for Linear Dynamic Procedure With the change from a two-period spectrum to the multipoint spectrum (Section 2.4, and C2.4), there is a greater possibility that the periods of the fundamental modes in stiff structures may be less than the period at the peak spectral acceleration. Depending on the soil type and location, certain sites do not reach peak spectral acceleration until almost 1 s when using the multipoint response spectra available from USGS (e.g., see the BSE-2E response spectrum for Site Class E in Oakland). Therefore, the response spectrum is modified as shown in Figure C7-4 to create a plateau at short periods when this occurs. This limit at short periods is out of concern that a small change in period, while on the ascending side of the spectrum could be unconservative if the actual period is slightly larger. Factors that could influence and lengthen the actual period of the building, relative to an analytical period determined from a mathematic model, include overestimating the stiffness of the primary lateral-force-resisting system, damage that softens the building's response, or soil–structure interaction. The 90% limit, rather than 100% of $S_{a,\max}$, recognizes that there is inelastic behavior that effective damping would increase, which would reduce the spectral acceleration. In addition, it is consistent with ASCE 7-22, Section 21.4.

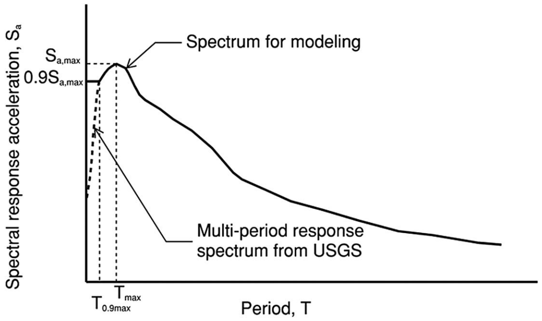


Figure C7-4. LDP horizontal response spectrum where fundamental period is less than $T_{0.9max}$.

While only adjusting the spectrum at the fundamental period is required, if the software does not permit modification of a single mode within the analysis, it is conservative to input the spectrum in the analysis program with the horizontal line at 90% of the maximum spectral acceleration as shown in Figure C7-4.

The two-period response spectrum of the exception in Section 2.4 is allowed only if the multipoint spectrum is not available at the site location per the reference in Section 2.4.1 and a site-specific response spectrum has not been developed.

When modeling the building elements, the stiffness of the elements should be carefully considered to most accurately represent the actual building behavior. Each material chapter contains recommended effective stiffness values for the majority of structural components. In addition, where required by Section 7.2.3.3, "Primary and Secondary Components," nonstructural component stiffnesses that can significantly affect the building behavior should be modeled. In particular, semirigid diaphragms with nonstructural toppings that are included with the mathematical model may need to account for the effect of the nonstructural topping as, in some cases, it can significantly increase the stiffness and lead to shorter periods with larger spectral accelerations. An alternative approach is to bound the stiffness in the analysis—once with the bare steel deck or bare wood diaphragm and once with the topping stiffness added.

C7.4.2.2.3 Response Spectrum Method for Linear Dynamic Procedure The LDP includes two analysis methods—namely, the response spectrum method and the response history method. The response spectrum method uses peak modal responses calculated from dynamic analysis of a mathematical model. Only those modes contributing significantly to the response need to be considered. Modal responses are combined using rational methods to estimate total building response quantities. The response history method involves a time-step by time-step evaluation of building response, using discretized recorded or synthetic earthquake records as base motion input. Pairs of ground motion records for simultaneous analysis along each horizontal axis of the building should be consistent. Consistent pairs are the orthogonal motions expected at a given site based on the same earthquake. Guidance for correlation between two sets of ground motion acceleration histories is provided in the US Nuclear Regulatory Commission Regulatory Guide 1.92 (USNRC 1976).

C7.4.2.3 Determination of Forces and Deformations for Linear Dynamic Procedure ASCE 41 does not require scaling the base shear obtained from a dynamic analysis to a percentage of the static base shear, unlike ASCE 7. This lack of a minimum base shear has been intentional since the first prestandard, FEMA 273. ASCE 41 uses a performance-based design approach and is not expecting the entire building to behave to the same ductility factor, while ASCE 7 is applying an *R*-factor to the entire structure. Each component in an ASCE 41 design is evaluated based on its individual ductility, which may lead to a higher design force than under a similar ASCE 7 design (if comparing designs that are both using an MCE_R seismic hazard) even without the base shear scaling. ASCE 7 scales base shear to provide a minimum system-level strength to provide a specific probability of collapse. The ASCE 41 linear analysis procedures are displacement-based, in which the pseudolateral forces are intended to push the structure to its maximum displacement. The pseudolateral forces from the LDP are modified by coefficients to approximate maximum displacements and therefore do not need to be further scaled to align with the LSP.

Per ASCE 7, Chapter C12, another reason for scaling is due to concerns of incorrect modeling. By correctly modeling the system, this issue can be corrected rather than requiring base shear scaling. Guidance on modeling different systems is given in the various material chapters of ASCE 41. It is also recommended that engineers verify that the model behavior makes sense for the given material types and nonstructural components.

C7.4.2.3.2 Diaphragms for LDP Diaphragms in the LDP are treated similar to diaphragms in the LSP. Refer to C7.4.1.3.4 for discussion of the requirements for evaluating diaphragms. There are two only major differences between the LDP and LSP for diaphragm evaluation. The first is the ability to use the story forces from the LDP in Equation (7-27) to determine the diaphragm inertial forces. The second is the ability to extract diaphragm inertial forces plus forces due to discontinuous vertical seismic-force-resisting elements or forces transferred through the diaphragm due to changes in stiffness of the vertical seismic-force-resisting elements directly from a three-dimensional LDP model where the diaphragms have been explicitly modeled and mass is distributed over the diaphragm at every floor. Forces in a stiff diaphragm are sensitive to how the diaphragm is modeled and how well the model is meshed. If the user chooses to model the diaphragm with elastic elements, they should consider the mesh and understand how forces can change based on mesh refinement.

C7.4.3 Nonlinear Static Procedure

C7.4.3.1 Basis of the Procedure The NSP is a sequential nonlinear procedure that tracks the global deformation of the building through a mathematical model which consists of elements with nonlinear force–displacement relationships representing the component action's behavior. Linear increasing forces are applied to the mathematical model, which allows for redistribution of forces as members reach their yield strength. The model tracks the post-yield deformation of the elements, which are then compared to acceptance criteria.

The target displacement is intended to represent the maximum displacement likely to be experienced for the selected Seismic Hazard Level. Because the mathematical model accounts directly for effects of material inelastic response, the calculated internal forces are reasonable approximations of those expected for the selected Seismic Hazard Level.

C7.4.3.2 Modeling and Analysis Considerations for Nonlinear Static Procedure The most important part of the NSP mathematical model is the nonlinear force–displacement relationships assigned to the deformation-controlled components. Section 7.5.1 describes the standard force–displacement relationships used in the standard. Those backbone curves are based on test data. Section 7.6 provides direction on how to establish these relationships. Chapters 8 through 12 present parameters to establish backbone curves for many common building components. However, there are inconsistencies in the parameters in Chapter 8 through 12 as a result of the continued development of the standard. Many of the parameters have not been updated since the standard's predecessor document FEMA 273. In FEMA 273, the backbone curves were based on second-cycle envelopes of hysteretic curves. Supplement 1 to ASCE 41-06 changed to base backbone curves on first-cycle envelopes. When that change was made, the legacy parameters were not updated. Additionally, there has been significant research since the publication of FEMA 273, which has resulted in major updates to some component actions, such as welded steel moment frame connections in FEMA 356 and concrete columns in ASCE 41-06, Supplement 1 and ASCE 41-17.

The provisions require that a valid range of modeling be established for every deformation-controlled component action. In many instances, test data will not be readily available to establish the valid range of modeling. The value that defines Point E, which is the maximum deformation specified in this standard, can be very conservative. There are very few component tests that push component subassemblages far enough into the strength degraded portion to establish a true Point E in Figure 7-9 relative to the amount of tests that displace components to Point C in Figure 7-9. In many cases, the parameter that defines Point E in Chapters 8 through 12 was extrapolated from limited testing or based on judgment. Therefore, the standard permits component actions to deform past Point E provided the strength and stiffness of the component action are degraded to a negligible value. Negligible value is specified instead of zero to recognize that degrading to zero has numerical instability issues in most commercial software. The value used should be significantly less than the strength at Point E.

C7.4.3.2.1 General Requirements for Nonlinear Static Procedure The requirement to carry out the analysis to at least 150% of the target displacement is meant to encourage the engineer to investigate likely building performance and behavior of the model under extreme load conditions that exceed the analysis values of the Seismic Hazard Level under consideration. The engineer should recognize that the target displacement represents a mean displacement value for the selected Seismic Hazard Level and that there is considerable scatter about the mean. Estimates of the target displacement may be unconservative for buildings with low strength compared with the elastic spectral demands.

The Simplified NSP of ASCE 41-06 is no longer included as an analysis option because it is often difficult to implement. Analysis using the Simplified NSP makes it difficult to properly satisfy the requirements of later ASCE 41 editions. Defining the force–deformation characteristics, primary versus secondary components, and the appropriate acceptance criteria is often challenging and potentially erroneous. The use of elastic-plastic backbone curves with the NSP of Section 7.3.2.1 should be permitted, with postprocessing to prove that the initial elastic-plastic assumption is appropriate.

When the strength degradation of components is not explicitly modeled, the $\mu_{\max}$ factor cannot be reliably estimated, and dynamic instability cannot be assessed beyond comparing component acceptance criteria with the corresponding demand. Elastic-plastic component action modeling of the Simplified NSP may miss potential failure mechanisms, particularly for taller buildings.

C7.4.3.2.2 Component Modeling for Nonlinear Static Procedure All points of potential inelastic action in any potential time or incremental displacement step should be captured explicitly in all modeled components or otherwise shown to have representative effects on load distribution and deformation demands on modeled structural components.

Nonlinear component actions should be modeled to capture prescribed force–deformation relationships in Chapters 8 through 12 or representative experimentally obtained component-level cyclic response. Fiber-type distributed plasticity elements using unidirectional material stress-deformation relationships may not appropriately capture component behavior by not including effects of shear-flexure interaction, reinforcement bar slip or buckling in concrete components, local buckling in steel components, or other local effects. Calibration of fiber models may be achieved through modification of the material stress-deformation relationships, discretization of fibers, integration lengths or mesh, hinge length definition over which nonlinear action is captured, or by combining with other elements in series or parallel. Effective stiffness, strain hardening, and critical deformations associated with component peak strength, significant lateral strength degradation, residual strength, total loss of lateral strength, and loss of gravity load resistance should be shown to be in agreement with component-level response using the modeling provisions of Chapters 8 through 12 or experimentally obtained cyclic behavior in accordance with Section 7.6.

Consideration of interactions with other modeled components and boundary conditions should be made with respect to fiber element models to represent calibrated component-level behavior. For example, rigid diaphragm constraints should not be placed among fiber element nodes because such constraints would restrict axial deformations that would otherwise be imposed in the fiber elements.

C7.4.3.2.4 Lateral Load Distribution for Nonlinear Static Procedure The distribution of lateral inertial forces determines relative magnitudes of shears, moments, and deformations within the structure. The actual distribution of these forces is expected to vary continuously during earthquake response as portions of the structure yield and stiffness characteristics change. The extremes of this distribution depend on the severity of the earthquake shaking and the degree of nonlinear response of the structure. More than one seismic force pattern has been used in the past as a way to bound the range of actions that may occur during actual dynamic response. Research in FEMA 440 (2005) has shown that multiple force patterns do little to improve the accuracy of nonlinear static procedures and that a single pattern based on the first-mode shape is recommended.

C7.4.3.2.5 Idealized Force–Displacement Curve for Nonlinear Static Procedure The idealized force–displacement curve is developed using an iterative graphical procedure to balance the areas below the actual and idealized curves up to Δ_d such that the idealized curve has the properties defined in this section. The definition of the idealized force–displacement curve was modified from the definition in FEMA 356 (2000b) based on the recommendations of FEMA 440 (2005).

C7.4.3.3 Determination of Forces, Displacements, and Deformations for Nonlinear Static Procedure

C7.4.3.3.2 Target Displacement for Nonlinear Static Procedure This standard presents the coefficient method for calculating target displacement. Other procedures can also be used. Section C3.3.3.3 of FEMA 274 (1997b) and FEMA 440 (2005) present additional background information on the coefficient method and another acceptable procedure referred to as the capacity spectrum method.

The C_0 coefficient accounts for the difference between the roof displacement of a MDOF building and the displacement of the equivalent single-degree-of-freedom (SDOF) system. Using only the first-mode shape (ϕ_1) and elastic behavior, coefficient C_0 is equal to

$$\begin{aligned} C_0 &= \phi_{1,r} \frac{\{\phi_1\}^T[M]\{1\}}{\{\phi_1\}^T[M]\{\phi_1\}} \\ &= \phi_{1,r}\Gamma_1 \end{aligned} \qquad \text{(C7-3)}$$

where

$\phi_{1,r}$ = Ordinate of mode shape 1 at the roof (control node),
$[M]$ = Diagonal mass matrix, and
Γ_1 = First modal mass participation factor.

Because the mass matrix is diagonal, Equation C7-3 can be rewritten as

$$C_0 = \phi_{1,r} \frac{\sum_1^N m_i \phi_{i,n}}{\sum_1^N m_i \phi_{i,n}^2} \qquad \text{(C7-4)}$$

where

m_i = Mass at level i, and
$\phi_{i,n}$ = Ordinate of mode shape i at level n.

If the absolute value of the roof (control node) ordinate of each mode shape is set equal to unity, the value of coefficient C_0 is equal to the first-mode mass participation factor.

Explicit calculation of C_0 using the actual deflected shape may be beneficial in terms of lower amplification of target displacement. The actual shape vector may take on any form, particularly because it is intended to simulate the time-varying deflection profile of the building responding inelastically to the ground motion and is likely to be different from the elastic first-mode shape. If this method is used, the mass participation factor, Γ_1, must be calculated using the actual deflected shape as the shape vector in lieu of the mode shape.

Use of the tabulated values, which are based on a straight-line vector with equal masses at each floor level, is approximate (particularly if masses vary much over the height of the building) and may be overly conservative.

Coefficients for estimating the target displacement have been modified based on the recommendations contained in FEMA 440 (2005).

FEMA 440 (2005) concluded that the previous cap on the C_1 factor was not appropriate, and a simplified equation was recommended based on μ_{strength}, effective period, T_e, and the site class factor, a, with a revised cap at $T = 0.2$ s. FEMA 440 (2005) recommended site class factors for Site Classes B, C, and D only. The site class factor for Site Class A was set equal to that for B, and the site class factor for Site Classes E and F was set equal to that for D. The use of the simplified C_1 equation to estimate displacements for soft-soil sites, including classes E and F, has higher uncertainty because of high dispersions of the results in studies of SDOF oscillators on soft soils. See FEMA 440 (2005) for more discussion on uncertainties related to the C_1 equation.

The C_2 factor was revised to better account for the effects of cyclic degradation of stiffness, as recommended in FEMA 440 (2005). For buildings with systems that do not exhibit degradation of stiffness and/or strength, the C_2 coefficient can be assumed to be 1.0. This assumption would include buildings with modern concrete or steel special moment-resisting frames, steel eccentrically braced frames, and buckling-restrained braced frames as either the original system or the system added during seismic retrofit.

The C_3 coefficient has been eliminated and replaced with a maximum strength ratio, $\mu_{\max}$, which is intended to measure dynamic instability. Where the value for $\mu_{\max}$ is exceeded, an NDP analysis is recommended to capture strength degradation and dynamic P-Δ effects to confirm dynamic stability of the building. As recommended in FEMA 440 (2005), the NDP analysis should include the in-cycle or cyclic strength or stiffness degradation in the hysteretic models of the components as required. The effective negative post-yield slope ratio, α_e, was introduced in FEMA 440 (2005) as a variable necessary to determine the maximum strength ratio, $\mu_{\max}$, that a building can have before dynamic instability is a concern. The negative slope caused by P-Δ effects, $\alpha_{\text{P-}\Delta}$, is based on the restoring force needed to balance the overturning moment caused by the weight of the structure displaced by an amount Δ, acting at the effective height of the first mode. It can be determined using structural analysis software by comparing the stiffness results of an analysis run with P-Δ effects to one run without P-Δ effects considered.

C7.4.3.3.4 Diaphragms for Nonlinear Static Procedure Diaphragms can be explicitly included in the NSP model with nonlinear force–deformation relationships to allow nonlinear behavior in the diaphragms. If this is done, the load pattern used to displace the structure must be applied to the diaphragms in addition to or instead of the vertical elements of the seismic-force-resisting system to ensure that the displacement demands in the diaphragms are properly captured. Nonlinear modeling of diaphragms is sensitive to the number of unique elements used to represent the diaphragm. The user is encouraged to investigate this and confirm that they have sufficiently meshed the diaphragm's linear and nonlinear elements to properly capture the diaphragm's nonlinear behavior.

If diaphragms are not explicitly modeled with nonlinear properties in the NSP, they are treated similarly to diaphragms in the LSP or are permitted to be evaluated separately using the LSP or LDP model. Refer to C7.4.1.3.4 for a discussion of the requirements for evaluating diaphragms. The provisions permit diaphragm forces to be estimated using Equation (7-27) with story forces from the NSP. Because the story forces change as the control node displacement changes, the provisions require Equation (7-27) be evaluated at the target displacement and at the displacement that produces the maximum force if that occurs before the target displacement. The diaphragm forces computed at the displacement that produces the largest base shear will likely approximate the maximum forces in the diaphragm. However, it is possible that forces in one or more diaphragms may be larger at a displacement after the one that produces the maximum base shear because of the way the vertical system elements have yielded and degraded. Therefore, the user should assess both cases when the target displacement is larger than the force that produces the maximum base shear. It may be possible that there is a displacement between the two points that produces a larger diaphragm force.

The provisions permit diaphragms classified as deformation-controlled components to be evaluated using an m-factor, provided the diaphragms are not transfer diaphragms. This hybrid approach of mixing linear analysis and nonlinear analysis is permitted for diaphragms in recognition that some limited nonlinearity in diaphragms spanning between vertical seismic-force-resisting elements may not change the overall behavior of the nonlinear analysis. This assumption may not be valid if the diaphragm is a transfer diaphragm or receives load from discontinuous seismic-force-resisting elements or if there is significant yielding in the diaphragm. The m-factor is capped at 2 to limit the amount of nonlinearity in the diaphragm. If the ductility demands in the diaphragm are greater than 2, the committee felt that it would be possible that yielding in the diaphragm could lead to a different building response than predicted with an elastic diaphragm or an idealized rigid diaphragm. The provisions also permit evaluating the diaphragms using a separate LSP or LDP model.

C7.4.4 Nonlinear Dynamic Procedure

C7.4.4.1 Basis of the Procedure The basis, modeling approaches, and acceptance criteria of the NDP are similar to those for the NSP. The main exception is that the response calculations are carried out using response history analysis. With the NDP, the displacements are not established using a target displacement but, instead, are determined directly through dynamic analysis using ground motion acceleration histories. Calculated response can be highly sensitive to characteristics of individual ground motions; therefore, the analysis should be carried out with more than one ground motion record. Because the numerical model accounts directly for effects of material inelastic response, the calculated internal forces are reasonable approximations of those expected for the selected Seismic Hazard Level.

C7.4.4.2.1 General Requirements for Nonlinear Dynamic Procedure The mathematical model for the NDP can use the same force–deformation relationships as are used in the NDP, but those relationships must also include . . . . The mathematical model of the component action shall result in reasonable agreement between the shape of the nominal and test hysteresis loop for each component type and the dissipated hysteretic energy. The modeled hysteresis should be checked against the measured hysteresis throughout the range of expected deformation demands.

C7.4.4.2.3 Nonlinear Response History Method for Nonlinear Dynamic Procedure Nonlinear modal response history (also called fast nonlinear analysis [FNA]) can be an efficient method to analyze structures that are predominantly linear elastic but have a limited number of predefined nonlinear link and/or support elements (Wilson 2010). The response of a structure using FNA depends on being able to adequately represent the nonlinear forces by the modal forces and requires the following special considerations:

1. Mass or mass moments of inertia should be present at all nonlinear degrees of freedom, and
2. The Ritz vector method should be used for the modal analysis.

An appropriate number of modes should be used in the modal analysis to represent adequately the nonlinear forces by modal forces. This representation can be accomplished by ensuring that the static modal load participation ratio of each nonlinear degree of freedom is 100%. An additional measure that can be used to determine the appropriate number of modes is the dynamic modal load participation ratio, but for many structures the ratio does not equal 100% because the method is not capturing the high-frequency response of each nonlinear degree of freedom, a result that may or may not affect the accuracy of the results. As a rule of thumb, the number of modes that should be calculated is equal to the nonlinear degrees of freedom multiplied by 2.5, a value that can be reduced if there are degrees of freedom that are constrained to each other.

The NDP FNA should follow from an appropriate FNA representing the response of the structure to gravity loads. This quasistatic FNA can be performed by applying the gravity load case as a ramp function while applying high modal damping.

The following criteria provide guidance on time step selection:

1. The analysis time step should not be greater than the step at which the ground motion acceleration histories are digitized.
2. The analysis time step should be less than or equal to $T/100$; T_{90}; and 0.01 s

where

T = Fundamental period of the building in the direction under consideration (judged by largest mass contribution), and

T_{90} = Period of the highest mode in the same direction as T to achieve 90% modal mass participation.

3. Use of a 50% smaller time step results in a difference in response of less than 10%.

Items 1 and 2 are based on NZS 1170.5:2004, Part 5: *Earthquake Actions—New Zealand* (SNZ 2004). For the direct-integration analysis method, selection of too large a time step can result in higher mode (short-period) responses not being captured or convergence to an incorrect solution, particularly for models exhibiting highly nonlinear characteristics. Guidance for correlation between sets of ground motion acceleration histories is provided in the US Nuclear Regulatory Commission Regulatory Guide 1.92 (USNRC 1976).

C7.4.4.2.4 Cyclic Response in Nonlinear Dynamic Procedure While enveloped in-cycle strength and stiffness degradation is captured sufficiently in the nonlinear static procedure using the modeling parameters prescribed in material chapters, the nonlinear dynamic procedure requires careful consideration of cycle-to-cycle response and representative energy dissipation. Representative hysteretic shapes capturing cycle-to-cycle stiffness degradation and related energy dissipation under load or deformation reversals are demonstrated in Figure C7-5. Low Pinching represents the behavior of components with low pinching such as material yielding actions in steel components or well detailed concrete columns subjected to low axial loads. Moderate Pinching represents the behavior of components with moderate pinching such as inelastic steel buckling modes or columns under moderate axial load. Significant Pinching represents the hysteretic behavior of components with significant pinching such as elastic steel buckling, a poorly detailed concrete column with high axial load, or the components sustaining sliding shear or splice failures. Rocking and elastically unloading systems will also display hysteretic shapes best represented by significant pinching. Figure C7-5 illustrates each of the four default states.

C7.4.4.2.5 Adaptive Models in NDP Adaptive force–deformation models can be used as a means to provide a better representation of the seismic response of the structure, but they require significant calibration. The model can adapt to mimic the difference between a monotonic loading and different types of cyclic loading. Section C7.6.1 discusses how significant loading protocol can be on the resulting force–deformation curve. Although many commercially available software packages do not currently have adaptive models, some of the more advanced finite-element analysis programs do. Section C7.6.5 discusses how adaptive force–deformation models differ from the force–deformation models specified in this standard.

C7.4.4.3 Determination of Forces and Deformations for Nonlinear Dynamic Procedure Where component response is a function of interacting actions, such as axial load and moment for a column or shear wall, response can be evaluated at the governing time step or by conservatively combining enveloped actions from each response history analysis, regardless of the time at which they occur.

Examples of component responses that are likely to be independent of the direction of action include shear about the same axis in a beam, column, or wall; plastic hinge rotation about the same axis in a symmetric shear wall or column; and building

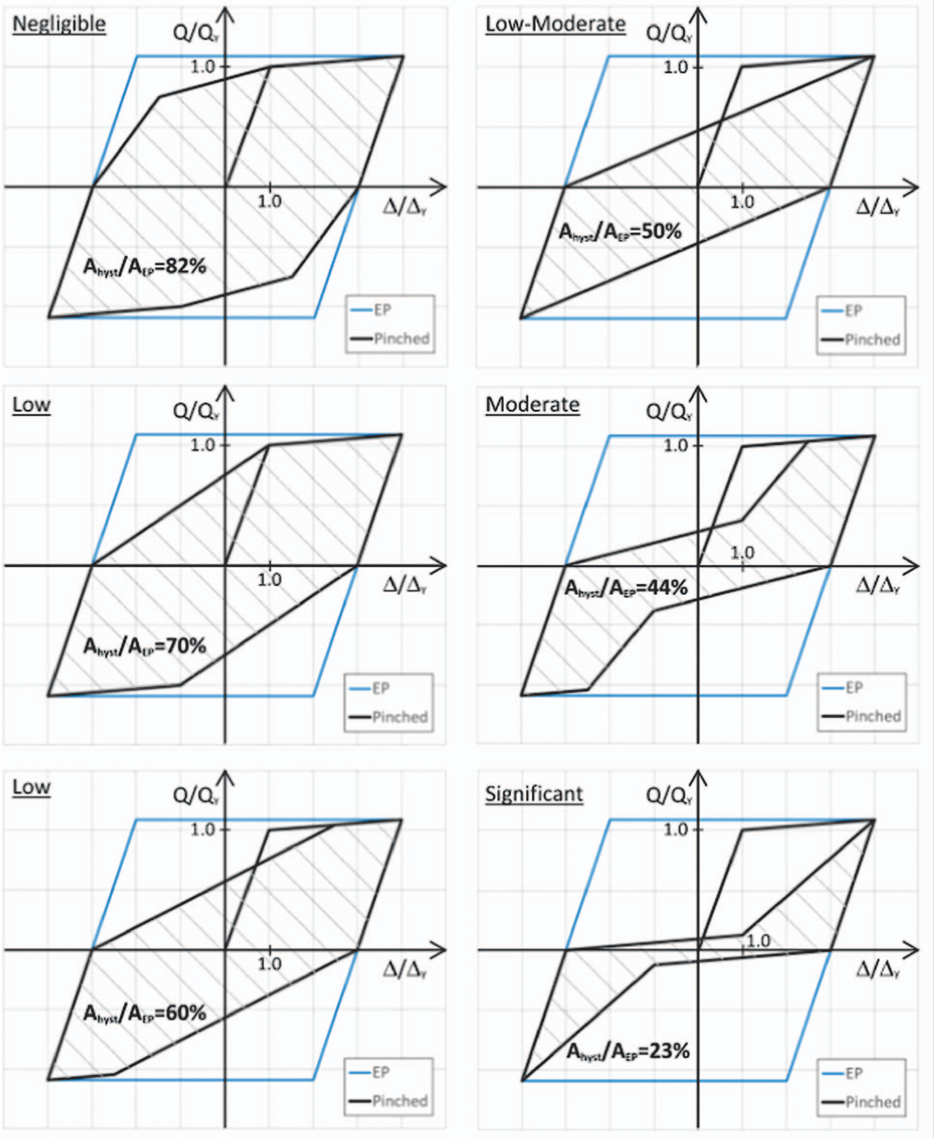


Figure C7-5. Inelastic hysteretic energy dissipated by representative inelastic actions under acceleration histories should be compared against other sources of analytical damping and characteristic values reported in literature to evaluate the adequacy of the modeled hysteretic behavior.

drifts as used for the evaluation and retrofit of nonstructural glazing systems or partitions.

For components that are sensitive to the direction of loading, forces and deformations should be determined such that the significance of positive and negative signed values is preserved during the processing of results. Examples of component responses that are likely to depend on the direction of action are axial tension versus compression in a column, positive and negative bending or plastic hinge rotation about the same axis in an asymmetrically reinforced concrete beam, plastic hinge rotation about the same axis in an asymmetric shear wall (e.g., L- or T-shaped), and relative displacement perpendicular to a building joint (pounding).

Section 7.4.4.3 outlines how averaged and maximum results should be determined from an NDP analysis. The removal of the option to use the maximum results from a suite of three ground motions, and deletion of ASCE 41-17, Table 7-1 Factor χ for Calculation of Out-of-Plane Wall Anchorage Forces, negates the need to define how maximum results should be determined from an NDP analysis.

C7.4.4.3.2 Diaphragms for Nonlinear Dynamic Procedure Diaphragms can be explicitly included in the NDP model with nonlinear force–deformation relationships to allow nonlinear behavior in the diaphragms. Nonlinear modeling of diaphragms is sensitive to the number of unique elements used to represent the diaphragm. The user is encouraged to investigate this and confirm that they have sufficiently meshed the diaphragm's linear and nonlinear elements to properly capture the diaphragm's nonlinear behavior. Mass should be assigned at every mesh point to properly capture the distribution of mass over the diaphragm and properly capture the forces in the diaphragm resulting from the floor accelerations from the ground motion records.

The provisions permit using the floor acceleration directly from the nonlinear analysis model to determine diaphragm forces. The provisions do not specify where on the diaphragm the acceleration should be computed. That is intentional because the point where the acceleration should be computed needs to be determined in the context of the diaphragm. Often it will be the center of mass of the diaphragm, but there are instances where the diaphragm span between vertical seismic force-resisting elements may not be at the center of mass and that is where it may be more appropriate to take the acceleration. Therefore, judgment is required to identify the point used to determine the floor acceleration.

If diaphragms are not explicitly modeled with nonlinear properties in the NDP, they are treated similar to diaphragms in the LDP or are permitted to be evaluated separately using and LSP or LDP model. Refer to C7.4.1.3.4 for a discussion of the requirements for evaluating diaphragms. It is also permitted to determine the diaphragm forces based on the change in forces in the vertical seismic-force-resisting elements.

The provisions permit diaphragms classified as deformation-controlled components to be evaluated using an *m*-factor, provided the diaphragms are not transfer diaphragms. The *m*-factor is capped at 2 in recognition of the potential inaccuracies of this hybrid nonlinear/linear procedure. If the ductility demands in the diaphragm are greater than 2, the committee felt that it would be possible that yielding in the diaphragm could lead to a different building response than predicted with an elastic diaphragm or an idealized rigid diaphragm. The provisions also permit evaluating the diaphragms using a separate LSP or LDP model.

C7.4.4.4 Damping for Nonlinear Dynamic Procedure Target damping ratios should be implemented considering both the expected linear elastic and nonlinear response of the structure to avoid overdamped solutions. If the period of the structure is expected to lengthen, then the damping ratio should also be limited to not greater than the target equivalent viscous damping ratio at long periods (e.g., 1.5*T* to 2.0*T*). Consistent with the ground motion scaling procedures in Chapter 2, the range of 0.2 times and 1.5 times the fundamental period is recommended for anchoring Rayleigh damping models at the target equivalent viscous damping ratio. Where equivalent viscous damping models are combined (e.g., Rayleigh damping used in conjunction with modal damping) for response history analysis, the equivalent viscous damping ratio should not exceed the target elastic equivalent viscous damping ratio specified in this section in the period range of 0.2 times and 1.5 times the fundamental period. The provisions for equivalent viscous damping methods are based on PEER/ATC-72-1, PEER TBIv2.03, Chopra and McKenna (2016a), and NZS 1170.5:2004 Part 5: *Earthquake actions—New Zealand* (SNZ 2004).

In the context of choosing a damping model to use in response history analysis, consideration should be given to plasticity models and viscous damping assumptions. Response of models using concentrated or lumped plasticity (zero length) elements (Figure C7-6) has been shown to be sensitive to damping model assumptions. Spurious damping forces have been observed in concentrated plasticity models when used in conjunction with initial-stiffness-proportional Rayleigh damping models. Such spurious damping forces can result in effective viscous damping exceeding the target damping, on the order of three times the yield moment of adjoining structural elements (Chopra and McKenna 2016b), which can in turn lead to an underestimation of dynamic response (Figure C7-7). Although the use of tangent-stiffness-proportional Rayleigh damping may substantially diminish such spurious damping forces, this approach is not recommended because it lacks physical basis and has difficult conceptual implications such as negative damping coefficients associated with the negative tangent stiffness of degrading components. Use of modal damping, which uses a damping matrix constructed by superposition of modal damping matrixes, may eliminate spurious damping forces. When such modal damping is used, damping ratios must be specified for all modes that are expected to contribute significantly to structural response (Chopra and McKenna 2016b). Response of distributed plasticity elements has been shown to be less sensitive to damping assumptions compared to concentrated plasticity elements. However, even such elements may become more sensitive to damping assumptions at deformation responses approaching collapse (Chopra and McKenna 2016b, Hall 2016). Some other methods to avoid spurious damping forces are condensing-out the degrees of freedom that would generate spurious damping forces (Bernal 1994, PEER/ATC 2010) or enforcing upper-bounds on the stiffness proportional damping terms (Hall 2005, Powell 2008, PEER/ATC 2010). The user is referred to PEER/ATC 72-1 (2010) for a more detailed discussion on spurious damping forces and methods to avoid them.

Figure C7-6. Beam with concentrated plasticity hinges.

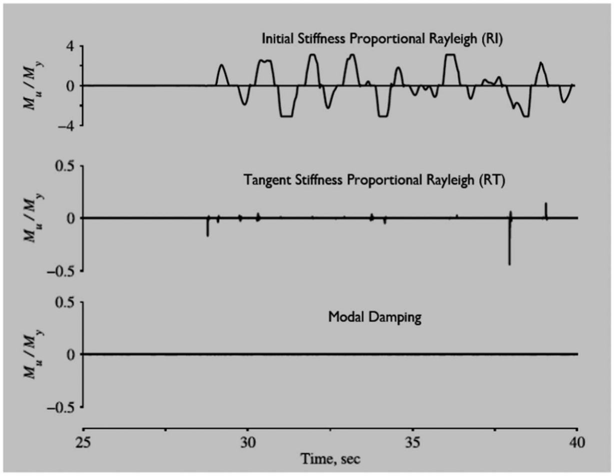


Figure C7-7. Example of spurious damping forces.

Use of the mass-proportional damping terms may lead to unrealistically large forces, and an underestimation of response, in structures with large rigid body motion. This effect may be significant in analyses of tall buildings, where drifts in the upper portions of the building are causecd, in part, by deformations that occur in lower levels of the building (Hall 2005, PEER/ATC 2010).

C7.5 ACCEPTANCE CRITERIA

C7.5.1 General Requirements The linear analysis procedures are intended to provide a conservative estimate of building response and performance for the selected Seismic Hazard Level. Because the actual response of buildings to earthquakes is typically nonlinear, nonlinear analysis procedures should provide more accurate representations of building response and performance. In recognition of the improved estimates of nonlinear analysis, the acceptance criteria for nonlinear procedures are more accurate and less conservative than those for linear procedures. Buildings that do not comply with the linear analysis acceptance criteria may comply with nonlinear acceptance criteria. Therefore, performing a nonlinear analysis is recommended to minimize or eliminate unnecessary seismic retrofit. Design professionals are encouraged to consider the limitations of linear procedures and to pursue nonlinear analyses where linear acceptance criteria are not met.

C7.5.1.1 Deformation-Controlled and Force-Controlled Actions Acceptance criteria for primary components that exhibit Type 1 behavior typically are within the elastic or plastic ranges between Points 0 and 2, depending on the performance level. Acceptance criteria for secondary components and all components in nonlinear analyses that exhibit Type 1 behavior can be within any of the performance ranges.

Acceptance criteria for primary and secondary components exhibiting Type 2 behavior are within the elastic or plastic ranges, depending on the performance level.

Acceptance criteria for primary and secondary components exhibiting Type 3 behavior are always within the elastic range.

Table C7-1 provides some examples of possible deformation- and force-controlled actions in common framing systems. Classifications of deformation- or force-controlled actions are specified for foundation and framing components in Chapters 8 through 12.

A given component may have a combination of both deformation- and force-controlled actions.

Classification as a deformation-controlled action is not up to the discretion of the user. Deformation-controlled actions have been defined in this standard by the designation of *m*-factors or nonlinear deformation capacities in Chapters 8 through 12. Where such values are not designated and component testing justifying Type 1 or 2 behavior is absent, actions are to be taken as force controlled. Any component action included in a nonlinear model as linear elastic without a nonlinear force–displacement relationship should be treated as a force-controlled action. There are specific provisions for nonlinear analyses when certain force-controlled actions may be reclassified as deformation controlled if their nonlinear force–deformation curve is explicitly included in the nonlinear model. When actions are included elastically in the nonlinear model, the model cannot adjust the response of the structure if demands on elastically modeled actions exceed their capacity and the results of the model may not properly capture the behavior of the structure.

Table C7-1. Examples of Possible Deformation-Controlled and Force-Controlled Actions.

Component	Deformation-Controlled Action	Force-Controlled Action
Moment Frames		
• Beams	Moment (*M*)	Shear (*V*)
• Columns	—	Axial load (*P*), *V*
• Joints	—	V^a
Shear walls	*M*, *V*	*P*
Braced Frames		
• Braces	*P*	—
• Beams	—	*P*
• Columns	—	*P*
• Shear link	*V*	*P*, *M*
Connections	*P*, *V*, M^b	*P*, *V*, *M*
Diaphragms	*M*, V^c	*P*, *V*, *M*

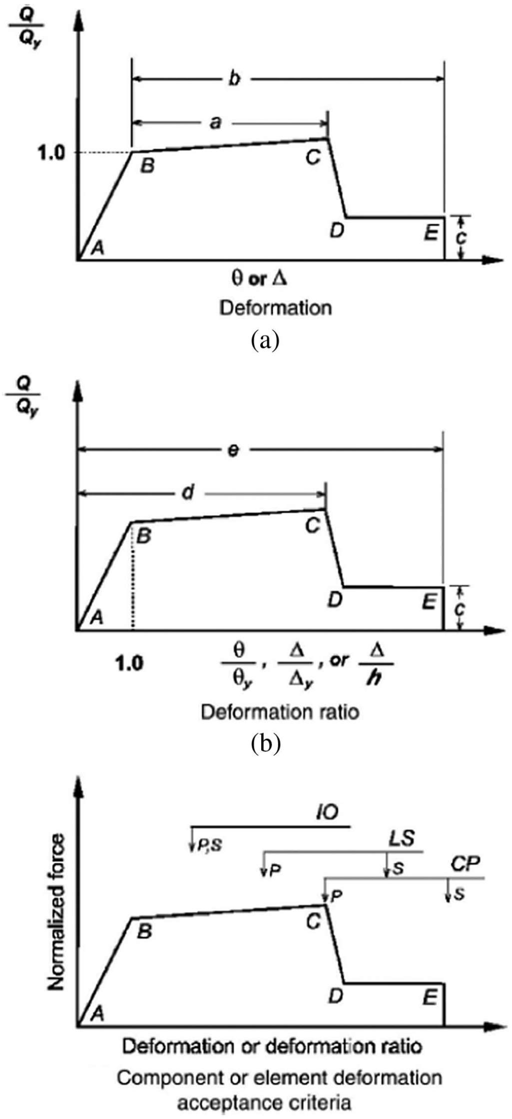
[a] Shear may be a deformation-controlled action in steel moment frame construction.

[b] Axial, shear, and moment may be deformation-controlled actions for certain steel and wood connections.

[c] If the diaphragm carries lateral loads from vertical seismic-force-resisting elements above the diaphragm level, then *M* and *V* shall be considered force-controlled actions.

Figure C7-8 shows the generalized force-versus-deformation curves used throughout this standard to specify element modeling and acceptance criteria for deformation-controlled actions in any of the four basic material types. Linear response is depicted between Point A (unloaded element) and an effective yield Point B. The slope from Point B to Point C is typically a small percentage (0% to 10%) of the elastic slope and is included to represent phenomena such as strain hardening. Point C has an ordinate that represents the strength of the element and an abscissa value equal to the deformation at which significant strength degradation begins (Line CD). Beyond Point D, the element responds with substantially reduced strength to Point E. At deformations greater than Point E, the element seismic strength is essentially zero.

The sharp transition as shown on idealized curves in Figure C7-8 between Points C and D can result in computational difficulty and an inability to converge where it is used as modeling input in



(a)

(b)

(c)

Figure C7-8. Generalized component force–deformation relations for depicting modeling and acceptance criteria.

nonlinear computerized analysis software. For some types of suddenly degrading components (e.g., pre-Northridge connection fracture), this is reflective of the observed component behavior. However, to avoid this computational instability, a small slope (e.g., 10 vertical to 1 horizontal) may be provided to the segment of these curves between Points C and D. Alternatively, the slope may be based on data from testing of comparable specimens. (e.g., for reinforced concrete components, it may be acceptable to connect Points 2 and 3 in Figure 7-5 for Type 1 components). See PEER/ATC 72-1 (2010) and NIST GCR 17-917-045 (NIST 2017) for additional guidance.

For some components, it is convenient to prescribe acceptance criteria in terms of deformation (such as θ or Δ), whereas for others it is more convenient to give criteria in terms of deformation ratios. To accommodate these different conventions, two types of idealized force-versus-deformation curves are used in Figure C7-8a, b. Figure C7-8a shows normalized force (Q/Q_y) versus deformation (θ or Δ) and the parameters *a*, *b*, and *c*. Figure C7-8b shows normalized force (Q/Q_y) versus deformation ratio (θ/θ_y, Δ/Δ_y, or Δ/h) and the parameters *d*, *e*, and *c*. Elastic stiffnesses and values for the parameters *a*, *b*, *c*, *d*, and *e* that can be used for modeling components are given in Chapters 8 through 12, 14, and 15. Acceptance criteria for deformation or deformation ratios for primary components (P) and secondary components (S) corresponding to the target Building Performance Levels of Collapse Prevention (CP), Life Safety (LS), and Immediate Occupancy (IO) as shown in Figure C7-8c are given in Chapters 8 through 12, 14, and 15.

For nonlinear procedures, it is permitted to allow some components that are force controlled to be reclassified as Type 3 deformation controlled. An example of this reclassification is failure of the welds that connect the brace to the gusset plate in a steel-braced frame system. In this case, acceptable performance may still be achieved provided that no gravity load collapse occurs and the remaining seismic-force-resisting system is adequate for the building to meet the selected performance level.

Eventually some critical portion of the gravity-load-resisting system governs the collapse limit for the building (e.g., column rotations, or gravity framing connection CP rotation limits). Nonlinear provisions are intended to make sure that when components fail, they are able to redistribute the forces to other structural elements in the building. In many buildings, almost all elements participate in the structure's seismic-force-resisting system. As the structure is subjected to increasing demands, some of these elements may begin to fail and lose strength much sooner than others. If a structure has sufficient redundancy, it may be permissible to allow failure of some of these elements, as long as this failure does not result in loss of gravity-load-carrying strength or overall stability.

It is also important to ensure that this type of reclassification is done to elements of the structure that do not support significant gravity loads because their failure could lead to loss of gravity load support and localized collapse. However, elements that support gravity load can be reclassified if it can be demonstrated that an alternate load path for the gravity load support is present and can be maintained at the maximum anticipated seismic displacement.

In these cases, it is important to consider the potential effect of overstrength on the system. For the braced frame example, a completely different mechanism may result if the brace welds are stronger than assumed. To capture this potential outcome, it is required that the analysis be repeated with stronger "yield" strength and all components be rechecked. The quantity Q_{CE}/Q_y is used as an approximate means to establish the upper-bound strength.

C7.5.1.2 *Critical and Noncritical Actions* The standard designates component actions as critical or noncritical based on the consequence of the component action's failure. When a component action's failure results in a disproportionate collapse involving either multiple bays or multiple stories, such as the loss of a column or a transfer girder, such component's action would be considered critical. What constitutes a multibay collapse is straightforward in most buildings but can be difficult in bearing wall buildings with irregularly spaced walls. In such cases, and for situations where bay spacings are abnormally large, the user should consider percentage of floor area that is tributary to the component under consideration to determine whether they would consider the loss of that component critical or not. If there is an alternate load path to resist gravity load after the component action fails, then the action can be classified as noncritical, because multiple bays or stories will not have collapsed. Additionally, if a component action's failure significantly changes the behavior of the lateral-force-resisting system by creating a torsional strength or weak story irregularity or by reducing the lateral force-resisting strength by 15% or more when compared to the original strength of the story, it is considered critical. Examples of this are brace connections where there is no redundancy in the braced frame at that story and the failure of that connection results in a torsional irregularity, a weak story, or a reduction in story strength of 15% or more.

C7.5.1.3 *Expected and Lower-Bound Strengths* In Figure 7-5, the strength of a component is affected by inherent variability of the strength of the materials making up the individual components and differences in work quality and physical condition. See Chapters 8 through 12, 14, and 15 for specific direction regarding the calculation of expected and lower-bound strengths of components.

C7.5.1.4 *Material Properties* Where calculations are used to determine expected or lower-bound strengths of components, expected or lower-bound material properties, respectively, shall be used.

C7.5.2 Linear Procedures

C7.5.2.1 Forces and Deformations

C7.5.2.1.1 Deformation-Controlled Actions for Linear Static Procedure or Linear Dynamic Procedure Because of possible anticipated nonlinear response of the structure, the actions as represented by Equation (7-36) may exceed the actual strength of the component to resist these actions. The acceptance criteria of Section 7.5.2.2.1 take this overload into account through use of a factor, *m*, that is an indirect measure of the nonlinear deformation capacity of the component.

C7.5.2.1.2 Force-Controlled Actions for Linear Static Procedure or Linear Dynamic Procedure The basic approach for calculating force-controlled actions for evaluation or retrofit differs from that used for deformation-controlled actions because nonlinear deformations associated with force-controlled actions are not permitted. Therefore, force demands for force-controlled actions should not exceed the force capacity (strength) of the deformation-controlled actions in the load path delivering load to or from the element with the force-controlled action under consideration.

Ideally, an inelastic mechanism for the structure is identified, and the force-controlled actions, Q_{UF}, are determined by limit analysis using that mechanism. This approach often produces a conservative estimate of the actions, even if an incorrect mechanism is selected. Where it is not possible to use limit (or plastic) analysis, or in cases where forces do not produce significant nonlinear response in the building, it is acceptable to determine the force-controlled actions for evaluation or retrofit using Equation (7-37) or (7-38).

Dividing by the minimum DCR is meant to reduce the demands on the force-controlled actions from the unreduced elastic pseudo seismic force to an approximation of the demands that would occur owing to yielding in the deformation-controlled actions elsewhere in the load path. If the LDP is used, the pseudo seismic force will already have accounted for potential reduction in forces caused by higher mode effects. The minimum DCR was selected over an average or the maximum because of the uncertainty in which deformation-controlled actions in the load path would be the ones to limit the force on the force-controlled action under consideration. Using an average DCR may underpredict the amount of force being delivered to the specific component under consideration. In some cases, several components in the load path may have high DCR, but the specific component adjacent to the component with the force-controlled action under consideration has a low DCR and that specific deformation-controlled action is the one that most affects the force-controlled action under consideration.

Often when the pseudo-seismic forces are applied perpendicular to vertical seismic-force-resisting-elements, there is some force that may be imparted to the elements. That force is often less than the component's yielding, meaning that $\mathrm{DCR}_{\min}$ would need to be taken as 1.0. When computing the DCRs, the intent is to only consider the DCR due to pseudo-seismic forces acting in the direction that matches the orientation of the vertical element of the seismic-force-resisting system the component action under consideration is part of.

Judgment is required when determining which components and which actions should be included in the load path delivering force to the component with the force-controlled action being evaluated. In a moment frame or braced frame structure, the majority of the elements in the stories above should be considered. The elements in the story below may be considered in lieu of the elements in the story above if there will be yielding in the story that will preclude the accumulation of seismic forces in the elements above. It is not necessary to consider the elements both above and below the component. In another example, both shear and moment in a shear wall might be considered deformation-controlled actions, but only one of the actions will yield. The other action's force will be limited by the action that yields. Therefore, this higher DCR should be the value considered for this component along with other components' controlling action DCRs should be considered in the load path.

The DCR of the floor or roof diaphragm framing into the story should be considered if it will significantly limit the forces to the specific element, such as a strut attached to that diaphragm. However, the yielding of one floor diaphragm should not be used if there are multiple stories delivering load to the element, because the diaphragm would only limit the forces being delivered from that specific floor level. When the load path is complicated or has significant variation in DCRs, using a mechanism analysis, as permitted in the first option to compute Q_{UF}, or a nonlinear analysis may be prudent to better understand the forces being delivered to the component for the force-controlled action being assessed.

When the $DCR_{\min} < C_1 C_2/\chi$, components in the load path may be essentially elastic. In that case, the demand should be based on the pseudo seismic force, modified to eliminate the C_1 and C_2 displacement amplification factors and multiplied by the χ factor. When the load path delivering forces to the component with the force-controlled action being assessed is essentially elastic, the coefficient χ adjusts the actions obtained from an analysis undertaken at the Life Safety and Immediate Occupancy Structural Performance Levels to provide a margin relative to Collapse Prevention that is consistent with those prescribed by the m-factors for deformation-controlled actions and Section 7.6.3. In cases where the BPOE or BPON is the selected performance objective, force-controlled actions will likely be controlled by the evaluation of the performance level at the BSE-2E or BSE-2N seismic hazard. The ratios between the BSE-2E and BSE-1E, or BSE-2N and BSE-1N seismic hazard parameters being greater than or equal to 1.5, is larger than the χ-factor adopted for the Life Safety and Immediate Occupancy Structural Performance Levels.

In the 2017 and earlier editions of the standard, a J-factor was used to limit the magnitude of force-controlled actions. The J-factor was defined as $DCR_{\min}$ is now, with an alternate to use explicit values based on level of seismicity. Coupling of ductility in a structure to level of seismicity is not correct. These two items are not always related. There is not always a correlation between level of seismicity and the amount of ductility within a load path, especially when the deformation-controlled components have limited ductility. Eliminating the seismicity-dependent alternate J-factor values also addresses the issue of potentially using a J-factor larger than the ductility of the deformation-controlled members, leading to potential failure of the force-controlled member prior to yielding. Throughout the standard there are deformation-controlled components that have m-factors less than 2.0, the maximum alternate J-factor in previous editions of the standard.

C7.5.2.2 Acceptance Criteria for Linear Procedures It is common practice in engineering to use the term demand-capacity ratio(DCR) as a measure of acceptability. In this situation, typically a value of less than unity is defined as acceptable; a value equal or greater than unity is defined as unacceptable. However, Equation (7-16) specifies DCR as Q_{UD}/Q_{CE}, which is a measure of ductility demand, not acceptability. To preserve the standard's use of the term DCR, but to avoid confusion, the term Acceptance Ratio is introduced by the standard. An Acceptance Ratio less than or equal to unity is considered acceptable; an Acceptance Ratio greater than unity is considered unacceptable.

C7.5.3 Nonlinear Procedures

C7.5.3.2 Acceptance Criteria for Nonlinear Procedures

C7.5.3.2.1 Unacceptable Response for Nonlinear Dynamic Procedure This section defines the criteria for determining unacceptable responses and allows a maximum of one unacceptable response per 11 motions under certain circumstances. An unacceptable response can be an indicator of global instability of the structure or a collapse or simply that the response exceeds a level where the analytical model can be considered a reliable predictor of performance. Along with other acceptance criteria for deformation- and force-controlled actions, this requirement helps ensure that collapse has a suitably low probability of occurrence and is not encountered in any of the ground motion runs that are used in computing average response. The conditions under which a response is considered to be unacceptable include (1) nonconvergence of analysis solution, which could indicate collapse or other problems with the model; (2) when the deformation in a deformation-controlled element exceeds the valid range of modeling unless the component action drops its lateral-force-resisting strength to 5% or less of the yield capacity and can either maintain gravity load support or simultaneously loses the ability to support gravity loads in the model, and the model adapts to that without instability; (3) when a critical force-controlled element that is modeled linearly exceeds its expected capacity; (4) when other nonmodeled elements, primarily gravity elements, exceed their gravity load capacities; and (5) when critical failure mechanisms of structural elements may not be adequately evaluated, or represented, within the analytical model when subjected to large dynamic transient story drift demands.

The limit of 6% drift as an unacceptable response parameter was chosen based on judgment of the committee resulting from concerns about analyses being capable of capturing dynamic instability at large drifts. This is consistent with requirements in ASCE 7 in which the peak transient drift of all analyses must be less than 150% of the limit on mean, which for Risk Category I and II buildings is 6%. This is not to say that all buildings whose analysis indicate a 6% peak transient drift will become unstable and collapse, but that further investigation to confirm the validity of the analysis should be performed. The provisions contain a caveat that permits analysis beyond 6% provided there is some validation. Buildings that are short or have very small gravity loads can remain stable at such large deformations (FEMA 2020a, 2020b, 2020c).

The exception allows one unacceptable response to be discarded for every 11 ground motion records when Structural Performance Level is Limited Safety or Collapse Prevention. This exception relaxes the need to have all the records converge and achieve acceptable response in recognition of a target of 90% reliability of achieving those performance levels. The 90% reliability is based on the presumption that achieving Collapse Prevention provides for a 10% probability of collapse comparable to the performance of a Risk Category I or II building designed to ASCE 7 under $\mathrm{MCE_R}$ shaking intensity. Haselton et al. (2017) discuss how one unacceptable response out of 11 provides a reasonable expectation that the probability of collapse is not greater than 10%. Numerous analyses must be performed to reliably quantify the probability of collapse. A 10% collapse probability goal is not necessarily met even if zero unacceptable responses are observed in a set of 11 analyses. Even if a building has a 10% probability of collapse, there is some chance that one unacceptable response will be observed in a set of 11 analyses (i.e., a "false positive"). Therefore, an acceptance criterion of "no unacceptable responses" would be violated quite often by a building that meets the 10% collapse probability goal. This is not to give the impression that 11 records alone are sufficient to conduct a full reliability assessment. It is not.

The Life Safety Performance Level is defined as providing a margin of safety against collapse, taken as approximately 1.3 (= 1/0.75) compared to the Collapse Prevention Performance Level. This equates to approximately a 5% probability of collapse, given the shaking intensity for which Life Safety performance is targeted. Haselton et al. (2017) demonstrated through statistical analysis that one ground motion suite of 11 possibly being unacceptable and possibly indicating a collapse would not provide a 5% probability of collapse, which is around what the performance level would imply by providing a 1.3 margin of safety against collapse.

If more than 11 ground motions are used for analysis, then additional unacceptable responses may be permissible. Two unacceptable responses would be permitted if 22 or more motions are used, and three unacceptable responses are permissible when 33 or more motions are used. However, the unacceptable response must come from suites of 11. For example, if one uses a conditional mean spectra approach with two suites of 11 records, the analysis cannot have two unacceptable responses from one suite of 11 and none from the other suite of 11. Conversely, where the analysis requires consideration of mass eccentricity, and where this is accounted for in a three-dimensional model with a separate analysis for each mass offset (resulting in a total number of analyses that is equal to the number of ground motions multiplied by mass offset cases), then additional unacceptable responses may be permissible. Four unacceptable responses would be permitted for a suite of 11 ground motions multiplied by four separate mass eccentricities (a total of 44 individual analyses). Similar increases in the number of unacceptable responses may be applied for other requirements that require multiple analyses for a given set of ground motion records, such as bounding properties in seismic isolation or damping.

In general, this standard uses mean demands to evaluate acceptance. The distribution of demands obtained from a suite of nonlinear analysis typically approximates a lognormal distribution. In such distributions, given typical dispersions, the mean demand will be approximately 110% to 120% of the median demand. Therefore, the standard adopts the procedure that when one such response is encountered, it is acceptable to discard this analysis and compute primary and secondary component demands as 120% of the median demand, of all analyses including the nonconvergent case, but not less than the mean demand, calculated from the analyses with acceptable response. When computing the median, the unacceptable response(s) should be considered larger than the median, because it is assumed that an unacceptable response may indicate a collapse causing very large deformations. For example, in a suite of 11 records with one unacceptable response, the median value would be the sixth response in ascending order of the acceptable responses.

The valid range of modeling for deformation-controlled elements may exceed the Collapse Prevention Performance Level limit if the response of the element is known reliably beyond this limit. In addition, it is generally recommended that all elements be modeled using their expected properties, and that unacceptable responses that are caused by force-controlled elements that exceed their expected capacity may be resolved by strengthening the element or by modeling the failing element using nonlinear elements that account for the applicable strength deterioration. Refer to the commentary in ASCE 7-16, Chapter C16, for additional discussion.

C7.5.3.2.2 Acceptance Criteria for Deformation-Controlled Actions for NSP or NDP Where all components are explicitly modeled with full backbone curves, the NSP or NDP can be used to evaluate the full contribution of all components to the seismic force resistance of the structure as they degrade to residual strength values. Acceptance criteria for nonlinear procedures are based on the provisions in Section 7.6.3. Values provided for Immediate Occupancy, Life Safety, and Collapse Prevention in Chapters 8 through 12, 14, and 15 are typically based on those provisions. Some acceptance criteria may have been altered from the provisions of Section 7.6.3 by judgment of the standards committee. Section C7.6.3 provides a detailed description of how those criteria were arrived at.

The standard is revised to eliminate the need to consider local deformation criteria for Collapse Prevention when the component is classified as ordinary. The reason for this relates to the definition of the Collapse Prevention Performance Level—the structure is on the verge of total or partial collapse. There may be extensive local damage, which could occur from the loss of gravity-load-carrying ability or almost total loss of lateral-force-resisting ability of specific components. However, if the element in question loses its ability to support gravity loads and it causes a multiple bay collapse, that would not meet the performance objective.

Although force–deformation curves typically possess moment-rotation relationships derived from experimentally obtained component behavior and rotations are monitored against prescribed acceptance criteria, fiber-type modeling behavior is dependent on the fiber material force–deformation relationships over a defined integration length. As such, there are several ways by which acceptance criteria may be monitored in a fiber model: a rotation gauge may be used to monitor rotation over a defined hinge length in a four-node panel fiber element, or alternatively strains measured over a defined hinge length may be monitored directly or converted to curvature in the component. In cases where rotation is not directly monitored to be consistent with prescribed rotations in Chapters 8 through 12 or based on experimentally obtained acceptance criteria in Section 7.6, curvatures or strains must be converted and calibrated to representative rotations for monitoring acceptance in nonlinear procedures.

C7.5.3.2.3 Acceptance Criteria for Force-Controlled Actions for Nonlinear Static Procedure or Nonlinear Dynamic Procedure The gamma factors account for the variability in the response of buildings to ground motion and attempt to provide 90% reliability on the given performance objective. Refer to the commentary to Chapter 16 in FEMA 1050 (2015) for discussion and additional resources on the statistical derivation of the gamma factors. These factors have been modified from what is in ASCE 7 to account for the pairing of the force demands with lower-bound capacities, as is required in ASCE 41. This factor accounts for both variability between records in a nonlinear response history and for the material variability in the deformation-controlled actions delivering load to the force-controlled action.

The 2013 edition and earlier editions of ASCE 41 would deem a building to meet the given performance objective with multiple records failing force-controlled elements. An example is a nonductile concrete building where the ground columns have four of the seven records showing shear failure in high axial load columns but the average is slightly less than the lower-bound capacity. Additionally, there is currently nothing in the standard that addresses the possibility that strengths of the deformation-controlled elements in the mathematical model actually underpredict the strengths in the actual building. The gamma factor increases with the consequence of failure of the force-controlled action, as shown in Table 7-8 for critical and noncritical actions. For noncritical force-controlled actions, it is taken as 1.0 since the demands are compared against lower-bound material properties and the consequence of those elements being overloaded would not likely lead to the collapse of the building. For critical force-controlled actions, the average force is amplified by 1.5. Gamma is only required when the force-controlled behavior of a component is not explicitly modeled with nonlinear properties per Section 7.5.1.1 and is treated but elastic.

When plastic mechanism analysis is used to limit the force on a component, the envelope of the forces produced by all likely plastic mechanisms should be obtained to ensure that the largest possible force is considered. However, mechanism analysis has been shown to underestimate shear in shear wall structures, which is why the exemption exists.

Another factor is added to the force-controlled actions to account for performance level. The performance level factor provides for an additional margin of safety against collapse in the Life Safety Structural Performance Level, which is part of the definition of the performance level, and the Immediate Occupancy Structural Performance Level. The product of this factor and the gamma factor is capped to limit the amount of amplification to not provide overly conservative levels of reliability. As an alternate to amplifying the demand by the performance level factor, χ, the user is permitted to perform the analysis with the ground motion records amplified by χ and compute demands on the force-controlled elements using Equation (7-41) with $\chi = 1.0$. The alternate explicitly assesses the demand on the force-controlled actions at earthquake shaking intensity amplified by the margin of safety against collapse the performance levels greater than Collapse Prevention seek. This may produce a demand lower than multiplying the force from the analysis by χ because the force is based on system yielding, which stops increasing with increasing hazard intensity. If the analysis using ground motion records amplified by χ produces more unacceptable responses than are permitted for the performance level being considered, then this method cannot be used because of concern that the demands on the force-controlled actions may be underestimated because of the unacceptable responses.

C7.6 EXPERIMENTALLY DERIVED MODELING PARAMETERS AND ACCEPTANCE CRITERIA

This section provides guidance for developing appropriate data to evaluate construction materials and detailing systems not specifically addressed by this standard or to update parameters within the standard. This standard specifies stiffnesses, *m*-factors, strengths, and deformation capacities for a wide range of components. Where other documents are developed to be used in conjunction with ASCE 41, the provisions of this section should be followed to develop modeling parameters and acceptance criteria.

To the extent practical, this standard has been formatted to provide broad coverage of various common construction types present in the national inventory of buildings. However, it is fully anticipated that in the course of evaluating and retrofitting existing buildings, construction systems and component detailing practices that are not specifically covered by this standard will be encountered. Furthermore, it is anticipated that additional research and new methods and materials not currently in use will be developed that may have direct application to building retrofit. This section provides a method for obtaining the needed analysis parameters and acceptance criteria for elements, components, and construction details not specifically included in this standard. It is intended to be used for both the development of modeling parameters and acceptance criteria for general use on projects and for project-specific testing programs.

The approach taken in this section is similar to that originally used to derive the basic analysis parameters and acceptance criteria contained in this standard for various components, except that some component actions had no or incomplete experimentation data available. The required force–deformation curves were derived by developers of this standard, either directly from research testing available in the literature or based on the judgment of engineers knowledgeable about the behavior of the particular materials and systems.

C7.6.1 Criteria for General Use Parameters This section provides direction on how to develop modeling parameters and acceptance criteria for general use. This is different from the directions for project-specific testing found in the 2017 and earlier editions of the standard. The provisions for project-specific testing are provided in Section 7.6.2. This section is for developing new modeling parameters and acceptance criteria or to update existing values provided in the standard, for other standards that are intended to be used with this standard, or for those provided in product literature for a proprietary component. Ideally, one would have sufficient test data for the specific component action over all possible boundary conditions and for all possible configurations of the component. The test data may be augmented by analytical modeling, but the parameters cannot be based solely on analytical modeling.

Often a specific component action is affected by many parameters, such as the specific construction material detailing, corollary actions (i.e., axial force and shear when investigating moment), and member properties. The effect of each of these parameters on the force–displacement relationship should be assessed as part of the development of the modeling parameters, with guidance given to adjust for the corollary parameters and the range of configuration and corollary parameters over which the modeling parameters are valid. For example, the behavior of a reinforced concrete column when subjected to moment is dependent on the axial load, the transverse reinforcement in the column, and the ratio of moment to shear. Section C10.4.2.2.2 discusses how the various parameters were considered in developing the modeling parameters in Table 10-8.

In cases where a component has multiple actions that affect its force–deformation relationship, there are two options for how to represent this. One option is to develop force–deformation relationships for each action. The other option is to develop a unified force–deformation relationship using one of the two actions as the surrogate for both actions. Either approach is acceptable. The test program should clearly indicate when one action is intended to represent multiple actions and provide direction on how to account for the effects of both actions on the force–deformation relationship.

Although desirable, it is often not possible to test every possible configuration and combination of corollary actions on a component. It is also common to have data from different tests with different configurations, boundary conditions, and loading protocols. Such data can be used to fill in gaps for specific tests. In developing general modeling parameters, this data should be normalized and combined. Judgment needs to be exercised regarding when to include or exclude a test in the general test data. Consideration should be given to aggregating test data from different sample size, because size effects can alter the response of a component action. This should be investigated and confirmed before combining normalized test data of different size specimens.

The goal of any testing program should be to understand the component action over the entire deformation range up to and, in some cases, beyond the point at which the component can resist any lateral force to the point where it loses its ability to support gravity loads. This necessitates testing to failure while the subassemblage has a load or applied force to simulate gravity load concurrent with its resistance to lateral forces. The standard recognizes failure can be defined differently and put forward two distinct limit states. The first is the point at which the specimen resists negligible lateral force, taken as less than 5% of the yield strength. The second is the point at which the component loses its ability to support gravity loads, when a component does support gravity loads. When devising a testing program, it may unclear which of these limit states will come first, so the testing program should be set up to assess both. Unfortunately, most existing testing did not test components to such extreme deformation levels. The standard treats the maximum deformations from the testing program as "failure" when the tests have not been conducted to the point where the action loses the ability to resist lateral forces or the component loses the ability to support gravity loads. Using the maximum deformation from the test as the valid range of modeling is conservative, which is why there is a permission to establish the valid range of modeling from similar tests or through analysis that has been calibrated to tests at such extreme deformations.

There are many factors to consider when normalizing test data to create backbone curves that can be used with the provisions of Section 7.6.3. One of the more significant is the loading protocols used in various tests. Historically, subassemblage testing was conducted using fully reversed cyclic load protocols, such as those defined in ATC-24 (ATC 1990). The standard does not specify which loading protocols to use. Figure C7-9 illustrates different types of loading protocols. The representative earthquake loading protocols mimic actual irregular earthquake response. The loading protocol has significant influence on the resulting envelope of the force–displacement relationship (backbone curves), as depicted in Figure C7-10. For this case, the backbones were essentially the same out to about 2% drift, but they differ significantly for larger drifts, depending on the protocol. The standard loading protocol, fully reversed cyclic loading using numerous cycles, produced backbones with the smallest drift capacities.

In one widely used standard loading protocol, fully reversed cyclic as described in the Applied Technology Council's *Guidelines for Seismic Testing of Components of Steel Structures* (ATC 1992), the specimen is subjected to a series of quasistatic, fully reversed cyclic displacements that are incremented from displacement levels corresponding to elastic behavior, to those at which failure of the specimen occurs. Many of the component action force–deformation relationships in this standard have been derived based on this loading protocol. However, more recent research found that a backbone curve derived from an envelope of cyclic laboratory test data can be conservative (FEMA-440A 2009a). The reason is that fully reversed cyclic loading can differ from the deformation histories at near-collapse. For many cases at near-collapse, the importance of cyclic deterioration

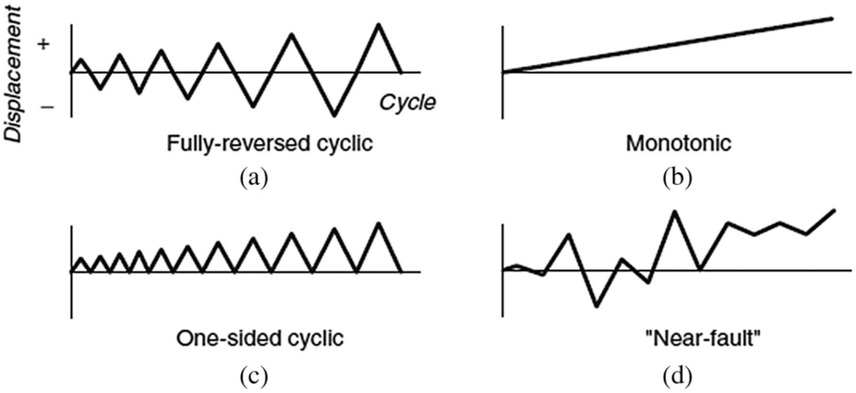


Figure C7-9. General types of loading protocols.

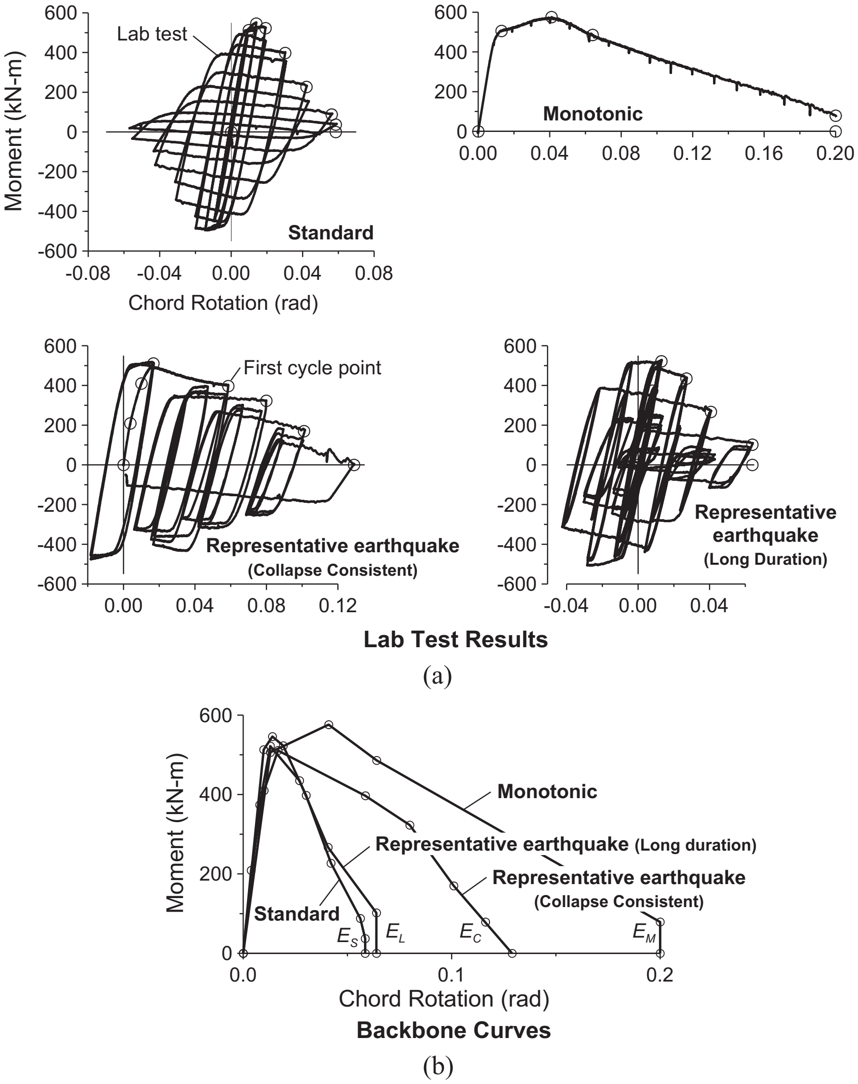


Figure C7-10. Envelopes from four different types of tests: standard (fully reversed cyclic protocol), monotonic, representative earthquake (collapse consistent), and representative earthquake (long duration).
Source: Suzuki and Lignos (2015, 2019).

diminishes because of ratcheting of the response in which the lateral deformations typically increase in one direction (one-sided response). Figure C7-11 shows examples of earthquake response having few large one-sided undulations that may be best simulated by a representative earthquake loading protocol (Figure C7-9d). Hence, there is a need to complement conventional component tests, which are usually based on stepwise increasing symmetric loading histories, with tests whose loading history pays specific attention to behavior close to collapse. Additional discussion on the importance of loading protocols can be found in Krawinkler (2009), FEMA P-440A (2009a), FEMA P-695 (2009c), and PEER/ATC 72-1 (2010).

Many past subassemblage tests were performed with objectives other than development of backbone curves. For example, tests performed for component qualification (e.g., AISC 2010a) are intended to provide evidence that a component satisfies certain ductility requirements, and data from such tests may be insufficient for backbone curve formulation. It may be advantageous to perform representative earthquake tests to supplement existing test data from fully reversed cyclic tests to have a better description of behavior over a full range, including near-collapse conditions.

Use of monotonic tests is prohibited, except in the case where one is developing an adaptive hinge model that can reproduce different loading protocols explicitly. There are situations in which cyclic loading reveals key types of component deterioration. Three examples are large tensile cracking of concrete, fracture of steel, and buckling of steel. Figure C7-12 shows examples where results from monotonic and fully reverse cyclic loading protocols differ significantly. When large tensile cracks form in concrete, the internal rebar elongates, but on cyclic reversal, the bars deform in compression, possibly causing local buckling to the rebar, spalling of concrete, or other effects. Many steel elements, particularly welds, have been shown to fail because of low cycle fatigue. This phenomenon is not generally characterized by monotonic testing because it requires load reversal to initiate the fatigue crack. Buckling of steel elements may locally deform in compression; and cracks, tears, and ultimate fracture may occur when the damaged steel is subjected to reversed tensile loading. These local damages may significantly

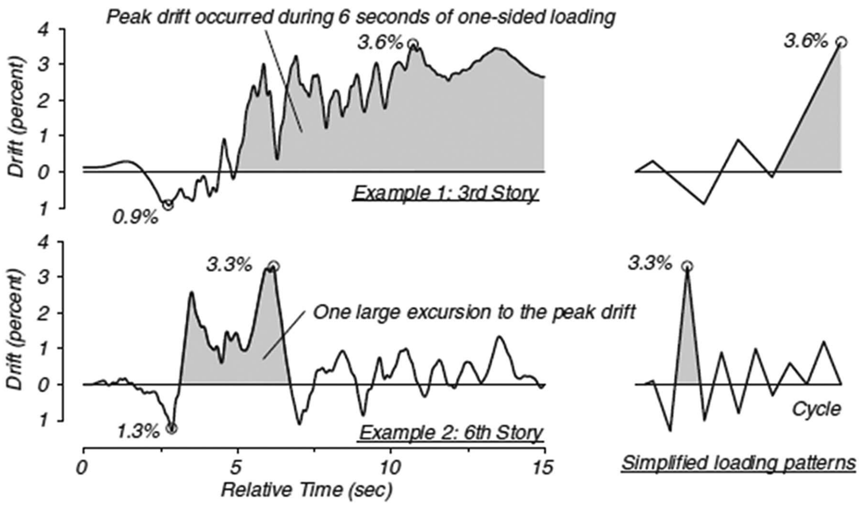
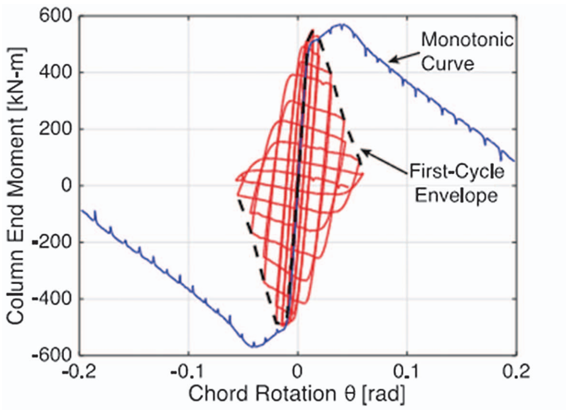
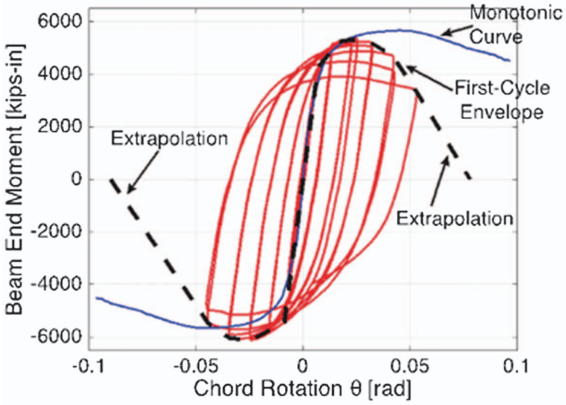


Figure C7-11. Time histories of drift response from two analysis runs from a computer model of an eight-story steel eccentric braced frame building.
Source: Maison and Speicher (2016).



Wide flange steel beam-column under constant axial load

(a)



Steel beam with RBS as part of fully restrained beam-to-column connections

(b)

Figure C7-12. Monotonic and first-cycle enevelope curves for various steel components.
Source: Suzuki and Lignos (2015), Tremblay et al. (1997).

reduce the inelastic deformation capacity and resistance and increase the deterioration noted in the system. As a result, if only monotonic data are available, they should be used with consideration of consequences of possible cyclic strength degradation.

When alternate general use parameters are proposed to be used in conjunction with the standard, some level of vetting must be done to confirm that the parameters have, in fact, been developed based on the rules in this section. Ideally the alternate parameters would be developed by another consensus standard's committee and published in that group's standard, such as AISC 342 or ACI 369. Other times the values may be provided by a product manufacturer or developed through a literature review for a specific project. In those cases, it is desirable that the development of those parameters has undergone some level of peer review, similar to parameters derived based on project-specific testing.

C7.6.2 Criteria for Individual Project Testing In some cases, a specific project warrants testing of a specific component, either an existing configuration or a proposed retrofit configuration. This section provides criteria to develop a testing program to develop modeling parameters and acceptance criteria for specific projects. This section is not intended to be used to take a project-specific test and apply it to general provisions, although a project-specific test may be included in a larger data set used to develop general use parameters based on Section 7.6.1.

C7.6.2.1 Experimental Setup The test specimen should replicate, as much as practical, the geometry and boundary conditions as in the actual building. Consideration should be given to the possible influence of gravity loads on the component lateral force resistance. The use of multiple test data allows some of the uncertainty with regard to actual behavior to be defined. It is required to have at least two tests with the same loading protocol consistent with the customary practice of having multiple specimens when component testing. A specific loading protocol has not been recommended, because selection of a suitable loading protocol depends on the anticipated failure modes, the sequences of the subassemblage, and the character of excitation it is expected to experience in the real structure, as well as conformance to standards for testing of a particular system, assembly, or component as applicable. In selecting an appropriate loading protocol, it is important that sufficient increments of loading be selected to adequately characterize the force–deformation behavior of the subassemblage throughout its expected range of performance. A loading protocol that uses cyclic loading to reflect design-level demands followed by a monotonic push to component failure may be an effective way to achieve this goal.

The standard recommends that the loading protocol used be representative of the deformations the component action will undergo when the structure is subjected to the seismic hazard being evaluated and account for factors like intensity and shaking duration. Tests should always proceed to a failure state so that the margin against failure of the subassemblage can be understood. Additional discussion on the importance of loading protocols can be found in Krawinkler (2009), FEMA P-440A (2009a), FEMA P-695 (2009c), and PEER/ATC 72-1 (2010).

C7.6.2.2 Data Reduction and Reporting It is important that data from experimental programs be reported in a uniform manner so that the performance of different subassemblies may be compared. The data reporting requirements specified are the minimum thought to be adequate to allow development of the required analysis parameters and acceptance criteria for the various evaluation and retrofit procedures. Some engineers and researchers may desire additional data from the experimentation program to allow calibration of analytical models and to permit improved understanding of the probable behavior of the subassemblies in real structures.

C7.6.3 Modeling Parameters and Acceptance Criteria for Nonadaptive Force–Deformation Curves A multistep procedure for developing design parameters and acceptance criteria for use with both the linear and nonlinear procedures is provided. The basic approach consists of the development of an approximate story seismic force–deformation curve and gravity load resistance curve for the component action, based on the experimental data.

In developing the representative component action force–deformation curve from the experimentation, use of a backbone curve is required. This curve takes into account, in an approximate manner, the strength and stiffness deterioration commonly experienced by structural components. The loading protocol used in subassemblage testing can have a large effect on the response envelope for a component, and protocols must realistically reflect demands caused by actual earthquake loadings throughout the component's expected range of performance. The backbone curve is taken as the average of the cyclic test envelopes.

Figure 7-6 distinguishes between component ductile and rapid strength loss behavior. Ductile behavior occurs when a cyclic test has positive to moderately negative in-cycle tangent stiffness throughout the test (Figure 7-6a). Accordingly, the peak displacement attained was set by the loading protocol and not by actual component failure (where element seismic strength is essentially zero). Significant rapid strength loss indicates occurrence of component deterioration associated with rapid decline in component resisting force with increasing deformation within a loading cycle and is taken here as when the component in-cycle tangent stiffness attains a large negative value (Figure 7-6b). When such rapid strength loss occurs and there is no residual strength or the deformation at a residual strength is small before the component completely loses the ability to support gravity loads, the component should be classified as force-controlled. It is possible that a component exhibits rapid strength loss but then has appreciable residual strength and deformation capacity. An example is a pre-Northridge moment connection (Figure C7-13), where the weld between the beam flange and the column flange may fracture with very little plastic deformation, but there is significant residual displacement, albeit at a greatly reduced strength. Actions such as this should be considered deformation-controlled.

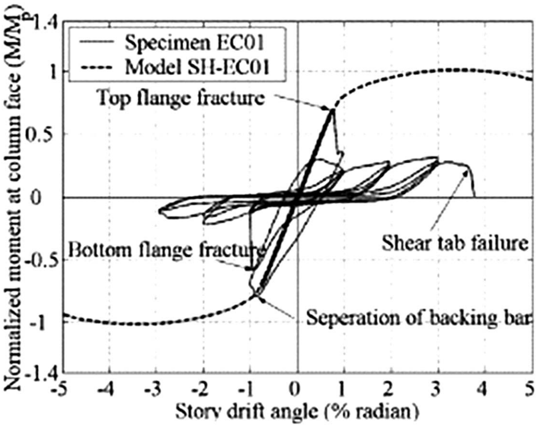
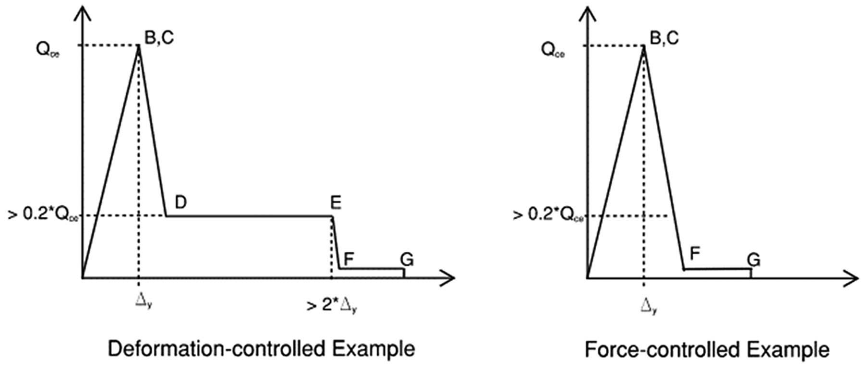


Figure C7-13. Rapid strength loss with residual deformation.
Source: Kim et al. (2008).

In some cases, test data will show that a component action exhibits sudden loss of strength. Whether that means the action should be considered force-controlled depends on whether the action degrades to a residual strength and exhibits additional deformation beyond, or the action reaches a point where it resists negligible lateral force or support gravity load shortly after failure. Figure C7-14 shows the provisions, which allow components with residual strength of at least 20% of the initial strength that sustain that residual strength for at least twice the deformation of the point of maximum strength.

To develop component action force–deformation relationships, data from individual tests must be normalized and aggregated. The standard uses the mean of similar component tests to develop the component action backbone curve. In previous editions of the standard, only a six-point multisegment curve shown in Figure 7-8a was permitted. This curve is retained because many of the existing modeling parameters are based on it. In addition, the standard permits the use of a more detailed multisegment curve or a smooth curve. If a smooth curve is used, the specific points on the multisegment linear curve still need to be derived because they are used to develop the acceptance criteria.

The first generation of component backbone curves in this standard's predecessor document, FEMA 273, used second-cycle envelopes of the hysteretic curves developed from such testing. In Supplement 1 to the 2006 edition of this standard, this section was updated to require the use of overall envelope of the hysteresis instead of second cycle. This change typically produces larger estimates of the modeling parameters. However, most of the modeling parameters in the standard were not revised to reflect this change. The 2017 edition of the standard relaxed the requirements for loading protocol to permit protocols that better represent demands from actual earthquakes. The user is cautioned when developing parameters based on new test data, especially if loading protocols other than the standard reverse cyclic protocol is used. It is possible that the newly tested component parameters may be significantly less conservative than the component action parameters for the other elements in the structure. Mixing backbone curves derived from different loading protocols or from second cycle versus first cycle can lead to inaccurate representations of the inelastic response of the structure because the more



(a)

(b)

Figure C7-14. Force-versus deformation-controlled backbone curves.

recent components appear to have more deformation capacity than those based on older parameters.

In some cases, regression analysis may be used to develop modeling parameters from large sets of data. The standard recommends the equations predict the median value of the data and the coefficient of variation be reported. Median is chosen over mean in recognition that large data sets may not follow a normal distribution. In developing equations, the data may need to be subdivided based on criteria such as failure mode or component size. The user is cautioned against aggregating too many different subassemblage test sizes or tests that exhibit failure modes. Aggregating too much data may skew the regression analyses compared to smaller data sets that have more consistent failure mechanisms.

Another topic that comes up when developing parameters for force–deformation curves from test data is whether to report plastic deformation or total deformation. The standard permits either and does not provide direction. The user should consider the test data and determine if the deformations where the component action behavior changes are sensitive to the yield point. If so, then plastic rotation should be used. If, however, the action is independent of the yield point, then total deformations should be used.

Acceptance criteria for deformation-controlled actions are based on the component backbone curves. In general, the local criteria correspond to significant changes in the component response. For the Immediate Occupancy criteria, the first part of the criteria is the deformation where visible damage is observed. Because Immediate Occupancy presumes that a structure will be deemed safe to reoccupancy, the criteria seek to limit damage that would lead someone to question the integrity of the structure. As an alternate, the Immediate Occupancy limit is stipulated as the 10th percentile of the point at which the component response transitions from a yielding strain-hardening behavior to a degraded behavior. This point is usually correlated with a change in damage state of the component under the action being considered, so providing a 90% confidence the component is not at that point was considered reasonable for the Immediate Occupancy limit. The previous alternate definition of the Immediate Occupancy, being 50% of the plastic deformation between yield (Figure 7-8, Point B) and the point at which strength degradation occurs (Point C) is retained in recognition that few component actions have an extensive test data set to develop a reliable coefficient of variation.

A significant change in the 2023 edition of the standard is the explicit definition of the Damage Control Performance Level criteria. Previously, the limit was taken as the average of the Immediate Occupancy limit and the Life Safety limit. This presented an issue where there was significant difference between the Immediate Occupancy point and the Life Safety limit, with the halfway point predicting significant damage that may not be consistent with the performance implied by Damage Control. The Damage Control limit is, therefore, set at the mean representation where the component would transition from the yielding, strain-hardening response to a strength degradation response corresponding to Point C. Because the transition from strain hardening to strength degradation often coincides with a noticeable change in damage, this point was considered a reasonable limit for Damage Control.

Another significant change in the 2023 edition of the standard is the permission to explicitly consider the point at which the component loses the ability to support gravity loads for Life Safety and Collapse Prevention. For Collapse Prevention, the limit is set as the 25th percentile when that point is reached. For Life Safety, the criteria is set at a more conservative reliability of the 10th percentile of the point at which that point is reached. The percentages were determined based on judgment. Section C7.5.3.2.1 discusses the goal of providing a 90% reliability of achieving Collapse Prevention. The 25th percentile of loss of gravity load support of an individual element was felt to be conservative enough, in recognition of the large coefficient of variation seen in limited test data where the loss of vertical load was established. Similar to Immediate Occupancy, the previous definitions of Life Safety and Collapse Prevention limits related to Point F in recognition of a lack of test data for most components to reliably establish the point where loss of gravity load carrying occurs and a coefficient of variation of that point.

Few, if any, experimental tests have been carried out to the condition where the component cannot support gravity loads. Such tests are now encouraged by this standard so that this key information will become available and used in future editions. Figure C7-15 shows how points indicating loss of lateral force (F) and loss of gravity load support (G) may be determined in practice. Point E is taken at 5% of the force at B. Point F is taken as the maximum deformation in the test. Point G is also taken as the maximum deformation used in the test. even if the component is still resisting gravity loads.

For linear procedures, the acceptance criteria for primary components are only based on the response of the component action up to the point of strength degradation, Point C in Figure 7-9. For secondary components, the limits are similar to the corresponding ones for nonlinear procedures, with the exception of the Damage Control limit. The primary component Collapse Prevention limit is based on Point C. The Life Safety limit is 75% of the Collapse

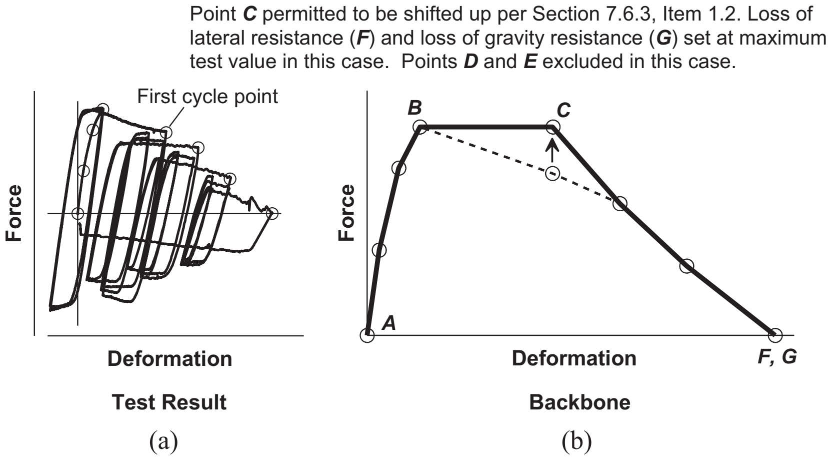


Figure C7-15. Construction of backbone curve from laboratory test of steel column.

Prevention Limit but not greater than the corresponding limit in the nonlinear procedure. For Damage Control, the limit is taken as the halfway point between the Life Safety primary component limit and the Immediate Occupancy limit and is the same for both primary and secondary components. All the performance limit points are multiplied by a factor of 0.75 to provide an additional level of conservatism in recognition of the imprecision of linear analyses.

C7.6.4 Modeling Parameters and Acceptance Criteria for Component Actions Based on Experimental Data for Fiber Models Fiber modeling force–deformation response and acceptance criteria should incorporate consideration of local effects not captured explicitly in the fiber material stress-deformation relationships. Mechanisms leading to softening or strength degradation in the experimental component response shall be represented and considered in the fiber model, especially for actions such as global buckling or shear-flexure interaction. Acceptance criteria for fiber models should be expressed in terms of curvature, rotation, strain, or displacement. Such acceptance criteria often require a discrete length over which they should be measured. This should be identified with the acceptance criteria. Additional information on phenomena specific to distributed plasticity fiber models is discussed in Sections 7.5 and C7.5.

C7.6.5 Modeling Parameters and Acceptance Criteria for Component Actions Based on Experimental Data for Adaptive Force–Deformation Models in the Mathematical Model Adaptive hinge models have force–deformation relationships that depend on the loading protocol. The provisions for developing loading nonadaptive force–deformation curve models should be followed but with the normalization and calibration done explicitly for different loading protocols. Section C7.6.1 discusses different loading protocols and how important they can be on the resulting force–deformation curve. Ideally, the model is calibrated to a monotonic loading protocol, a standard fully reverse cyclic protocol, and a third loading protocol, such as the representative earthquake protocol in Figure C7-9. The model should be validated to properly adjust the force–deformation relationship as the loading protocol changes.

CHAPTER C8
FOUNDATIONS, SUBSURFACE SOIL, AND GEOLOGIC SITE HAZARDS

C8.1 SCOPE

This chapter provides geotechnical engineering provisions for building foundations, soils, and seismic–geologic site hazards. Acceptance criteria for the behavior of the foundation system and foundation soils for shallow foundations are evaluated for soil bearing and the structural footing in addition to the behavior assessment of the superstructure. Deep foundations where the flexibility of the soil is included in the analysis acceptability is measured through its effect on the superstructure assessment in addition to the strength of the deep foundation.

Geotechnical requirements for buildings that are suitable for deficiency-based evaluation and retrofit are included in Chapter 5. Structural engineering issues of foundation systems are discussed in the chapters on steel (Chapter 9), concrete (Chapter 10), masonry (Chapter 11), and wood (Chapter 12).

C8.2 SITE CHARACTERIZATION

The guidance of the Authority Having Jurisdiction over historical matters should be obtained if historic or archaeological resources are present at the site.

C8.2.1 Subsurface Soil and Foundation Information

C8.2.1.1 Subsurface Soil Conditions Prescriptive procedures may be used to estimate the short-term expected soil bearing capacity of the foundations when construction documents are available and necessary information on allowable or expected soil bearing capacity values are specified. The prescriptive method can also be used in cases where soil bearing is not known, but bearing values are estimated based on the original design loads on the footings. Site-specific assessments may be required to determine foundation bearing capacity and/or stiffness, depending on the analysis method selected for evaluation.

Acquiring this additional information involves determining unit weights, shear strength, friction angle, compressibility characteristics, soil moduli, and location of water table. In addition, the relative density of cohesionless soils and the relative strength of cohesive soils should also be provided for a quantitative description of the soils.

Specific foundation information developed for an adjacent or nearby building may be useful if subsurface soils and groundwater conditions in the site region are known to be uniform. However, less confidence will result if subsurface data are developed from anywhere but the site of the building being evaluated or retrofit. Adjacent sites where construction has been done recently may provide a guide for evaluation of subsurface conditions at the site being considered. Sources of existing geotechnical information are discussed in Section C3.2.4.

C8.2.1.2 Foundation Conditions

C8.2.1.2.1 Structural Foundation Information Shallow foundation types may consist of isolated, combined, or continuous spread footings, and mat foundations. Deep foundations may consist of driven piles, cast-in-place concrete piers, and auger-cast piles. Intermediate foundations may consist of shallow foundations on rammed aggregate piers.

Foundation configuration information includes dimensions and locations, depths of embedment of shallow foundations, pile–shaft tip elevations, and variations in cross section along the length of the pile for tapered piles and belled caissons.

Foundation material types include concrete, steel, and wood. Foundation installation methods include cast-in-place and open- or closed-end driving.

C8.2.1.2.2 Foundation Loads Foundation overturning loads are obtained from the analysis procedures from Chapter 7. For linear procedures, loads to the foundation are pseudo-force demands. For nonlinear procedures, loads to the foundation represent realistic actual loads based on yielding in the superstructure.

C8.2.1.3 Load–Deformation Characteristics of Subsurface Soil under Seismic Loading Traditional geotechnical engineering considers soil load–deformation characteristics attributable to long-term dead loads plus frequently applied live loads. In most cases, long-term settlement governs foundation design. Short-term (earthquake) load–deformation characteristics have at times been used for design; however, such relationships may not be found in the older geotechnical reports for existing buildings. The use of long-term loads for earthquake performance assessment may lead to an underestimation of system stiffness; the requirements in Section 8.4 or further geotechnical investigation may be more suitable for short-term loads.

Foundation load–deformation characteristics may be required, depending on the analysis method selected for evaluation of the foundation system. Even when the superstructure is analyzed using the fixed-base procedures, if the foundation demands are transferred to another software platform for further analysis of the foundation as a beam-on-elastic soil system, the soil stiffness properties are required. Soil stiffness values for short duration loading are typically much stiffer than those used in typical settlement calculations, which include immediate and long-term settlement. Secant stiffness moduli values are used when the analysis is linear, so an effective stiffness is warranted. Initial elastic stiffness is permitted to be used for nonlinear dynamic procedures when the soil stiffness varies as the analysis progresses. Starting with the secant stiffness results in artificially softer soil and could give inaccurate results.

C8.2.1.4 Soil Shear Modulus and Poisson's Ratio Parameters Equations (8-1) through (8-4) are obtained by Kramer

(1996) and Seed et al. (1986). The numerical coefficients in Equations (8-2) through (8-4) are different from the sources because the source equations were restricted to a specific system of units. Atmospheric pressure, p_a, has been included in the equations to make them dimensionally consistent and valid for any system of units.

Equation (8-5) is obtained from Salgado (2008). It was intended to be used to evaluate the effect of mean effective stress on the friction angle of the soil. Equation (8-5) is adopted to determine a reasonable spatially averaged mean effective stress for use in Equations (8-2), (8-3), and (8-4) for estimation of the shear modulus. The soil shear modulus determined from Equation (8-3) is based on the standard penetration $(N_1)_{60}$ value and based on the void ratio when determined using Equation (8-4). Mean effective stress refers to the average of the three principal effective stresses. Equation (8-4) underestimates the shear modulus if the shear wave velocity (v_{s0}) is measured before consolidation under the expected vertical loads on the footings. Because Equation (8-5) may sometimes produce estimates of the spatially averaged mean effective stress smaller than the overburden, and this was not considered reasonable, Equation (8-6) introduces a lower bound.

The reduction factors for shear modulus in Table 8-1 approximately account for the modulus reduction of the soil caused by nonlinearity associated with ground shaking.

Poisson's ratio for soil varies from 0.1 to 0.5 for dry to saturated states. The higher the Poisson's ratio, the stiffer the soil resistance to short-term dynamic loads. For saturated clays the upper-bound value is permitted, and an average value of 0.25 is used for other soil states.

The initial shear modulus, G_0, is derived from information provided in the geotechnical report such as the shear wave velocity, standard penetration test, or the effective vertical shear stress based on foundation dimensions. The effective shear modulus, G, can then be determined as a function of the ground shaking intensity. The ratio of G/G_0 is then used to calculate the effective secant stiffness properties of the soil to account for nonlinear effects based on ground shaking intensity. As a result, there will be a different effective shear modulus for each hazard level, BSE-1E, BSE-2E, or BSE-1N, BSE-2N.

For Site Classes D and E in areas of stronger ground shaking conditions, the soil stiffness values in the table may be too soft because the values represent fully degraded modulus characteristics for the given peak ground acceleration level. It is therefore permitted to perform site-specific studies to determine the effective shear modulus of the soil.

Where the nonlinear dynamic procedure is used with a flexible foundation, it is permitted to use G equal to G_0 where the soil response is modeled considering hysteretic stiffness and strength degrading effects and gap-growth characteristics.

C8.2.2 Seismic–Geologic Site Hazards Geologic site hazards are a function of the seismic hazard and the site conditions. Some hazards may only be relevant during very strong seismic shaking. Therefore, the hazards must be assessed under the same Seismic Hazard Level for which the building is being analyzed.

Initially there may be maps or other published reports to indicate that a specific site may be susceptible to earthquake-induced geologic hazards such as liquefaction, fault rupture, or landsliding. If there is any indication that there might be the potential for any of the geologic site hazards listed in this section at a building site, a geotechnical investigation that includes in situ sampling should be performed. The purpose of that in situ geotechnical investigation is to determine with greater accuracy the potential for and extent of geologic site hazards present.

C8.2.2.1 Fault Rupture Buildings that straddle active faults should be assessed to determine if retrofit is warranted, possibly to reduce the collapse potential of the structure given the likely amount and direction of fault displacement.

C8.2.2.2 Liquefaction Soil liquefaction is a phenomenon in which a soil below the groundwater table loses a substantial amount of strength and stiffness because of strong earthquake ground shaking or other rapid loading. Recently deposited (i.e., geologically young) and relatively loose natural soils and uncompacted or poorly compacted fill soils are potentially susceptible to liquefaction. Loose sands and silty sands are particularly susceptible; loose silts and gravels also can liquefy. Dense natural soils and well-compacted fills have low susceptibility to liquefaction. High-plasticity fine-grained soils are generally not susceptible, except for highly sensitive clays found in some geographic regions.

Liquefaction analysis of level or mildly sloping ground consists of the following steps: (1) liquefaction susceptibility based on soil characteristics and water table depth; (2) liquefaction triggering (or potential) based on soil capacity (liquefaction resistance) and seismic demand (cyclic stress ratio); and (3) evaluation of consequences of liquefaction, for example, lateral spreading and liquefaction-induced settlement. When a building is located adjacent to a slope or retaining structure, an analysis of liquefaction of sloping ground may be required. This process consists of the following steps: (1) liquefaction susceptibility to define contractive (strain-softening) soils; (2) liquefaction triggering; and (3) posttriggering stability.

Liquefaction susceptibility of level and mildly sloping ground. Specific soil and water conditions determine whether a soil is susceptible to being liquefied under rapid loading. These conditions include the following:

1. Deposit type and age: These criteria are described in Table 8-2.
2. Soil type: Generally, soils with plasticity indexes less than about 10 (coarse-grained gravelly sands, sands, silty sands, and nonplastic silts, as well as lean clayey silts and silty clays) are susceptible to liquefying, depending on the seismic demand.
3. Soil density or consistency: Coarse-grained, nonplastic soils are not susceptible to liquefaction if they are dense to very dense. Lean, fine-grained soils generally are not susceptible to liquefaction if they are stiff to hard (i.e., if they have low water content).
4. Depth to water table: Only saturated soils are susceptible to liquefaction. Furthermore, if the water table is considerably below the foundation or ground surface, liquefaction effects are unlikely to manifest at the surface or affect the overlying structure.

Liquefaction triggering using cyclic stress procedure for level and mildly sloping ground. The potential for liquefaction to occur depends on both the soil capacity (or liquefaction resistance) and the seismic demand. Although various methods are available to evaluate liquefaction triggering (Youd et al. 2001), the most commonly used approach is the empirical cyclic stress Idriss method, proposed by Seed and Idriss (1971) and Whitman (1971). The state of practice using the cyclic stress method is described by Youd et al. (2001). The current version of the procedure uses the standard penetration test (SPT) blow count, cone penetration test (CPT) tip resistance, or shear wave velocity (V_s) to evaluate liquefaction resistance, although SPT or the combination of CPT and SPT are widely preferred. Using penetration resistance (rather than shear wave velocity) to assess liquefaction potential is considered a reasonable engineering approach because many of the factors that affect penetration resistance affect liquefaction resistance of sandy soils similarly, and because the cyclic stress method is based on the observed performance of soil deposits during worldwide historical earthquakes (Youd et al. 2001, Cetin et al. 2004, Idriss and Boulanger 2008). Idriss and Boulanger (2008) provide an updated perspective on evaluation of triggering, consequences, and mitigation of soil liquefaction during earthquakes.

Lateral spreading of level and mildly sloping ground. Lateral spreads are ground-failure phenomena that can occur on level ground adjacent to shallow declivities (i.e., river banks) or mildly sloping ground (in general, slopes less than 6%) underlain by liquefied soil. Earthquake ground shaking affects the stability of mildly sloping ground containing liquefiable soils as the combined seismic inertia forces and static shear stresses exceed the strength of the liquefiable soils. Temporary instability manifests as lateral downslope movement that can potentially involve large land areas. For the duration of ground shaking associated with moderate to large earthquakes, there could be many such occurrences of temporary instability, producing an accumulation of downslope movement. The resulting movements can range from a few inches or less to tens of feet, and they are characterized by breaking up of the ground and horizontal and vertical offsets.

Methods to evaluate lateral spreading displacements include empirical, semiempirical, and numerical. The most widely used empirical procedure is that proposed by Bartlett and Youd (1992) and updated by Youd et al. (2002). This procedure estimates lateral displacements as a function of strength of shaking (magnitude and peak ground acceleration) and characteristics of loose sediments (thickness, grain size, and fines content of sandy soils with normalized SPT blow count less than 15). Other empirical methods include those proposed by Rauch and Martin (2000) and Bardet et al. (2002). Various semiempirical methods based on laboratory measurements of shear strain have been proposed by Ishihara and Yoshimine (1992), Zhang et al. (2004), and Idriss and Boulanger (2008). Olson and Johnson (2008) proposed a semiempirical method that uses a Newmark sliding-block analysis in conjunction with the liquefied shear strength ratio proposed by Olson and Stark (2002), which allows the use of site-specific ground motions to estimate lateral displacements. Updike et al. (1988), Egan et al. (1992), and USACE (1995) previously proposed similar approaches. In addition to these empirical and semiempirical procedures, more complex numerical deformation analyses can be performed using various constitutive models, including UBCsand (Beaty and Byrne 1998, Puebla 1999), Norsand (Jefferies and Been 2006), or the effective-stress model proposed by Yang et al. (2003). Idriss and Boulanger (2008) describe a method for integration of strain potential to determine a lateral displacement index for lateral spreading.

Liquefaction-induced settlement of level and mildly sloping ground. Liquefaction involves the generation of excess pore-water pressure. As these pore-water pressures dissipate, the liquefied soil reconsolidates and surface settlements occur. Differential settlements commonly occur because of lateral variations in soil stratigraphy and density. These differential settlements can be quite large, particularly when influenced by lateral spreading or bearing capacity failure. Settlements may range from a few inches, where thin layers liquefy, to a few feet, where thick, loose soil deposits liquefy.

Several semiempirical methods are available to estimate liquefaction-induced settlements, including Tokimatsu and Seed (1987), Ishihara and Yoshimine (1992), and Zhang et al. (2002). These methods are largely based on laboratory-measured volumetric or axial strains associated with pore-water pressure dissipation. Dashti et al. (2010) discuss the influence of shallow building foundations on liquefaction-induced settlements, but this approach is not routine and should be carried out by a geotechnical specialist.

Liquefaction-induced lateral earth pressures on level ground. Liquefaction of soils adjacent to building walls increases the lateral earth pressures against the wall. The lateral earth pressure can be approximated as a fluid pressure having a unit weight equal to the saturated unit weight of the soil plus the inertial forces on the soil equal to the hydrodynamic pressure by using the Westergaard procedure described in Ebeling and Morrison (1992) or another procedure.

Evaluating potential for flotation of buried structures below level ground. A common phenomenon accompanying liquefaction is the flotation of tanks or structures that are embedded in liquefied soil. The potential for flotation of a buried or embedded structure can be evaluated by comparing the total weight of the buried or embedded structure with the increased uplift forces occurring because of the generation of liquefaction-induced pore-water pressures.

Liquefaction susceptibility of sloping ground. Flow liquefaction can occur in liquefied soils subjected to static driving shear stress larger than the liquefied shear strength, for example, ground slopes greater than 6%, below embankments, or below building foundations, and can involve displacements ranging from a few feet to hundreds of feet or more. Liquefaction susceptibility in sloping ground involves evaluating whether contractive (strain-softening) soils are present below the structure and can be accomplished by comparing penetration resistance to threshold penetration resistances by using the thresholds proposed by Ishihara (1993), Baziar and Dobry (1995), or Olson and Stark (2003). If soils susceptible to liquefaction are not present, flow liquefaction is not possible.

Liquefaction triggering of sloping ground. If susceptible soils are present, liquefaction triggering analyses must be performed. Liquefaction triggering can be evaluated in terms of yield strength ratios, as proposed by Olson and Stark (2003), or by applying sloping ground and overburden stress corrections (K_α and K_σ, respectively) as proposed by Seed and Harder (1990), Seed et al. (2003), and Idriss and Boulanger (2008). However, these approaches involve considerable uncertainties and should be carried out by a geotechnical specialist.

Posttriggering stability. If liquefaction is triggered in sloping ground, the potential for a flow slide can be evaluated using conventional limit equilibrium slope stability using an approach that satisfies force and moment equilibrium (e.g., the Morgenstern and Price, Spencer, or generalized limit equilibrium methods). The liquefied soils should be assigned a liquefied shear strength for the stability analysis. The liquefied shear strength can be assigned using the recommendations from Seed and Harder (1990) or Olson and Stark (2002). Such calculations should be carried out by a geotechnical specialist.

Posttriggering bearing capacity failure. The occurrence of liquefaction in soils supporting foundations can result in bearing capacity failures and large, plunging-type settlements. In fact, any buildup of pore-water pressures in a soil still reduces soil strength (i.e., softens the soil) and decreases the bearing capacity.

The potential for bearing capacity failure beneath a spread footing or mat foundation depends on the depth of the liquefied (or softened liquefied) layer below the footing, the size of the footing or mat, and the applied load (including any eccentricity in the applied load). If lightly loaded small footings are located sufficiently above the depth of liquefied materials, bearing capacity failure may not occur. The foundation bearing capacity for a case where a footing or mat is located some distance above a liquefied layer can be assessed by evaluating using the liquefied shear strength (Seed and Harder 1990, Olson and Stark 2002), softened shear strengths, and/or drained or undrained shear strength of nonliquefied strata (as appropriate), then applying bearing capacity formulations for layered systems (e.g., Meyerhof 1974, Hanna and Meyerhof 1980, Hanna 1981). The capacity of friction pile or pier foundations can be similarly assessed based on the strengths of the liquefied, softened liquefied, and nonliquefied strata penetrated by the foundations. Such calculations involve considerable uncertainties and should be carried out by a geotechnical specialist.

Lateral earth pressures imposed by lateral spreading or flowing ground. During lateral spreading or flow failures, large lateral forces can be applied to building foundations, causing lateral movement of the structure or significant damage to pile foundations. There are no widely accepted methods to evaluate lateral spreading forces, although some techniques are available. As a result, such calculations involve considerable uncertainties and should be carried out by a geotechnical specialist.

C8.2.2.2.1 Liquefaction-Affected Structural Evaluation Soil liquefaction can significantly alter the ground motion that a building experiences, in addition to reducing the strength and stiffness of the soil supporting the building. To properly assess the implication of liquefaction, the structure should be analyzed first by assuming that liquefaction does not occur. This method provides for the upper-bound structural response and accounts for the fact that liquefaction may not occur during a seismic event, even if the site investigation indicates that the site has the potential for liquefaction.

The second analysis is intended to assess the performance of the structure during the seismic event while foundation soils are liquefied. During that response, the ground shaking, and thus the foundation input motions, are different than they would be if liquefaction did not occur. Also, the foundation strength and stiffness are reduced, which could lead to additional deformations in the structure, and they should be explicitly modeled and evaluated. However, there are no widely accepted methods to perform effective stress-based site response analysis, although some techniques are available. As a result, such calculations involve considerable uncertainties and should be carried out by a geotechnical specialist.

C8.2.2.2.2 Postliquefaction Structural Evaluation Differential settlement and lateral spreading caused by liquefaction can have significant effects on a structure. The movement of the foundation elements can pull the structure apart and cause local or global collapse. The structure's ability to accommodate such deformations of the foundation elements must be assessed.

The analysis in this section is similar to analyses used when assessing a building for progressive collapse caused by the loss of a column. In that type of analysis, a column or multiple columns are removed, then the structure is analyzed to assess how the loads redistribute and whether the deformations induced on the structural elements as the loads redistribute are within acceptable limits. In this analysis, which must explicitly account for the nonlinear behavior of the structure similar to a nonlinear static pushover analysis, settlement and lateral movements are imposed on a foundation element or groups of foundation elements. After that, the superstructure elements are checked to confirm that those elements designated as deformation controlled have deformations within acceptable limits, and those elements designated as force controlled are not stressed beyond their capacity. In addition, the analysis should confirm that no structural elements unseat as a result of the anticipated deformations.

Because the number of foundation elements affected by the liquefaction-induced differential settlement are different for each building and may even be different depending on the Seismic Hazard Level at which liquefaction is being considered, an explicit number of iterations of this type of analysis cannot be specified. The design professional, subject to the approval of the Authority Having Jurisdiction, must determine how many iterations are required based on the specific site characteristics and the building configuration. The number of iterations must sufficiently demonstrate that the building can perform within the acceptable bounds of the performance level being targeted in the evaluation or retrofit design.

C8.2.2.3 Settlement of Nonliquefiable Soils Settlement of nonliquefiable soils may accompany strong ground shaking and can be damaging to structures. In saturated soils, these settlements occur as a result of generation of some excess pore-water pressure and subsequent reconsolidation after shaking, whereas in dry sands, these settlements occur as a result of vibration. Types of soil susceptible to liquefaction (i.e., relatively loose natural soils, or uncompacted or poorly compacted fill soils) also generally experience differential settlement. Methods proposed by Tokimatsu and Seed (1987), Ishihara and Yoshimine (1992), and Zhang et al. (2002) can be used for nonliquefied saturated coarse-grained soils; methods proposed by Tokimatsu and Seed (1987) and Stewart et al. (2001) can be used for nonliquefied dry coarse-grained soils; and methods proposed by Stewart et al. (2004) can be used for nonliquefied fine-grained soils.

C8.2.2.4 Landsliding If no blocks of rock are present at the site but a cliff or steep slope is located nearby, then the likely performance of the cliff under earthquake loading should be evaluated. The earthquake loading condition for cliff performance must be compatible with the earthquake loading condition selected for the Performance Objective for the building.

Some sites may be exposed to hazards from major landslides moving onto the site from upslope, or retrogressive removal of support from downslope. Such conditions should be identified during site characterization and may pose special challenges if adequate investigation requires access to adjacent property.

Anderson et al. (2008) provide a method for one to determine a seismic coefficient and factor of safety for such analysis.

C8.3 MITIGATION OF SEISMIC–GEOLOGIC SITE HAZARDS

Opportunities exist to improve seismic performance under the influence of some site hazards at reasonable cost; however, some site hazards may be so severe that they are economically impractical to include in risk-reduction measures. The discussions presented in this section are based on the concept that the extent of site hazards is discovered after the decision for seismic retrofit of a building has been made. However, the decision to retrofit a building and the selection of a Performance Objective may have been made with full knowledge that significant site hazards exist and must be mitigated as part of the retrofit.

Possible mitigation strategies for seismic–geologic site hazards are presented in the following sections.

Fault rupture. If the structural performance of a building evaluated for the calculated ground movement caused by fault rupture during earthquake fails to comply with the requirements for the selected performance level, mitigation schemes should be used that include one or more of the following measures to achieve acceptable performance: stiffening of the structure and/or its foundation, strengthening of the structure and/or its foundation, and modifications to the structure and/or its foundation to distribute the effects of differential vertical movement over a greater horizontal distance to reduce angular distortion.

Large movements caused by fault rupture generally cannot be mitigated economically. If the structural consequences of the estimated horizontal and vertical displacements are unacceptable for any performance level, either the structure, its foundation, or both, might be stiffened or strengthened to reach acceptable performance. Measures are highly dependent on specific structural characteristics and inadequacies. Grade beams and reinforced slabs are effective in increasing resistance to horizontal displacement. Horizontal forces are sometimes limited by sliding friction capacity of spread footings or mats. Vertical displacements are similar in nature to those caused by long-term differential settlement.

Liquefaction. If the structural performance of a building evaluated for the calculated ground movement caused by liquefaction during an earthquake fails to comply with the requirements for the selected performance level, then one or more of the following mitigation measures should be implemented to achieve acceptable performance.

Modification of the structure. The structure should be strengthened to improve resistance against the predicted liquefaction-induced ground deformation. This solution may be feasible for small ground deformations.

Modification of the foundation. The foundation system should be modified to reduce or eliminate the differential foundation displacements by underpinning existing shallow foundations to achieve bearing on deeper, nonliquefiable strata or by stiffening a shallow foundation system by a system of grade beams between isolated footings, or any other approved method.

Modification of the soil conditions. One or more of the following ground improvement techniques should be implemented to reduce or eliminate the liquefaction under existing buildings: soil grouting (either throughout the entire liquefiable strata beneath a building or locally beneath foundation components), soil mixing, installation of drains, or installation of permanent dewatering systems.

Other types of ground improvement widely used for new construction are less applicable to existing buildings because of the effects of the procedures on the building. Thus, removal and replacement of liquefiable soil or in-place densification of liquefiable soil by various techniques is not applicable beneath an existing building.

Mitigation of lateral spreading. Large soil volumes should be stabilized, and/or buttressing structures should be constructed.

If the potential for significant liquefaction-induced lateral spreading movements exists at a site, then the mitigation of the liquefaction hazard may be more difficult. This difficulty occurs because the potential for lateral spreading movements beneath a building may depend on the behavior of the soil mass at distances well beyond the building and immediately beneath it.

Differential settlement compaction. If the structural performance of a building evaluated for the calculated differential settlement during earthquake fails to comply with the requirements for the selected performance level, then one or more mitigation measures similar to those recommended for liquefaction should be implemented to achieve acceptable performance.

Landslide. If the structural performance of a building evaluated for the calculated ground movement caused by landslide during earthquake fails to comply with the requirements for the selected performance level, then one or more of the following mitigation measures should be implemented to achieve acceptable performance:

1. Regrading,
2. Drainage,
3. Buttressing,
4. Structural improvements,
5. Gravity walls,
6. Tieback–soil nail walls,
7. Mechanically stabilized earth walls,
8. Barriers for debris torrents or rock fall,
9. Building strengthening to resist deformation,
10. Grade beams,
11. Shear walls, and
12. Soil modification or replacement (grouting and densification).

Flooding or inundation. If the structural performance of a building evaluated for the effects of earthquake-induced flooding and inundation fails to comply with the requirements for the selected performance level, then one or more of the following mitigating measures should be implemented to achieve acceptable performance:

1. Improvement of nearby dam, pipeline, or aqueduct facilities independent of the building;
2. Diversion of anticipated peak flood flows;
3. Installation of pavement around the building to reduce scour; and
4. Construction of a seawall or breakwater for tsunami or seiche protection.

C8.4 SHALLOW FOUNDATIONS

It is assumed that foundation soils are not susceptible to significant strength loss caused by earthquake loading from overturning and sliding actions. In general, soils have considerable ductility unless they degrade significantly in stiffness and strength under cyclic loading, which can occur during rocking generated by overturning action on the foundation when the critical contact area as defined in Section 8.4.4.1 exceeds approximately one-half of the total footing area.

Foundation overturning is action that causes rotation, plowing of the tip of the footing into the soil, plunging, or vertical displacement (settlement or uplift) of the foundation, or a combination thereof. *Foundation overturning* is differentiated from *overturning* itself as defined in Chapter 1, which is an action that determines the axial tension demand on the connection of the superstructure to the foundation. With this assumption, the provisions of this section provide an overview of the requirements and procedures for evaluating the ability of foundations to withstand the imposed seismic loads without excessive deformations.

C8.4.1 Selection of Evaluation Procedures There are three overarching procedures (Sections 8.4.3, 8.4.4, and 8.4.5) that can be used to evaluate the foundation system (Figure C8-1). Many cases are covered by the simplified procedure, in Section 8.4.3. For the remaining cases, the decision of how to model foundation fixity is paramount to an accurate analysis.

Foundation Degree of Fixity: Fixed versus Flexible. The determination of whether foundation movement is an important consideration for a myriad of building types and configurations is a complicated and complex question that it is impractical to develop prescriptive code provisions, similar to those of ASCE 7, for each case. The cases, as defined in Section 8.4.1, where fixed-base assumptions are able to capture accurate performance outcomes include

1. Partial retrofits as permitted by Section 2.4.5 where the foundation or effects of foundation flexibility are not

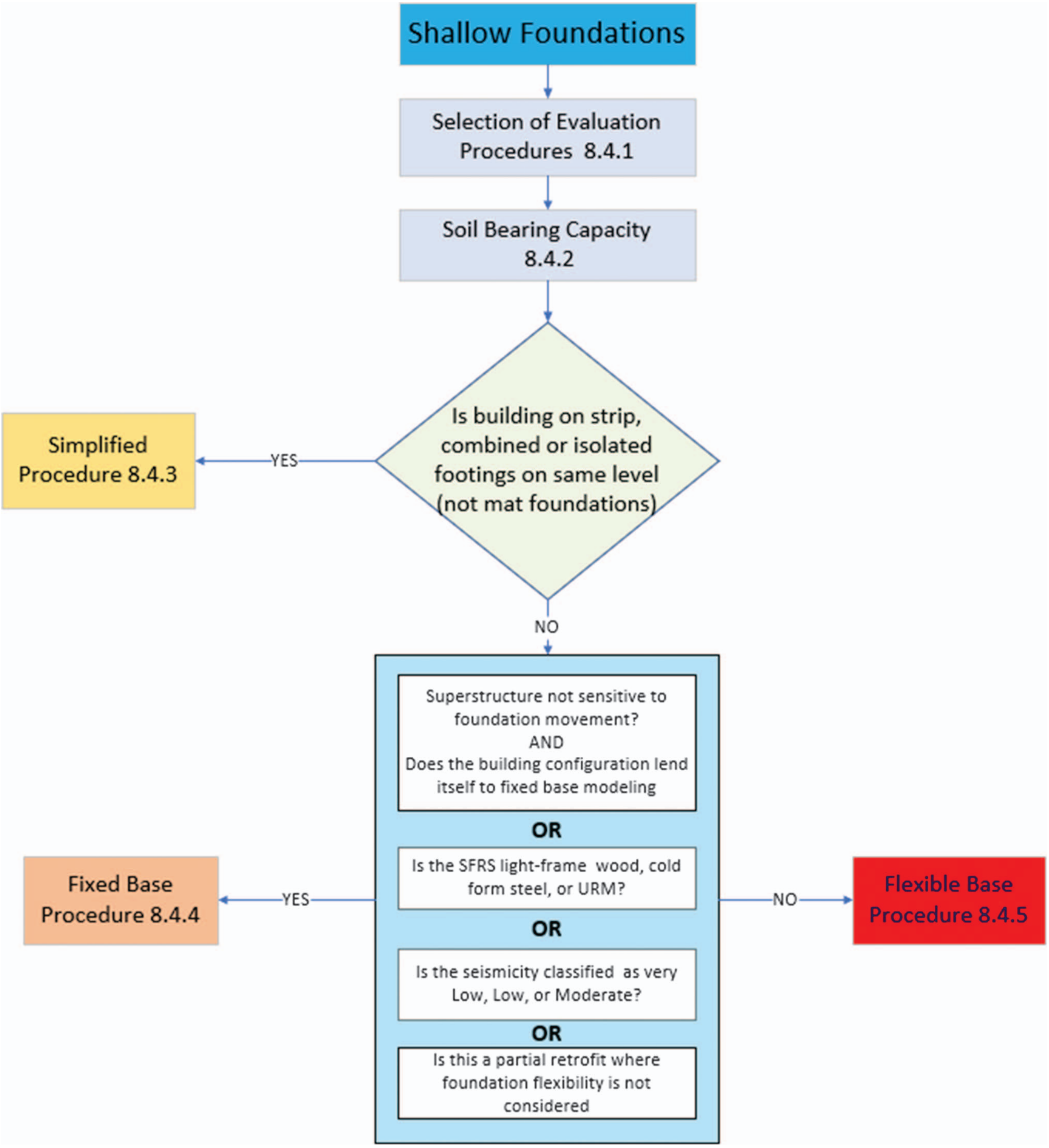


Figure C8-1. Foundation evaluation procedures for buildings on shallow foundations.

considered as part of the Performance Objective. These partial retrofits can be either voluntary or mandatory. In general, Tier 2 deficiency-only retrofits or Tier 3 evaluations would lend itself to fixed-base modeling where the building configuration does not dictate flexible-base modeling.

2. Buildings located in Very Low, Low, or Moderate seismicity from Table 2-6 since modeling the foundation flexibility may not significantly impact the deformations of the superstructure when the ground motion is relatively small.
3. Building of light frame construction (wood or cold-formed steel), where the superstructure is flexible relative to the foundation.
4. Unreinforced masonry buildings, where yielding/failure of the superstructure is likely to occur prior to where the rotation of the foundation due to soil yielding is a concern.
5. Box-stepped foundations on gradually sloping sites of wood or light frame construction, which have level top of foundation walls as shown in Figure C8-2.

There are buildings however that do not lend themselves to fixed-base modeling. Foundation deformations that can cause additional inelastic deformations in the superstructure may not be assessed in a fixed-based model. Potential collapse mechanisms of superstructure secondary components may be overlooked. These cases where fixed-base procedures are not recommended are subdivided into two categories:

1. Buildings where foundation types and configuration vary significantly. Modeling of foundation lateral and/or vertical flexibility is recommended for

    (a) Buildings where the superstructure lateral-force-resisting elements are simultaneously supported on deep and shallow foundations: Overturning deformations of the lateral-force-resisting elements supported on deep foundation elements, which resist uplift or compression, may differ significantly from the overturning deformations of lateral-force-resisting elements supported on shallow foundations. This can cause differential settlements that affect the force and deformations in the superstructure.

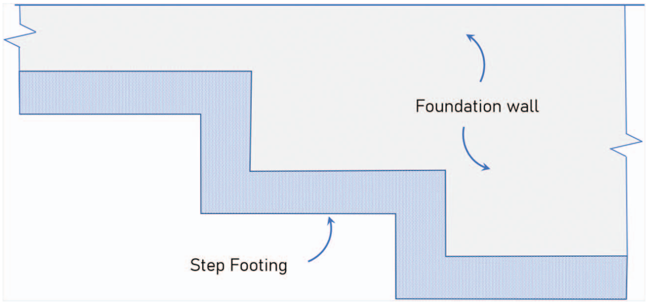


Figure C8-2. Stepped footing where lateral and vertical soil flexibilities are not required to be modeled.

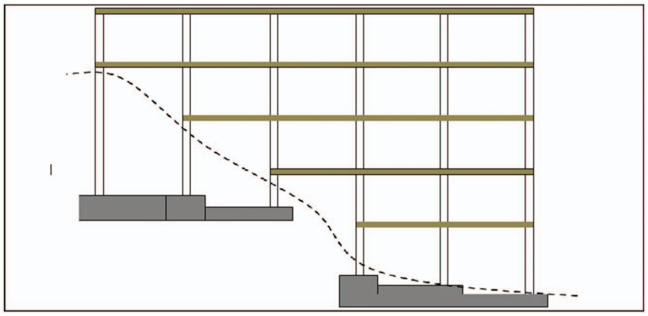


Figure C8-3. Footing elevations at different heights; lateral and vertical soil flexibilities are required to be modeled.

    Where nearly all the lateral forces are resisted by either the shallow or deep foundation elements, it is expected that superstructure demands owing to differential settlement, or differences in strength and/or stiffness, can be accommodated by redistribution of forces without distress such that the targeted Performance Objective is maintained while using a fixed-base model. One such case may be where a seismic retrofit consists of adding micropiles or deep foundation elements to resist overturning solely. Nonbattered micropiles resist mostly rocking action and provide little lateral resistance such that the differential lateral movement is limited, and superstructure deformations caused by rocking behavior are considered small.

    (b) Buildings with nontrivial elevation difference of the bottom of unconnected footings that support lateral-force-resisting elements (Figure C8-3). When this type of building support condition exists, a fixed-base model will report the lateral force demands resisted mostly at the highest fixed foundation level. Adding flexibility into the model will more adequately predict and distribute the lateral force demands.

    When most of the building is supported at the higher elevation (first floor in Figure C8-4) and gravity loads to the lower elevation (ground floor) are a minimal portion of the total weight of the building, a fixed-base procedure may be used. The lateral demands may effectively be resolved by the foundations and superstructure at the upper level, so a fixed-base procedure can be used.

2. Buildings where superstructure elements are sensitive to base rotations include

    (a) Cantilever shear wall or braced frame buildings with different wall or bracing heights. Buildings with such configurations as shown in Figure C8-5 make it potentially difficult to get a reasonable estimate of the relative stiffness and load distribution with a fixed-base model.

    There are however conditions where the superstructure response is minimally affected by foundation flexibility modeling assumptions and modeling the foundation as a fixed base is acceptable. Such conditions are exemplified when

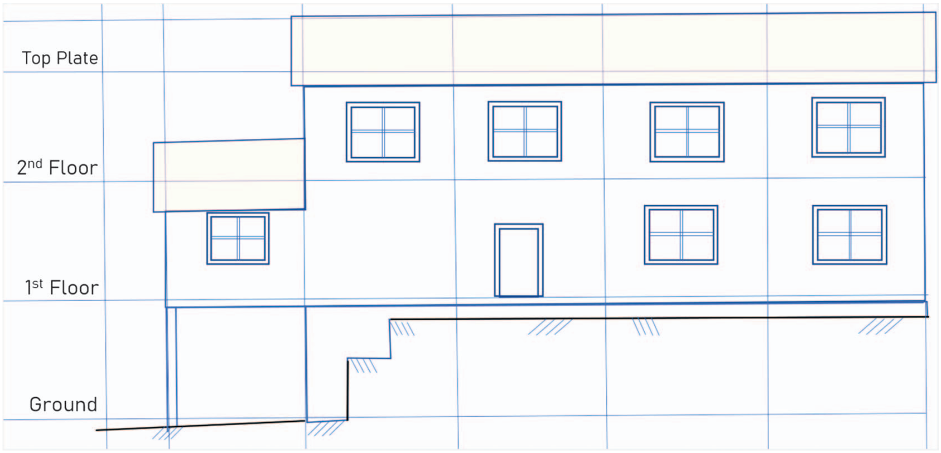


Figure C8-4. Building with a modest portion at a different foundation elevation.

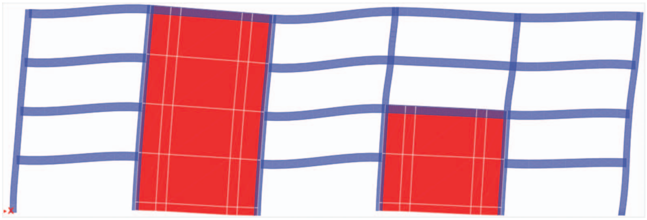


Figure C8-5. Building with lateral-force-resisting elements of different heights.

- The shorter wall/bracing elements resisting system resist a small portion of the base shear of the building, implying that the taller more flexible elements still resist a significant majority of the overturning demand on the structure.
- The foundation overturning capacity is greater than the lateral yield capacity of the short wall or short braced frame.
- The building has a full basement with a nearly complete system of perimeter basement walls, at least one-story in height, that can resist the overturning and sliding actions. Where there is a complete load path with an abundance of basement strength and stiffness, the inelastic deformations of the superstructure will be structurally insignificant from foundation movement.
- Gravity elements can accommodate additional drift caused by foundation movement, such as heavy timber beam-column connections or non-moment steel connections.
- The building has flexible diaphragms where the redistribution of loads to shorter walls through the diaphragm is limited.
- The shorter walls uniformly distribute the stiffness and prevent a soft or weak story irregularity.

(b) Buildings where the seismic-force-resisting system consists of full height cantilever shear walls, coupled shear walls, or stiff braced frames. Unless the limitations of Sections 8.4.3 or 8.4.4 are met, fixed-base procedures may not be appropriate for these systems because foundation flexibility significantly affects the stiffness of these seismic-force-resisting systems and/or changes the displacement pattern with height, therefore, demands on secondary elements in the superstructure may be underestimated. For example, fixed-base models for concrete shear walls on independent spread footings may minimize deformation demands on the walls themselves but could underestimate the demands on other secondary components in the buildings, such as beams and columns in moment frames, which may be sensitive to additional building movement as shown in Figure C8-6, unless the independent spread footings are sufficiently stiff and strong to justify the fixed-base modeling, or the secondary components can accommodate additional movement not accounted for in the fixed-base model. Examples of buildings with secondary components that are not sensitive to movement may include heavy timber beam/column connections or non-moment steel connections.

Flexible-base models provide a better force distribution in the superstructure than a fixed-base model when appropriate soil properties and modeling techniques are used. In some cases, flexible-base foundation models are able to provide both improved superstructure demands and a more efficient result. Nonlinear modeling where foundation rocking occurs has been shown to demonstrate better performance of the building by resulting in lower demands on the primary seismic-force-resisting elements, provided that secondary elements have sufficient ductility to accommodate the additional foundation movement as shown in Figure C8-7.

Regardless of whether a fixed-base procedure is permitted, it may still be prudent to consider including foundation flexibility in conditions where the superstructure deformations are sensitive to base rotations, in particular, for enhanced Performance Objectives and where improved reliability may be desired.

C8.4.2 Expected Soil Bearing Capacities In the past, geotechnical engineers tended to make conservative assumptions to determine the soil bearing capacities of soil for foundation design. Traditionally, a factor of 3 was often used as a minimum acceptable factor of safety against soil bearing failure. In many cases, however, foundation dimensions are controlled by settlement, not capacity, considerations. If allowable pressures were controlled by long-term settlements, then allowable pressures may be much smaller than expected capacities under dynamic loading situations. It is important to obtain information on actual factors of safety in the determination of the expected capacities. This result may be obtained from prescriptive methods (Section 8.4.2.1), past geotechnical reports, or based on new site-specific geotechnical investigations, such as in situ plate bearing testing or near full-scale foundation element testing. Because the prescriptive soil bearing values or the values determined from soil exploration and testing are based on information from the construction documents or site-specific testing, respectively, the knowledge factor, κ, is set equal to 1.0.

In projecting expected capacities and load–deformation characteristics, it is also important to understand the soil bearing pressures that the foundations are exhibiting under the building gravity loads or have experienced during past seismic loading conditions and whether the foundations have performed adequately.

C8.4.2.1 Prescriptive Expected Soil Bearing Capacities When the allowable soil bearing pressures for dead load plus live load used to design the foundation are indicated either on the drawings or on previous geotechnical reports, the prescriptive soil bearing capacities can be estimated assuming that a factor of safety of 3 was used. Typically allowable soil bearing values for dead and live loads only are based on limiting settlement, and those allowable capacities that include transient loads such as wind or seismic load include a stress increase; therefore, the values for dead plus live load should be used with Equation (8-7) in this section.

Equation (8-8) provides a means by which one can estimate the soil bearing capacities when there is no information on the original design foundation values or original geotechnical report.

This method is based on the fact that foundations traditionally have been designed with a factor of safety of 3 for gravity loads. Therefore, given the absence of any visible distress in the superstructure owing to differential foundation settlement, it is reasonable to assume the factor of safety of 2.5 on the sustained dead load plus 0.4 of the unreduced live load. The previous factor of 1.5 Q_G in ASCE 41-17 was judged to be too conservative and not consistent with the factors of safety used in the design. The soil bearing capacity q_c for the foundation is the average value of the sustained dead load plus 0.4 of the unreduced live load on the foundation system divided by the sum of the areas of all the footings supporting the load. The prescriptive expected soil bearing capacity may also be calculated based on the axial load on the individual footings.

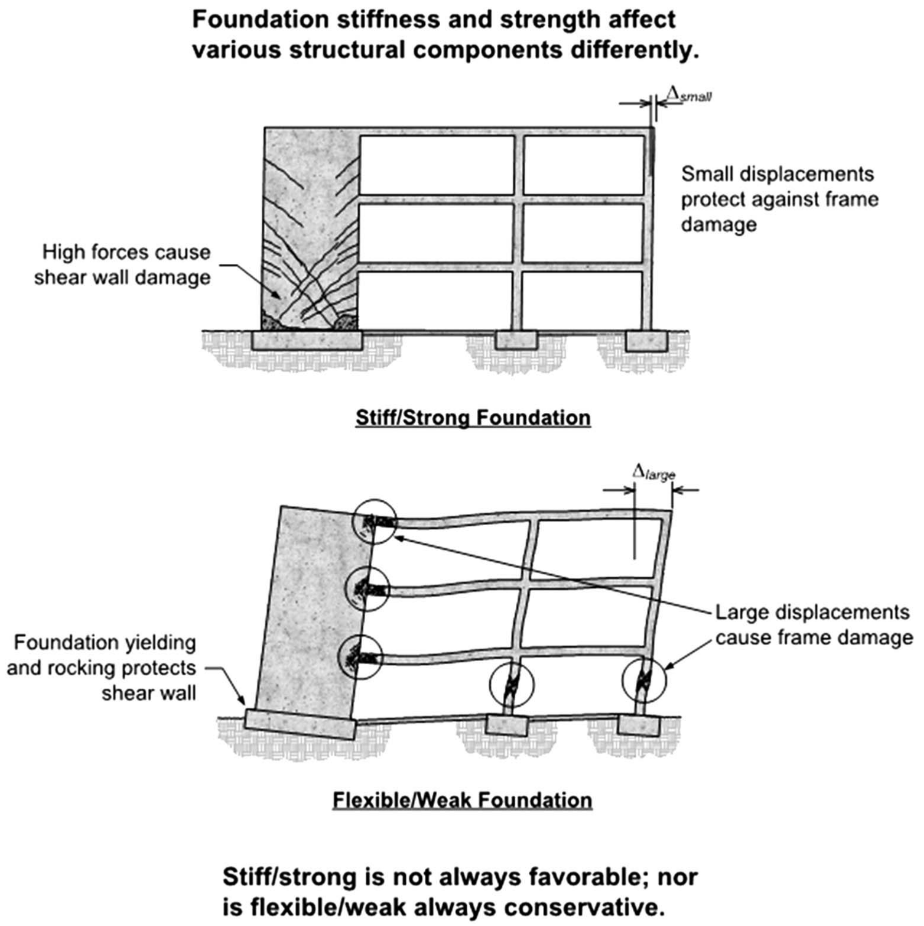


Figure 2-6 The significant impact of soil flexibility on a reinforced concrete shear wall system (from ATC, 1996).

Figure C8-6. Impact of Soil Flexibility on Reinforced Concrete Shear-Wall System.
Source: FEMA P-2091, Figure 2-6 (ATC 1996).

The expected short duration loading soil strength q_{cDA} has a factor of two amplification from the prescriptive soil bearing capacity q_c under static loads to account for the strength increase in soils because rate effects and soil bearing capacities are usually determined based on settlement and not the ultimate short-term soil bearing capacity. This amplification factor can range anywhere from about 1.3 to 2.0, depending on the type of soil and moisture content. The upper-bound value was used for consistency with the existing provisions and m-factors used in the acceptance criteria. This amplification factor of 2 replaces the upper-bound factor for ultimate soil bearing capacity and is different from the customary one-third increase for seismic when allowable soil bearing pressures are provided.

Additional information on typical allowable soil bearing capacities for various subsurface conditions can be found in NAVFAC DM-7.01 (NAVFAC 1986b) and NAVFAC DM-7.02 (NAVFAC 1986a). Those referenced allowable values can be adjusted by Equation (8-7) and compared with what is obtained through the use of the method in Section 8.4.2.1, Item 2 to confirm the reasonableness of this method.

These provisions are not intended to be used in lieu of a subsurface geotechnical site investigation (where available or otherwise required) nor proof or verification load tests, as may be required to establish the capacity of new or existing foundations. This includes foundation systems that are specified based on structural performance criteria (e.g., minimum strength and stiffness), such as micropiles, and subject to proof and/or verification testing at the building site to establish the dependable capacity. The expected capacity used in evaluation and design of such foundation systems should not exceed that established through such building and site-specific methods.

C8.4.2.2 Site-Specific Capacities The geotechnical site investigation should focus attention on the regions of soil below and near the foundations at locations where the strength of the soil is expected to be mobilized.

C8.4.3 Simplified Procedure A simplified procedure is introduced in ASCE 41-23 for evaluation of the foundation system of buildings on shallow foundations on relatively level ground, without getting into the complexities of the rest of the chapter, or for sloping sites with slopes greater than 10%. In this procedure, where the foundation consists of strip footings supporting gravity and lateral loads at multiple locations along the footing, the foundation should be discretized into rectangular segments supporting the elements of the lateral-force-resisting system without consideration of the bends at corners or other foundation plan geometric irregularities, as shown in Figure 8-1. Dividing the strip footing into individual segments allows the

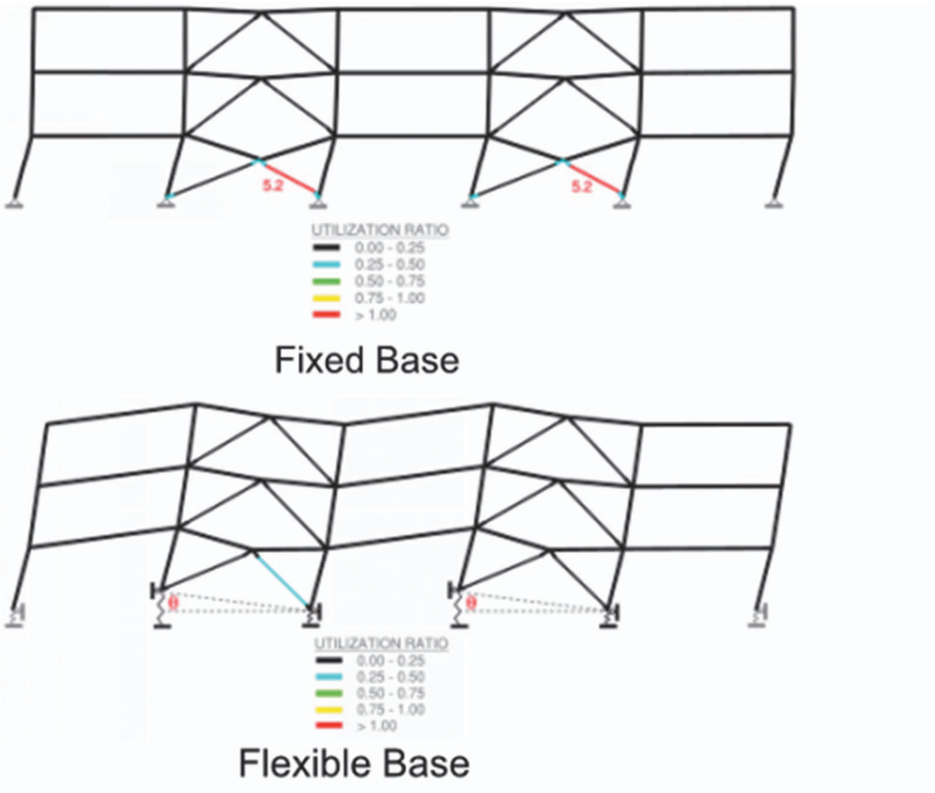


Figure 2-5 The significant impact of soil flexibility on a coupled braced frame system (from FEMA, 2019). Utilization ratio is the acceptance ratio as defined in FEMA (2018) and is similar to the demand-to-capacity ratio.

Figure C8-7. Impact of Soil Flexibility on a Coupled Braced Frame System.
Source: FEMA P-2091, Figure 2-5 (ATC 1996).

user to conservatively and quickly evaluate the adequacy of the foundation using Equation (8-10) without a more accurate estimate of the axial load on the foundation as required by Section 8.4.4.1.1, Equation (8-14).

Soil acceptance is based on gravity axial load and foundation overturning demand on the segment of footing being less than *m*-factor times the moment capacity of that footing. The structural integrity of the footing is determined based on the ability of the footing to resist demands using a bearing pressure q_c under the footing. Footing acceptance is evaluated depending on the action (moment or shear) on the footing with the requirements in the material chapters. Restoring shears and moments should be applied to the ends of the foundation segments when checking reinforcement to get an accurate representation of the moment and shears on the foundation segment.

Although there is a discrepancy between the requirements for overturning stability or soil bearing and the evaluation of the footing, case studies have shown this approach to give reasonable outcomes. The demands on the foundation are the pseudo seismic forces demands and do not reflect any reduction resulting from inelastic deformations in the superstructure. In addition, concentration of the resisting soil pressure applied at the ends of the foundation segment results in the maximum demand at the critical sections of that foundation segment.

Because the foundation moment capacity is dependent on the applied axial load on the footing, this procedure is not permitted when the pseudo seismic axial demand on individual isolated footings exceeds 0.2 times the gravity load on the footing (Figure C8-8). For these conditions the foundation should be evaluated using the fixed-base or flexible-base procedures.

C8.4.4 Fixed-Base Procedure In many cases, the foundation flexibility is not modeled explicitly. For these cases, two things must be considered: global overturning stability and yielding of the footing at the soil–foundation interface. The provisions in this section are intended to supplant the global overturning stability check in Section 7.2.9. The foundation is assessed on a component level, and the overall stability of the gravity load and seismic-force-resisting system is deemed adequate by means of satisfying the component action assessment for the foundation soil and restoring dead load. The acceptance criteria (*m*-factors) are limited such that localized displacements at the soil–foundation interface can occur without introducing structurally significant deformations where P-Δ or deformation compatibility becomes an issue.

When a fixed-base model is permitted to be used, the demands on the soil and forces counteracting potential uplift of the foundation must be checked per this section to determine if there is excessive deformation occurring caused by yielding of the soil or uplift of the foundation. This check is performed with the *m*-factors provided and using approximate expected vertical loads to determine the moment capacity, because overturning strength is dictated by the level of axial load present during the earthquake and soil bearing capacity under the foundation. If the

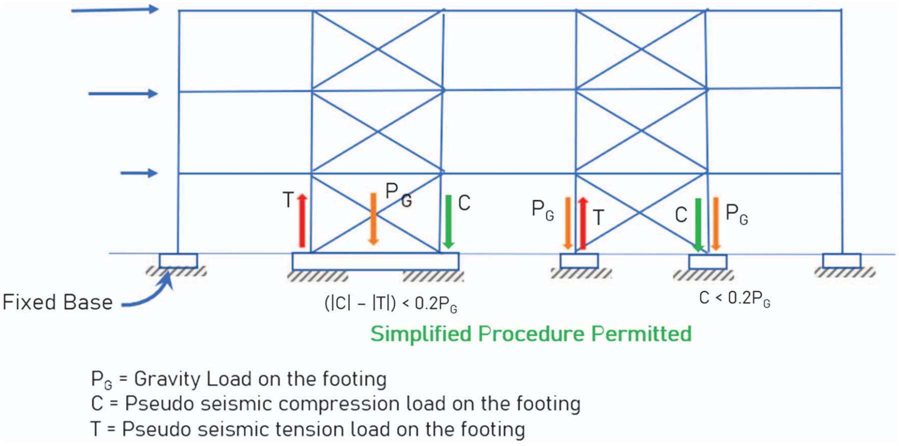


Figure C8-8. Example showing when the simplified procedure may be used as a function of the pseudo-seismic axial load.

foundation overturning demand does not exceed the *m*-factor augmented moment capacity and the earthquake uplift load does not exceed the *m*-factor augmented dead load, then no further analysis is required. However, when the acceptance criteria are not satisfied, there is potential for increased localized vertical deformation at the soil–foundation interface that could affect the behavior of the building through additional imposed drifts on the gravity framing system or through transfer of load to other seismic-force-resisting elements. For those instances, either the flexibility of the foundation should be modeled using a flexible base and *m*-factors in Table 8-7 or the foundation should be proportioned to be large enough that the requirements of this section are satisfied. For cases where the foundation supporting a frame consists of multiple isolated footings coupled by the superstructure above, the footing area A_f may be taken as the summation of all the frame footings, and the expected vertical load P_{UD} may be calculated as the cumulative sum of the vertical forces acting on all the footings.

The deformation compatibility checks that one gets with a flexible-base foundation are more important for existing buildings because there may not be explicit mitigation measures for deformation compatibility that are typically provided by the prescriptive detailing requirements in building code provisions for new buildings. Ideally, the flexibility at the soil–foundation interface should be included in the analytical model to capture potential stiffness modifications for the structural system and to represent more accurate dynamic characteristics and acceleration demands. For conditions where significant loading at the soil–foundation interface may lead to vertical settlement the lower *m*-factors derived from Table 8-7 would apply when $A_c/A > 0.4$. Provisions in subsequent sections of this chapter address modeling the soil–foundation interface as a flexible base.

C8.4.4.1 Linear Procedures When linear procedures are used for the evaluation of the building, loads to the foundation are obtained from procedures used from Chapter 7.

C8.4.4.1.1 Isolated Spread Footings Depending on the type of foundation, and the rigidity of the foundation relative to the soil, an appropriate evaluation procedure is selected. The flowchart (Figure C8-9) shows the various options that could be used to evaluate the foundation using linear procedures where the building is modeled using a fixed-base assumption.

C8.4.4.1.1.1 Foundation Overturning Moment Capacity The general approach to determining overturning moment capacity for any footing is provided at the start of the section, followed by guidance on specific cases.

The general method to determine the overturning moment capacity, M_{CE}, for all footings, including nonrectangular and I-shaped footings, can be obtained by first determining the critical contact area, A_c, and integrating the product of the bearing capacity times the distance from the centroid of the axial load on the footing over the critical contact.

For unidirectional loading in one of the orthogonal directions, *x*- or *y*-, of the footing, M_{CE} is determined using principles of mechanics taking the summation of the overturning resistance about the centroid of the contact area.

The expected axial seismic load may be determined from several methods, and often the simpler method may overestimate the axial load and require more detailed investigation. The expected gravity axial load at the soil footing interface need not be bounded by the load factors of Equations (7-1) and (7-2). The derivation of $DCR_{\max}$, as defined in Section 8.4.4.1.1.1, is based on a level of force reduction in recognition of yielding occurring in the load path to the foundation, in which case the actual forces being imposed on the foundation are less than the unreduced, pseudo-elastic force level. If this analysis approach results in a calculation that is deemed too conservative, either a more detailed, accurate capacity-based evaluation can be performed, or the engineer should use a more detailed analysis, such as flexible-base or nonlinear analysis.

C8.4.4.1.1.1.1 Rectangular Footings Equation (8-14) is derived from the general equation for a rectangular footing. The overturning capacity is calculated based on the summation of moment resistance about the center of the soil bearing compression block as shown in Figure C8-10.

In cases where the footing provides overturning resistance beyond the boundaries of the individual footing in question (Figure C8-11), such as with a continuous grade beam or spandrel beam just above grade, principles of mechanics are used to determine the overturning moment capacity by summing all restoring actions including gravity loads and the capacity of the footings:

$$\Sigma M_A = 0 \rightarrow M_{CE} = P_U\left(\frac{L_f}{2} - \frac{L_c}{2}\right) + V_{GB1}\left(L_f - \frac{L_c}{2}\right) + V_{GB2}\left(\frac{L_c}{2}\right) \qquad \text{(C8-1)}$$

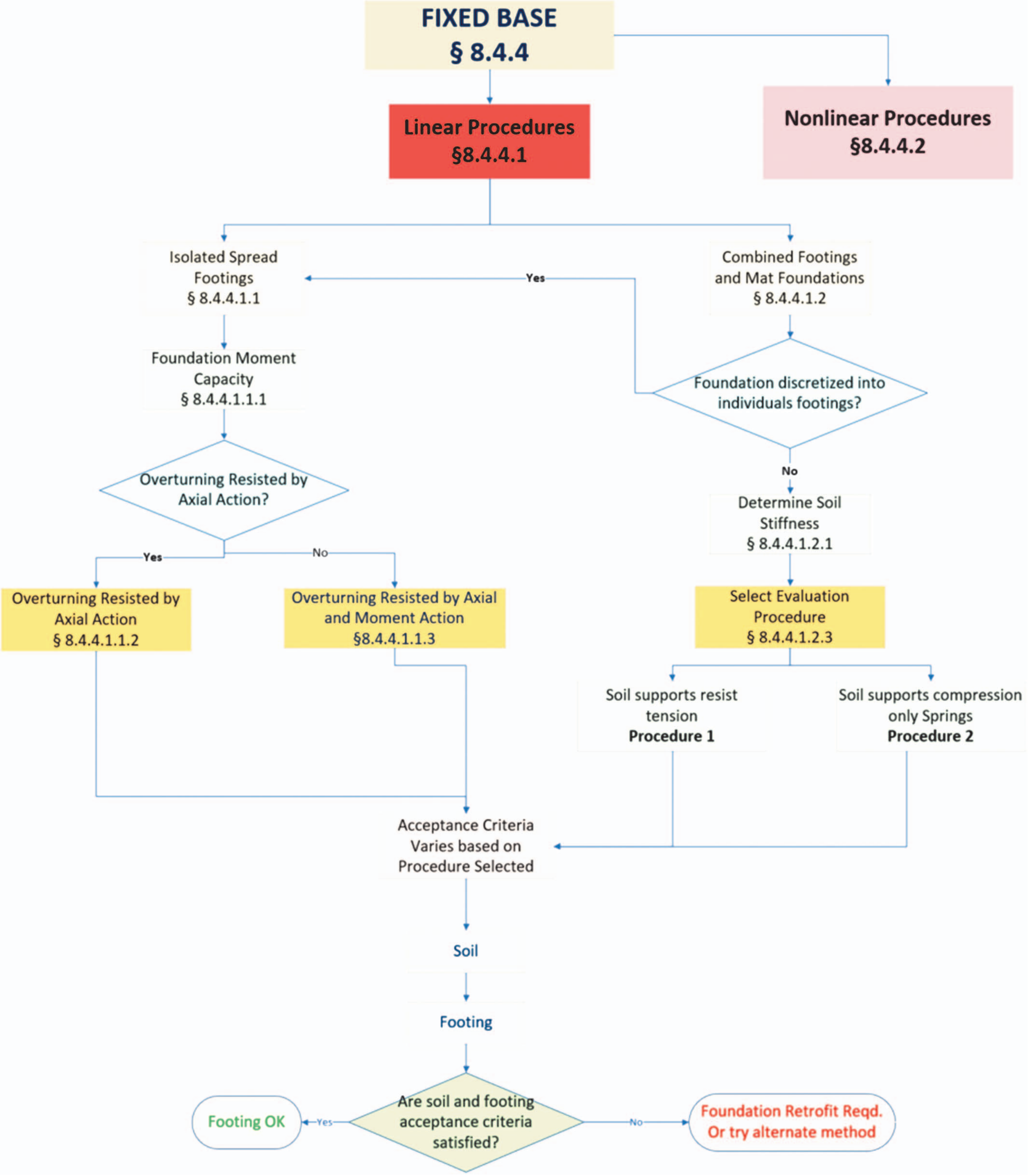


Figure C8-9. Flowchart for evaluation of foundations using linear procedures for buildings modeled as a fixed base.

For shallow strip or isolated spread footings supporting multiple structural members, either principles of mechanics or explicit mathematical modeling can be used to evaluate the overturning demand and capacity.

Where principles of mechanics are used, and the combined footing is rigid relative to the soil for foundations supporting multiple structural members, the overturning demand action, M_{UD}, is calculated as the sum of individual overturning axial forces on each member. The expected gravity load, P_U, is the sum of vertical loads on each member. The resisting moment capacity is the summation of all restoring loads multiplied by their eccentricity to the center of rotation (Figure C8-12).

$$M_{CE} = P_{U1}\left(L_1 - \frac{L_c}{2}\right) + P_{U2}\left(L_2 - \frac{L_c}{2}\right) + P_{U3}\left(L_3 - \frac{L_c}{2}\right) + V_{GB1}\left(L_f - \frac{L_c}{2}\right) + V_{GB2}\left(\frac{L_c}{2}\right) \tag{C8-2}$$

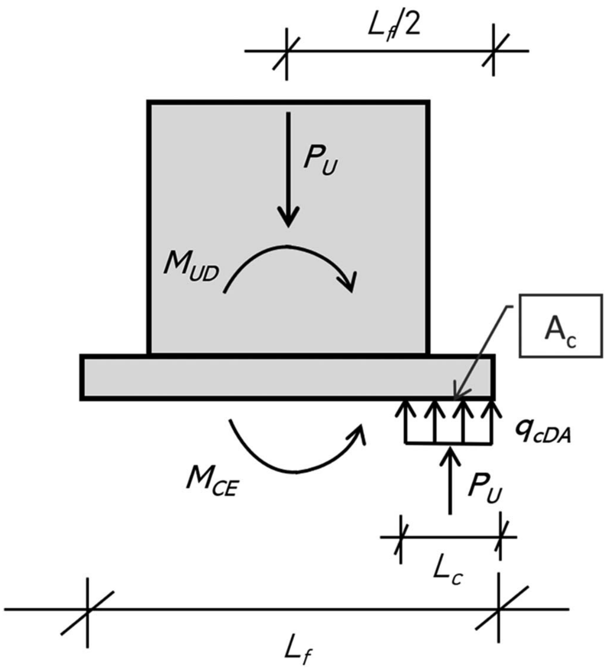


Figure C8-10. Overturning and resisting forces on an isolated spread footing.

C8.4.4.1.1.1.2 I-Shaped Footings For I-shaped footings, Equation (8-14) is valid for rocking about the major principal axis, provided the length L_c calculated using Equation (8-16) is less than the length of the flange. Where L_c exceeds the flange length, an adjustment is required to calculate the moment capacity accounting for the reduced width beyond the length of the flange required to resist the axial load P_U on the footing.

C8.4.4.1.1.1.3 All Other Footings and Footings with Bidirectional Moment For foundations with loading in each orthogonal direction (*x*- and *y*-directions) (Figure C8-13), overturning acceptance criteria is satisfied when the requirements of Equation (8-21) or Equation (8-23) are met. From Equation (8-23), acceptance is satisfied when the sum of the squares of the acceptance in each orthogonal direction *x* and *y*, and from Equation (8-21), the total overturning moment divided by $m\kappa$ is compared with the total moment capacity in the resultant bending direction. Equation (8-21) gives a good approximation of the acceptance ratio (AR) but tends to be slightly on the unconservative side for AR less than 1.0 but accurate when the AR = 1.0.

For irregular plan shaped combined footings, such as shown in Figure C8-14, even with the application of uniaxial overturning moments, it will almost always result in nonrectangular soil bearing pressure areas.

In Equation (8-21), the overturning moment demands in one of the orthogonal directions, *x* or *y*, is first reduced by the *m*-factor for overturning, and the moment capacity in the orthogonal

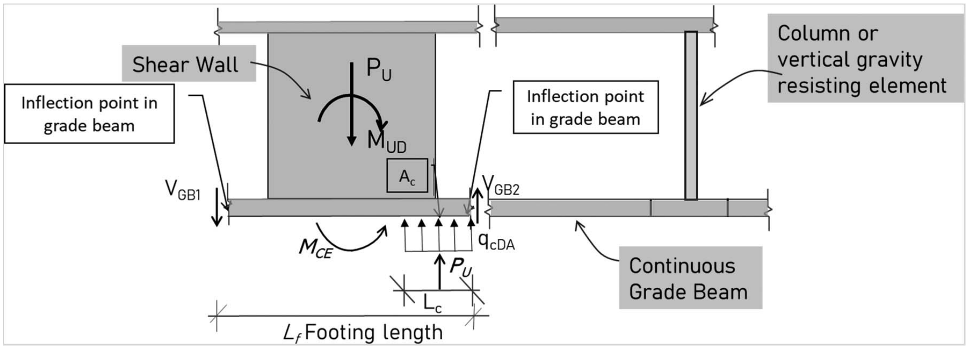


Figure C8-11. Overturning and resisting forces on an isolated spread footing with grade beam resistance.

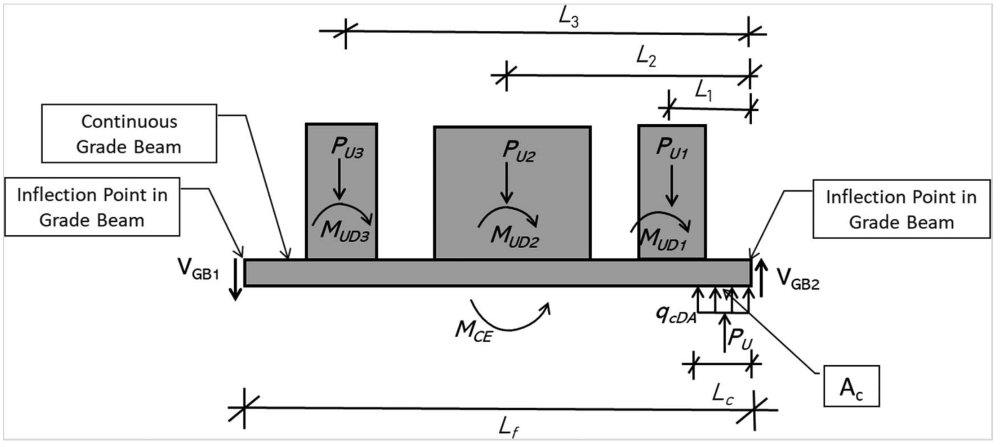


Figure C8-12. Overturning and resisting forces from multiple structural members on an isolated spread footing segment with grade beam resistance.

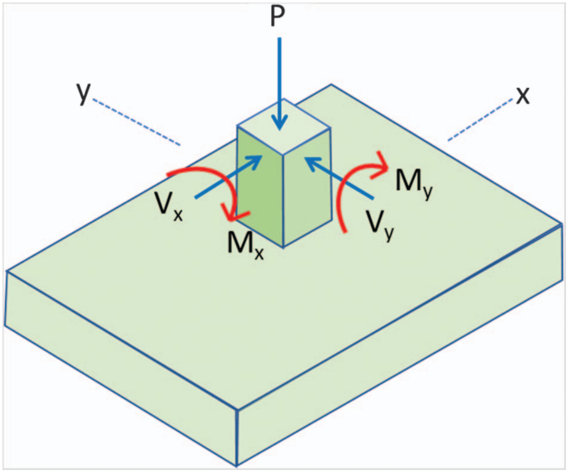


Figure C8-13. Foundations with loading in two orthogonal directions, *x* and *y*.

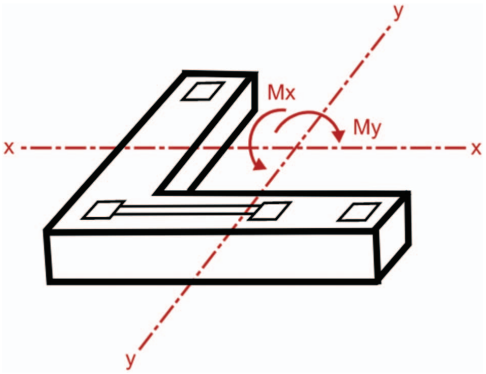


Figure C8-14. Footing under inherent biaxial loading.

direction is derived by simultaneously solving the equations of equilibrium for axial load and moment to define the boundaries of the critical contact area.

If the soil pressure block under the footing resisting the applied loads is discretized into individual segments, the moment capacity is a sum of the volume of soil pressure for the area multiplied by the distance of the centroid of the individual soil pressure blocks from the centroid of the footing cross section. The equations of equilibrium for axial load and moment about the *x*-axis can then be written as

$$q_{cDA}\sum_{i=1}^{n} A_i = P_U \tag{C8-3}$$

and

$$q_{cDA}\sum_{i=1}^{n} A_i y_i = P_U Y_{c.g.} + M_{UD,x}/m = 0 \tag{C8-4}$$

where

A_i = Area of cross section *i* resisting axial load P_{UF},
y_i = Distance from centroid of cross section *i* of the footing to the *x*-axis,
x_i = Distance from centroid of cross section *i* of the footing to the *y*-axis,
n = Total number of areas resisting the axial load P_{UF},
$Y_{c.g.}$ = Distance from the centroid of the footing to the edge of the footing in the direction of loading along the *y*-axis,
$M_{UD,x}$ = Component of applied moment in the *x*-direction or minor axis of overturning, and
$M_{UD,y}$ = Component of applied moment in the *y*-direction or major axis of overturning.

Given the boundaries of the critical contact area, the moment capacity $M_{CE,y}$ is obtained from the following expression:

$$M_{CE,y} = q_c \sum_{i=1}^{n} A_i x_i - P_U X_{c.g.} \tag{C8-5}$$

Therefore, the total moment capacity of the foundation is

$$M_{CE} = \sqrt{\left(\frac{M_{UD,x}}{m}\right)^2 + (M_{CE,y})^2} \tag{C8-6}$$

and the moment demand on the footing defining the critical contact area in the *x*-direction becomes its capacity, or

$$M_{CE,x} = M_{UD,x}/m \tag{C8-7}$$

where

$X_{c.g.}$ = Distance from the centroid of the footing to the edge of the footing in the direction of loading along the *x*-axis,
$M_{CE,x}$ = Moment capacity of the foundation in the *x*-direction,
$M_{CE,y}$ = Moment capacity of the foundation in the *y*-direction, and
$M_{UD} = \sqrt{M_{UD,x}^2 + M_{UD,y}^2}$.

The biaxial moment capacity for two of the most common shapes of the critical contact area A_c forming the soil pressure block with a bearing capacity q_c for a rectangular footing is shown in the two cases below. Additional information and moment capacities for less common patterns and for footings of an angle cross section can be found in Lobo (2021).

Case 1: Biaxial moment capacity of a rectangular footing where the zero-pressure line intersects two opposite edges of the footing

The critical contact area A_c under the footing required to resist the axial load can be discretized into an area consisting of a rectangular and a triangular cross as shown in Figure C8-15.

Applying Equation (C8-5) and from Figure C8-15, the *y*-axis moment capacity $M_{y,CE}$ can be determined as shown in Equation (C8-8):

$$M_{y,CE} = q_c B_f \left\{ L_1 \left(X_{c.g.} - \frac{L_1}{2} \right) + \frac{1}{2} L_2 \left(X_{c.g.} - L_1 - \frac{L_2}{3} \right) \right\} \tag{C8-8}$$

where

$$L_2 = \frac{6}{q_c B_f}\left(P_U - \frac{2(P_U Y_{c.g.} - M_x)}{B_f} \right)$$

and

$$L_1 = \frac{P_U}{q_c B_f} - \frac{L_2}{2}$$

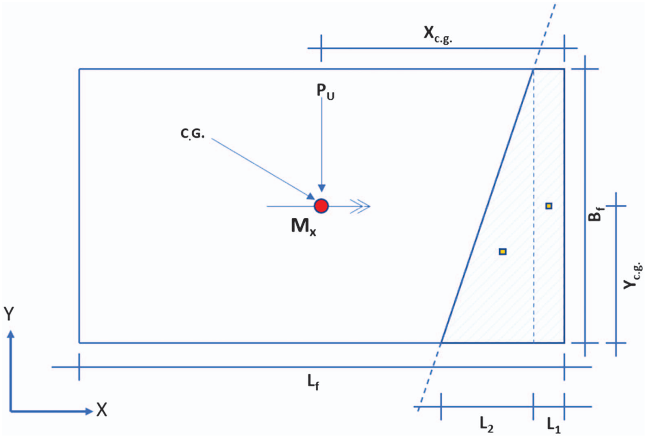


Figure C8-15. Critical contact area A_c of a rectangular footing, when the zero-pressure line intersects two opposite edges of the footing.

Case 2: Biaxial moment capacity of a rectangular footing where the zero-pressure line intersects two adjacent edges of the footing

If the zero-pressure line of the soil pressure block intersects two adjacent edges of the footing as sown in Figure C8-16, the ultimate moment in the orthogonal direction $M_{y,CE}$ is given by the following expressions:

$$M_{y,CE} = \frac{1}{2} q_{cDA} L_x L_y \left(X_{c.g.} - \frac{L_x}{3} \right) \qquad \text{(C8-9)}$$

where

$$L_y = 3\left(Y_{c.g.} - \frac{M_x}{P_U} \right)$$

and

$$L_x = 2\left(\frac{P_U}{q_c L_y} \right)$$

For a given axial load, the normalized moment capacities in each orthogonal direction for a footing of rectangular section, as shown in Figure C8-17, there is a minimal reduction in moment capacity where moments in the orthogonal direction are less than 20% of the moment capacity in that direction. Therefore, the effects of biaxial moments are permitted to be ignored for orthogonal moments less than 20% of the capacity in that direction.

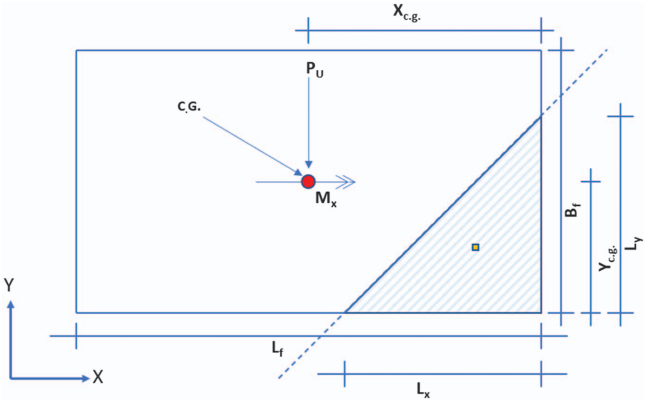


Figure C8-16. Critical contact area A_c of a rectangular footing, when the zero-pressure line intersects two adjacent edges of the footing.

C8.4.4.1.1.2 Overturning Forming Axial Load Action There are conditions where compression or uplift caused by overturning on isolated spread footings is associated with coupled foundations, such as those supporting columns in a braced or moment frame (Figure C8-18). In these cases, overturning demand results in axial actions, tensions, and compression, on the foundation. When the moment demand on the isolated spread footing, M_{Ftg}, is less than 20% of associated *m*-factor times the moment capacity of the foundation, the local moment may be ignored. Foundation acceptance is evaluated assuming axial compressive demand acts over the full area of the footing against the soil bearing capacity multiplied by the *m*-factor. Overturning stability is evaluated by comparing uplift (or tension) demand with the gravity dead load multiplied by the associated *m*-factor. The vertical elements in net tension should be checked to ensure adequate capacity to engage the dead load of the footing and slab-on-grade tributary to the footing.

C8.4.4.1.1.3 Overturning Induced Moment and Axial Load Actions Load demands on isolated spread footings from seismic overturning action often results in moment and axial load on the footing. These demands are resisted by the supporting soil. When axial loads are high relative to the soil bearing capacity, the footing tends to plow into the soil with repeated cyclic moment action on the footing (Figure C8-19). The resisting soil pressure redistributes back and forth from a uniform to a triangular or rectangular soil pressure block under the footing, for footings considered rigid relative to soil. Foundation acceptance is based on the footing being strong enough to resist the demands without yielding and inelastic deformations limited to the soil. Where foundation yielding is expected to occur, the foundation should be evaluated using the flexible-base procedures in Section 8.4.5 or the combined footing requirements in Section 8.4.4.1.2.

C8.4.4.1.1.3.1 Acceptance Criteria for Soil Bearing and Overturning The acceptance for soil bearing is satisfied when the overturning demand M_{UD} is less than *m* times the moment capacity using either Equation (8-21) or (8-23) and a knowledge factor κ for soil bearing of 1.0. When the overturning resistance from footing structural actions is included in the calculation of overturning capacity, the *m*-factor from Table 8-4 may be applied to the calculated M_{CE} for fixed-base procedures. There may be situations in which the bearing capacity *m*-factor is greater than the *m*-factor for component actions for the structural foundation (e.g., from Chapter 10 for concrete foundations) or where the structural foundation is required to be assessed as force controlled. In these cases, it is still acceptable to use the *m*-factor from Chapter 8 for the assessment of the soil.

Buildings modeled using the fixed-base assumption, with the lateral-force-resisting elements supported on I-shaped or rectangular footings, where coupled seismic tension and compression action occur simultaneously on the footing, it can be shown that for A_c/A ratios less than 0.2, if the center portion of the footing is ignored (Figure C8-20) such that overturning action is considered to be resisted by axial compression and uplift action at the ends of the footing, the acceptance criteria for axial load action in accordance with Section 8.4.4.1.1.1 gives a more favorable result. This is because the *m*-factor for uplift stability is greater than the *m*-factor for overturning stability. A comparison of the acceptance ratios for soil bearing for overturning stability between the results using a fixed-base procedure and flexible-base procedure shows a big difference in the acceptance ratios for the foundations in the two cases. For consistency with the acceptance

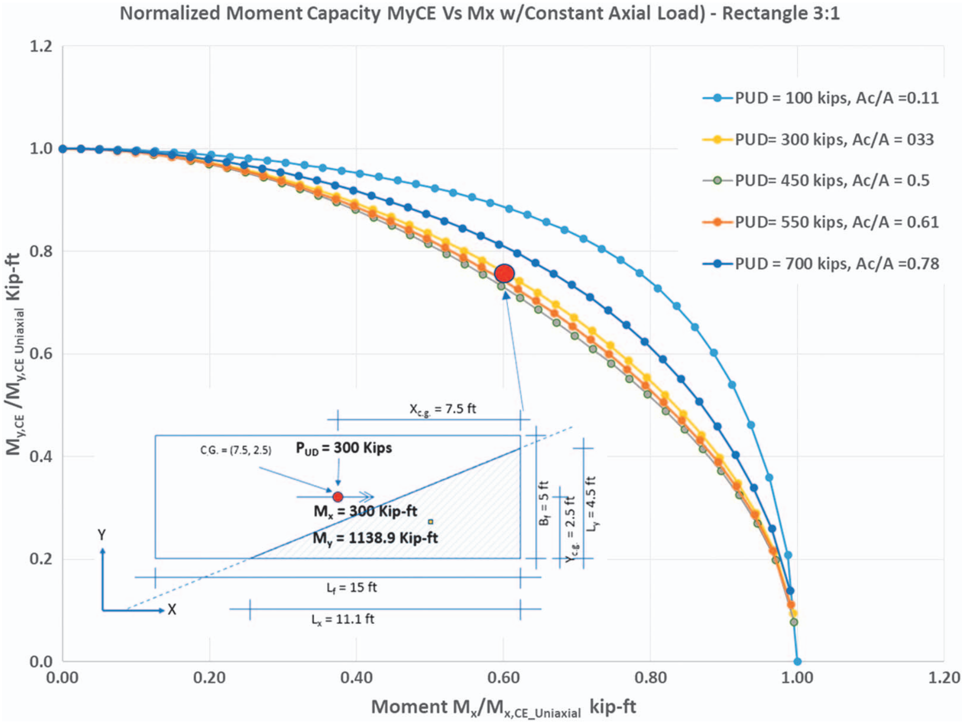


Figure C8-17. Normalized orthogonal moment capacities for a rectangular footing.

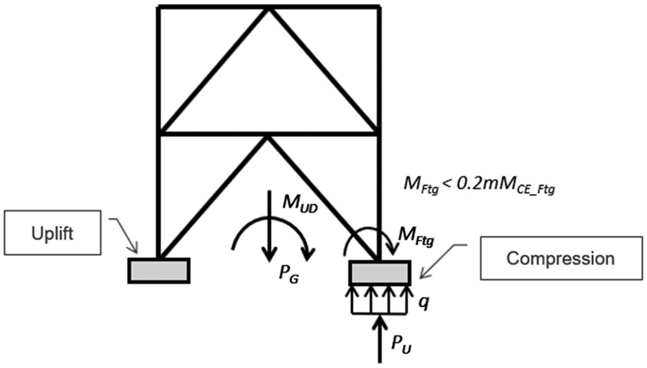


Figure C8-18. Overturning resisted by coupled axial load actions on footings.

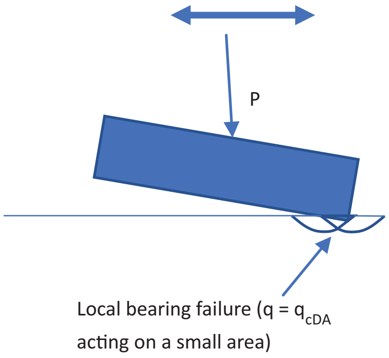


Figure C8-19. Local bearing failure due to rocking action.

criteria for buildings where the foundations are modeled using the flexible-base procedure while still maintaining a level of conservatism when using the fixed-base procedure, an exception is added to permit the center portion of the footing to be ignored for soil bearing acceptance.

C8.4.4.1.1.3.2 Acceptance Criteria for the Structural Footing Evaluation of footings for rocking action is performed using a capacity-based approach where internal forces in the footing are determined based on the application of the expected soil bearing capacity without the increase for short-term loading by a rectangular compression block on the rocking footing (Figure C8-21). Component actions on the structural foundation are evaluated

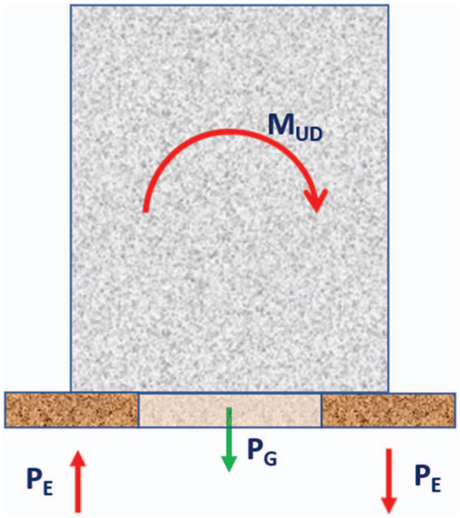


Figure C8-20. Foundation overturning demands on a rectangular or I-shaped footing.

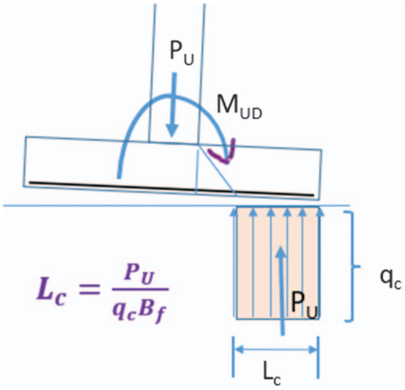


Figure C8-21. Soil pressure distribution under the footing used for evaluating the footing strength.

in accordance with Chapters 9 through 12 corresponding to the foundation material.

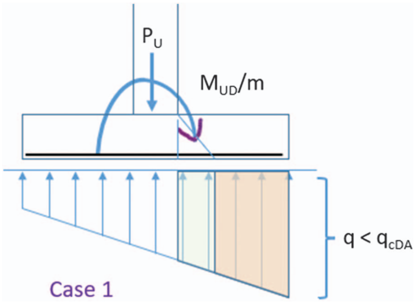
Where component actions in the structural foundation are required to be evaluated as force controlled, the structural foundation assessment will either justify adequate strength in the footing to provide overturning resistance, or strengthening of the footing is required. Where the structural foundation does not meet the component acceptance criteria, the footing is not considered rigid relative to the soil, and the requirements in this section cannot be used in calculating the overturning capacity for soil bearing.

For structural footings that do not satisfy the strength requirements in Chapters 9 through 12 corresponding to the foundation material, additional checks may be performed to verify adequacy of the footing based on the magnitude of the seismic load.

The shear and moment demand on the footing may be further evaluated such that the acceptance criteria for soil bearing is still satisfied. This is illustrated next for rectangular footings, where the footing should be evaluated considering the following cases as applicable.

Case 1: Uniform or Trapezoidal Distribution of Soil Pressure

This condition is applicable when the soil pressure q distributed along the length from $Q_{\max}$ to $Q_{\min}$, determined from Equation (C8-10), satisfies the requirement that no portion of the soil is in tension, $Q_{\min} > 0$ and the $Q_{\max} < q_{cDA}$, as follows:



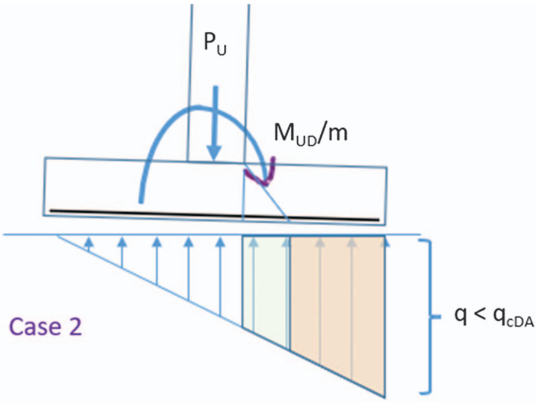
$$0 \leq q < q_{cDA}$$

where

$$Q_{max/min} = \frac{P_U}{A_f}\left(1 + \frac{6e_{AC}}{L_f}\right);\ when\ e_{AC} = \frac{M_{CE}}{P_U} \leq L_f/6 \quad \text{(C8-10)}$$

Case 2: Triangular Distribution of Soil Pressure

This condition is applicable when the soil pressure q linearly distributed along the length goes from $Q_{\max}$, determined in Equation (C8-11), to zero and satisfies the requirement that $Q_{\max} < q_{cDA}$.



$$0 \leq q < q_{cDA}$$

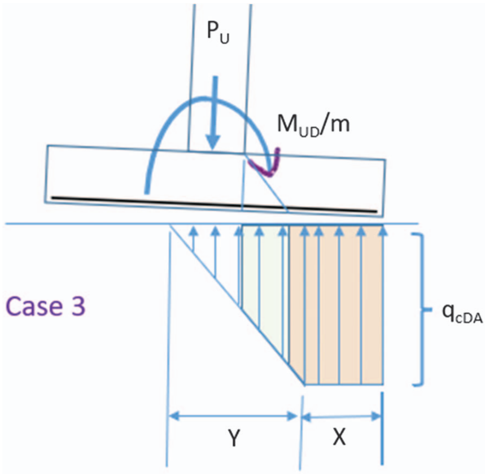
where

$$Q_{\max} = \frac{2P_U}{3B_f\left(\frac{L_f}{2} - e_{AC}\right)};\ when\ \frac{L_f}{6} \leq e_{AC} \leq \frac{L_f}{2} \quad \text{(C8-11)}$$

$$Q_{\min} = 0\ at\ L' = 3\left(\frac{L_f}{2} - e_{AC}\right) \leq L_f$$

Case 3: Rectangular and Triangular Distribution of Soil Pressure

This condition may be used if the conditions in this section are met when the soil pressure distribution of the seismic demands are not satisfied using either Case 1 or 2.



A rectangular distribution of soil pressure with $q = q_{cDA}$ shall be applied over an area for a distance X [Equation (C8-12)] from footing end toward the neutral axis followed by a triangular distribution over a distance Y [Equation (C8-13)] with $q_{cDA} \geq q \geq 0$, where

$$X = \frac{P_U}{q_{cDA}B_f} - \frac{1}{2}Y \quad \text{(C8-12)}$$

$$Y = \sqrt{12\{P'L_f - 2M' - P'^2\}} > 0 \quad \text{(C8-13)}$$

and

$$X + Y < L_f$$

where

$$P' = \frac{P_U}{q_{cDA} B_f}$$

and

$$M' = \frac{M_{UD}/m}{q_{cDA} B_f}$$

C8.4.4.1.2 Combined Footings, Mat Foundations, and Isolated Spread Footings A common practice for new buildings designed using ASCE 7 is to use an elastic, fixed-base building model to design the superstructure, and a separate model with an elastic mat on compression-only springs to design the foundation. Different proprietary software are typically used for this two-step analysis approach; the structure is modeled with a fixed base, and the reactions are then transferred to another foundation analysis program to determine the soil bearing pressure distribution and to design the foundation structure. The demands are based on reduced forces, as determined through the application of a global force reduction factor, and compression-only soil springs are used to represent the soil and soil–structure interface.

For existing building evaluation and retrofit using ASCE 41, the standard uses unreduced force demands and treats each component action as either force or deformation controlled. For deformation-controlled actions, the capacity is increased by an m-factor that varies depending on its ductility capacity. The unreduced demand is then compared to an amplified capacity on a component action basis. See Chapter 2 of FEMA P-2006 for a discussion of the differences between ASCE 7 and ASCE 41 provisions. Although the linear procedures are meant to be elastic procedures, practice has the tools and propensity to incorporate geometric nonlinearity (soil separation from footing) into the design process. Therefore, the standard has addressed this by permitting two procedures, which are discussed separately.

A flowchart of the steps for evaluation of buildings on combined footings or mat foundations is shown in Figure C8-22.

C8.4.4.1.2.1 Foundations Idealized as Individual Footings Buildings on combined or strip footings may have their foundation system evaluated by idealizing the foundation component directly supporting the vertical elements of the lateral-force-resisting elements of the superstructure into isolated spread footing segments. Demands from the vertical elements supported by these foundation segments from the fixed-base analysis are individually applied to the foundation component. The extent of each foundation segment beyond the ends of the vertical lateral-force-resisting element is limited to the location of the expected point of contraflexure or the maximum length beyond which yielding of the foundation occurs when a uniform soil pressure equal to q_c is applied from the edge of the idealized segment toward the centroid of the foundation segment over a distance that balances the applied axial load on the foundation segment. This extension should not be greater than half the clear distance between the edge of the vertical element on this foundation component and the next adjacent vertical component on the foundation. These idealized foundation components are created similar to the idealization used in the simplified procedure, except that the resistance provided by the interconnecting foundation component may be added to the foundation moment capacity, which is amplified by the m-factor when checking foundation acceptance. Additional information on idealizing the foundation segments is found in Section C8.4.5.2.2.1.

C8.4.4.1.2.2 Foundations Evaluated in a Separate Analysis from the Superstructure Where a fixed-base assumption is used and the foundation consists of combined footings or mat foundations, demands from the superstructure are extracted and analyzed outside the model used for evaluation of the superstructure to check the foundation. Because of the indeterminate nature of foundation systems with combined footings and mat foundations, they are typically analyzed using computer models where the footings are supported on individual vertical soil springs or distributed area springs, also called Winkler springs, as a beam-on-elastic foundation. The foundation model with springs is analyzed elastically where the springs resist tension and compression, or nonlinearly with yielding or nonyielding compression-only springs.

C8.4.4.1.2.2.1 Soil Stiffness See Section C8.4.5.2.2.1.

C8.4.4.1.2.2.2 Soil Strength See Section C8.4.2, Expected Soil Bearing Capacities.

C8.4.4.1.2.2.3 Acceptance Criteria for Soil Bearing and Overturning Acceptance criteria is satisfied if the foundation meets the necessary criteria from either Procedure 1 or Procedure 2, depending on whether the soil springs resist tension or behave nonlinearly as compression-only spring, as shown in the flowchart in Figure C8-22.

For both procedures, consideration should be given to the local magnitude of combined footing or mat foundation rotations determined in the separate beam-on-elastic foundation analyses and on its effect on the superstructure above when the superstructure is evaluated using a fixed base.

Procedure 1: Soil Springs Resist Tension and Compression Unreduced, pseudo-elastic demands and elastic compression-tension soil springs are used in this procedure.

This procedure may be used for fixed-base footing assessment or any linear flexible-base evaluation of soil and the footing. Flexible-base procedures in Section 8.4.5 may be used to justify adequacy of the foundation when the acceptance criteria for the fixed-base procedure is not satisfied. This requirement is intended to ensure reasonably accurate seismic forces and displacement demands on the structure for all methods.

Soil spring stiffnesses or modulus of subgrade reaction, which act in both tension and compression, used in the analysis model are derived from Figure 8-2 by discretizing a continuous or mat foundation into individual “effective” footings that are interconnected, or from Equation (8-24), or as provided by the geotechnical engineer. With the mat foundation now represented with flexural and shear flexibility and the distributed springs, the pseudo-elastic reaction forces from the base of the building are applied.

The total foundation rotation demand is obtained at the base of the wall, or from the bottom of two columns that form a braced frame, or from columns of moment frames or other lateral-force-resisting systems. To assess the soil bearing, the rotation demand is compared directly to the total rotation acceptance values in Table 8-8. The soil bearing is deemed to comply if the overturning rotation demand is less than the rotation values in Table 8-8.

For this linear-elastic procedure, the soil pressure distribution and structural actions of the foundation are based on an unreduced elastic earthquake level, which includes soil remaining in contact with the footing, which increases the restoring force on the foundation.

Procedure 2: Soil Springs are Compression-Only An alternative to Procedure 1 is to apply the earthquake forces divided by an m-factor per Table 8-5 along with the gravity loads per the applicable load combinations in Chapter 7.

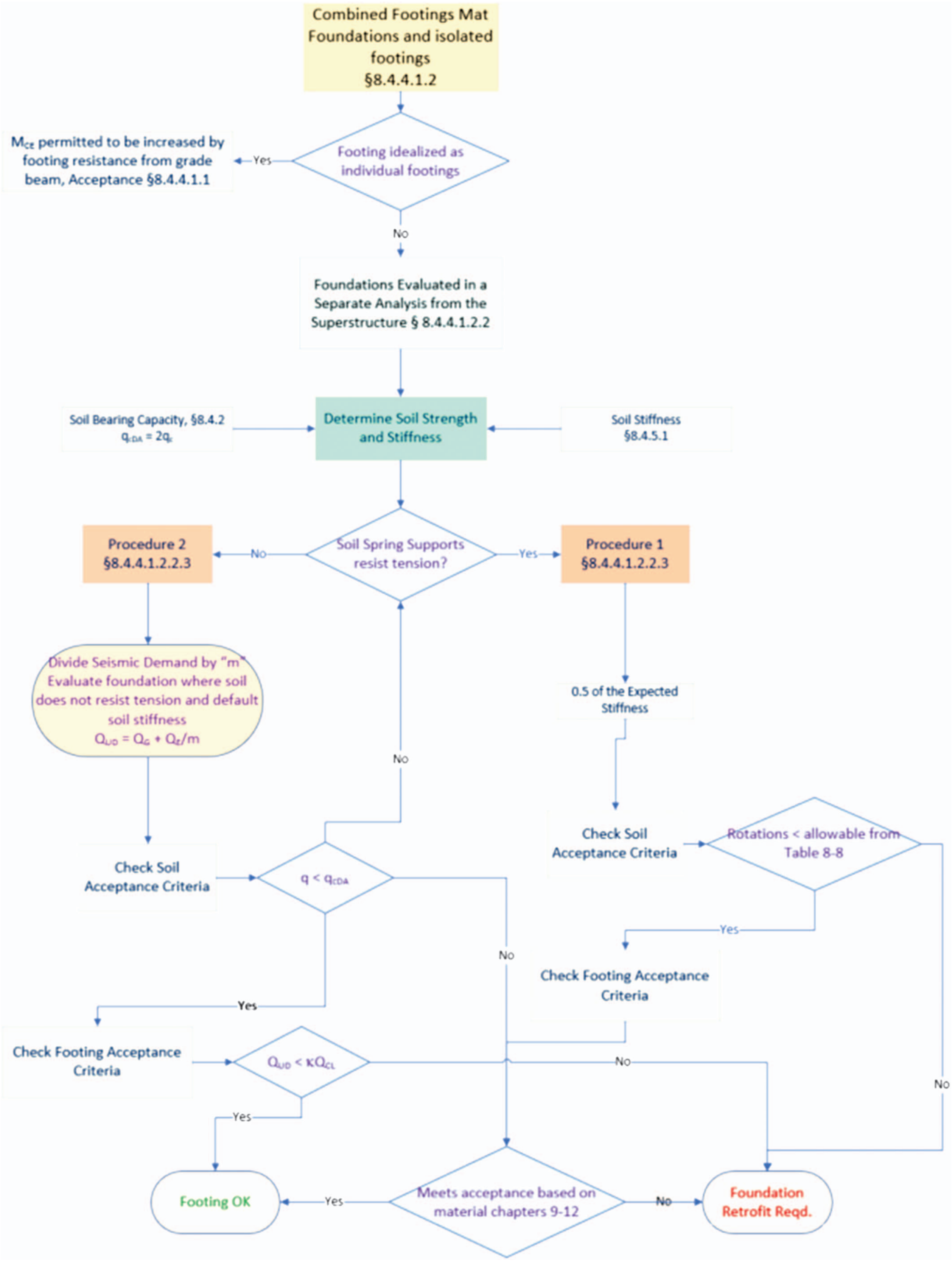


Figure C8-22. Evaluation process for buildings on combined footings, mat foundations, and isolated spread footings.

In Procedure 2, if the spring capacity in compression is not capped or does not yield, and the soil pressure distribution is triangular with the maximum pressure occurring at the edge of the footing when the footing is rigid relative to the soil. Because the soil pressure distribution is triangular, the maximum soil pressure at the loaded edge of the footing can be increased because limiting it to the maximum soil pressure is conservative. For uniaxial and biaxial loading on the footing with appropriate meshing of the footing, an adjustment factor between 1.33 and 1.69 gives a one-to-one correspondence when the soil pressure gradient is parallel to the loaded edge and when the maximum pressure is at the corner and goes to zero along the two adjacent edges of the corner. These factors can be easily derived by equating the volumes of the soil pressure blocks for the uniform and triangular pressure distributions, given that the centroids of the soil pressure forming the critical contact area A_c for the two soil pressure distributions are the same location.

C8.4.4.1.2.2.4 Acceptance Criteria for the Structural Foundation The structural footing is evaluated to the requirements of the appropriate foundation material in Chapters 9 through 12 with component actions classified as force or deformation controlled as specified in those chapters. If Procedure 2 is used for evaluation of soil bearing, for component actions classified as deformation controlled, no further reduction by m is permitted because the demands have already been reduced by the soil m-factor.

C8.4.4.2 Nonlinear Procedures In many cases, the inelasticity in the superstructure governs the response of the building when the foundations and supporting soil are rigid and nonyielding relative to the superstructure. In these cases, there is limited deformation in the superstructure caused by deformations from the foundation structure or supporting soil, and a nonlinear analysis is permitted for analysis of the superstructure with the foundations being modeled as a fixed base. The superstructure demands or base reactions are input as a load on a foundation plan and evaluated as deformation controlled using compression-only springs for acceptance without the soil m-factors to reduce the demand.

Consideration should be given to the local magnitude of combined footing or mat foundation rotations determined in the separate foundation analyses with allowable values in Table 8-8 to assess its effect on the superstructure above when the superstructure is evaluated using a fixed base.

C8.4.5 Flexible-Base Procedure For footings on soft or loose soils, foundation deformations may not be reasonably captured by linear analysis procedures without considering the flexibility of the foundation structure in the analysis. The soil support is represented by spring coefficients equal to the modulus of subgrade reaction and modeled as beams on elastic supports or Winkler spring foundations. For linear analysis procedures, potential gapping between the soil and the footing due to overturning demands is restrained as the spring coefficients resist both tension and compression. To account for this resistance in the analysis, the stiffness of the springs is taken as one-half of the expected elastic stiffness.

Where uplift occurs, the footing generally is governed by rocking about the leading edge of the footing. Existing spread footings may yield before attaining the full rocking capacity, which would result in a soil bearing pressure more closely aligned with a flexible footing condition.

Case studies have shown that modeling the foundations as compression-only springs combined with elastic superstructure with pseudo seismic demands gives unreasonable and incorrect distribution of demands in the superstructure. Therefore, this procedure is limited to buildings that behave essentially as elastic where the maximum DCR in the superstructure is less than or equal to 1.5 when using linear procedures for analysis of the superstructure. When superstructure DCRs exceed 1.5, gapping between the soil and footing should only be used when the inelasticity of the superstructure is also considered in the analysis as the demands are limited to the actual force that can be delivered to the foundation.

This section applies to both linear and nonlinear analysis procedures for foundation because both soil strength and stiffness are required for evaluation of the foundation. When linear analysis procedures are used, soil stiffness is required to be explicitly modeled, and soil strength is used to establish acceptance. For nonlinear procedures, both strength and stiffness are included in the analysis model, which vary with magnitude of loading based on a predefined backbone curve or from a hysteretic model for the NDP.

C8.4.5.1 Soil Stiffness The stiffness calculations in Figure 8-2 are based on classic beam-on-elastic foundation principles and are intended as uncoupled discrete soil springs located and acting on one isolated footing at a single point, whereas Equation (8-24) is the unit vertical stiffness intended for distributed vertical soil springs below a footing. For both, no loss of contact between the footing and soil is assumed.

Earthquake loading typically results in an inelastic soil–structure response, where the soil separates from the bottom of the footing and yielding of the soil in bearing potentially occurs. Soil stiffness may also be represented as a unit stiffness equal to the modulus of subgrade reaction of the soil. Where soil information is not reasonably determined by prescriptive means, this information is provided by the geotechnical engineer based on soil borings and other geotechnical test methods.

Soil stiffness may be taken as the secant stiffness at the expected soil deformation and associated yield force for the linear procedures.

For all nonlinear procedures, initial foundation stiffness which softens with increased loading should be used in conjunction with gapping between the soil and the footing to satisfy equilibrium for a more accurate force distribution in the superstructure.

C8.4.5.2 Linear Procedures The flexural and shear flexibility of the footing should be included in the model. The footing supports can be provided by discrete soil springs that represent a tributary area of contact and be distributed uniformly across the footing–soil interface, or they can be distributed area springs with a spring coefficient equal to the modulus of subgrade reaction. No gapping at the soil–foundation interface is permitted when linear procedures are used for demands to the superstructure or the foundations unless the superstructure DCRs are less than 1.5.

C8.4.5.2.1 Isolated Spread Footings

C8.4.5.2.1.1 Soil Stiffness For linear procedures, where soil springs resist tension and compression, evaluation of soil bearing caused by overturning action is based on soil springs modeled with an expected stiffness equal to the stiffness multiplied by 0.5. The reduced stiffness is to compensate for any potential uplift prevented because of the tension resistance provided by the springs. The additional settlement is not expected to significantly impact superstructure forces because settlement is expected to be uniform, because the stiffnesses for all springs are equally reduced.

C8.4.5.2.1.2 Soil Strength Determination of soil strength uses the same procedure for a fixed-base analysis.

C8.4.5.2.1.3 Acceptance Criteria Overturning action is caused by coupled vertical forces acting on independent foundations or a moment applied to a foundation. For the former, both the downward and upward actions are assessed independently. The m-factors in Table 8-7 were derived to limit foundation settlements to acceptable values. The m-factors in Table 8-7 for LS and CP were based on the experimental observation by Deng et al. (2012) that earthquake-induced foundation settlements for rectangular rocking footings ($M/H > L_f$) were invariably less than 1% of the footing length, L_f, if the value of A_c/A_f is less than approximately 1/8. Thus, large m-factors are allowed if $A_c/A_f < 0.20$. It was also observed by Deng et al. (2012) that settlements rapidly accumulate because of cyclic loading if the value of $A_c/A_f > 0.5$ (footings with a static factor of safety with respect to bearing capacity less than 2). Therefore, m-factors are reduced for $A_c/A = 0.5$; there is no m-factor on the axial compression for $A_c/A = 1$ because the footing is loaded to capacity by axial loads alone.

The experimental data presented by Deng et al. (2012) were limited to rectangular footings with aspect ratios near 2. For larger aspect ratios, with rocking loading the small edges of the footing, settlements are expected to be greater; the parameter b/L_c was introduced to account for this effect. The parameter b represents the minimum width of the ends of the footing, $b = B_f$ for rectangular footings, and $b = t_f$ for I-shaped footings. B_f, L_f, and t_f are defined in Figure 8-4.

Because few experimental data are available for rocking or overturning on footings with I-shape, the m-factors were reduced for I-shaped footings. The missing area ratio, $(A_{\text{rect}} - A_f)/A_{\text{rect}}$ is defined to quantify the extent of the effect of the I-shape. For L-shaped footings, which commonly occur at corners of buildings, use of the acceptance criteria for rectangular footings shall be permitted with the appropriate footing shape parameters based on the direction of loading.

Case (d) in Figure 8-4 applies to I-shaped footings with a very thin "web" (perhaps a thin shear wall that connects two rectangular footings). Case (d) may also represent a composite footing consisting of two or more separate footings connected by a coupling beam, shear wall, or a frame in the aboveground structure. Where the foundation supporting a frame consists of isolated spread footings, Case (d) shall be used; the footing area A_f may be taken as the summation of all the frame footings, and the expected vertical load P_U may be calculated as the cumulative sum of the vertical forces acting on all the footings.

In cases where isolated spread footings are coupled from the structure above and the foundation action results in uplift forces, the overturning capacity is taken as the expected dead load multiplied by the corresponding m-factor for that footing. This local foundation assessment ensures that the global overturning behavior is limited to stable lateral deformation. The use of lower-bound stiffness or stiffness values reduced to half when determining the uplift action simulates the loss of contact between the soil and structure at those locations. An average stiffness between zero and the compression stiffness is used to capture the average response, similar to that performed in the industry when modeling abutments on bridges. This approximation is deemed sufficiently accurate, and no further reduction on the lower bound is required, provided that all the foundations that resist the overturning actions use the lower-bound stiffness.

C8.4.5.2.2 Combined Footings, Mat Foundations, and Foundations Idealized as Isolated Footings

C8.4.5.2.2.1 Soil Stiffness Foundation systems with combined footings and mat foundations, are typically analyzed using computer models where the footings are supported on individual vertical soil springs or distributed area springs, also called Winkler springs, or a beam-on-elastic foundation as shown in Figure C8-23. The foundation model with springs is analyzed elastically where the springs resist tension and compression or nonlinearly with yielding or nonyielding compression-only springs, provided primary component DCRs in the superstructure are less than or equal to 1.5.

When the entire building or portion thereof is supported by a mat foundation over multiple bays (Figure C8-24), the modulus of subgrade reaction using the whole width of the mat foundation determined from Section 8.4.5.1 gives unrealistically low values for the soil spring stiffness per unit tributary area of foundation. Therefore, four alternate methods are provided for an equivalent width to be used in the determination of the soil spring stiffness, which gives a more reasonable estimate of the actual stiffness of the soil under the mat.

1. The effective width is zoned to coincide with the column grid lines and limited by the typical bay width (Figure C8-25). Widths for end bays are typically less than the interior bays resulting in greater stiffness at the perimeter than in the center portion of the mat. Methods to determine foundation stiffness using the procedures in ACI 336.2R may also be used.
2. The effective width B'_f used to determine the soil spring stiffness is determined based on the minimum footing area required to support 1.5 times the gravity axial load at each location of the vertical structural component on the mat and the allowable soil pressure (Figure C8-26).
3. The effective width is based on the geometry and spacing of the vertical structural components of the mat and the thickness of the mat (Figure C8-27).
4. Other rational procedures based on settlement of the mat from finite-element modeling of the soil continuum that include geometry and rigidity of the mat are also permitted.

C8.4.5.2.2.2 Soil Strength See Section C8.4.2, Expected Soil Bearing Capacities.

C8.4.5.2.2.3 Acceptance Criteria When the foundation is modeled with soil springs that resist both tension and compression the combined footing may be discretized into individual foundation segments, and the acceptance criteria for the soil bearing is evaluated similar to the acceptance criteria for isolated footings. The overturning moment capacity may be increased including the resistance provided by the interconnecting grade beams. Alternatively, the requirements of Procedure 1 in Section 8.4.4.1.2.3 can be used to evaluate the foundation system.

If Procedure 2 is used to evaluate the foundation system, the demands on the foundation from the superstructure are required to be transferred to a separate computer model for evaluation of

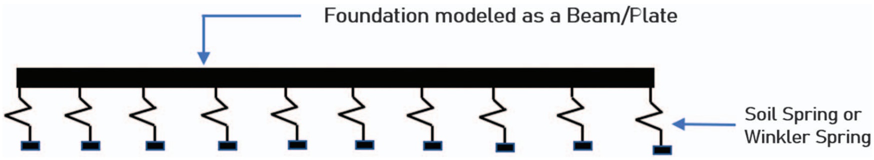


Figure C8-23. Foundation on soil springs or Winkler springs.

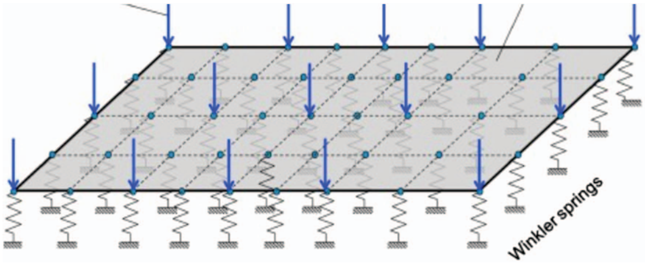


Figure C8-24. Mat foundation on Winkler springs.

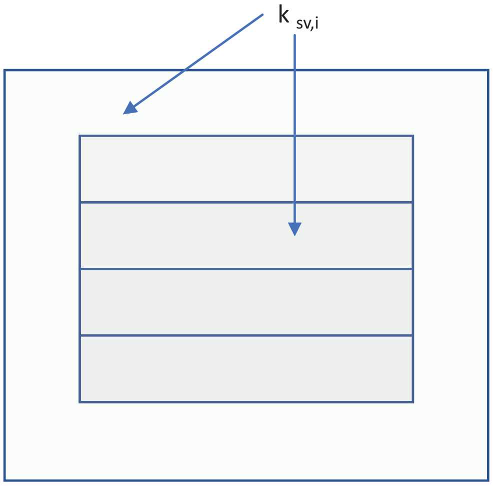


Figure C8-25. Foundation widths zoned to coincide with column grid lines.

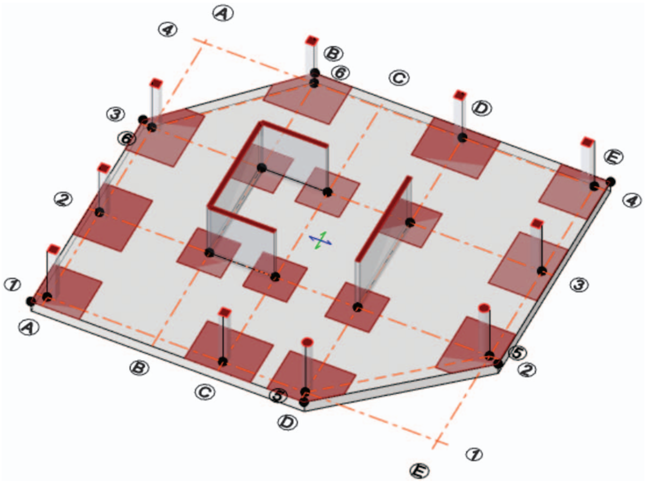


Figure C8-26. Foundation widths sized based on allowable soil bearing required to support 1.5 times the allowable gravity load.

the foundation system. The seismic demands may be globally reduced by the fixed-base *m*-factors from Table 8-5.

If the demands to the foundation are already reduced by the soil *m*-factor, the foundation structural component cannot be evaluated by further reducing demands by the *m*-factor from the material chapters for deformation-controlled actions.

C8.4.5.3 Nonlinear Procedures Explicit models of foundations account for the capacity and stiffness of each foundation element.

Load–deformation characteristics are required where the effects of foundations are to be taken into account in NSPs (pushover), or NDPs (time history). Foundation load–deformation parameters characterized by both stiffness and capacity can have a significant effect on both structural response and load distribution among structural components.

For axial and shear load–deformation behavior of foundations, an equivalent elastoplastic representation is acceptable. Rocking behavior of foundations can be represented by a trilinear relationship and depends on the footing shape.

The behavior of a shallow foundation that uplifts as opposed to slides is not subject to as much uncertainty as the behavior of a shallow foundation that deforms because of bearing capacity failure or sliding. The overturning capacity is largely controlled by the vertical load and the dimensions of the footing. These factors are not affected by variability in soil properties. Nonlinear rocking modeling parameters and acceptance criteria are described in Section 8.4.5.3.

It is important that geotechnical engineers report the average expected soil characteristics and any factor of safety applied to arrive at design values for soil strength and stiffness. In the past, design values recommended by geotechnical engineers were often consistent with lower-bound strengths. If such reduced values were used by the structural engineer as expected values, the application of the prescribed upper- and lower-bound variations would not achieve the intended aim.

Earlier versions of the standard required bounding on soil strength and stiffness to be considered when soil properties were explicitly modeled. This introduced a lot of conservatism in the design to account for the uncertainty in the properties of the soil. The requirement for bounding has been removed in this version standard for strength and stiffness for the following reasons.

Strength. As stated previously, geotechnical engineers generally provided recommendations of lower-bound strengths. For sands, the allowable values were based on settlement, and there is additional strength before soil failure occurs. For clays the ultimate strength is lower, however, there is a difference between immediate settlement and long-term settlement. Consolidated soils have higher strength than unconsolidated soil. Fluctuation in the groundwater table also influences the soil strength. High groundwater tables can reduce the soil bearing strength by as much as 50%. A recent study by Alencar et al. (2021) shows that groundwater affects soil strength by about 20% for softer soils and 7% for stiff soil. The difference between soft and stiff soils can be 400%. This could be the one of the reasons why high bounding uncertainty in strength was introduced in the prestandard to ASCE 41. The assumption made here is that these fluctuations in historic high groundwater table are already incorporated in the allowable soil bearing values provided by the geotechnical engineer at the start of construction. Designers should use caution when applying the strength and stiffness values in ASCE 41 in new construction. If the soil properties are reasonably accurately determined at the start of construction and appropriate factors of safety applied, the variation in soil bearing capacity is in line with the variation in other structural systems for which ASCE 41 routinely does not require bounding of strengths and stiffnesses.

In addition, soils have a dynamic amplification factor on strength of about two times the static values. Therefore, the short-term dynamic amplification factor to account for this strength increase is applied when soil bearing resistance is determined.

If we are fairly reasonable on strength, then having bounding on top of that with a factor of 4 is unjustified. For linear

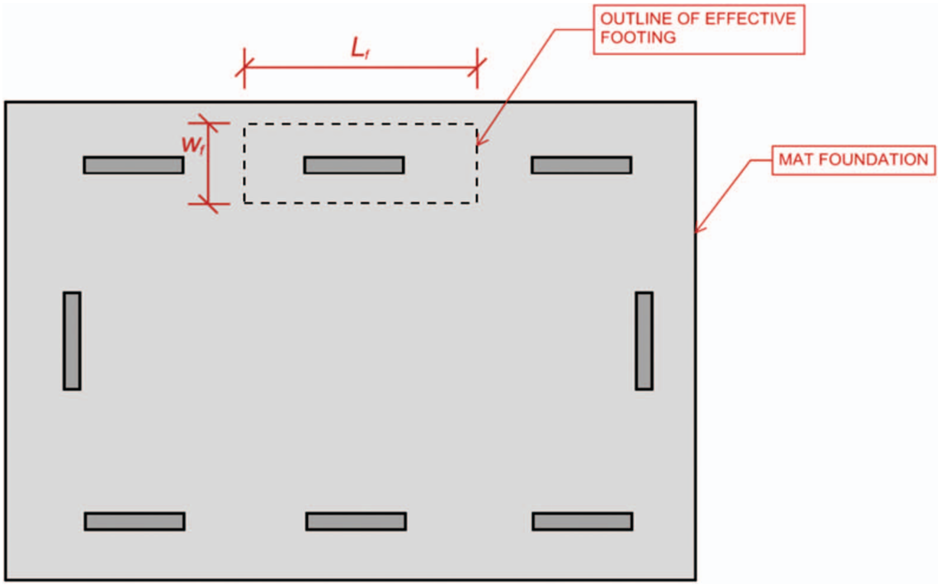


Figure C8-27. Foundation widths sized based on mat thickness and geometry of vertical elements on the mat.

procedures using fixed base or flexible-base assumptions, upper-bound values are already permitted. This implies there is a different set of rules for linear versus nonlinear procedures for soil strength. Halving the strength for nonlinear procedures is unrealistic because the dynamic resistance of soils for short-term loads,which is real, is not taken advantage of.

Stiffness. Soil stiffness values as used in the standard are already reduced for site class when the G/G_0 ratio is applied. These values represent the secant stiffness for fully degraded soil at higher ground shaking magnitudes for Site Classes D and higher. In addition, consolidated soils have higher strength than unconsolidated soil. Therefore, the stiffness values are low to begin with, and using an additional factor of two as further reduction is unreasonable.

Lower-bound stiffness for elastic analysis for flexible base is used to counteract the fact that gapping is prevented, that is, soil resists tension. For the condition in which the soil does not resist tension, the rotation value is in line with the values in Table 8-8 when soil does not get too soft. For higher values of S_{DS}, the ratios are too small already. Therefore, using lower stiffness values via bounding was also considered to be unreasonable. A fixed-base analysis provides an upper bound on soil stiffness values. Studies by various researchers have shown that doubling of soil stiffness values minimally change the initial period of the building. Requiring a bounding analysis is therefore not mandated because the expected stiffness values that use the effective shear modulus of the soil give results within the margin of the error to not alter the target performance of the building.

Consideration of foundation rocking. Buildings may rock on their foundations in an acceptable manner, provided that the structural components can accommodate the resulting displacements and deformations. Consideration of rocking can be used to limit the force input to a building.

The design professional is directed to the work of Housner (1963), Priestley et al. (1978), Yim and Chopra (1985), FEMA 274 (1997b), Makris and Roussos (1998), Makris and Konstantinidis (2001), Gajan et al. (2010), and Deng et al. (2012). Significant discrepancies between nonlinear dynamic analysis and response spectrum methods occur for both rocking systems and more conventional hinging systems when large deformations (e.g., P-Δ effects) become significant. Gajan et al. (2010) show that rocking on soil dissipates considerable energy associated with plastic deformations of the soil and that the energy dissipation is not well described using the theory of inelastic collisions.

C8.4.5.3.1 Modeling Parameters for Nonlinear Static Procedure The acceptance criteria (total footing rotation angle) in Table 8-8 were derived to limit foundation settlements to acceptable values. The values are only applicable if the acceptable story drifts are $\geq 1\%$. The allowable rotations in Table 8-8 for LS and CP were based on the experimental observation by Deng et al. (2012) that earthquake-induced foundation settlements for rectangular rocking footings ($M/H > L_f$) were invariably less than 1% of the footing length, L_f, if the value of A_c/A_f is less than 1/8. Thus, large rotations are allowed if $A_c/A_f < 0.13$. It was also observed by Deng et al. (2012) that settlements rapidly accumulate because of cyclic loading if the value of $A_c/A_f > 0.5$ (footings with a static factor of safety with respect to bearing capacity less than 2); therefore, allowable rotations are set to be less than 0.004% for $A_c/A_f = 0.5$ at the Life Safety Performance Level. Zero footing rotation is acceptable if $A_c/A_f = 1$ because the footing is loaded to capacity by axial loads alone.

The experimental data presented by Deng et al. (2012) were limited to rectangular footings with aspect ratios L_f/B_f between 1/2 and 2. For larger aspect ratios, with rocking loading the small edges of the footing, settlements are expected to be greater; the parameter b/L_c was introduced to account for this effect. The parameter b represents the minimum width of the ends of the footing, $b = B_f$ for rectangular footings, and $b = t_f$ for I-shaped footings. B_f, L_f, and t_f are defined in Figure 8-4.

Because few experimental data are available for rocking foundations on footings with I-shape, the allowable rotations were reduced for I-shaped footings. The missing area ratio, $(A_{\text{rect}} - A_f)/A_{\text{rect}}$, is defined to quantify the extent of the effect of the I-shape.

Case d in Figure 8-4 applies to I-shaped footings with a very thin "web" (perhaps a thin shear wall that connects the two rectangular footings). Case (d) may also represent one composite footing consisting of two separate rectangular footings connected by a coupling beam or a shear wall in the aboveground structure.

The "web" of the "I" should be sufficiently stiff to ensure that the rectangular footings would rotate about the same point.

C8.4.5.3.2 Modeling Parameters for Nonlinear Dynamic Procedure Table 8-8 does not provide a means to explicitly account for self-centering associated with rocking, nor does it account for the magnitude of hysteretic damping. However, the footing rotation angle limits in Table 8-8 do implicitly limit seismic settlement. If nonlinear dynamic analysis is to be conducted, nonlinear inelastic Winkler-style foundation springs should be used, which can account for hysteretic damping, gapping, and self-centering effects (NIST GCR 12-917-21 [NIST 2012a], Gajan et al. 2010). Using softer Winkler springs for footing middle zones and stiffer Winkler springs at footing end zones, to match the vertical and rotational stiffness of classic elastic soil spring equations, may result in negating the full extent of the permanent seismic settlement that would occur otherwise in the Winkler spring modeling under large footing rotations.

Modification of the response spectrum because of kinematic interaction effects may be considered but damping associated with soil–structure interaction should not be included in the selection of the input motion. Damping elements with constant radiation damping coefficients shall not be placed in parallel with nonlinear yielding elements. It is often acceptable to use Rayleigh damping in parallel with the springs with $[C] = \alpha_M \cdot [M] + \beta_k \cdot [K_T]$, where $[M]$ is the mass matrix and $[K_T]$ is the tangent stiffness matrix, with α_M and β_k determined to provide the appropriate damping ratio over the desired frequency range (PEER/ATC 72-1 2010).

C8.4.5.3.3 Acceptance Criteria Superstructure and foundation inelasticity are included in the analysis, and demands to the foundations are the expected demands based on the desired performance. When the NSP is used, acceptability of soil displacements is limited to the rotation limits in Table 8-8, and the foundation structural element are evaluated as force controlled, because this is based on the maximum force delivered to the element.

For nonlinear dynamic procedures, the nonlinear soil deformation should already be included in the analysis; therefore, the acceptability of soil displacements is based on the ability of the structure to accommodate the displacements within the acceptance criteria for the selected Performance Objective. If deformations in the foundations are excessive or the soil springs do not capture seismic settlement or permanent gapping/deformation, the foundation rotation limits in Table 8-8 should be used for soil acceptance criteria. The structural footing acceptance is similar to acceptance for the NSP.

C8.4.6 Shallow Foundation Lateral Load For footings subjected to lateral loads, the base traction strength is given by $V = C + N\mu$, where C is the effective cohesion force (effective cohesion stress, c, times footing base area), N is the normal (compressive) force, and μ is the coefficient of friction. If included, side traction is calculated in a similar manner, but it is considered on one side of the footing only. The coefficient of friction is often specified by the geotechnical consultant. In the absence of such a recommendation, μ may be based on the minimum of the effective internal friction angle of the soil and the friction coefficient between soil and foundation from published foundation references. The ultimate passive pressure strength is often specified by the geotechnical consultant in the form of passive pressure coefficients or equivalent fluid pressures. The passive pressure problem has been extensively investigated for more than 200 years. As a result, countless solutions and recommendations exist. The method used should, at a minimum, include the contributions of internal friction and cohesion, as appropriate.

As shown in Figure 8-6, the force–displacement response associated with passive pressure resistance is highly nonlinear. However, for shallow foundations, passive pressure resistance generally accounts for much less than half of the total capacity. Therefore, it is adequate to characterize the nonlinear response of shallow foundations as elastic–perfectly plastic using the initial, effective stiffness and the total expected capacity. The actual behavior is expected to fall within the upper and lower bounds prescribed in this standard.

The model represented in Figure 8-6 does not include parameters for the planar dimensions of the foundation element (width and length), or dependence on soil type. As a result, this simplified model can considerably underestimate strength and stiffness. In lieu of using the default properties of Figure 8-6, it is acceptable to use more advanced methods, such as the one presented in *Investigation of the Resistance of Pile Caps and Integral Abutments to Lateral Loading* (Mokwa and Duncan 2000).

C8.5 DEEP FOUNDATIONS

C8.5.1 Pile Foundations

C8.5.1.1 Stiffness Parameters Because the passive pressure resistance of pile caps may be a significant part of the total capacity strength, it may not be appropriate to base the force–displacement response on the initial, effective stiffness alone. Instead, the contribution of passive pressure should be based on the passive pressure mobilization curve provided in Figure 8-6. In lieu of using the default properties of Figure 8-6, which can considerably underestimate strength and stiffness, it is acceptable to use more advanced methods, such as the one presented in Mokwa and Duncan (2000).

Although the effects of group action and the influence of pile batter are not directly accounted for in the form of the preceding equations, it can be reasonably assumed that the latter effects are accounted for in the range of uncertainties that must be considered. The method presented in Mokwa and Duncan (2000) does quantify pile group effects.

C8.5.1.2 Capacity Parameters The lateral capacity of a pile cap should be calculated in the same way that the capacity of a shallow foundation is computed, except that the contribution of base traction should be neglected. Section C8.4.6 provides a more detailed description of the calculation procedure. The method presented in Mokwa and Duncan (2000) provides a comprehensive approach to calculating the capacity contribution of pile caps to the lateral resistance of piles and pile groups.

C8.5.2 Drilled Shafts Where the diameter of the shaft becomes large [>24 in. (610 mm)], the bending and the lateral stiffness and strength of the shaft itself may contribute to the overall capacity. This size is obviously necessary for the case of individual shafts supporting isolated columns.

C8.6 SOIL–STRUCTURE INTERACTION EFFECTS

Foundation flexibility is covered in Section 8.4.5. SSI effects that serve to reduce the shaking input to the structure relative to the free-field motion (kinematic interaction and damping) are covered in this section. Procedures for calculating kinematic and damping effects were taken from recommendations in FEMA 440 (2005) and have been included in FEMA 368 (2001) and FEMA 450, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (2004) for a number of years. Further discussion of SSI effects can be found in FEMA 440 (2005) and NIST GCR 12-917-21 (ATC 2012).

C8.6.1 Kinematic Interaction

C8.6.1.1 Base Slab Averaging For base slab averaging effects to occur, foundation components must be interconnected with grade beams or concrete slabs. The concept of base slab and the basis for the provisions in ASCE 41-06 (2007) can be found in FEMA 440 (2005). The basis for the current equation and additional background material can be found in NIST GCR 12-917-21 (ATC 2012). The fundamental basis for base slab averaging is that the base slab is very stiff and stronger than the vertical elements of the lateral-force-resisting system to allow a filtering of high-frequency (short-period) ground motions. If the base slab is too flexible or yields before the vertical elements of the lateral-force-resisting system, then that filtering cannot occur.

These effects are most pronounced on softer sites. Therefore, a requirement of Site Class C, D, or E is placed on the provisions' use. Previous editions of the standard did not permit the use of base slab averaging as derived from equations estimating the reduction in response parameter, but current consensus is that that limitation was not necessary. For Site Class F, it is likely that base slab averaging does occur, but the significant amount of nonlinearity that occurs on a Site Class F makes it difficult to correlate these equations to. A more detailed analysis than simply using the equations in this section to understand the effects of base slab averaging must be conducted to understand the effects on Site Class F.

The equations that predict the reduction from base slab averaging rely heavily on the period of the building, so the building period must include the flexibility of the foundation so that the period is not underestimated, leading to an unconservative reduction. Yielding of elements in the superstructure can cause the fundamental period to lengthen. That is why the provisions require the use of the effective fundamental period as opposed to the elastic fundamental period. For the nonlinear static procedure, that period is explicitly calculated from the pushover curve. For the nonlinear dynamic procedure, the response spectrum can be modified by the ratio of response spectra (RRS) at each period and the ground motions selected and scaled to that modified response spectrum, because the effects on the demand parameters caused by structural yielding will be explicitly picked up in the model.

The underlying models have only been studied up to an effective size of 260 ft (79.2 m), which is why that limitation has been placed on Equation (8-30).

Because the reduction can become quite significant and there has not been a thorough study of this phenomenon, a 0.75 factor is applied to temper the reductions.

The method has not been rigorously studied for buildings on piles; however, it is considered reasonable to extend the application to pile-supported structures in which the pile caps are in contact with the soil and are laterally connected to one another.

C8.6.1.2 Embedment The embedment effect model was largely based on studies of buildings with basements. The recommendations can also be applied to buildings with embedded foundations without basements where the foundation is laterally connected. However, the embedment effect factor is not applicable to embedded individual spread footings.

As with base slab averaging, this reduction relies heavily on the period of the building; the building period must include the flexibility of the foundation so that the period is not underestimated, leading to an unconservative reduction. Also, because the reduction can become quite significant and there has not been a thorough study of this phenomenon, a 0.75 factor is applied to temper the reductions.

C8.6.2 Foundation Damping Soil–Structure Interaction Effects Foundation damping effects tend to be important for stiff structural systems such as shear walls and braced frames, particularly where they are supported on relatively soft soil sites, such as Site Classes D and E. The procedure is conservative where foundation aspect ratios exceed 2:1 and where foundations are deeply embedded ($e/r_x > 0.5$), but it is potentially unconservative where wall and frame elements are close enough so that waves emanating from distinct foundation components destructively interfere with each other across the period range of interest.

See FEMA 440 (2005) and NIST GCR 12-917-21 (2012) for further discussion of foundation damping SSI effects, including limitations. This procedure is based on theoretical equations that assume a rigid foundation system. For the procedure to be applicable, the foundation system and base slab must be stiff with respect to the vertical elements of the lateral-force-resisting system. Furthermore, the foundation elements (footings, slabs, grade beams) cannot yield before the vertical elements of the lateral-force-resisting system, because yielding foundation elements create a softer foundation system and deviate too much from the assumed condition of the theoretical equations.

The provisions of this section are based on the computation of an effective damping ratio based on a first-mode response. For most buildings analyzed using the LSP, LDP, and NSP that have periods small enough for foundation damping to be applicable, this is a fair assumption. However, because the foundation damping equations contained in this section are based on an elastic structure, they should not be used to reduce a target response spectrum that ground motion response histories are scaled to. If the design professional wishes to use foundation damping with the NDP, the explicit modeling of the damping at the soil–foundation interface is required. NIST GCR 12-917-21 (ATC 2012) provides guidance on how to do this. It also provides greater discussion for calculating foundation damping for different directional responses of a building, if more detail is desired than the provisions herein supply.

The provisions are prohibited for use with deep foundations. This prohibition is not because foundation damping does not occur with deep foundations. The equations in these provisions are based on shallow foundations. The prediction that damping for deep foundations requires additional modifications to the equations was deemed outside of the scope of these provisions. The user is referred to NIST GCR 12-917-21 (ATC 2012) for discussion on how to explicitly include foundation damping with deep foundation systems.

C8.6.2.1 Radiation Damping for Rectangular Foundations The radiation damping is most significant for rigid buildings situated on soft soils. The preceding equations are based on the stiffness of the subsurface media, as measured by the shear wave velocity, and the ratio of the stiffness of the flexible-base soil–structure system to the stiffness of the structure as if it were a rigid body sitting atop the soil (which is a way to measure the stiffness of the soil). The greater the difference between the periods of the soil–structure system compared to the period of a rigid body on the soil, the less radiation damping will occur because the primary movement of the building will be in the structure, not at the soil–foundation interface. Additionally, the greater the period lengthening for the flexible-base soil–structure system compared to a fixed-base condition, the more that deformations at the soil–foundation interface are significant and radiation damping is likely to occur. Figure C8-28, taken from NIST GCR-12-917-21 (2012), shows graphically how the

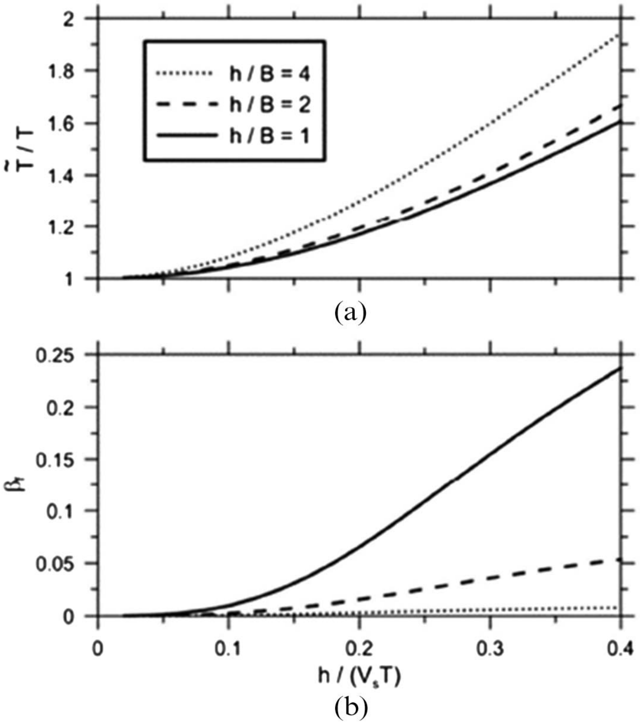


Figure C8-28. Plots of period lengthening and radiation damping versus structure-to-soil stiffness ratio for different aspect ratios.
Source: Figure 2-2 in ATC (2012).

increase of that period lengthening and the softness of the site leads to increase in the radiation damping.

C8.7 SEISMIC EARTH PRESSURE

There has been little evidence of failures in engineered basements in past or recent earthquakes despite the fact that seismic increment pressure was not included in design codes until the 2006 IBC (Lew et al. 2010). Most reported damage occurred in poorly constructed, nonengineered walls or as a result of soil failure. Significant damage has been observed in waterfront structures such as wharfs, primarily caused by liquifaction in loose saturated granular soils. These structures are not within the perview of ASCE 41 standards. Based on this observation, evaluation of subterranian walls may be limited to those structures where the performance goal is Immediate Occupancy or Damage Control. Observable damage to walls in Immediate Occupancy structures may result in evacuation following a large earthquake.

Past evaluation of seismic increment has been based on the original experimentation by Mononobe and Matsuo in 1929 and subsquent experimental and/or analytical modifcations to that approach. The 1929 experimentation was based on shake-table testing that did not account for wall yielding, frequency content of the soil, damping, and soil–structure interaction. The Mononobe–Okabe (M-O) approach simlified the analysis to assume the acceleration of an assumed wedge of soil. The shape of the wedge of soil varied from triangular (similar to active pressure assumptions), to rectangular, to inverted triangular. Depending on the assumed shape of the soil wedge, the resultant force on the wall varied from 1/3H, to 1/2H, to 2/3H, respectivly, resulting in large differences in the calculated flexural demand on the basement wall.

More recent experimental centrifuge reseach that includes the effects of the surrounding soil indicates to the 1/3H as the best approximation of the horizontal force resultant. It has also been noted that the M-O approach provides highly conservative results for peak ground accelerations greater than 0.6*g*.

The Mononobe–Okabe approach has been favored because of its simplicity. Seismic earth pressures may be estimated using Mononobe–Okabe as modified by Seed and Whitman for a building wall retaining unsaturated, level soil above the groundwater table using Equation (C8-14):

$$\Delta_p = 0.4 k_h \gamma_t H_{rw} \qquad \text{(C8-14)}$$

where

Δ_p = Additional earth pressure caused by seismic shaking, which is assumed to be a uniform pressure;
k_h = Horizontal seismic coefficient in the soil, which may be assumed equal to $S_{XS}/2.5$;
γ_t = Total unit weight of soil; and
H_{rw} = Height of retaining wall.

The resultant may be assumed at $1/3H_{rw}$.

FEMA/NEHRP (2020) provides a more complex approach that better acounts for the physical mechanisms using soil–structure interaction.Seismic earth pressure against basement walls depend on relative wall–soil displacement and is a result of kinematic and soil–structure interactions as shown in Figures C8-29 and C8-30.

This approach, while more complicated than Mononobe–Okabe, includes both kinematic and inertial soil–structure interaction effects to estimate the relative displacement between the soil and wall that produces soil pressure on the wall. This approach, when compared to available test data, demonstrates substantially better results when compared to the classical Mononobe–Okabe approach.

The approach requires a seismic hazard analysis to determine peak ground velocity, a deaggregation of the hazard, wall dimensions, and the shear wave velocity profile of the backfill soil against the wall. The evaluation is limited to a single frequency and amplitude of soil excitation, a planar wall (no

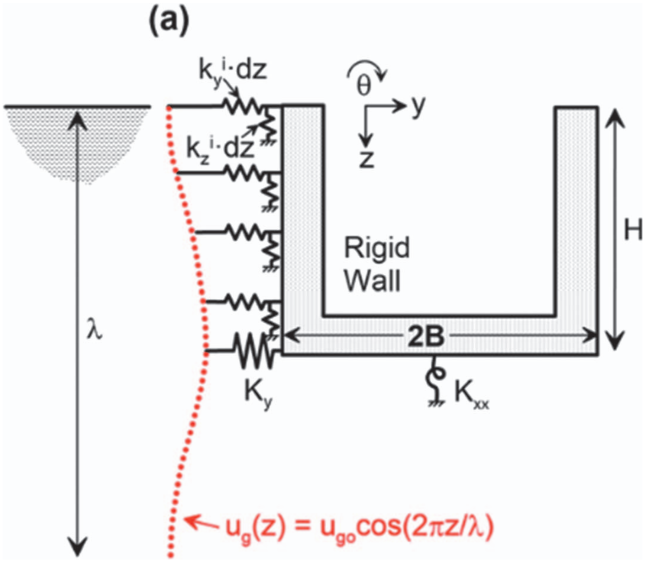


Figure C8-29. Kinematic seismic earth pressure model.
Source: FEMA/NEHRP (2023). Resource Paper 4: "Seismic lateral earth pressures," Figure 3.

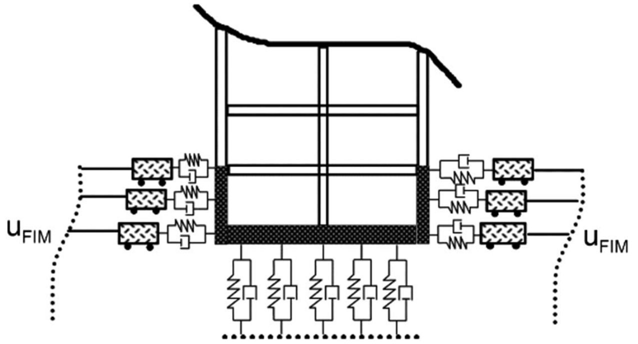


Figure C8-30. Inertial seismic earth pressure model.
Source: FEMA/NEHRP (2023). Resource Paper 4: "Seismic lateral earth pressures," Figure 9.

tie-backs or soil nails), and no soil failure such as liquifaction or slope instability.

The *m*-factors in Table 8-11 are calculated based on a simple flexural model that assumes fixity at the base and pin at the top of the wall. The prototype model assumes equal vertical flexural reinforcement on both faces of the wall. The *m*-value represents the allowable scale factor that increases the total soil loading from the initial-yield moment at the base of the wall to second-yield moment within the height of the wall.

Collapse Prevention is determined when either

1. Plastic flexural capacity is exceeded at two locations, the base and interior to the span, creating an unstable mechanism; or
2. Plastic flexural capacity is exceeded at the base when concrete crushing occurs at a strain equal to 0.003.

The plastic flexural capacity is sensitive to axial loading and flexural reinforcing ratio. For light axial loaded walls, there is larger available ductility to safely resist the primary flexural forces. As the axial loads increase, the available ductility reduces. Life Safety and Immediate Occupancy *m*-factors are based on Chapter 7 recommendations with the Collapse Prevention *m*-factors shown for completeness.

C8.8 FOUNDATION RETROFIT

Guidance for modification of foundations to improve seismic performance is provided as follows:

Soil material improvements. Improvement in existing soil materials may be effective in the retrofit of foundations by achieving one or more of the following results: (1) improvement in vertical bearing capacity of footing foundations, (2) increase in the lateral frictional resistance at the base of footings, and (3) increase in the passive resistance of the soils adjacent to foundations or grade beams.

Soil improvement options to increase the vertical bearing capacity of footing foundations are limited. Soil removal and replacement and soil vibratory densification usually are not feasible because they would induce settlement beneath the footings or would be expensive to implement without causing settlement. Grouting may be considered to increase bearing capacity. Different grouting techniques are discussed in FEMA 274, Section C4.3.2 (1997b). Compaction grouting can achieve densification and strengthening of a variety of soil types and/or extend foundation loads to deeper, stronger soils. The technique requires careful control to avoid causing uplift of foundation components or adjacent floor slabs during the grouting process. Permeation grouting with chemical grouts can achieve substantial strengthening of sandy soils, but the more fine-grained or silty the sand, the less effective the technique becomes. Jet grouting could also be considered. These same techniques also may be considered to increase the lateral frictional resistance at the base of footings.

Soil improvement by the following methods may be effective in increasing the passive resistance of soils adjacent to foundations or grade beams: removal and replacement of existing soils with stronger, well-compacted soils or with treated (e.g., cement-stabilized) soils; in-place mixing of existing soils with strengthening materials (e.g., cement); grouting, including permeation grouting and jet grouting; and in-place densification by impact or vibratory compaction. In-place densification by impact or vibratory compaction should be used only if the soil layers to be compacted are not too thick, and vibration effects on the structure are tolerable.

Shallow foundation retrofit. The following measures may be effective in the retrofit of shallow foundations:

1. New isolated or spread footings may be added to existing structures to support new structural elements such as shear walls or frames.
2. Existing isolated or spread footings may be enlarged to increase bearing or uplift capacity. Consideration of existing contact pressures on the strength and stiffness of the modified footing may be required unless uniform distribution is achieved by shoring and/or jacking.
3. Existing isolated or spread footings may be underpinned to increase bearing or uplift capacity. Underpinning improves bearing capacity by lowering the contact horizon of the footing. Consideration of the effects of jacking and load transfer may be required.
4. Uplift capacity may be improved by increasing the resisting soil mass above the footing.
5. Mitigation of differential lateral displacement of different portions of a building foundation may be carried out by provision of interconnection with grade beams, reinforced grade slabs, or ties.

Deep foundation retrofit. The following measures may be effective in the retrofit of deep foundations consisting of driven piles made of steel, concrete, wood, cast-in-place concrete piers, or drilled shafts of concrete.

Shallow foundations of spread footings or mats may be provided to support new shear walls or frames or other new elements of the lateral-force-resisting system, provided that the effects of differential foundation stiffness on the modified structure are analyzed and meet the acceptance criteria.

New wood piles may be provided for an existing wood pile foundation. A positive connection should be provided to transfer the uplift forces from the pile cap or foundation above to the new wood piles. Existing wood piles should be inspected for deterioration caused by decay, insect infestation, or other signs of distress before undertaking evaluation of existing wood pile foundations.

Driven piles made of steel, concrete, wood, cast-in-place concrete piers, or drilled shafts of concrete may be provided to support new structural elements such as shear walls or frames and or supplement the vertical and lateral capacities of existing pile and pier foundation groups.

However, driving new piles may induce settlement in the existing foundation elements, and that possibility should be considered when designing the retrofit. Because of that problem, pin piles or auger-cast piles may be preferable because they can be installed without inducing much settlement to the existing structure.

CHAPTER C9
STEEL AND IRON

C9.1 SCOPE

Techniques for repair of earthquake-damaged steel components are not included in this standard.

The linear static procedure (LSP) presented in Chapter 7 is most often used for the analysis of cold-formed steel (CFS) light-frame buildings; however, properties of the idealized inelastic performance of various components and connections are included so that nonlinear procedures can be used if desired.

The evaluation and assessment of various structural components of CFS light-frame buildings are found in Section 9.4. For a description and discussion of connections between the various components and elements, see Section 9.4.2.2. Properties of shear walls are described in Section 9.6, along with various retrofit or strengthening methods.

C9.2 REFERENCE STANDARD FOR STRUCTURAL STEEL, COMPOSITE STEEL–CONCRETE, AND CAST AND WROUGHT IRON

Prior to the 2023 edition of this standard, provisions regarding the condition assessment of structural steel, composite steel–concrete, and cast and wrought iron components, their stiffness and strength characteristics, and their permissible capacities were provided in Chapter 9. Starting in 2022, this information is published in AISC 342, which is referenced by Chapter 9 in lieu of reprinting the information contained in AISC 342. As such, Section 9.4 and beyond focus only on cold-formed steel components.

C9.3 MODIFICATION TO THE REFERENCE STANDARD FOR STRUCTURAL STEEL, COMPOSITE STEEL–CONCRETE, AND CAST AND WROUGHT IRON

The publication of AISC 342 generally precedes this standard. Consequently, there may be modifications to AISC 342 that are needed to align the two standards.

C9.4 MATERIAL PROPERTIES AND CONDITION ASSESSMENT FOR COLD-FORMED STEEL

C9.4.1 General The extent of in-place materials testing and condition assessment that must be accomplished is related to availability and accuracy of construction and as-built records, the quality of materials used and construction performed, and the physical condition of the structure. Data such as the mechanical properties of material used in component and connection fabrication may be effectively used to reduce the amount of in-place testing required. The design professional is encouraged to research and acquire all available records from original construction.

For CFS light-frame construction, the extent of in-place materials testing and condition assessment that must be accomplished is related to availability and accuracy of construction documents and as-built records, the quality of materials used and construction performed, and physical condition. A specific difficulty with light-frame construction is that structural components are often covered with other components, materials, or finishes; in addition, their behavior is influenced by past loading history. Knowledge of the mechanical properties of material used in original component or connection fabrication is invaluable and may be effectively used to reduce the amount of in-place testing required. The design professional is encouraged to research and acquire all available records from the original construction, including design calculations.

CFS connection configuration also has an important influence on response to applied loads and motions. A large number of connector types exist; the most prevalent are screws. An understanding of connector configuration and mechanical properties must be gained to properly analyze the anticipated performance of the building.

CFS light-frame construction has evolved over the years; cold-formed steel framing is a common building material for residential and commercial structures in the United States. It has often been used for the framing of roofs and floors and in combination with other materials. Establishing the age and recognizing the location of a building can be helpful in determining what types of seismic-force-resisting systems may be present.

Based on the approximate age of a building, various assumptions can be made about the design and features of construction. Older light-frame structures that predate building codes and standards usually do not have the types of elements considered essential for predictable seismic performance. In these conditions, new elements generally have to be added, or the existing elements have to be upgraded to obtain predictable performance.

If the age of a building is known, the code in effect at the time of construction and the general quality of the construction usual for the time can be helpful in evaluating an existing building. The level of maintenance of a building may be a useful guide in determining the structure's capacity to resist loads.

In more recent times, CFS light-frame studs, joists, and trusses have become popular. Seismic-force resistance is either provided by diagonal strap bracing attached to the studs and top and bottom tracks or by structural panels attached with sheet metal screws to the studs and the top and bottom track in a manner similar to that of wood construction. The CFS light-frame studs and joists vary in size, thickness, and configuration, depending on the manufacturer and the loading conditions.

C9.4.2.1 Material Properties Configuration (including base steel thickness) and mechanical properties affect strength and stiffness of CFS light-frame members and connections. Coatings intended for corrosion protection, such as zinc or paint, do not contribute significantly to the structural behavior and should be excluded when determining material properties. Minimum design material properties should be documented on original construction documents and in many cases are identified on the installed components.

C9.4.2.1.1 Default Mechanical Properties and Nominal or Specified Properties of Cold-Formed Steel Light-Frame Construction Actions associated with CFS light-frame components generally are deformation-controlled; thus, expected strength is used most often. Lower-bound values are used with components supporting discontinuous shear walls, connectors, and axial compression of individual frame components, which are force-controlled. Material properties listed in this chapter are expected-strength values. If lower-bound material properties are needed, they should be taken as mean minus one standard deviation values or they can be adjusted from expected-strength values in accordance with Section 9.4.2.5.

C9.4.2.2 Component Properties

1. **Elements.** Structural elements of the seismic-force-resisting system are composed of primary and secondary components, which collectively define element strength and resistance to deformation. Behavior of the components—including shear walls, beams, diaphragms, columns, and braces—is dictated by physical properties such as area; mechanical properties; thickness, depth, and slenderness ratios; lateral-torsional buckling resistance; and connection details.

   The actual physical dimensions should be measured. Modifications to members should be noted, including holes. The presence of corrosion or deformation should be noted.

   These primary component properties are needed to properly characterize building performance in the seismic analysis. The starting point for establishing component properties should be the available construction documents. Preliminary review of these documents should be performed to identify vertical-load (gravity-load) and seismic-force-resisting elements and systems, and their critical components and connections. Site inspections should be conducted to verify conditions and to ensure that remodeling has not changed the original design concept. In the absence of a complete set of construction documents, the design professional must thoroughly inspect the building to identify these elements, systems, and components, as indicated in Section 9.4.3.
2. **Connections.** The method of connecting the various components of the structural system is critical to its performance. The type and character of the connections must be determined by a review of the plans and a field verification of the conditions.

C9.4.2.3 Test Methods to Quantify Mechanical Properties To obtain the desired in-place mechanical properties of materials and components, including expected strength, it is often necessary to use proven destructive and nondestructive testing methods.

Section 9.4.2.5 addresses these established default strengths and distortion properties. This information may be used, together with tests from recovered samples or observation, to establish the expected properties for use in component strength and deformation analyses. Where possible, the load history for the building needs to be assessed for possible influence on component strength and deformation properties.

To quantify material properties and to analyze the performance of archaic CFS light-frame construction, shear walls, and diaphragm action, more extensive sampling and testing may be necessary. This testing should include further evaluation of load history and corrosion effects on properties and an examination of wall and diaphragm continuity and of the suitability of in-place connectors.

Where it is desired to use an existing assembly and little or no information about its performance is available, a cyclic load test of a mock-up of the existing structural elements can be used to determine the performance of various assemblies, connections, and load transfer conditions. See Section 7.6 for an explanation of the backbone curve and the establishment of alternative modeling parameters.

C9.4.2.4 Minimum Number of Tests To quantify expected strength and other in-place properties accurately, a minimum number of tests must be conducted on representative components. The minimum number of tests is dictated by available data from original construction, the type of structural system used, desired accuracy, and quality or condition of in-place materials. Visual access to the structural system also influences testing program definition. As an alternative, the design professional may elect to use the default strength properties in accordance with Section 9.4.2.5. However, using default values without testing is only permitted with the linear analysis procedures. It is strongly encouraged that the expected strengths be derived through testing of assemblies to model behavior accurately.

Removal of coverings, including the exterior wall covering, fireproofing, and partition materials, is generally required to facilitate sampling and observations.

Component types include studs, track, joists, straps, and sheathing and are found in gravity and seismic-force-resisting systems. The observations shall consist of each connector type present in the building (e.g., screws, bolts, and straps), such that the composite strength of the connection can be estimated.

C9.4.2.5 Default Mechanical Properties References to applicable ASTM standards can be found in AISI S100 (2020a) for cold-formed steel components in general and AISI S240 (2020b) for cold-formed steel components utilized specifically in light-frame construction.

Table 9-1 is a condensed version of similar information provided in AISI S400 (2020c). The default deflection values at yield and peak capacity for two plies of fastened steel sheet under shear are based on the work of Moen et al. (2016). The selected values are median values for monotonic testing. Cyclic testing exhibited significantly greater deformations, and variation across ply thickness and fastener diameter and head details were also observed. If more refined data is required, Moen et al. (2016) provides a more detailed prediction method, or testing should be conducted.

C9.4.3 Condition Assessment

C9.4.3.1 General The physical condition of existing components and elements and their connections must be examined for degradation. Degradation may include environmental effects (e.g., corrosion, fire damage, or chemical attack) or past or current loading effects (e.g., overload, damage from past earthquakes, fatigue, fracture, or buckling). The condition assessment should also examine for configuration problems observed in recent earthquakes, including effects of discontinuous components;

improper screwing, welding, or bolting; poor fit-up; and connection problems at the foundation level. Often, unfinished areas, such as attic spaces, basements, and crawl spaces, provide suitable access to structural components and can give a general indication of the condition of the rest of the structure. Invasive inspection of critical components and connections is typically required.

Component orientation, plumbness, and physical dimensions should be confirmed during an assessment. Connections in steel components, elements, and systems require special consideration and evaluation. The load path for the system must be determined, and each connection in the load path(s) must be evaluated. This evaluation includes diaphragm-to-component and component-to-component connections. The strength and deformation capacity of connections must be checked where the connection is attached to one or more components that are expected to experience significant inelastic response. Anchorage of exterior walls to roof and floors in concrete and masonry buildings, for which diaphragms are used for out-of-plane loading, requires detailed inspection.

The condition assessment also affords an opportunity to review other conditions that may influence cold-formed steel elements and systems and overall building performance. Of particular importance is the identification of other elements and components that may contribute to or impair the performance of the system in question, including infills, neighboring buildings, and equipment attachments. Limitations posed by existing coverings, wall and ceiling space insulation, and other conditions should also be defined such that prudent retrofit measures may be planned.

C9.4.3.2 Scope and Procedures For cold-formed steel elements and components, accessibility constraints may necessitate the use of instruments such as a fiberscope or video probe to reduce the amount of damage to covering materials and fabrics. The knowledge and insight gained from the condition assessment is invaluable for understanding load paths and the ability of components to resist and transfer loads. The degree of assessment performed also affects the knowledge factor, which is discussed in Section 9.4.4.

Direct visual inspection provides the most valuable information because it can be used to identify any configuration issues, it allows measurement of component dimensions, and it identifies the presence of degradation. The continuity of load paths may be established by viewing components and connection condition. From visual inspection, the need for other test methods to quantify the presence and degree of degradation may be established.

The scope of the removal effort is dictated by the component and element design. For example, in a strap-braced wall, exposure of several key connections may suffice if the physical condition is acceptable and the configuration matches the construction documents. However, for shear walls and diaphragms, it may be necessary to expose more connection points because of varying designs and the critical nature of the connections. For encased walls for which no construction documents exist, it is necessary to indirectly view or expose all primary end connections for verification.

The physical condition of components and connectors may also support the need to use certain destructive and nondestructive test methods. Devices normally used for the detection of reinforcing steel in concrete or masonry may be used to verify the diagonal strap bracing and hardware located beneath finish surfaces.

C9.4.3.3 Basis for the Mathematical Building Model The acceptance criteria for existing components depend on the design professional's knowledge of the condition of the structural system and material properties, as previously noted. Certain damage—such as water staining, evidence of prior leakage, corrosion, and buckling—may be acceptable. The design professional must establish a case-by-case acceptance for such damage on the basis of capacity loss or deformation constraints. Degradation at connection points should be carefully examined; significant capacity reductions may be involved, as well as a loss of ductility.

C9.5 GENERAL ASSUMPTIONS AND REQUIREMENTS FOR COLD-FORMED STEEL

C9.5.1 Stiffness

C9.5.1.2 Use of Nonlinear Procedures for Cold-Formed Steel Light-Frame Construction The generalized force–deformation relation for cold-formed steel components is similar to that used for structural steel. However, some care must be taken particularly with the definition of the end of the linear portion, that is, Point B. For structural steel, this is typically the point of initial yielding; however, for cold-formed steel components, this point may be below the point of first yield and is thus defined appropriately for each component in this standard.

C9.5.2 Strength and Acceptance Criteria

C9.5.2.2 Deformation-Controlled Actions The relative magnitude of the m-factors alone should not be interpreted as a direct indicator of performance. The stiffness of a component and its expected strength, Q_{CE}, must be considered where evaluating expected performance.

Tables 9-2 and 9-3 provide acceptance criteria relevant to cold-formed steel light-frame construction. To evaluate the criteria, expected strength and stiffness are addressed in Section 9.6 for cold-formed steel light-frame construction shear wall systems; in Section 9.8 for cold-formed steel light-frame construction, strap-braced wall systems; and in Section 9.7 for cold-formed steel moment-frame systems. Expected strength and stiffness for individual cold-formed steel flexural members are also addressed in Section 9.8 in relation to generic moment-frame response. Tables 9-2 and 9-3 also provide acceptance criteria for cold-formed steel framed wood structural panel sheathed diaphragms, and cold-formed steel steel-to-steel shear connections. These provisions are not addressed explicitly as a separate cold-formed steel component; however, they are addressed by this standard through the clause in this section. Strength and stiffness of a cold-formed steel framed wood structural panel sheathed diaphragm may be determined from AISI S400 (2020c). Strength of a cold-formed steel steel-to-steel shear connection may be determined from AISI S100 (2020a), and stiffness of a cold-formed steel steel-to-steel shear connection is provided in Section 9.4.2.5.

C9.5.2.3 Force-Controlled Actions This section now recognizes that strengths can be obtained experimentally. This is consistent with the strength derivation permitted in Section 9.5.2.2 for deformation-controlled actions and it is intended to allow the user to use either the experimental testing per Section 7.6 or in-situ sampling/testing per Section 9.4.2.3.

The maximum forces developed in yielding shear walls and diaphragms are consistently 1.5 to 2 times the yield force. Other components and connectors exhibit similar overstrength.

C9.5.3 Connection Requirements in Cold-Formed Steel Light-Frame Construction In considering connections in this standard, connectors are distinguished from bodies of connections and bodies of connection hardware. Connectors, which consist of the screws, welds, and bolts used to link pieces of a connection assembly together, are considered to be force-controlled since they possess little inherent deformation capacity. Connection ductility is derived from the bodies of the connections or bodies of connection hardware. The ductility in a light-frame shear wall or diaphragm assembly comes from the bearing of the connectors on a clip, plate, or sheathing material. In bolted connections, ductility is provided by yielding of the metal around the bolt hole. Brittle failure can occur in the bodies of connections, or in the bodies of connection hardware. Therefore, the connectors are assumed to be force-controlled, and the bodies of connections and bodies of connection hardware are considered force-controlled for fracture limit states and deformation-controlled for bearing and yielding limit states. Where determining the demand on force-controlled portions of the connection assembly, use of a limit-state analysis to determine the maximum force that can be delivered to the connection is recommended.

Where computing the strength of connections, all potential limit states should be considered, including those associated with the bodies of connections, the bodies of connection hardware, and connectors with which the assembly may be composed. For example, in addition to the strength of a tie-down device itself, limit states for the stud screws, foundation bolts, and net section of the end post should be considered. The controlling condition determines the expected or lower-bound strength of the connection.

C9.5.5 Retrofit Measures Special attention is required where connections such as bolts and screws are encountered.

Wood structural panels are used to provide lateral strength and stiffness to most modern CFS light-frame buildings and are generally recommended for the retrofit of horizontal diaphragms and shear walls of existing buildings. The system relies on the in-plane strength and stiffness of the panels and their connection to the framing. Panels are connected by screwing into the same structural member to create, in effect, one continuous panel. The various panels are described in Section 9.6.2. The performance of the structural panels is dependent to a great degree on the attachment to the framing. The attachment spacing and effectiveness should be investigated if the existing panels are expected to withstand significant loads. If fasteners are to be added to existing panels, they should be the same size as the existing fasteners.

C9.6 COLD-FORMED STEEL LIGHT-FRAME CONSTRUCTION, SHEAR WALL SYSTEMS

C9.6.1 General The behavior of cold-formed steel light-frame shear walls is complex and influenced by many factors; the primary factor is the wall sheathing. Provisions for combination of dissimilar materials on opposite sides of the wall require coordination of *m*-factors and modeling parameters for default shear wall types. Where test data are available, there is no restriction on consideration of strength and stiffness of the wall assembly sheathed on opposite sides with dissimilar materials. AISI S400 (2020c) provides additional guidance on the strength and stiffness of cold-formed steel framed shear walls sheathed with dissimilar materials.

Wall sheathings can be divided into many categories (e.g., brittle, elastic, strong, weak, good at dissipating energy, or poor at dissipating energy). In many existing buildings, the walls were not expected to act as shear walls (e.g., a partition wall).

A major factor influencing the behavior of shear walls is the aspect ratio of the wall. AISI S240 (2020b) and AISI S400 (2020c) limit the aspect ratio (height-to-width) for wood structural panel shear walls to 2:1 for full design shear capacity and permit reduced design shear capacities for walls with aspect ratios up to 4:1. The interaction of the floor and roof with the wall, the end conditions of the wall, and the redundancy or number of walls along any wall line would affect the wall behavior for walls with the same aspect ratio. In addition, the rigidity of the tie-downs at the wall ends has an important effect in the behavior of narrow walls.

The presence of any but small openings in shear walls causes a reduction in the stiffness and strength because of a reduced length of wall available to resist seismic forces. Special analysis techniques and detailing are required at the openings. The presence or addition of chord members around the openings reduces the loss in overall stiffness and limits damage in the area of openings. AISI S240 (2020b) and AISI S400 (2020c) cover design of shear walls with openings.

For cold-formed steel light-frame shear walls, the important limit states are sheathing failure, connection failure, tie-down failure, and excessive deflection. Limit states define the point of Life Safety. To reduce damage or retain usability immediately after an earthquake, deflection must be limited (see Chapter 1). The ultimate capacity is the maximum capacity of the assembly, regardless of the deflection.

AISI S213 (2012), AISI S240 (2020b), and AISI S400 (2020c) for cold-formed steel light-frame shear walls require capacity protection for the chord studs, anchorage, and collectors of the shear wall to ensure that the designated energy-dissipating mechanism is triggered and maintained. In an ASCE 7-based seismic design, this is achieved by designing the chord studs, anchorage, and collectors at Ω_E force levels. All applicable load combinations, including superposition of gravity and lateral load, still apply. In an ASCE 41-based seismic design, the capacity protection is achieved by designing the chord studs, anchorage, and collectors for the forces delivered to components from the expected strength of the shear wall as well as superposed gravity loads, per Chapter 7.

C9.6.2 Types of Cold-Formed Steel Light-Frame Construction, Shear Wall Systems

C9.6.2.1 Existing Cold-Formed Steel Light-Frame Shear Walls Cold-formed steel light-framed shear walls are a relatively new form of construction; before the introduction of shear wall design standards in the 1990s, common practice was generally to mimic wood construction. AISI S400 (2020c) provides guidelines for current best practices in cold-formed steel light-framed shear walls.

C9.6.2.2 Enhanced Cold-Formed Steel Light-Frame Shear Walls Possible retrofit methods for cold-formed steel light-frame shear walls include increasing attachment to existing sheathing and/or replacing existing sheathing on one or both sides. Strength and stiffness of the sheathing, connectors, shear wall boundary members, diaphragm, and foundation all must be checked to ensure that they meet the newly intended demands.

C9.6.3 Stiffness, Strength, Acceptance Criteria, and Connection Design for Cold-Formed Steel Light-Frame Construction Shear Wall Systems

C9.6.3.1.1 Stiffness of Wood Structural Panels The deflection at yield, defined here as Point B in Figure 9-1, is determined based on the secant stiffness at 40% of the expected strength, calculated at a force level of 80% of the expected strength. The deflection expression in AISI S400 (2020c) is nonlinear; the use of the secant stiffness at 40% is a practical simplification. It is possible to use the complete nonlinear curve up to the expected wall strength.

C9.6.3.1.3 Acceptance Criteria for Wood Structural Panels Acceptance criteria provided in Tables 9-2 and 9-3 were developed based on the experimental data that support AISI S400 (2020c).

C9.7 COLD-FORMED STEEL MOMENT-FRAME SYSTEMS

C9.7.3.1 Generic Cold-Formed Steel Moment Connection Often cold-formed steel members are not expected to develop moment-frame systems; this is because (1) connections are not typically detailed to transmit moment, and (2) cold-formed steel members are typically assumed to have limited rotational capacity. Existing AISI standards do not provide the rotational capacity of cold-formed steel members. This standard provides a means to assess the expected strength and rotational capacity of cold-formed steel members. Cold-formed steel members with inelastic reserve (i.e., expected strengths greater than the moment at first yield, M_y) often have substantial rotation capacity. The generalized force–deformation curve of Figure 9-1 is used with the notation as provided in Figure 9-2. Rotational capacity in both local buckling and distortional buckling must be assessed. The provisions assume that the member is adequately braced against lateral-torsional buckling. The provided expressions are based on the work of Ayhan and Schafer (2016).

C9.7.3.1.1 Strength of Generic Cold-Formed Steel Moment Connection Expected strength for the peak capacity, M_2, is the same as the nominal strength predictions in AISI S100 (2020a). See AISI S100 (2020a) for detailed explanations of local and distortional buckling and how to establish the elastic critical moments M_{crl} and M_{crd}.

C9.7.3.1.4 Connections for Cold-Formed Steel Generic Moment Connection Standard moment-frame connections do not exist for cold-formed steel systems; therefore, testing is required. Deformations in the connection are likely to be of a similar order of magnitude as member deformations and thus should be considered in any approximation of total system deformations. The connection may be designed as a force-controlled component, and deformations may be pursued primarily in the member—as envisioned in this section through providing member strength and stiffness properties per Sections 9.7.3.1.1 and 9.7.3.1.2. As an alternative, the connection may be designed as a deformation-controlled component, and deformations may be pursued primarily in the connection itself, as in the special bolted moment-frame system discussed in the next section.

C9.7.3.2 Cold-Formed Steel Special Bolted Moment Frame AISI S400 (2020c) provides complete design details for a bolted cold-formed steel moment-frame system that is also recognized in ASCE 7. The system uses hollow structural section (HSS) columns and cold-formed steel beams and relies on bearing deformations at the beam-to-column bolted connections to dissipate energy.

C9.7.3.2.3 Acceptance Criteria for Cold-Formed Steel Special Bolted Moment Frame Experiments conducted to develop AISI S400 (2020c) provide the necessary data to develop *m*-factors and modeling parameters, but to date the information has not yet been processed.

C9.8 COLD-FORMED STEEL LIGHT-FRAME CONSTRUCTION, STRAP-BRACED WALL SYSTEMS

C9.8.1 General AISI S400 (2020c) provides guidance on the strength and stiffness of cold-formed steel light-frame construction with strap-braced walls. Strap-braced walls designed to the 2009 and 2012 editions of AISI S213 or 2015 and later editions of AISI S400 have specific capacity-based design protections for the flat strap to ensure yielding of the flat strap similar to the expected-strength provisions of this standard. For 2015 and later editions of AISI S400, wall aspect ratios may be greater than 2.0, but additional checks on the chord studs are required. Thus, aspect ratios greater than 2.0 are only appropriate for new flat strap-braced wall systems. Testing has shown that one or two layers of gypsum board panel attached over the top of a strap and to the same framing affects the response, and thus separate acceptance factors, modeling parameters, and acceptance criteria have been provided for this case.

C9.8.2 Types of Cold-Formed Steel Light-Framed Construction with Strap-Braced Walls

C9.8.2.1 Existing Cold-Formed Steel Light-Frame Construction with Strap-Braced Walls Flat strap-braced walls are a popular form of design for cold-formed steel light-frame construction, particularly for wind resistance. Design of flat strap-braced walls generally follows basic mechanics. Specific wall design standards were introduced by AISI in the 1990s, and by 2009, capacity-based considerations for the straps and boundary members of the wall were introduced in AISI S213 (2012), now AISI S400 (2020c).

C9.8.2.2 Cold-Formed Steel Light-Frame Construction with Enhanced Strap-Braced Walls Possible retrofit methods for cold-formed steel light-frame construction with enhanced strap-braced walls include adding compatible bracing to both sides of a wall and/or replacing existing straps with straps detailed to ensure yielding on one or both sides. Strength and stiffness of the sheathing, connectors, shear wall boundary members, diaphragm, and foundation must be checked to ensure that they meet the newly intended demands.

C9.9 CFS DIAPHRAGMS

AISC 342 (2022a), Chapter G, now contains the latest provisions on bare steel deck diaphragms, steel deck diaphragms with reinforced concrete structural topping, and steel deck diaphragms with unreinforced structural concrete topping or lightweight insulating concrete. These provisions are applicable for buildings with CFS seismic-force-resisting systems.

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CHAPTER C10
CONCRETE

C10.3 MODIFICATIONS TO THE REFERENCE STANDARD

Modify Commentary Sections of ACI 369.1 Chapters 3 and 7, and Notation by replacing the referenced commentary in ACI 369.1 with the amended commentary in this section.

C10.3.1 General Assumptions and Requirements. ***Replace Section C3.1 of ACI 369.1 with the italicized text as follows.***

ACI 369.1 Chapter 3

C3.1.1 General *Brittle or low-ductility failure modes typically include behavior in direct or nearly direct compression; shear in slender components and in-component connections; torsion in slender components; and reinforcement development, splicing, and anchorage. The stresses, forces, and moments acting to cause these failure modes should be determined from a limit-state analysis, considering probable resistances at locations of nonlinear action.*

C3.1.2 Stiffness *For columns with low axial loads (below approximately* $0.1A_gf_c'$*), deformations caused by bar slip can account for as much as 50 percent of the total deformations at yield. Further guidance regarding calculation of the effective stiffness of reinforced concrete columns that include the effects of flexure, shear, and bar slip can be found in Elwood and Eberhard (2009). Flexure-controlled wall stiffness can vary from approximately* $0.15E_{cE}I_g$ *to* $1.0E_{cE}I_g$*, depending on wall longitudinal reinforcement and axial load (Abdullah 2019). A method for calculating wall stiffness, which provides compatibility with fiber section analysis, is offered in C7.3.1.*

C3.1.2.1 Linear procedures. *The effective flexural rigidity values in Table 3.1.2.1 for beams, columns, and walls account for the additional flexibility from reinforcement slip within the beam-column joint or foundation before yielding. The values specified for columns were determined based on a database of 221 rectangular reinforced concrete column tests with axial loads less than* $0.67A_gf_c'$ *and shear span-depth ratios greater than 1.4. Measured effective stiffnesses from the laboratory test data suggest that the effective flexural rigidity for low axial loads could be approximated as* $0.2EI_g$*; however, considering the scatter in the effective flexural rigidity and to avoid under-estimating the shear demand on columns with low axial loads,* $0.3EI_g$ *is recommended in Table 3.1.2.1 (Elwood et al. 2007). In addition to axial load, the shear span-depth ratio of the column influences the effective flexural rigidity. A more refined estimate of the effective flexural rigidity can be determined by calculating the displacement at yield caused by flexure, slip, and shear (Elwood and Eberhard 2009).*

Wall stiffness values were extracted from experimental data sets for shear-critical walls included in the dataset assembled by Abdullah (2019); these data were used to update wall cracked and uncracked stiffness values in Table 3.1.2.1. The flexural rigidity of cracked walls was found to be significantly influenced by axial load. For simplicity in linear procedures, a single value of $0.25E_{cE}I_g$ *was selected for the effective wall flexural rigidity, which corresponds approximately to the value from experimental tests for walls with an axial load ratio of* $0.1A_gf_{cE}'$*. This effective rigidity from Table 3.1.2.1 includes the effects of bond slip along with flexural cracking, and a further reduction in stiffness is not required to account for bond slip. The method for deriving effective stiffness based on a constant yield curvature methodology discussed in C7.3.1 has an explicit term related to bond slip in Equation C7.3.1d, however this additional flexibility should only be used with methods that separate flexural stiffness independently from bond slip.*

Previous versions of the standard specified a shear rigidity of $0.4E_{cE}A_w$ *for uncracked walls based on first principles for relating elastic shear modulus to modulus of elasticity with an assumed Poisson's ratio of 0.25 for concrete. Analysis of experimental data showed that shear rigidity was lower, likely due to shrinkage cracking, and that the average shear rigidity was approximately* $0.3E_{cE}A_w$*. Shear and flexural cracked stiffness coefficients were derived relative to the load-deformation data from the Abdullah 2019 database using cantilever wall flexibility idealizations to generalized yield: flexure-controlled walls assumed a shear flexibility and adjusted moment of inertia to match the idealized yield deformation; while for shear, a subset of shear-controlled walls were isolated which had experimental reported cracking points for initial shear flexibility. Additional commentary is given in C7.3.1.*

The modeling recommendations for beam-column joints (6.2.2.1) do not include the influence of reinforcement slip. When the effective stiffness values for beams and columns from Table 3.1.2.1 are used in combination with the modeling recommendations for beam-column joints, the overall stiffness is in close agreement with results from beam-column subassembly tests (Elwood et al. 2007).

The effect of reinforcement slip can be accounted for by including rotational springs at the ends of the beam or column elements (Saatcioglu et al. 1992). If this modeling option is selected, the effective flexural rigidity of the column element should reflect only the flexibility from flexural deformations. In this case, for axial loads less than $0.3A_gf_{cE}'$ *the effective flexural rigidity can be estimated as* $0.5E_{cE}I_g$*, with linear interpolation to the value given in Table 3.1.2.1 for axial loads greater than* $0.5A_gf_{cE}'$

Because of low bond stress between concrete and plain reinforcement without deformations, components with plain longitudinal reinforcement and axial loads less than $0.5A_gf_{cE}'$ *can have lower effective flexural rigidity values than in Table 3.1.2.1.*

C3.1.2.2.3 While there are a variety of element types and analytical methods available for simulation of nonlinear load-deformation relationships generalized in Figure 3.1.2.2.3, it is noted that analytical results can be sensitive to abrupt changes in assigned stiffness within and between components. Convergence issues may also be encountered, for example in rigid-plastic hinges relative to semi-rigid-plastic hinges, and care should be taken to confirm loading and unloading stiffness are representative of expected component behavior and as prescribed in Chapters 4 through 12.

Alternatively, it shall be permitted to base the nonlinear load-deformation relation derived from laboratory test data for components or subassemblages (1) subjected to gravity load effects and lateral load or deformation histories similar to those expected for building components and (2) exhibiting response modes similar to those expected for building components. Where experimental data sets are used to define nonlinear action-deformation relations, simulated analytical deformation demands shall not exceed the maximum tested deformation imposed on the component or subassemblages used for model calibration.

C3.1.2.2.3. Several nonlinear modeling parameters in this standard … are related to the applied axial load on the member. It is noted that the effect of asymmetry on strength and deformation capacity are most critical in evaluating performance at ultimate deformation demands, and this standard permits the use of average cracked section properties based on gravity load effects for evaluating stiffness as a practical simplification. Where axial loads vary significantly due to earthquake effects, accurate simulation of stiffness and deformation capacity requires that these model parameters be updated during the analysis to reflect the changing axial loads. Most structural analysis software used for nonlinear analysis simulate changes in axial and flexural rigidity due to changes in axial load but do not include shear flexibility as a function of axial load. If such models are used, the effects of earthquake axial load on the force-deformation backbone relations may be incorporated by applying differing modeling parameters in each loading direction that correspond to the earthquake axial loads in each direction. The earthquake axial loads should be estimated in either direction of loading at anticipated drift levels. This process may be iterative, starting with preliminary modeling parameters based on gravity loads to obtain estimates of earthquake-induced axial loads. The modeling parameters would then be updated based on those axial loads that include earthquake effects. This process results in non-symmetric backbone relations with respect to direction of loading.

C3.1.2.2.4. The effective stiffness coefficient for beams in Table 3.1.2.2.4 was inferred from test data and was calibrated to provide the mean secant stiffness to yield of beam subassemblies subjected to cyclic loading. It is intended for use with the nonlinear analysis procedures (NSP and NDP). The effective stiffness coefficient for beams in Table 3.1.2.2.4 was not calibrated to produce estimates of drift demand of linear systems similar to those calculated using nonlinear dynamic analyses (Sozen 2013).

Govindan (2018) performed a regression analysis of data from 58 beam tests and found that the effective stiffness coefficient of elements with axial load ratios $P/f'_{cE}A_g$ between 0 and 0.125 had a mean of 0.22 with a coefficient of variation of 35%. In the analysis by Govindan, an effective stiffness coefficient of 0.20 corresponded to the 45th percentile of the test data, a coefficient of 0.25 corresponded to the 64th percentile, and 81% of beams had an effective stiffness coefficient lower than 0.3. Effective stiffness in the regression analysis performed by Govindan was defined on the basis of the secant to the yield point of experimental data, defined by extrapolating a secant to the deformation at 75% of yield to the yield moment.

Owing to the manner in which it was derived, the proposed effective stiffness coefficient for beams includes flexibilities related to flexure, shear, and reinforcement slip. Calculated deformation related to shear at yield for the beam data set compiled by Govindan (2018) based on an elastic shear modulus equal to $0.4E_{cE}$ were very small compared with the total deformation at yield, ranging between 0.4% and 2.1% of the chord rotation at yield, with a mean of approximately 1.2%.

Wall stiffness values obtained from a large data sets of wall tests (Abdullah 2019) were used to update the wall cracked and uncracked stiffness values in Table 3.1.2.2.4 in the 2022 version of this standard. … For shear-controlled walls, the deformation at shear yield is provided as a nonlinear modeling parameter in 7.4.1.1.2. It is noted that the wall shear stiffness values are reported in terms of E_{cE}, and not shear modulus, G, used by most finite element formulations, whereby a 0.4 coefficient is associated with elastic shear stiffness incorporating an embedded Poisson ratio of 0.25. This version of the standard reduced the coefficient from the traditional elastic shear stiffness of $0.4E_{cE}A_w$ based on regression analysis from the Abdullah 2019 wall database. Additional commentary is given in C7.3.1.

C3.1.2.2 Nonlinear procedures, *The generalized load-deformation relations shown in Figure 3.1.2.2.3 are described by linear response from Point A (unloaded component) to an effective yield Point B, then a linear response at reduced stiffness from Point B to C, then reduction in lateral strength to Point D, then response at residual strength to Point E, and loss of the residual strength or gravity load carrying capacity thereafter. Typically, the response shown in Figure 3.1.2.2.3 is associated with flexural response or tension response. In this case, the resistance at $Q/Q_{yE} = 1.0$ is the yield value, and subsequent strain hardening is accommodated by hardening in the load-deformation relation as the member is deformed toward the expected strength. Where the response shown in Figure 3.1.2.2.3 is associated with compression, the resistance at $Q/Q_{yE} = 1.0$ typically is the value where concrete begins to spall, and strain hardening in well-confined sections can be associated with strain hardening of the longitudinal reinforcement and an increase in strength from the confinement of concrete. Where the response shown in Figure 3.1.2.2.3 is associated with shear, the resistance at $Q/Q_{yE} = 1.0$ typically is the value at which the design shear strength is reached and, typically, no strain hardening follows.*

The deformations used for the load-deformation relation of Figure 3.1.2.2.3 should be defined in one of two ways, as follows:

(a) ***Deformation, or Type I:*** *In this curve, deformations are expressed directly using terms such as strain, curvature, rotation, or elongation. The parameters $a_{n\ell}$ and $b_{n\ell}$ refer to deformation portions that occur after yield, or plastic deformation. The parameter $c_{n\ell}$ is the reduced resistance after the sudden reduction from C to D. Parameters $a_{n\ell}$, $b_{n\ell}$, and $c_{n\ell}$ are defined numerically in various tables in this standard. Alternatively, parameters $a_{n\ell}$, $b_{n\ell}$, and $c_{n\ell}$ can be determined directly by analytical procedures justified by experimental evidence.*

(b) ***Deformation ratio, or Type II:*** *In this curve, deformations are expressed in terms such as shear angle and tangential drift ratio. The parameters $d_{n\ell}$ and $e_{n\ell}$ refer to total deformations measured from the origin. Parameters $c_{n\ell}$, $d_{n\ell}$, and $e_{n\ell}$ are defined numerically in various tables in this standard.*

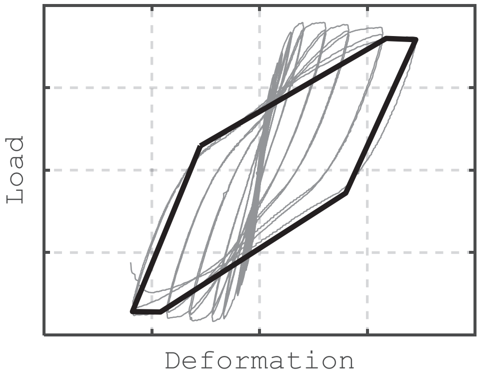
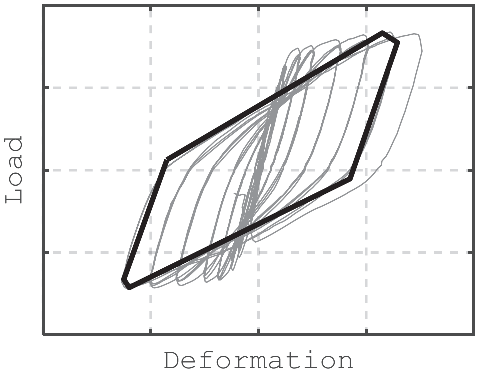
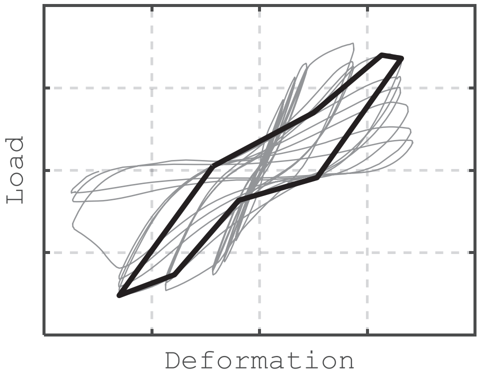
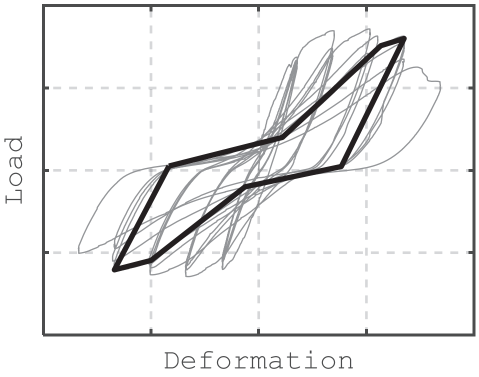
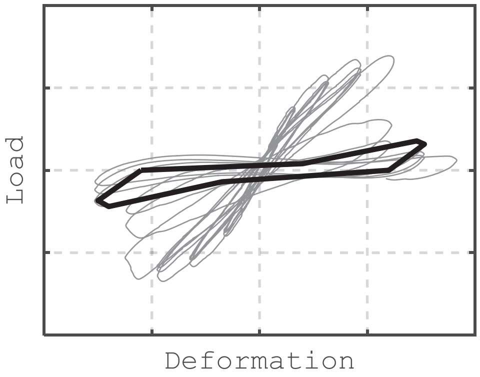
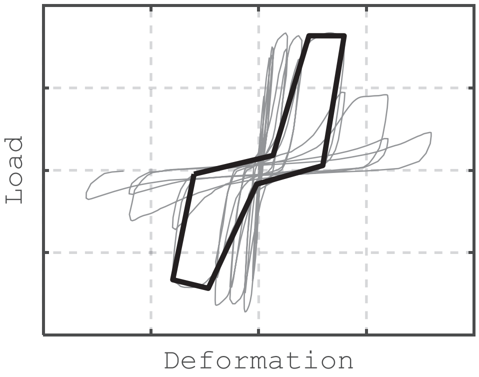
Pinching Level	Hysteresis Plots (Columns)	Hysteresis Plots (Walls)
Low: Type A	**Sokoli and Ghannoum, 2016** 	**Dasti et al., 2017** 
Moderate: Type B	**Kim et al., 2011** 	**Christidis and Trezos, 2017** 
Significant: Type C	**Woods and Matamoros, 2010** 	**Anoda, 2014** 
— Experiment ▬ Numerical Model (one loop)		

Figure C3.1.2.2a. General hysteresis types.

Alternatively, parameters $c_{n\ell}$, $d_{n\ell}$, and $e_{n\ell}$ can be determined directly by analytical procedures justified by experimental evidence.

Provisions for determining alternative modeling parameters and acceptance criteria based on experimental evidence are given in ASCE 41, 7.6.

Displacement demands determined from nonlinear dynamic analysis are sensitive to the rate of strength degradation included in the structural model. However, the available experimental results for establishing the strength degradation relationship are limited and exhibit wide ranges of behavior. Unless there is experimental evidence of sudden strength loss for a particular component under consideration, the use of a model with a sudden strength loss from Point C to D in Figure 3.1.2.2.3 can result in overestimation of the drift demands for a structural system and individual components. A more realistic model for many concrete components has a linear degradation in resistance from Point C to E, as shown in Figure 3.1.2.2.3 with a dashed line.

Strength loss that occurs within a single cycle can result in dynamic instability of the structure, whereas strength loss that occurs between cycles is unlikely to cause such instability.

Figure 3.1.2.2.3 does not distinguish between these types of strength degradation and may not accurately predict the displacement demands if the two forms of strength degradation are not properly considered.

Common shapes of the hysteresis under force or deformation reversals shown in Figure C3.1.2.2a illustrate stiffness degradations and energy dissipation under cyclic loading. Type A represents the behavior of the components with low pinching such as columns subjected to low axial loads adequately detailed for toughness. Type B represents the behavior of the components with moderate pinching such as a column under moderate axial load. Type C represents the hysteretic behavior of the components with significant pinching such as a poorly detailed column with high axial load or the components sustaining sliding shear or splice failures.

Some commercially available analysis software has features that enable calculating the fraction of the total ground motion input energy dissipated through different sources, such as inelastic hysteretic behavior of deformation-controlled actions and structural damping. Where this information is available it should be compared with characteristic values reported in the literature to evaluate the adequacy of the model . . . .

Nonlinear fiber elements formulated solely on the basis of stress-strain relationships directly obtained from tensile and compressive tests of axially loaded reinforcing steel coupons or concrete cylinders may not completely represent the nonlinear behavior of reinforced concrete components because uniaxial material tests do not include phenomena such as bar slip, bar buckling, cover spalling, and cyclic response. Because the objective is to accurately reproduce member response, stress-strain relations and mesh configuration of fiber elements should be adjusted to verify that the calculated response obtained with the numerical model produces action-deformation envelop curves similar to those obtained with the modeling parameters provided in this standard or obtained from experimental data. Pugh et al. (2017), Lowes et al. (2016), and Marafi et al. (2019) provide guidance on regularization of concrete and steel material models to minimize mesh-sensitivity in models that comprise solid elements, layered and fiber shell elements, and beam-column elements with fiber-type sections.

Fiber-type section models are often used to simulate the nonlinear cyclic response of concrete members controlled by flexure. Fiber-section models provide explicit simulation of the impact of axial load variation on component response and are appropriate for use for columns and walls for which variation in axial load during earthquake loading may alter strength, stiffness, and deformation capacity. When constructing these models, the cross-section of a structural component is discretized using fibers with constitutive behavior representative of unconfined and confined concrete, as well as reinforcing steel bars. Fiber-section models should reproduce the action-deformation response of components to produce numerical models that accurately simulate the response of the system. To that effect, even though fiber-section models are calibrated to accurately reproduce component behavior, local responses such as curvatures or material strains may not be adequately reproduced.

At hinge locations, axial deformation and curvature of each fiber section along the length of the hinge are multiplied by an integration length, which is often an assumed plastic hinge length, to compute an axial load-axial deformation or moment-rotation response history, respectively. To obtain a computed response that is in substantial agreement with the results of physical tests, the fiber discretization of a section must be sufficient to represent the stress and strain field at the section level, and concrete and reinforcement material models must represent characteristic features of the cyclic material stress-strain histories, where material response includes strain softening. The softening portion of the stress-strain response history must be defined accounting for the energy dissipated during softening and the integration length associated with the fiber-section.

Fiber-type section models may be embedded in zero-length hinge elements, in finite-length hinge elements and in beam-column elements with integration schemes that assume distributed or lumped plasticity. For components for which deformations due to response modes other than flexural and axial response modes are significant, it is necessary to combine the fiber-type models with models that simulate the other significant deformation modes such as shear and reinforcing bar slippage.

A limitation of typical fiber-type section models, which may significantly impact simulation results, is the assumption that

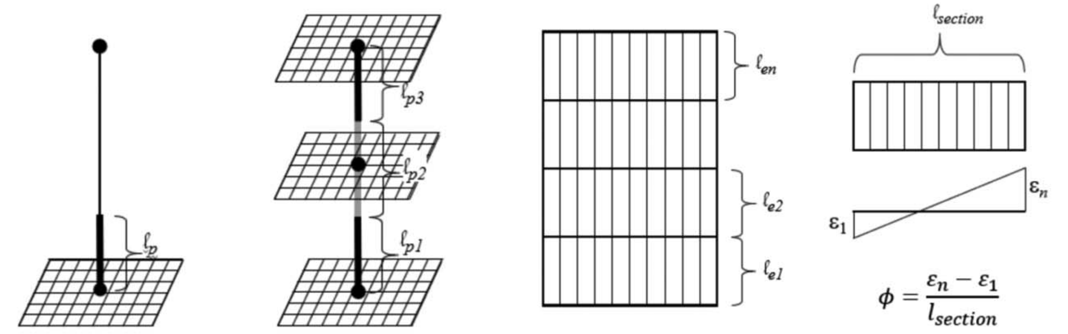


Figure C1a: lumped-plasticity beam-column line-element, 2D fiber section element located at element critical section

Figure C1b: distributed-plasticity beam-column line-element, 2D fiber sections located at element ends and distributed within element

Figure C1c: fiber shell element model of planar wall; mesh comprises a single element along the horizontal length of the wall and multiple elements along the vertical height of the wall

Figure C1d: calculation of fiber section curvature from extreme fiber strains

$$\theta = \phi_{e1} * l_p \text{ for } lp < l_{e1}$$

$$\theta = \phi_{e1} * l_{e1} + \phi_{e2} * (l_p - le_1) \text{ for } le_1 < l_p < le_1 + le_2$$

Figure C3.1.2.2b. Nonlinear wall modeling options.

plane sections remain plane and perpendicular to the neutral axis. Using this assumption, it is not possible to directly simulate flexure-shear interaction or deformation associated with slip of anchored longitudinal reinforcement.

Nonlinear axial deformations of fiber sections may not be compatible with those in lumped plastic hinges with axial force and bending moment interaction. It is not recommended for the design professional to embed a frame element with lumped plastic hinges in a fiber-type model for the simulation of an embedded column or pilaster in a wall unless force-deformation behavior of the wall-column assembly model is consistent with modeling requirements in Chapter 7 for walls or experimental data.

Within this standard, rotation is the primary measure used to define component response and acceptance criteria. When the nonlinear response of a structure is simulated using a modeling approach other than a concentrated hinge, defined by a moment-rotation or story shear-deflection history, it is necessary to convert between the rotations specified in this standard and the deformation measures employed within the model. To make these conversions, it is recommended that the component rotations specified within this Standard be compared with a rotation computed using (1) curvature at the critical section and, as needed, curvature computed at additional sections, as well as (2) a hinge length (l_p) equivalent to that specified within this standard or justified by experimental evidence for the hinge rotation being considered. Figure C3.1.2.2b shows an example of this conversion for a wall modeled using multiple approaches.

NIST GCR 17-917-45 and -46 also provide state-of-the-art recommendations for nonlinear modeling of structural elements where nonlinear response is highly dependent on the loading protocol, especially for large inelastic deformation demands, for which adaptive hysteretic models are recommended, provided that implementation and use of an adaptive model are practical and feasible in analysis software. Modeling parameters in this standard are based on the experimental data of components subjected to cyclic loading and may underestimate energy dissipation and deformation capacities of deformation-controlled members subjected to limited load or deformation reversals.

For columns or walls under the combined effects of axial load and bi-directional lateral loads, the ratio of deformation demand to deformation capacity in the acceptance criterion of the principal directions, should be combined using an appropriate combination rule such as square root of the sum of the squares. Criteria to account for the interaction of shear forces acting along orthogonal axes is provided in 22.5.1 of ACI 318.

C10.3.2 Concrete Structural Walls. ***Replace Sections C7.1 through C7.7 of ACI 369.1 with the italicized text as follows.***

ACI 369.1 Chapter 7

C7.1 TYPES OF CONCRETE STRUCTURAL WALLS AND ASSOCIATED COMPONENTS

Concrete structural walls are typically vertical elements of rectangular cross-section or combinations of interconnected rectangular elements that serve as lateral load-resisting elements in concrete structures. Structural walls (or wall segments) have traditionally been considered slender, or controlled by flexure, if their aspect ratio [(h_w/ℓ_w (height/length)] is greater than 3.0 and considered squat, or controlled by shear, if their aspect ratio is less than 1.5. However, recent examination of a wall test database has demonstrated that wall failure modes are more closely related to shear and flexure demands and demand-capacity ratios, and that there is little correlation between failure mode and wall aspect ratio or shear span-to-depth ratio (Abdullah 2019, Pugh et al. 2017). As such, this standard classifies walls as either flexure- or shear-controlled based on shear and flexure demands and demand-capacity ratios.

Identification of component types in concrete structural wall elements depends, to some degree, on the relative strengths of the wall segments based on expected or measured material properties. Vertical segments are often termed wall piers, whereas horizontal segments can be called coupling beams or spandrels. The licensed design professional is referred to FEMA 306 for additional information regarding the behavior of concrete wall components. Selected information from FEMA 306 has been reproduced in Table C7.1 and Figure C7.1 to clarify wall component identification.

Walls or wall segments with flanged cross-sections can behave differently in terms of stiffness, strength, and ductility from those with rectangular cross-sections (Abdullah et al. 2022, Abdullah and Wallace 2021, Gulec and Whittaker 2011). Flanged walls include walls with barbell-shaped, C-shaped, T-shaped, and other non-rectangular-shaped cross-sections. A designation of walls with flanged or rectangular cross-sections is introduced based on the ratio of gross-section moment of inertia of the section bounded by the effective flange width defined in Section 3.1.3 (I_{g_flange}) to that of the rectangular portion of the cross-section in the direction of loading (I_{g_rect}). Flanged walls are designated as those with I_{g_flange}/I_{g_rect} equal, or greater, than 1.5. For rectangular sections, I_{g_flange}/I_{g_rect} equals to 1.0. For wall sections with I_{g_flange}/I_{g_rect} ranging between 1.0 and 1.5, linear interpolation between the parameters for rectangular and flanged sections is permitted based on the ratio of moments of inertia (I_{g_flange}/I_{g_rect}).

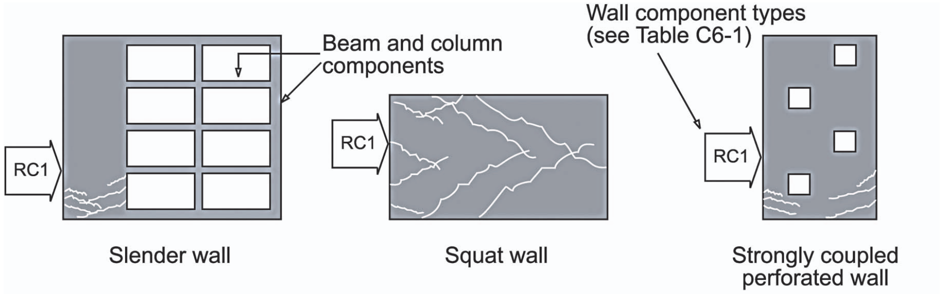
C7.1.1 Monolithic reinforced concrete structural walls and wall segments. *The wall reinforcement is normally continuous in both the horizontal and vertical directions, and bars are typically lap-spliced for tension continuity. The reinforcement mesh can also contain horizontal ties around vertical bars that are concentrated either near the vertical edges of a wall with constant thickness or in boundary members formed at the wall edges. The amount and spacing of these ties is important for determining how well the concrete at the wall edge is confined and, thus, for determining the lateral deformation capacity of the wall.*

In general, slender reinforced concrete structural walls are . . . governed by flexural deformations and tend to form a plastic flexural hinge near the base of the wall under severe lateral loading. The ductility of the wall is a function of the amount and distribution of longitudinal reinforcement, level of axial load, amount of lateral shear required to cause flexural yielding, lateral stability, . . . and transverse reinforcement in the boundary elements, including the ratio of the transverse reinforcement spacing to the diameter of the longitudinal reinforcing bars. In general, higher axial load and shear stresses reduce the flexural ductility and energy-absorbing capability of the wall. Short or squat structural walls are normally governed by shear deformations. These walls normally have a limited ability to deform beyond the elastic range and continue to resist seismic forces. Thus, these walls are typically analyzed either as displacement-controlled components with low ductility capacities or as force-controlled components.

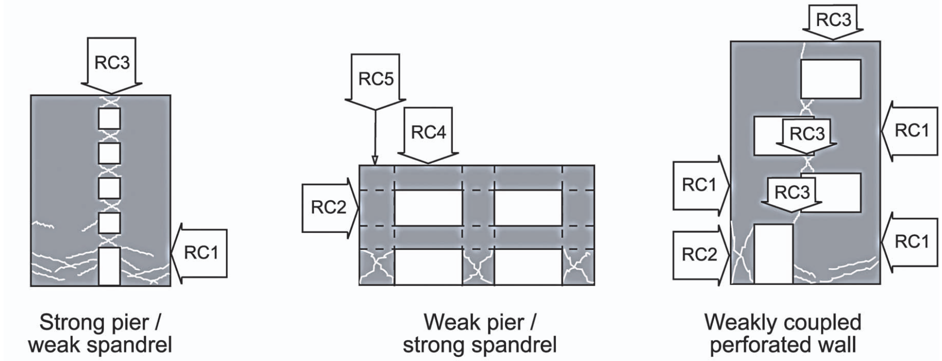
C7.1.2 Reinforced concrete columns supporting discontinuous structural walls. *In structural wall buildings, it is not uncommon to find that some walls are terminated either to create commercial space in the first story or to create parking spaces in*

Table C7.1. Reinforced Concrete Structural Wall Component Types.

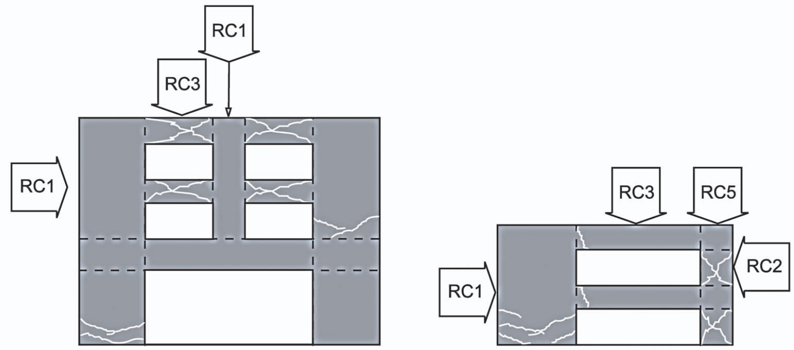
Component type per FEMA 306		Description	ASCE 41 designation
RC1	Isolated wall or stronger wall pier	Stronger than beam or spandrel components that can frame into it so that nonlinear behavior (and damage) is generally concentrated at the base, with a flexural plastic hinge or shear failure. Includes isolated (cantilever) walls. If the component has a major setback or cutoff of reinforcement above the base, this section should be also checked for nonlinear behavior.	Monolithic reinforced concrete wall or vertical wall segment
RC2	Weaker wall pier	Weaker than the spandrels to which it connects; characterized by flexural hinging top and bottom or shear failure.	
RC3	Weaker spandrel or coupling beam	Weaker than the wall piers to which it connects; characterized by hinging at each end, shear failure, or sliding shear failure.	Horizontal wall segment or coupling beam
RC4	Stronger spandrel	Should not suffer damage because it is stronger than attached wall piers. If this component is damaged, it should probably be reclassified as RC3.	
RC5	Pier-spandrel panel zone	Typically, not a critical area in RC walls.	Wall segment



(a) Cantilever Wall Mechanisms



(b) Pier / Spandrel Mechanisms



(c) Mixed Mechanisms

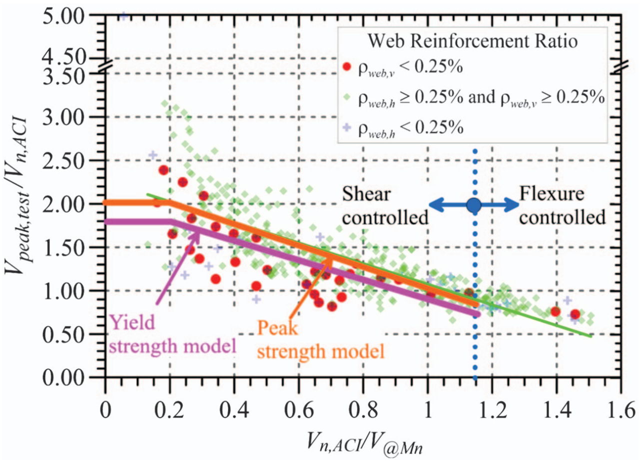
Figure C7.1. Identification of component types in concrete structural wall elements.
Source: FEMA 306 (ATC 1998).

the basement. In such cases, the walls are commonly supported by columns. Such designs are not recommended in seismic zones because very large demands can be placed on these columns during earthquake loading. In older buildings, such columns often have standard longitudinal and transverse reinforcement; the behavior of such columns during past earthquakes indicates that tightly spaced closed ties with well-anchored 135-degree hooks are required for the building to survive severe seismic forces.

C7.1.3 Reinforced concrete coupling beams. *Coupled walls are generally much stiffer and stronger than they would be if they acted independently. Coupling beams typically have a small span-depth ratio, and their inelastic behavior is normally affected by the high shear forces acting in these components. Coupling beams in most older reinforced concrete buildings commonly have conventional reinforcement that consists of longitudinal flexural reinforcement and transverse reinforcement for shear. In some more modern buildings, or in buildings where coupled structural walls are used for seismic retrofit, the coupling beams can use diagonal reinforcement as the primary reinforcement for both flexure and shear. The inelastic behavior of coupling beams that use diagonal reinforcement has been shown experimentally to be much better with respect to retention of strength, stiffness, deformation capacity, and energy dissipation capacity than the observed behavior of coupling beams with longitudinal reinforcement.*

C7.2.1 Flexural Strength. *The 1.15 overstrength in Section 7.2.1 represents the mean overstrength factor for measured experimental ultimate moment capacity relative to expected yield strength of flexure-controlled walls from Abdullah's database (2019), with a COV of 0.10 and 0.15 for Conforming and Nonconforming walls, respectively. The permission of a 1.15 factor applied to* f_{yE} *to represent strain hardening is not based on experimental data but is expected to result in similar overstrength calculation when using cross-sectional fiber analysis. This factor is intended to represent material overstrength effects relative to expected material properties. A 1.1 factor for converting from lower-bound to expected reinforcing steel properties, combined with a 1.15 factor for material post-yield strength gain, results in a similar value when compared to the traditional combined overstrength factor of 1.25 in Table 2.2.1.2.*

C7.2.2 Shear Strength. *The cracking shear strength is taken as the concrete contribution term to wall shear strength based on ACI 318 Chapter 18 for rectangular sections. Flanged walls were determined to exhibit higher cracking shear strength than rectangular walls based on a dataset of 290 wall tests, with 142 rectangular walls, 135 flanged or barbell walls, and 13 T-shaped or half-barbell walls (Abdullah and Wallace 2021, Abdullah et al. 2022). The shear strength relation in Chapter 18 of ACI 318 for structural walls (*$V_{CWall318}$*) was found to be related to moment demand. This is because the ACI 318-19 shear strength equation does not include the influence of axial load, amount and distribution of longitudinal reinforcement in the web and boundaries, and cross-section shape – variables that are considered in moment strength calculations and have been shown to impact wall shear strength (Abdullah and Wallace, 2021, Abdullah et al. 2022). Therefore, to obtain yield shear strength, an adjustment factor is provided to the ACI 318 relation as a function of the ratio of unadjusted shear strength (*$V_{CWall318E}$*) to shear demand at flexural yielding (*V_{MCyDE}*). It is noted this adjustment to the ACI 318 wall shear strength equation will result in a different shear strength calculated using this Standard relative to ACI 318: the intent of this standard is to provide a best-estimate of median expected component strength and accurate distribution of component mechanisms for reliability in deformation-controlled and force-controlled acceptance criteria, whereas ACI 318 targets a minimum reliability threshold in component strength calculations directly.*



Specimen Shear Strength versus Shear Demand Associated with Flexural Yielding. *Chapter 18 of ACI 318 requires that at least two curtains of web vertical and horizontal reinforcement be used in a wall if the shear demand exceeds* $2\lambda A_{cv}\sqrt{f'_c}$ *or if the aspect ratio is greater than, or equal to, 2.0. Experimental results by Hidalgo et al. (2002), Orakcal et al. (2009), and Abdullah (2019) show that there is no significant difference between the strength of walls with one or two curtains of web reinforcement or whether or not the wall satisfies the maximum spacing requirements of ACI 318. If plain concrete is encountered in an existing building, Chapter 14 of ACI 318 can be used to derive capacities.*

C7.2.3 Shear-Friction Strength. *Shear-friction strength should be evaluated at all possible failure planes along a wall or wall segment height, such as at the end of dowel bars, at an interface between dissimilar materials, or at an interface between two concretes cast at different times. Two equations are provided for shear-friction strength. The simpler Equation (7.2.4) is similar to the shear-friction strength equation in ACI 318, Chapter 22 but is applied with lower friction coefficients. Equation (7.2.3) is based on Equation (7.2.4) but accounts for the impact of moment demand on shear-friction strength and provides improved estimates of shear-friction strength for walls and wall segments (Abdullah et al. (2021b)). Results reported by Abdullah et al. (2021b) on a dataset of 70 shear-friction-controlled walls tested under reversed cyclic loading protocols, demonstrate that shear-friction coefficients at concrete interfaces transferring cyclic shear and moment demands are not significantly influenced by the type of interfaces, and exhibit shear-friction coefficients on the lower end of values for interfaces not cycled (ACI 318, 22.9.4.2). The provisions of ACI 318 for shear-friction strength were developed primarily based on results from "push-off" tests under monotonic loading protocols, which differ from wall loading conditions under earthquakes. For walls under relatively low moment demands, Equation (7.2.3) results in shear-friction strengths in closer agreement with those in ACI 318, Chapter 22. Additionally, the upper-limit on shear-friction strength in ACI 318, Table 22.9.4.4e of 800 psi is not justified by experimental evidence and is not considered in Equation (7.2.3) and (7.2.4) (Abdullah et al. 2021b). For walls with higher strength bars, the yield strength of the reinforcing bars was observed not to fully* *mobilize at the interface and is therefore limited to 75,000 psi. Equation (7.2.4) assumes that reinforcement is normal to the interface. For inclined reinforcement, adjustments to the equation should be made as provided in ACI 318. The shear-friction strength equations provided were calibrated for wall and wall segment potential interfaces sustaining cyclic flexural loading. They are not intended for interfaces with differing boundary conditions such as the vertical plane at slab-wall interfaces, which should be evaluated using the provisions for cast-in-place diaphragms.*

Results of a dataset of 53 shear-friction-controlled walls reported by Abdullah (2019) demonstrate that the upper limit of ACI 318, Table 22.9.4.4e significantly underestimates wall shear-friction strength, especially for walls with high strength concrete. As such, the upper-limit in ACI 318-19, Table 22.9.4.4e need not apply.

C7.3 LINEAR STATIC AND DYNAMIC PROCEDURES FOR STRUCTURAL WALLS AND WALL SEGMENTS

C7.3.1 *Modeling Slender structural walls and wall segments may be modeled using solid elements, shell elements, or beam-column elements. Accurate representation of stiffness, and strength can be achieved using appropriate element formulations and level of mesh refinement.*

Element stiffness recommendations for flexure-controlled structural walls are intended to provide a secant-to-yield stiffness, neglecting the effect of wall stiffness properties prior to flexural cracking on the calculated response. When significant flexural cracking is expected to occur, the initial wall stiffness is not considered to have a significant effect on calculated nonlinear deformations because demands generally exceed the cracking load during the first significant cycle of dynamic loading. In cases where little to no cracking is expected to occur, the licensed design professional may use iterative analytical techniques to obtain a more accurate approximation of the wall stiffness.

To calculate the effective stiffness to yield of flexure-controlled walls, the 2013 version of ASCE 41 recommended using an effective stiffness of 0.5 times the gross moment of inertia (I_g). However, experimental studies of slender walls pushed to yield-level drifts have shown lower stiffness reduction factors, in the range of 0.15 to 0.25 times the gross moment of inertia (PEER 2010, Panagiotou and Restrepo 2007, Priestley et al. 2007). The 2022 version of this standard updated the wall stiffness values to account for the axial load and longitudinal reinforcement ratio.

Results of a large dataset of flexure-controlled wall tests (Abdullah 2019) revealed that cracked effective flexural stiffness is significantly influenced by sustained axial load such that with axial load increasing from zero to $0.5A_g f'_{cE}$, cracked effective flexural stiffness increases, on average, by a factor of five (i.e., from 0.2 to $1.0E_{cE}I_g$). The results also showed that longitudinal reinforcement ratio has a significant influence on cracked effective stiffness, especially for walls with low to moderate sustained axial loads. The modifications to Table 3.1.2.1 and addition of Table 7.3.1 reflect the statistics and results of the dataset and provide a closer estimate of cracked effective flexural stiffness than a constant fraction of $E_{cE}I_g$ as has traditionally been the used. Taking the ratio of the values given in Table 7.3.1 to the experimental values from the database results in a mean of 1.05 and a coefficient of variation of 0.31.

For a given concrete cross section, studies have shown that yield curvature is not sensitive to reinforcing ratio and axial loads (Wallace and Moehle 1992). Equations that rely on the yield curvature to calculate the effective stiffness (Priestley and Kowalski 1998) have been shown to provide estimates of effective stiffness that are in reasonable agreement with experimentally measured values when axial loads and reinforcement ratios are relatively low. For the case where $N_{UG}/(A_g f'_{cE}) \leq 0.15$ and $\rho_l \leq 0.01$, the effective yield curvature ϕ_{yE} can be approximated for concrete walls with rectangular cross-section as:

$$\phi_{yE} = \frac{2f_{ylE}}{l_w E_s} \qquad (C7.3.1a)$$

For flexural deformations without the effect of bond slip, the effective flexural rigidity ($E_{cE}I_{eff}$) can be calculated in accordance with Equation (C7.3.1b)

$$E_{cE}I_{eff} = \frac{M_{CyGE}}{\phi_{yE}} \qquad (C7.3.1b)$$

where M_{CyGE} is evaluated using an applied axial load N_{UG}.

Alternatively, moment-curvature analysis can be used, and a more general relationship for wall flexural rigidity can be derived

$$E_{cE}I_{eff} = \frac{M_{fyGE}}{\phi_{fyE}} \qquad (C7.3.1c)$$

where M_{fyGE} is evaluated using an applied axial load N_{UG}.

Where inadequate anchorage or splices are present, the calculated moment strength used to evaluate wall flexural rigidity should be based on the reduced reinforcement capacity in accordance with 3.5.

When bond slip is expected at the interface between the structural wall and the anchoring element, the additional flexibility at the interface should be considered as a rigid-body rotation to account for bond slip of the longitudinal reinforcing bars within the foundation. Where this type of behavior is anticipated, the additional flexibility can be accounted for either implicitly by reducing the wall effective flexural rigidity or explicitly by introducing a flexible rotational spring. There are several methodologies available for approximating bond slip. Results presented by Abdullah (2019) show that walls with axial loads in excess of $0.2A_g f'_{cE}$ are likely to experience little to no bar slip/extension from the foundation. Therefore, no reduction factor due to bond slip should be considered for such walls.

For continuous walls, an acceptable approach for capturing the effects of bond slip is to modify the effective flexural rigidity of the wall in the story directly above the interface as follows:

$$E_{cE}I_{eff} = \frac{M_{fyGE}}{\phi_{fyE}}\left(\frac{h_1}{h_1 + \ell_{sp}}\right) \qquad (C7.3.1d)$$

Equation (C7.3.1d) assumes a constant yield curvature profile over the first-floor height h_1 and compares well against shake table testing from multi-story building prototypes. With this method, the flexibility associated with bar slip is lumped within the story above the interface, and only the moment of inertia over the height h_1 is modified for bond slip. Above the height h_1, Equation (C7.3.1c) can be used to estimate wall flexural rigidity using yield moments and curvatures at wall hinges or using the expected maximum moments and associated curvatures at the levels considered.

The strain penetration depth ℓ_{sp} in this equation is meant to approximate the length over which flexural longitudinal bar strains penetrate into the foundation system and can be approximated as follows for the purpose of approximating bar slip.

Equation (C7.3.1e) was derived assuming an average bond stress of $12\sqrt{f'_{cE}}$*, which was shown to be an appropriate estimate of average bar stresses into the foundation under earthquake excitations (Ghannoum and Moehle 2012). Other equations and methodologies have been proposed to account for strain penetration and deformations from bar slip (Priestley et al. 2007).*

$$\ell_{sp} = \frac{1}{48}\frac{f_{y\ell E}}{\sqrt{f'_{cE}}} d_b \qquad (C7.3.1e)$$

For plain bars, ℓ_{sp} *can be taken as twice the value obtained from Equation (C7.3.1e). As an alternative to modifying the flexural rigidity to account for bar slip, a rotational spring can be used to explicitly capture slip, where the spring stiffness is defined as*

$$K_R = \frac{2M_{fyGE}}{\phi_{fyE}\ell_{sp}} \qquad (C7.3.1f)$$

In place of ϕ_{fyE} *and* M_{fyGE}, ϕ_{yE} *and* M_{CyGE} *can be used in Equation (C7.3.1d) and Equation (C7.3.1f) to account for bar slip effects.*

Approximate closed-form methods can be used to calculate M_{CyGE} *for the purpose of estimating the effective flexural rigidity of planar walls, as shown in Equation (C7.3.1g) (Cardenas et al. 1973). Equation (C7.3.1g) was simplified to approximate the effects of the neutral axis depth and was verified against data from sectional analysis of about 900 walls, producing a mean of about 0.97 and coefficient of variation of 0.19.*

$$M_{CyGE} = 0.45A_{sl}f_{ylE}\ell_w\left(1 + \frac{N_{\text{UG}}}{A_{sl}f_{ylE}}\right) \qquad (C7.3.1g)$$

$E_{cE}I_{eff}$ *should be in the range of* $0.15E_{cE}I_g$ *and* $0.5E_{cE}I_g$ *when Equation (C7.3.1a) to (C7.3.1g) are used for cracked walls.*

C7.3.2 Acceptance Criteria. *Although previous versions of this standard have provided generalized guidance on classifying wall behavioral modes based on aspect ratio, a large experimental database (Abdullah 2019) has demonstrated that component classification is better achieved by considering the relative strengths of a member. A wall or wall segment is determined to be controlled by shear, shear-friction at the wall base, or flexure based on the ratio of the lower of the expected diagonal shear and shear-friction capacities to the maximum shear demand corresponding to the expected maximum flexural strength at the critical section (*V_{MCultE}*). The simplified wall expected shear strength equation,* $V_{CWall318E}$*, was used for the purpose of classification to be consistent with the distinction in the wall database (Abdullah 2019) and to avoid the potential for changing wall classification depending on directionality and other complications with the more robust* $V_{CydWallE}$ *shear strength equation introduced in this version of the standard.* $V_{CydWallE}$ *is to be used for comparison relative to shear actions and acceptance criteria, however, and the more complex calculation for* $V_{CydWallE}$ *prescribed in Section 7.2.2 is permitted for modeling and acceptance criteria where desired by the licensed design professional.*

The distribution of lateral forces along the height of a wall is required to compute V_{MCultE}*, which can be interpreted as a resultant lateral load applied at an effective height of the wall. In nonlinear analyses, shear amplifications from higher mode effects are typically captured for walls that develop their flexural strength. However, dynamic amplification of wall shear demands due to higher mode responses are not fully captured by the linear analysis procedures. ACI 369.1-17, 7.2.4.1 accounted for the impact of higher mode responses on shear demand by assuming a uniform distribution of lateral forces over the height of a wall, which is equivalent to assuming an effective wall height that is half the total wall height. Research has shown that dynamic shear amplification is strongly correlated with building period, which is a function of building height (e.g., Paulay EERI 1986, Munshi and Ghosh 2000, Fischinger et al, 2010, Kim and Wallace, 2016). A simplified dynamic shear amplification factor (*ω_v*) is therefore introduced to amplify shear demand on cantilever walls and account for higher mode effects. It is noted* ω_v *need not apply to non-cantilever walls for which* V_{MCultE} *corresponds to the development of the positive and negative maximum expected flexural strengths at opposite ends of the wall segment. This approach is aligned with the shear amplification procedures in ACI 318-19, 18.10.3. It is noted that the provision in ACI 318-19 also includes shear amplification due to moment overstrength. However, since the expected material strengths are used to compute* M_{CultE}*, a moment overstrength amplification factor is not considered here. A lower-bound limit of 0.007 times the wall height above the critical section measured in inches is imposed on* n_s *(number of stories above the critical section) to account for buildings with large story heights (i.e., greater than 12 ft).*

For walls with nonsymmetric section properties and/or axial loading about a bending axis, moment strengths and associated shear demands may be significantly different for loading in the opposite directions. As such, wall strengths and acceptance criteria should be defined differently in each opposite loading direction. Moment strength will typically be larger when the larger flange is in tension. m-factors will typically be lower for the same case since (1) for flexure-controlled walls, when the smaller flange is in compression, lower b and higher c_{GE} *values will be obtained, which results in lower m-factors (Tables 7.3.2d and 7.3.2e), (2) for shear-controlled walls, the larger moment strength will generate a lower ratio of shear strength to shear demand (*$V_{CWall318E}/V_{MCyDE}$*), which results in lower m-factors (Table 7.3.2c), (3) or shear-friction controlled walls, the larger moment strength will generate a lower shear-friction strength to shear demand ratio* $(V_{CyfWallSE}/\omega_v V_{MCyDE})$ *and lower m-factors for primary components (Table 7.4.2b), and (4) the larger moment strength will result in the wall classification per Table 7.3.2a tending closer to a shear-controlled outcome, which is more conservative. As a simplification to defining wall strengths and acceptance criteria differently in opposite loading directions, it is permitted to use the moment strength from either of the loading directions that results in the largest DCR and/or lowest m-factor obtained using Tables 7.3.2a through 7.3.2e. The more conservative wall classification and m-factor can be used for both loading directions. The largest DCR may be conservatively determined by combining the higher demand and the lower moment, shear, or shear-friction strengths from the opposite loading directions. Additional information for the derivation of acceptance criteria for shear-controlled walls is included in Section C7.4.1.1.2.*

A database for structural wall tests developed by Abdullah (2019) and new information from recent studies on the performance of structural walls were evaluated to propose updated modeling parameters and acceptance criteria for flexure-controlled concrete structural walls and wall segments. The database, which compiles detailed data on more than 1,000 reinforced concrete structural wall tests reported in the literature, included over 180 flexure-controlled walls with "Conforming" detailing and more than 250 flexure-controlled walls with

"Nonconforming" detailing. The Conforming walls are nearly or fully compliant with Special Structural Wall provisions of ACI 318, whereas the Nonconforming walls are those that do not satisfy the detailing requirements of Conforming walls as characterized in this section. In the case of walls that are asymmetrical about the centroid of the cross-section, the transverse reinforcement detailing requirements are checked for the compression boundary element. Demands are typically applied in orthogonal wall principal directions to determine the bounds on acceptance criteria in each principal axis. Detailed background information on the development of the modeling parameters and acceptance criteria given in Table 7.3.2b, Table 7.3.2d, and Table 7.3.2e are provided by Abdullah (2019), Abdullah and Wallace (2019), and Abdullah and Wallace (2020).

Moment-curvature results from a dataset of more than 900 walls presented by Abdullah (2019) show that a hinge rotation of 0.0025 rad represents an upper-bound yield hinge rotation for a hinge length equal to half the wall length. In recognition of flexibility due to bar slip/extension, this value of θ_{yE} can be increased to 0.003 rad as a conservative estimate of θ_{yE} for use in Table 7.3.2b in lieu of use of Equation (7.4.1.1.1). For convenience for the design professional, alternative tabulated m-factors are included in Table 7.3.2d and Table 7.3.2e for walls controlled by flexure, which were produced using the equations in Table 7.3.2b, a value of θ_{yE} of 0.003 rad, and nonlinear θ_{yE} θ_{yE}.

C7.4.1 Modeling. *Walls have a potential to exhibit nonlinear action in both shear and flexure when wall shear strength is between approximately 0.7 and 1.3 times the shear demand corresponding to the flexural yield strength of structural walls or wall segments, M_{Cy} (Abdullah 2019). In these cases, shear and flexure actions can be coupled, and both shear and flexure components may have measurable contributions to total wall deformations. Hence, for nonlinear modeling, if both rotational and translational elements in a particular member experience inelastic deformations simultaneously, the cumulative deformation prior to lateral strength degradation can be larger than prescribed for either action's strength-loss deformation parameter (d_{nl}). As such, the cumulative force-deformation envelope and acceptance criteria should be verified against the lowest permitted deformations from Table 7.4.1.1.1a, Table 7.4.1.1.1b, and Table 7.4.1.1.2. In order to capture the appropriate force-deformation envelope for these walls and walls segments, it may be necessary in nonlinear modeling to limit inelastic deformation to either the rotational or translational action, or employ multidimensional concrete constitutive models, such as solid element models or layered shell element models, to provide accurate simulation of strength and deformation capacity. If wall models employ one-dimensional material constitutive models (e.g., fiber elements), it may be necessary to adjust these one-dimensional material constitutive models to provide accurate simulation of strength and deformation capacity.*

For walls with nonsymmetric section properties and/or loading about a bending axis, a simplification is permitted to use the more conservative wall classification and deformation capacities to model the wall backbone for loading in both directions. However, the wall strengths assigned to the backbone should be direction-specific. Refer to Section C7.3.2 for a more detailed related discussion. The resulting simplification is generally conservative for the wall considered. However, by altering the force-deformation backbone of a wall, the distribution of forces between building members and the progression of failure may be altered.

C7.4.1.1 Nonlinear Static and Nonlinear Dynamic Procedures Employing Lumped-Plasticity Load-Deformation Models *ACI369.1-17 nonlinear modeling parameters for flexure-controlled walls and wall segments were given as plastic hinge rotations. However, the modeling parameters are now given as total hinge rotation (Figure 3.1.2.2.3(b)). This approach was chosen to avoid sensitivity of the modeling parameters to yield rotation and for consistency with shear-controlled wall modeling parameters. This approach also facilitates conversion between rotation and strain measures through a plastic hinge length. In the 2021 version of this Standard, two new modeling parameters were introduced, Parameters c'_{nl} and d'_{nl}, to represent the ratio of ultimate strength to yield strength and the total hinge rotation capacity at onset of residual strength, respectively, as shown in Figure 3.1.2.2.3b. Furthermore, the predictor variable given in the first column of Table 19 of ACI 369.1-17 (i.e., $[(A_s - A'_s)f_{yE} + P]/A_g f'_{cE}$ was found to not correlate well with wall deformation capacities; therefore, this variable was replaced in 2021 version of this standard with a slenderness parameter, $l_w c_{DE}/b_s^2$, which was found to significantly influence wall deformation capacities at Points C, D, and E of the backbone relation shown in Figure 3.1.2.2.3b (Abdullah and Wallace 2019, Abdullah 2019).*

The database used to develop modeling parameters in Table 7.4.1.1.1a and Table 7.4.1.1.1b for walls controlled by flexure contained only 10 wall tests with ρ_{lw} below 0.0025. The limited data, however, suggest that Nonconforming walls with such low longitudinal reinforcement ratios can exhibit substantially lower deformation capacity at lateral strength loss than those with higher longitudinal reinforcement ratios when subjected to relatively low compression demands (i.e., $l_w c_{DE}/b_s^2 < 10$), for which the failure mode is typically tension-fracture of longitudinal bars due to the significant tensile strains expected to develop in the extreme tension bars (Abdullah 2019). As such, Table 7.4.1.1.1b does not apply to walls or wall segments with ρ_{lw} lower than 0.001, which are considered plain concrete in accordance with 7.2. A reduction factor is applied for ρ_{lw} between 0.001 and 0.0025 and for low values of the parameter, $l_w c_{DE}/b_s^2$, which represents the level of compression demands in a wall or wall segment.

Table C7.4.1.1a and Table C7.4.1.1b present the statistics for the ratio of estimated modeling parameters obtained using Table 7.4.1.1.1a and Table 7.4.1.1.1b, respectively, divided by the experimental values of these parameters contained in the database.

The term b_s represents the width of the flexural compression zone of the wall section. For a planar wall, b_s is equal to t_w. The width of the flexural compression zone, b_s, for other conditions is illustrated in Figure C7.4.1.1. For cases with a large b_s, such as where the barbell or flange of a wall is in compression, deformation capacity is likely to be relatively large. However, cases with a barbell or flange in tension and a thin wall web in compression may result in large values of $\frac{l_w c_{DE}}{b_s^2}$ and higher shear demands such that lower deformation capacities are likely. For cases where b_s varies over c_{DE}, or where c_{DE} varies over b_s, representative or weighted average values of b_s and c_{DE} should be used, as illustrated in Figure C7.4.1.1.

C7.4.1.1.2. Structural Walls and Wall Segments Controlled by Shear. The recommended backbone shape and parameters provided for concrete structural walls differ from the general backbone description in Chapter 7 of ASCE 41. For walls controlled by shear, the load-deformation relationship in Figure 3.1.2.2.3c provides a better representation of the behavior than that in Figure 3.1.2.2.3b in which deformations related to shear are not negligible compared with the deformations related to flexure. The proposed nonlinear modeling parameters for walls *and wall segments controlled by shear represent the shear, or lateral translation, deformation component in a model in which the total lateral drift is calculated as the sum of contributions of components related to flexure, shear, and slip of the reinforcement. Note that variables f_{nl}, and g_{nl} in Figure 3.1.2.2.3c are not the same as those used in Chapter 7 of ASCE 41. The backbone parameters provided in Table 7.4.1.1.2 for nonlinear modeling of shear-controlled walls were developed based on data from 365 quasi-static, reversed, cyclic tests conducted on walls whose behavior was controlled by shear or flexure-shear (Abdullah and Wallace 2021, Abdullah et al. 2022). The shear deformation capacity at initiation of strength loss of walls controlled by shear, as represented by d_{nl}, was found to be governed by the shape of the cross-section and the ratio of wall shear strength to wall shear demand corresponding to its flexural strength. As the latter ratio approaches unity, wall behavior tends to shift from a shear-controlled behavior to a flexure-shear-controlled behavior (i.e., the wall experiences inelastic flexural deformation in addition to inelastic shear deformation prior to initiation of lateral strength loss). Although the d_{nl} values in Table 7.4.1.1.2 only represent shear deformation, the higher d_{nl} values in that table for walls with high ratios of shear strength to wall shear demand incorporate the inelastic flexural deformation. While the level of applied axial load was not found to significantly influence the point at initiation of strength loss (d_{nl}), it was found to be a critical parameter for the rate of strength degradation up to axial collapse, as represented by parameters d'_{nl} and e_{nl}.*

Table C7.4.1.1a. Statistics* of Modeling Parameters for Conforming Reinforced Concrete Structural Walls and Associated Components Controlled by Flexure.

Parameter	Mean	Median	Lognormal Standard Deviation	Coefficient of Variation, COV
$M_{CyGE,calculated}$/ $M_{CyGE,experimental}$	1.01	1.00	0.12	0.12
c'	1.03	1.02	0.10	0.10
c	1.15	0.84	0.97	0.84
d	0.98	0.95	0.17	0.17
d'	1.01	1.01	0.22	0.21
e	1.03	1.01	0.22	0.21

*The statistics are for the ratios of estimated-to-experimental values.

Table C7.4.1.1b. Statistics* of Modeling Parameters for Nonconforming Reinforced Concrete Structural Walls and Associated Components Controlled by Flexure.

Parameter	Mean	Median	Lognormal Standard Deviation	Coefficient of Variation, COV
$M_{CyGE,calculated}$/ $M_{CyGE,experimental}$	0.97	0.97	0.14	0.14
c'	1.03	0.97	0.15	0.15
c	1.22	1.00	0.95	0.78
d	0.95	0.93	0.22	0.23
d'	1.01	0.97	0.24	0.24
e	1.01	1.02	0.21	0.21

*The statistics are for the ratios of estimated-to-experimental values.

For nonsymmetric flanged sections, modeling parameters and acceptance criteria depend on the direction of loading and should be taken from flange or rectangular shape values based on the shape of the wall end that is under compression demands from moment and shear actions. It is permitted to define the force-deformation backbone relations for all wall section shapes based on rectangular section values in Table 7.4.1.1.2. This simplification is generally conservative for the wall considered in terms of delivering lower deformation capacities. However, by altering the wall force-deformation backbone of a wall, the distribution of forces between building members and the progression of failure sequences may be altered. It is noted that the dataset used to develop the modeling parameters and acceptance criteria for shear-controlled walls only contained a limited number of non-symmetric flanged sections.

Modelling parameters provided in Table 7.4.1.1.2 represent median values, except for cases where a simplified model with a limited bias was justified. Table C7.4.1.1.2 lists modeling parameter statistics (mean, median, distribution, STDV, and COV) for walls controlled by shear.

C7.4.1.1.3 Structural Walls and Wall Segments Controlled by Shear-Friction. *Test results indicate that shear-friction behavior at an interface is characterized by almost zero slip along the interface until the yield shear-friction strength defined in 7.2.3 is exceeded (Abdullah et al. 2021b). Where a lumped-plasticity translational element is used to simulate shear sliding along an interface but does not include the effects of diagonal shear deformations within the wall or wall segment, the load-deformation relationship of the element should be defined as presented in Figure 3.1.2.2.3d. Alternatively, if the elastic shear flexibility of the wall or wall segment is aggregated into the lumped-plasticity translational element used to simulate the nonlinear shear-friction behavior, the load-deformation relationship of the element should be defined as presented in Figure 3.1.2.2.3c using modeling parameters and stiffness values for shear-controlled walls up to Point B, and as presented in Figure 3.1.2.2.3d beyond Point B.*

The nonlinear modeling parameters for shear-sliding are presented in inches. The sliding deformation along an interface is a local behavior and is independent of wall height (Abdullah et al. 2021b). The modeling parameter b_{nl} in Table 7.4.1.1.3 estimates the slip at which lateral strength is lost at an interface due to deterioration of concrete at the sliding shear interface. Walls and wall piers sustaining sliding at an interface are expected to maintain gravity load carrying capacity beyond the slip defined by b_{nl}. Wall tests used to derive nonlinear modeling parameters in Table 7.4.1.1.3 did not sustain axial collapse and, therefore, could not be used to define the expected slip displacement at which axial collapse occurs (Abdullah et al. 2021b). The values of modeling parameter b_{nl} provided in Table 7.4.1.1.3 are based on experience and judgment. Under sustained transverse (out-of-plane) loads such as earth or fluid loads, a sliding interface may become unstable. b_{nl} is limited to a'_{nl} in Table 7.4.1.1.3 in such cases.

The shear-friction modeling parameters were calibrated for potential interfaces along the wall and wall segment height. They are not intended for use with interfaces having different

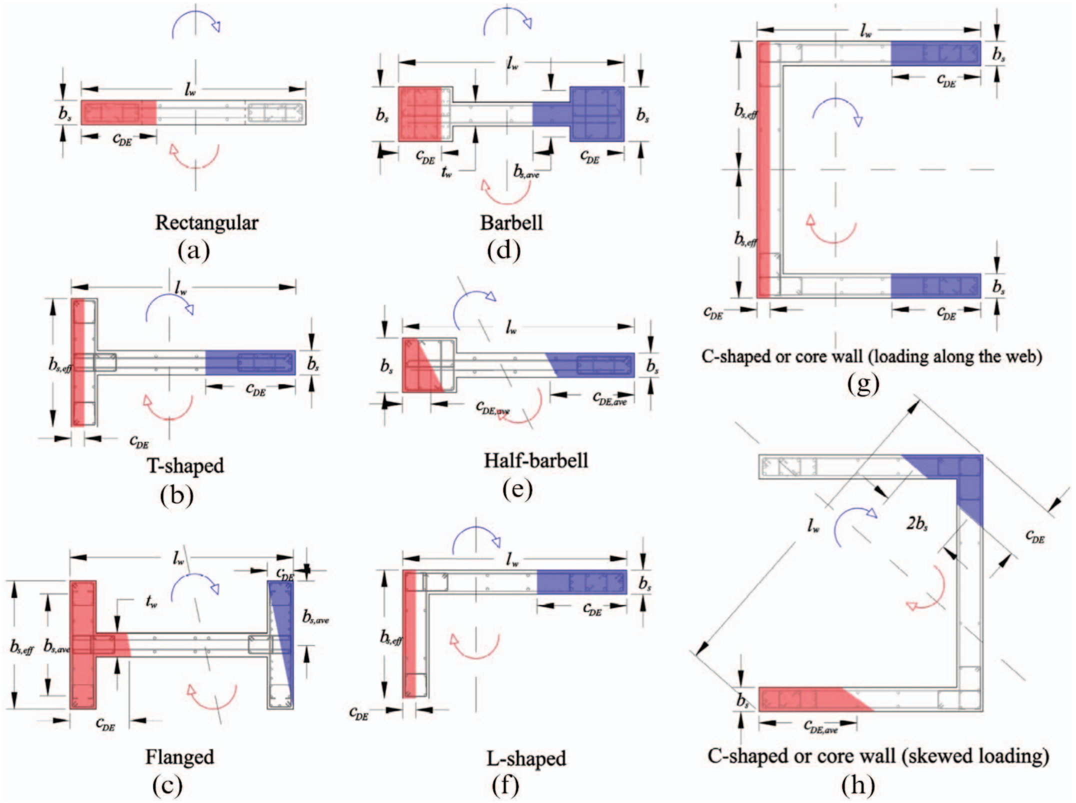


Figure C7.4.1.1. Definition of width (bs) and length (c_{DE}) of flexural compression zone (Abdullah and Wallace 2020). ($b_{s,ave}$ = average width of compression zone, $c_{DE,ave}$= average depth of neutral axis, and $b_{s,eff}$= effective with of wall flange; the blue and red arrows indicate the direction of bending).

Table C7.4.1.1.2. Modeling Parameter Statistics for Reinforced Concrete Structural Walls and Associated Components Controlled by Shear.

Parameter	Mean	Median	Standard Deviation	Coefficient of Variation, COV
$V_{CcrWall}$	1.09	1.01	0.40	0.37
$V_{CydWall}{}^{b}$	1.01	1.00	0.25	0.24
$c'_{nl}{}^{c}$	1.01	1.00	0.20	0.19
g_{nl}	1.31	1.15	0.60	0.46
d_{nl}	1.04	0.96	0.40	0.38
d'_{nl}	1.10	1.06	0.39	0.36
e_{nl}	1.06	1.04	0.37	0.35

[a] Statistics are for the ratios of estimated-to-experimental values.
[b] For values predicted by Equation (7.2.2c).

boundary conditions such as the vertical plane at slab-wall interfaces, which should be evaluated using the provisions for cast-in-place concrete diaphragms.

Table C7.4.1.1d presents the statistics for the ratio of estimated modeling parameters obtained using Table 7.4.1.1.3 divided by the experimental values of these parameters contained in the test database.

C7.4.1.2 Solid Element and Layered Shell Element Models Employ Multi-Dimensional Concrete Material Models. *Using these models, it is possible to simulate the interaction of flexure and shear demands and, thus, the potential for shear failure or reduced deformation capacity in flexure due to shear demand. However, in models employing traditional beam-column elements with fiber-type section models and fiber-shell element models, one-dimensional concrete and steel material models are used, and shear response is decoupled from the flexural response such that shear does not affect strength or deformation capacity. Thus, when beam-column element models with fiber-type section models and fiber-shell element models are used to simulate the response of walls, it is necessary to adjust material constitutive models to simulate reduced flexural deformation capacity or strength resulting from flexure-shear interaction. Various studies have evaluated the ability of these models to capture experimentally observed behavior including flexure-shear interaction (Orakcal and Wallace 2006, Massone et al. 2006, Kolozvari et al. 2014a, Kolozvari et al. 2014b, Kolozvari et al., 2019a).*

To accurately simulate the response of structural walls and wall segments using solid elements, layered and fiber-shell elements or beam-column elements with fiber-type section models it is necessary to use an element mesh that is sufficiently refined to accurately simulate stress and strain fields within the structure, as well as to use material constitutive models that accurately simulate the material response including the impact of confinement on concrete compressive strength and strain capacity, loss of concrete tensile strength due to cracking, and the onset of steel strength loss due to buckling and fracture. Accurate simulation of strength loss requires adjustment of material properties on the basis of the length over which deformations localize during failure, which is a function of mesh size. Section 3.2.1 includes requirements intended to result in accurate simulation of response. The following references provide guidance on modeling

Table C7.4.1.1d. Statistical Values[a] for Modeling Parameters for Reinforced Concrete Structural Walls and Wall Segments Controlled by Shear-Friction.

Parameter	Mean	Median	Lognormal Standard Deviation	Coefficient of Variation, COV
$V_{CyfWallSE}$[b]	1.01	0.97	0.20	0.20
$V_{CyfWallE}$[c]	1.05	1.09	0.26	0.25
c'_{nl}[b]	1.03	1.07	0.25	0.24
c'_{nl}[c]	0.99	0.98	0.20	0.21
c_{nl}	1.03	0.90	0.45	0.44
a_{nl}	1.06	1.07	0.35	0.33
a'_{nl}	1.02	1.00	0.41	0.41

[a] Statistics are for the ratios of estimated-to-experimental values.
[b] For values predicted by Equation 7.2.3.
[c] For values predicted by Equation 7.2.4.

the response of reinforced concrete structural walls and wall segments using multiple modeling techniques NIST GCR-17-917-45 (NIST 2018a), NIST GCR-17-917-46a,c (NIST 2018b), Pugh et al (2017), Marafi et al. (2019), Lowes et al. (2016), Lowes et al. (2019), Kolozvari et al. (2014a and 2014b), Orakcal and Wallace (2006), Kolozvari et al. (2018b), and Kolozvari et al. (2019).

For shear-friction controlled walls and wall segments, defined in accordance with 7.3.2, the nonlinear behavior is governed by shear sliding at the interface. Where shell elements are used to simulate both the elastic shear flexibility of the wall, or wall segment, aggregated with the nonlinear behavior of the shear-friction interface, the nonlinear modeling parameters for shear sliding provided in Table 7.4.1.1.3 should be converted from their units of inches to shear strain, by dividing them by the height of the shell element(s) aggregating the behaviors.

C7.4.1.1.3. Structural Walls and Wall Segments Controlled by Shear-Friction. Test results indicate that shear-friction behavior at an interface is characterized by almost zero slip along the interface until the yield shear-friction strength defined in Section 7.2.3 is exceeded (Abdullah et al. 2021b). Where a lumped-plasticity translational element is used to simulate shear sliding along an interface but does not include the effects of diagonal shear deformations within the wall or wall segment, the load-deformation relationship of the element should be defined as presented in Figure 3.1.2.2.3d. Alternatively, if the elastic shear flexibility of the wall or wall segment is aggregated into the lumped-plasticity translational element used to simulate the nonlinear shear-friction behavior, the load-deformation relationship of the element should be defined as presented in Figure 3.1.2.2.3c using modeling parameters and stiffness values for shear-controlled walls up to Point B, and as presented in Figure 3.1.2.2.3d beyond Point B.

The nonlinear modeling parameters for shear-sliding are presented in inches. The sliding deformation along an interface is a local behavior and is independent of wall height (Abdullah et al. 2021b). The modeling parameter b_{nl} in Table 7.4.1.1.3 estimates the slip at which lateral strength is lost at an interface due to deterioration of concrete at the sliding shear interface. Walls and wall piers sustaining sliding at an interface are expected to maintain gravity load carrying capacity beyond the slip defined by b_{nl}. Wall tests used to derive nonlinear modeling parameters in Table 7.4.1.1.3 did not sustain axial collapse and, therefore, could not be used to define the expected slip displacement at which axial collapse occurs (Abdullah et al. 2021b). The values of modeling parameter b_{nl} provided in Table 7.4.1.1.3 are based on experience and judgment. Under sustained transverse (out-of-plane) loads such as earth or fluid loads, a sliding interface may become unstable. b_{nl} is limited to a'_{nl} in Table 7.4.1.1.3 in such cases.

The shear-friction modeling parameters were calibrated for potential interfaces along the wall and wall segment height. They are not intended for use with interfaces having different boundary conditions such as the vertical plane at slab-wall interfaces, which should be evaluated using the provisions for cast-in-place concrete diaphragms.

Table C7.4.1.1d presents the statistics for the ratio of estimated modeling parameters obtained using Table 7.4.1.1.3 divided by the experimental values of these parameters contained in the test database.

***C7.4.2 Acceptance Criteria** Although wall elements employing nonlinear material fiber elements have become more common in practice in recent years, the usable strain limits in 3.3 are not intended to be used to develop strain-based acceptance criteria. Stress-strain models that compose fiber models must be modified to consider element mesh parameters and potential … localized wall behavior, for example cyclic fatigue, reinforcement buckling, reinforcement rupture, and shear-flexure interaction, such that predicted analytical response and related acceptance criteria are in general agreement with the generalized load-deformation values in 7.4.1.1. Lowes et al. (2016), Pugh et al. (2017), Marafi et al. (2019), and Lowes et al. (2019) demonstrate modification of material models for flexure-controlled, compression-controlled walls and different nonlinear modeling approaches.*

***C7.5.1 Modeling** For linear procedures, coupling beams shall be modeled using solid elements, shell elements, or beam-column elements that represent elastic response. Coupling beams that have diagonal reinforcement satisfying ACI 318-19 requirements commonly have a stable hysteretic response under large load reversals. Therefore, these members could adequately be modeled with beam elements used for typical frame analyses.*

C7.7 RETROFIT MEASURES FOR REINFORCED CONCRETE STRUCTURAL WALLS, WALL SEGMENTS, AND COUPLING BEAMS

The following measures can be effective in retrofitting reinforced structural walls, wall segments, coupling beams, and reinforced concrete columns supporting discontinuous structural walls:

*(a) **Addition of wall boundary elements:** Addition of boundary elements can be an effective measure in strengthening walls or wall segments that have insufficient flexural strength. These members can be either cast-in-place reinforced concrete components or steel sections. In both cases, proper connections should be made between the existing wall and the added components. The shear demand and shear capacity of the retrofitted wall should be reevaluated.*

*(b) **Addition of confinement jackets at wall boundaries:** Increasing the confinement of the wall boundaries by the addition of a steel or reinforced concrete jacket can be an effective measure in improving the flexural deformation capacity of a structural wall. For both types of jackets, the longitudinal steel should not be continuous from story to story unless the jacket is also being used to increase the flexural capacity. The minimum thickness for a concrete jacket should be 3 in. Carbon fiber wrap should be permitted for improving the confinement of concrete in compression.*

Table C7.4.1.1d. Statistical Values[a] for Modeling Parameters for Reinforced Concrete Structural Walls and Wall Segments Controlled by Shear-Friction.

Parameter	Mean	Median	Lognormal Standard Deviation	Coefficient of Variation, COV
$V_{CyfWallSE}{}^{b}$	1.01	0.97	0.20	0.20
$V_{CyfWallE}{}^{c}$	1.05	1.09	0.26	0.25
$c'_{nl}{}^{b}$	1.03	1.07	0.25	0.24
$c'_{nl}{}^{c}$	0.99	0.98	0.20	0.21
c_{nl}	1.03	0.90	0.45	0.44
a_{nl}	1.06	1.07	0.35	0.33
a'_{nl}	1.02	1.00	0.41	0.41

[a] Statistics are for the ratios of estimated-to-experimental values.
[b] For values predicted by Equation 7.2.3.
[c] For values predicted by Equation 7.2.4.

(c) ***Reduction of flexural strength:*** *Reduction in the flexural capacity of a structural wall to change the governing failure mode from shear to flexure can be an effective retrofit measure. It can be accomplished by saw-cutting a specified number of longitudinal bars near the edges of the wall.*

(d) ***Increased shear strength of wall****: Increasing the shear strength of the web of a structural wall by casting additional reinforced concrete adjacent to the wall web can be an effective retrofit measure. The new concrete should be at least 4 in. thick and should contain horizontal and vertical reinforcement. The new concrete should be properly bonded to the existing web of the structural wall. The use of carbon fiber sheets, epoxied to the concrete surface, should also be permitted to increase the shear capacity of a structural wall.*

(e) ***Confinement jackets to improve deformation capacity of coupling beams and columns supporting discontinuous structural walls:*** *The use of confinement jackets described previously as a retrofit measure for wall boundaries, and in Chapter 2 for frame elements, can also be effective in increasing both the shear capacity and the deformation capacity of coupling beams and columns supporting discontinuous structural walls.*

(f) ***Infilling between columns supporting discontinuous structural walls:*** *Where a discontinuous structural wall is supported on columns that lack either sufficient strength or deformation capacity to satisfy design criteria, making the wall continuous by infilling the opening between these columns can be an effective retrofit measure. The infill and existing columns should be designed to satisfy all the requirements for new wall construction, including any strengthening of the existing columns required by adding a concrete or steel jacket for strength and increased confinement. The opening below a discontinuous structural wall should also be permitted to be infilled with steel bracing. The bracing members should be sized to satisfy all design requirements, and the columns should be strengthened with a steel or a reinforced concrete jacket.*

All the aforementioned retrofit measures require an evaluation of the wall foundation, diaphragms, and connections between existing structural elements and any elements added for retrofit purposes.

C10.3.3 Concrete Foundations. ***Replace Sections C12.1 through C12.4 of ACI 369.1 with the italicized text as follows.***

ACI 369.1 Chapter 12

C12.1 TYPES OF CONCRETE FOUNDATIONS

Previous editions of the standard required concrete foundation components be considered force controlled. This requirement was relaxed to permit foundation components be treated similar to their analogous superstructure components as has been done for other ASCE 41 material provisions. As an example from other material provisions in ASCE 41, steel piles are treated like steel columns, which these provisions now permit for concrete piles.

Similarly, footings and mats are treated analogous to slabs. The reason for this is the similarity in component response and the factors that affect ductility, specifically punching shear ratio. In this case, the punching shear ratio is taken as the punching shear demand acting on the slab critical section from the column under N_{UG} divided by the direct punching shear capacity of the slab as defined by ACI 318. The only change to the slab provisions required for application in evaluation of footings and mats is to consider the top reinforcement as opposed to the bottom reinforcement when determining if there is continuous reinforcement passing through the column core. This approach would be similar in the case of a mat, whereby the column core from the slab provisions should be considered as the anchor bolts or dowels that lap with the column longitudinal reinforcement for application to mat foundations. Similar to slab-column frames, mat-column frames may be modeled using an effective beam width model, an equivalent frame model, or a finite element model as discussed in C4.4, where the primary distinction is that a subgrade stiffness for soil should be captured along the mat in accordance with Section 8.4.2.5. In linear procedures, the shear and moment actions resulting from the analysis are then compared against their acceptance criteria, force-controlled for shear actions and m in accordance with Section 4.4 for flexural actions. For nonlinear procedures, modeling the interaction can be accomplished using beam elements to represent the mat and a rigid-plastic torsional member to represent the shear transfer at the mat and column. If the shear (one-way or punching) is insufficient to develop the flexural capacity of the effective mat strip, force-controlled acceptance criteria is required.

C12.1.2 Deep Concrete Foundations

C12.1.2.1. Driven Concrete Pile Foundations. *In poor soils, or soils subject to liquefaction, bending of the piles can be the only dependable resistance to seismic forces.*

C12.1.2.2. Cast-in-Place Concrete Pile Foundations. *Segmented steel cylindrical liners are available to form the shaft in weak soils and allow the liner to be removed as the concrete is placed. Various slurry mixes are often used to protect the drilled shaft from caving soils. The slurry is then displaced as the concrete is placed by the tremie method.*

C12.2 ANALYSIS OF EXISTING CONCRETE FOUNDATIONS

Engineering judgment should be practiced when modeling the effects of the foundation elements. The determination of the appropriate boundary element type to be used can often be quickly performed by comparing the relative strengths and stiffness of the superstructure component with the foundation element. For example, the base of a column can typically be modeled as fixed *when it connects to a mat or pile foundation; similarly, the ends of a concrete structural wall can typically be modeled as pinned when connecting to shallow foundations. The engineer is permitted to use simple boundary elements (i.e., fixed or pinned) when they can be justified. A more rigorous approach is required when a simple approach cannot be justified. In lieu of a more rigorous analysis approach, the engineer may also consider "bounding" the analysis by utilizing both a fixed boundary element analysis approach and a pinned boundary element analysis approach.*

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CHAPTER C11
MASONRY

C11.1 SCOPE

The provisions of this chapter should be applied to solid or hollow clay-unit masonry, solid or hollow concrete-unit masonry, and hollow clay tile. The provisions of the chapter are applicable to reinforced masonry. For the purposes of this standard, reinforced masonry is defined in Chapter 1 and may differ from the requirements for reinforced masonry in TMS 402. Any discrepancies are caused by the intent of TMS 402 to apply to new construction, not existing structures. Stone or glass block masonry is not covered in this chapter, although the principles herein may provide guidance for stone or glass block masonry buildings in conjunction with project-specific component testing programs.

Techniques for repair of earthquake-damaged masonry components are not included in this standard. The design professional is referred to FEMA 306 (1998a), FEMA 307 (1998b), and FEMA 308 (1998e) for information on evaluation and repair of masonry wall components.

An alternate procedure for the evaluation and retrofit of unreinforced masonry (URM) bearing wall buildings with flexible diaphragms is contained in Section 16.2. The Performance Objective and limitations for that procedure are specified in Section 16.2.1. Resources for the evaluation and retrofit of adobe buildings include Tolles et al. (2002), CBSC (2013), and Blondet et al. (2011).

Evaluations and retrofits of URM bearing wall buildings and steel or concrete frame buildings with URM infill walls may result in margins against collapse that are difficult to quantify and at times small for the following reasons:

1. Buildings with URM walls are inherently brittle systems that can exhibit limited ductility capacity in certain configurations and modes of behavior;
2. URM walls rely on friction, overburden from supported loads and wall weights, and often highly variable material properties;
3. URM wall strengths and stiffnesses degrade with each additional cycle of response to motions, and thus they are vulnerable to incremental damage, particularly in larger-magnitude, longer-duration earthquakes and multiple aftershocks compared with damage from shorter-duration, isolated, and moderate earthquakes; and
4. Nonlinear modeling of URM walls is feasible, but experience to date suggests that analytical results do not always provide reliable estimates of performance because of variability in actual material strength and condition.

Owners, design professionals, and Authorities Having Jurisdiction over the safety of buildings with URM walls are forewarned to take these factors into consideration when managing their seismic risks.

As indicated in Chapter 1, great care should be exercised in selecting the appropriate retrofit approaches and techniques for application to historic buildings to preserve their unique characteristics.

C11.2 CONDITION ASSESSMENT AND MATERIAL PROPERTIES

C11.2.1 General Construction of existing masonry buildings in the United States dates back to the 1500s in the southeastern and southwestern regions, to the 1770s in the central and eastern regions, and to the 1850s in the western half of the nation. The stock of existing masonry buildings in the United States is composed largely of structures constructed since 1800. Because the types of units, mortars, and construction methods changed during this time, knowing the age of a masonry building may be useful to identify the characteristics of its construction. Although structural properties cannot be inferred solely from age, some background on typical materials and methods for a given era can help to improve engineering judgment and provide some direction in the assessment of an existing building. The design professional should be aware that values given in some existing documents are working stress values rather than the expected or lower-bound strengths used in this standard.

C11.2.2 Condition Assessment Buildings are often constructed with masonry veneer as an architectural finish, which may make the wall appear thicker than the actual structural thickness. In many areas of the country, the veneer wythe (in many parts of the world, the term *leaf* is used for wythe) is separated from the structural wall by an air space to provide ventilation and moisture control. This method is called cavity wall construction. In this case, the veneer may be anchored but does not add any strength to the assembly.

In areas of the southwest United States and along the California coast (as well as other regions), the veneer is placed directly against the building wall. It will be in a running bond pattern without a header course. Other patterns are also seen. If the veneer is not anchored or has a layer of building paper between it and the inner wythe, it cannot be considered as part of the structural wall.

Veneer on modern buildings may be adhered or anchored. In either case, the veneer is a weight to be considered but does not contribute to a wall's strength. In all cases, the veneer must be anchored to prevent it from detaching during an earthquake. Requirements for veneer are specified in Chapter 13.

Outer wythes that are bonded to the inner wythes with a regular pattern of header courses or by collar joints filled with mortar or grout and connected by wall ties are not veneer. In this case, the outer wythes are part of the structural wall and can be used in evaluating the height-to-thickness ratio of the wall.

See Section C11.3.2.1 and TMS 402, Section 5.1.4.2, for additional information regarding condition assessment of multiple wythes in walls developing effective composite action. URM cavity walls can also be evaluated for semicomposite behavior between the two layers without necessarily filling cavities (Walsh et al. 2015).

The design professional is referred to FEMA 306 (1998a), FEMA 307 (1998b), and FEMA 308 (1998e) for additional information regarding the condition of masonry. The classification of the condition of masonry requires consideration of the type of component, the anticipated mode of inelastic behavior, and the nature and extent of damage or deterioration. These documents also contain extensive information regarding the effects of damage on strength, stiffness, and displacement limits for masonry components. Included are damage classification guides with visual representations of typical earthquake-related damage of masonry components, which may be useful in classifying the condition of masonry for this standard. The severity of damage described in FEMA 306 (1998a), FEMA 307 (1998b), and FEMA 308 (1998e) is categorized as insignificant, slight, moderate, heavy, and extreme. Masonry in good condition has severity of damage not exceeding insignificant or slight, as defined by FEMA 306 (1998a). Masonry in fair condition has severity of damage not exceeding moderate. Masonry with heavy or extreme damage is classified as poor.

C11.2.2.2 Comprehensive Condition Assessment The following nondestructive tests may be used to plan the locations of destructive tests of reinforced and unreinforced masonry and to assist in the interpretation of the data produced by this testing.

Ultrasonic Pulse Velocity. Measurement of the velocity of ultrasonic pulses through a wall can result in the detection of variations in the density and modulus of masonry materials as well as the presence of cracks and discontinuities. Transmission times for pulses traveling through a wall (direct method) or between two points on the same side of a wall (indirect method) are measured and used to infer wave velocity.

The use of test equipment that has wave frequencies in the range of 50 kHz has been shown to be appropriate for the condition assessment of masonry walls. Use of equipment with higher-frequency waves is not recommended because the short wavelength and high attenuation are not consistent with typical dimensions of masonry units. Test locations should be sufficiently close to identify zones with different properties. Contour maps of direct transmission wave velocities can be constructed to assess the overall homogeneity of a wall elevation. For indirect test data, vertical or horizontal distance can be plotted versus travel time to identify changes in wave velocity (slope of the curve). Abrupt changes in slope identify locations of cracks or flaws.

Ultrasonic methods are not applicable for masonry of poor quality or low modulus or for masonry with many flaws and cracks. The method is sensitive to surface condition, the coupling material used between the transducer or receiver and the masonry, and the pressure applied to the transducer.

The use of ultrasonic pulse velocity methods with masonry walls has been researched extensively by Kingsley et al. (1987), Calvi (1988), and Epperson and Abrams (1989). A standard for the use of ultrasonic methods for masonry has been developed in Europe with the RILEM Committee, TC 127-MS (2001).

Mechanical Pulse Velocity. The mechanical pulse velocity test consists of impacting a wall with a hammer blow and measuring the travel time of a sonic wave across a specified gauge distance. An impact hammer is equipped with a load cell or accelerometer to detect the time of impact. A distant accelerometer is fixed to a wall to detect the arrival time of the pulse. Wave velocity is determined by dividing the gauge length by the travel time. The form and duration of the generated wave can be varied by changing the material on the hammer cap.

The generated pulse has a lower frequency and higher energy content than an ultrasonic pulse, resulting in longer travel distances and less sensitivity to small variations in masonry properties and minor cracking. The mechanical pulse method should be used in lieu of the ultrasonic pulse method where overall mean properties of a large portion of masonry are of interest.

The use of mechanical pulse velocity measurements for masonry condition assessments has been confirmed through research by Kingsley et al. (1987) and Epperson and Abrams (1989). Although no standard exists for mechanical pulse velocity tests with masonry, a standard for concrete materials, ASTM C597 (ASTM 2016d), does exist.

Impact Echo. The impact-echo technique can be useful for nondestructive determination of the location of void areas within grouted reinforced walls, as reported by Sansalone and Carino (1988). Commercial devices are available or systems can be assembled using available electronic components. Because this technique cannot be used to distinguish between a shrinkage crack at the grout–unit interface and a complete void in the grout, drilling of small holes in the bed joint or examination using an optical borescope should be performed to verify the exact condition.

Radiography. A number of commercial radiographic (X-ray) devices exist that can be used to identify the location of reinforcing steel in masonry walls. These devices are also useful for locating bed-joint reinforcing steel, masonry ties and anchors, and conduits and pipes. The better devices can locate a No. 6 bar at depths up to approximately 6 in. (150 mm); however, the limitations of this technique are such that for a 12 in. (300 mm) thick concrete masonry wall, a bar located off center cannot be found where access is limited to only one side of the wall. In most cases, these devices are not able to assist with determining the locations or lengths of reinforcing bar splices in walls and instead are best used to identify the location of single isolated bars. The devices become less useful where the congestion of reinforcing bars increases.

Infrared Thermography. Digital imagery in the infrared spectrum can be used to detect the presence of subsurface anomalies such as voids, hidden flues, and chimneys in masonry walls, and the locations of grouted and ungrouted cells (Dalrymple 2006).

Surface Penetrating Radar. The surface penetrating radar (SPR) method, also referred to as ground penetrating radar, involves the transmission of high-frequency microwave electromagnetic radio pulses into the object of interest and measuring the time elapsed between transmission, reflection off a discontinuity, and reception back at a surface radar antenna. A pulse of radar energy is generated on a dipole transmitting antenna that is placed on the surface of the wall. The resulting wave of electromagnetic energy propagates into the material where portions of it are reflected back to the surface at discontinuities. The discontinuities where reflections occur are created by changes in dielectric properties of the underlying material.

SPR can be used to detect voids and other defects in multiwythe masonry walls, locate horizontal and vertical reinforcing bars or embedded structural steel, locate grouted and ungrouted cells in concrete masonry walls, and evaluate effectiveness of injection repairs (Schuller 2003). Unlike impact-echo and ultrasonic signals, surface penetrating radar is also applicable to masonry in poor condition, because microwave energy can travel through air space to provide information beyond the first debond, crack, or other flaw.

Borescopic Investigations. Visual inspections of masonry walls can be conducted by drilling small-diameter holes and inserting a video device into the holes, for example, where required by Section 11.5.3.

C11.2.2.3 Supplemental Tests Ancillary tests are recommended, but not required, to enhance the level of confidence in masonry material properties or to assess condition. Possible supplemental tests are described as follows.

Surface Hardness. The surface hardness of exterior wythe (leaf) masonry can be evaluated using the Schmidt rebound hammer. Research has shown that the technique is sensitive to differences in masonry strength but cannot by itself be used to determine absolute strength. A Type N hammer [5,000 lb (2,268 kg)] is recommended for normal-strength masonry, whereas a Type L hammer [1,600 lb (726 kg)] is recommended for lower-strength masonry. Impacts at the same test location should be continued until consistent readings are obtained because surface roughness can affect initial readings.

The method is limited to tests of only the surface wythe (leaf). Tuck-pointing may influence readings, and the method is not sensitive to cracks.

Measurement of surface hardness for masonry walls has been studied by Noland et al. (1987).

Vertical Compressive Stress. In situ vertical compressive stress resisted by solid unreinforced masonry can be measured using a thin hydraulic flat jack that is inserted into a removed mortar bed joint. Pressure in the flat jack is increased until distortions in the brickwork are reduced to the precut condition. Existing vertical compressive stress is inferred from the jack hydraulic pressure, using correction factors for the shape and stiffness of the flat jack. For more information, refer to ASTM C1196 (2009).

The method is useful for measurement of gravity load distribution, flexural stresses in out-of-plane walls, and stresses in masonry veneer walls that are compressed by a surrounding concrete frame. The test is limited to only the face wythe of masonry.

No fewer than three tests should be done for each section of the building for which it is desired to measure in situ vertical stress. The number and location of tests should be determined based on the building configuration and the likelihood of overstress conditions.

Large-Scale Load Tests. Large-scale destructive tests may be undertaken on portions of a masonry component or element to (1) increase the confidence level on overall structural properties, (2) obtain performance data on archaic building materials and construction materials, (3) quantify effects of complex edge and boundary conditions around openings and two-way spanning behavior, and (4) verify or calibrate analytical models. Large-scale load tests do not necessarily have to be run to the ultimate limit state. They may have value for simply demonstrating structural integrity up to some specific performance level.

In situ large-scale tests are expensive and are typically limited to a single or a few samples, and test data must be extrapolated to the remainder of the system, based on a low confidence level. In situ tests may result in considerable local damage that requires substantial reconstruction near the sample location. In situ testing may prove unreasonably costly or impractical in certain situations because of several factors, such as time and space limitations and unavailability of portable testing facilities. On such occasions, it may be feasible to remove and transport masonry samples for laboratory testing. Procedures for removal and transportation of masonry samples are given in Building Science Series 62 (NBS 1977). Standards for laboratory test methods are published by ASTM.

Out-of-plane strength and behavior of masonry walls can be determined with air-bag tests. Behavior of test panels incorporating connections and edge details can be determined from such a test, in addition to flexural and arching properties of a solid or perforated wall. Strength and deformation capacity under in-plane seismic forces can be determined by loading an individual portion of wall that is cut free of the surrounding masonry. Loading actuators are reacted against adjacent and stronger portions of masonry.

Air-bag testing can provide insight regarding the out-of-plane strength of the wall but does not consider the dynamic characteristics of a cracked wall responding to out-of-plane demands. Where adequate wall-to-diaphragm connections are present, the dynamic out-of-plane stability of URM walls is best evaluated using shake table testing with realistic boundary conditions.

Visual and nondestructive surveys should be used to identify locations for test samples.

Standards for laboratory test methods are published by ASTM.

C11.2.2.4 Condition Enhancement Enhancing existing masonry elements can be an integral step for effective retrofits and, as such, it should be conducted as part of the condition assessment before extensive material testing is undertaken and retrofits are implemented. Replacement materials, brick, and mortar should be compatible with the original materials in terms of mechanical properties, as well as porosity and water vapor permeability. Many historic masonry buildings have been severely damaged by using incompatible materials that have very different strength, density, and stiffness than the original materials.

For filling voids in masonry, cementitious or lime-based grouts are preferable because they are more compatible with the base material than epoxy. To minimize the risk of displacing masonry elements, a low-pressure injection is preferable, with pressures typically limited to about 5 to 10 lb/in.^{2} (34.5 to 69.0 kPa). Users are cautioned about injecting epoxies into voids. Bursting of structural material has inadvertently been caused by epoxy having a flash set and substantial expansion when a critical volume is injected.

C11.2.2.5 Pointing or Repointing of Unreinforced Masonry Walls For guidance on pointing or repointing, see NPS (1998), BIA (2005), National Research Council Canada (2008), and ASTM C270, Appendix X3 (2014).

C11.2.3 Properties of In-Place Materials and Components

C11.2.3.3 Masonry Compressive Strength The three test methods are further described in Section C7.3.2.1 of FEMA 274 (1997c). As an alternative to the test methods given in this section of this standard and for buildings constructed with materials similar to those specified in TMS 602, the expected masonry compressive strength may be deduced from a nominal value prescribed in TMS 602. Nominal values prescribed in TMS 602 are based on the unit-strength method and are more conservative than values obtained from prism testing. Furthermore, the unit-strength method of TMS 602 was developed based on data from masonry constructed after the 1950s, and its application to earlier masonry construction may not be appropriate. Old masonry is often characterized by low strength values. Testing is recommended for masonry constructed before the 1950s. Underestimating masonry compressive strength, such as by using default values, can be unconservative when determining demands on frame members and connections of masonry infilled frames.

C11.2.3.4 Masonry Elastic Modulus in Compression Default values of elastic modulus in accordance with TMS 402 are based on a scalar coefficient multiplied by the expected masonry compressive strength, f_{me}. The elastic modulus, E_{me}, shall be calculated as the slope of the stress–strain curve between 5% and 33% of the expected masonry compressive strength, f_{me}. Wolde-Tinsae et al. (1993) have shown that the scalar coefficients adopted by TMS 402 are appropriate when f_{me} is based on the unit-strength method, whereas lower scalar values were found when f_{me} is based on prism tests. Using f_{me}, as determined in Section 11.2.3.3 of this standard, to determine E_{me} in accordance with Section 11.2.3.4 will overestimate the elastic modulus. The alternative using ASTM C1197 (2020a) can provide more accurate and reliable estimates of elastic moduli.

C11.2.3.5 Masonry Flexural Tensile Strength The flexural tensile strength of older brick masonry walls constructed with lime mortars may often be neglected. However, the term “lime mortar” is often not consistently defined and may be misunderstood. Mortar Types S, N, and O use lime in different proportions. The term *lime mortar* is commonly used to refer to mortars that have lime as the primary binding agent. The concept that weathering of mortar is attributed to “lime mortar” can be misleading because unwashed sand is also a common reason for weathering of mortar.

The three test methods for out-of-plane bending, including guidance on field implementation of the bond wrench method for in situ testing, are further described in Section C7.3.2.3 of FEMA 274 (1997c) and Hamid and Schuller (2019). For in-plane bending, flexural stress gradients across the section width are much lower than for out-of-plane bending. Thus, data from tests described in this section are conservative and should be used only in lieu of data on in-plane tensile strength.

C11.2.3.6 Unreinforced Masonry Shear Strength

C11.2.3.6.1 Determination of Expected URM Shear Strength by Testing for Bed-Joint Shear Strength Expected shear strength of URM components can be inferred from in situ measurements of bed-joint shear strength using the in-place shear test detailed in ASTM C1531.

The method is limited to tests of the face wythe. When the test unit is pushed, resistance is provided across not only the bed-joint shear planes but also the collar-joint shear plane. Because seismic shear is not transferred across the collar joint in a multiwythe (multileaf) masonry wall, the estimated shear resistance of the collar joint must be deducted from the test values. This deduction is achieved by multiplying the v_{te} term by 0.75 in Equation (11-2), which for a typical clay unit is the ratio of the areas of the top and bottom bed joints to the sum of the areas of the bed and collar joints. In cases where the collar joint does not contribute to the shear strength, the 0.75 factor need not be applied.

The effect of friction at the particular location of the masonry element being evaluated is accounted for by increasing the bed-joint shear capacity by the addition of the term P/A in Equation (11-2). The sum is then multiplied by a reduction factor equal to 0.75, which is an adjustment to indirectly account for the expected difference between in situ and tested strengths. The 1.5 factor in Equation (11-2) reduces the tested shear to average shear on the wall or wall pier. The shear stress, based on bed-joint sliding, is calculated by $v = VQ/Ib$ where Q is zero at the edge of the cross section and maximum at the center of the element.

C11.2.3.6.2 Alternative Procedures for Determining Expected URM Shear Strength by Testing for Tensile Splitting Strength Expected shear strength of URM components can also be inferred from tensile splitting tests as detailed in ASTM C1531(2019), ASTM C496 (1996), and ASTM E519 (2015). These alternatives are potentially useful where access for ASTM C1531 tests is restricted or where mortar strengths are expected to be higher than masonry unit shear strengths.

The effect of friction at the particular location of the masonry element being evaluated is accounted for by increasing the tensile splitting shear capacity by the addition of the term P/A in Equation (11-5). The sum is then multiplied by a reduction factor equal to 0.75, which is an adjustment to indirectly account for the expected difference between in situ and tested strengths. The 1.5 factor in Equation (11-2) reduces the tested shear to average shear on the wall or wall pier. The shear stress, based on tensile splitting, is calculated by $v = VQ/Ib$ where Q is zero at the edge of the cross section and maximum at the center of the element.

C11.2.3.6.3 Determination of Lower-Bound URM Shear Strength by Testing for Bed-Joint Shear Strength In walls where collar-joint mortar does not contribute to the shear strength, the 0.75 factor modifying v_{tL} need not be applied.

C11.2.3.7 Masonry Shear Modulus Shear stiffness of reinforced masonry should ideally be taken as a fraction of the uncracked shear stiffness value. However, the relationship between the shear modulus and the modulus of elasticity for reinforced masonry has historically been given as $0.4E_m$, although little experimental evidence exists to support this relationship (see Commentary 4.2.2 in TMS 402). Table 10-5 of this standard provides similar guidance for concrete shear walls that are typically assumed to be cracked.

Laboratory tests of URM shear walls (Epperson and Abrams 1989, Abrams and Shah 1992) have found that the shear modulus of URM does approach the value of 0.4 times the elastic modulus in compression, as given by the theory of elasticity for isotropic, elastic members. This value is limited to elastic uncracked behavior of the URM. After cracking, the shear stiffness is known to reduce substantially as sliding along bed joints develops or as diagonal tension cracks open. Because these nonlinear effects cannot be related to the elastic modulus in compression, the $0.4E_m$ value is only appropriate for uncracked URM. Shear stiffness of postcracked URM can be taken as a fraction of the initial shear stiffness. Test data by Atkinson et al. (1989) provide estimates of shear stiffness based on a frictional mechanism along bed joints.

C11.2.3.8 Steel Reinforcement Yield Strength Properties Over the years, lap splice requirements for reinforced masonry have evolved. The development length for deformed bars in TMS 402 Equation (6-1):

$$l_d = \frac{0.13 d_b^2 f_y \gamma}{K \sqrt{f'_m}}$$

In earlier editions of TMS 402, masonry development lengths and lap splices were only a function of the bar size and the yield stress of the reinforcement. The preceding formula now accounts for the masonry cover (K), and masonry compressive strength (f'_m) in addition to the bar size (γ and d_b) and yield stress of the reinforcement (f_y).

Lap splices are defined as the development length (required $l_s = l_d$), but not less than 12 in. (305 mm).

Research by Blake et al. (1995) led to the development of the TMS 402 lap splice equation. The underlying failure mode is to induce bar yielding before splitting to provide ductility. It is expected the lap splice will develop 1.25 $A_b f_y$. There was no cyclical testing. Blake et al. point out that lesser laps have resulted in masonry splitting—a nonductile response. Therefore, existing deformed bars that do not provide sufficient lap splice length to meet the TMS 402 requirements are deemed to be force controlled.

Based on the testing reported, reduced lap lengths exhibit a linear relationship to the full capacity.

Although it is expected that few reinforced masonry (RM) walls were constructed with plain bars, criteria are provided that require the lap splice length to be twice that of an equivalent deformed bar. Absent any testing, a factor of two was selected to be conservative unless testing or calculations are provided to justify shorter development and lap splice lengths. The shorter length should never be less than that for a deformed bar of equivalent diameter.

C11.2.3.9 Minimum Number of Tests The number and location of material tests should be selected to provide sufficient information to adequately define the existing condition of materials in the building. Test locations should be identified in those masonry components that are determined to be critical to the primary path of seismic-force resistance.

C11.2.3.10 Default Properties Default properties for masonry based on the tables in current code provisions are applicable to buildings built with materials similar to those specified in current codes or references. Where materials are different (i.e., type of mortar, unit strength, air entrainment), testing should be carried out to characterize the material properties.

Default values of compressive strength in Tables 11-2a and 11-2b are based on the unit-strength method in TMS 602 and a strength reduction factor of 0.6 for unreinforced masonry and 0.9 for reinforced masonry to reflect typical lower bounds. The unit-strength method in TMS 602 provides masonry compressive strength values based on the clay/concrete masonry unit compressive strength and type of mortar (M, S, or N). Default values for flexural tensile strength in Tables 11-2a and 11-2b are based on TMS 402 strength values and a strength reduction factor of 0.6 to reflect typical lower bounds. Lime mortar, traditionally made from lime, sand, and water, was commonly used in masonry construction throughout the country until the early twentieth century. This mortar is characterized by low strength and can be easily scraped away from the joints by hand with a metal tool. Analytical methods can be used to determine the components and their ratios for existing mortar. For more information on mortar analysis, refer to Schnabel (2009). Table 11-2c provides default values for old unreinforced clay masonry constructed with lime mortar. Default values in Table 11-2c are based on research by Lumantarna et al. (2014), Palmer and Hall (1931), and Palmer and Parsons (1934). A strength reduction factor of 0.6 was applied to compressive and flexural tensile strengths in Table 11-2c to reflect typical lower bounds. The 80% factor to obtain default masonry shear strengths in Tables 11-2a and c are expected to provide typical lower bounds. Comparison of default masonry shear values with values that may be obtained from Equation (11-1) shows that if in-place shear tests are undertaken, a significant increase in strength over default values is possible.

C11.3 MASONRY WALLS

Expected yield strength of reinforcing steel, as specified in this standard, includes consideration of material overstrength and strain hardening.

Component drift ratios are the ratio of differential displacement, Δ_{eff}, between each end of the component over the effective height, h_{eff}, of the component. Depending on the geometry of the wall or wall pier configuration, the elevations at which these parameters are determined may vary within the same wall element, as shown in Figure C11-1.

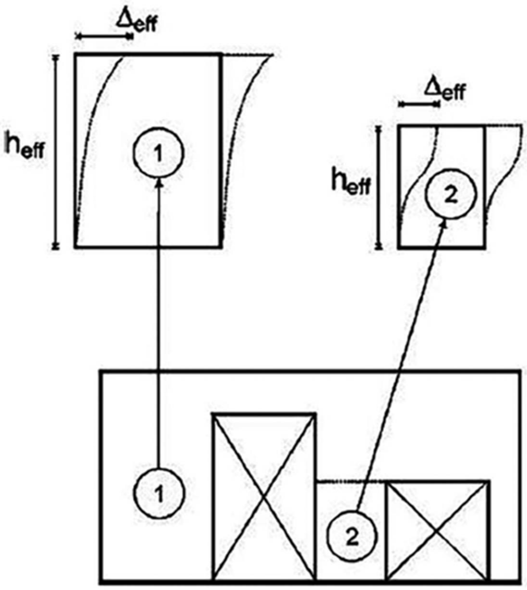


Figure C11-1. Effective height and differential displacement of wall components.

Note: h_{eff} = effective height of the component under consideration; Δ_{eff} = differential displacement between the top and bottom of the component. Depending on the wall and pier geometry, the elevations at which these parameters are defined may vary in the same wall assembly.

C11.3.1 Types of Masonry Walls Any of these categories of masonry elements can be used in combination with existing, retrofitted, or new seismic-force-resisting elements of other materials such as steel, concrete, or timber.

C11.3.1.2 New Masonry Walls Standards and guidelines for seismic design of new masonry walls are found in TMS 402, ASCE 7, and FEMA P-750 (2009b).

C11.3.1.3 Retrofitted Masonry Walls Methods of retrofitting masonry walls are intended to improve performance of masonry walls subjected to both in-plane and out-of-plane seismic forces and are described as follows.

Infilled Openings in Unreinforced Masonry. An infilled opening may be considered to act compositely with the surrounding masonry if the interface of new and old masonry has greater shear strength than the expected shear strength of the old masonry. This interface shear strength shall be verified experimentally.

Stiffness assumptions, strength criteria, and acceptable deformations for masonry walls with infilled openings should be consistent with unretrofitted solid masonry walls; differences in elastic moduli and strengths for the new and old masonry walls should be considered for the composite section.

Enlarged Openings. Openings in URM shear walls may be enlarged by removing portions of masonry above or below windows or doors.

Openings are enlarged to increase the height-to-length aspect ratio of wall piers so that the limit state may be altered from force-controlled to deformation-controlled actions. This method is only applicable to URM walls.

Stiffness assumptions, strength criteria, and acceptable deformations for URM walls with enlarged openings shall be reassessed to reflect the final condition of the wall. Load paths for structural elements supporting walls over existing or new openings such as lintels or arches must be addressed in such alterations.

Shotcrete. An existing unreinforced masonry wall with an application of shotcrete may be considered as a composite section if the bond between the shotcrete and the masonry is adequate to force a common strain in the composite materials. Stresses should be determined by relative elastic moduli. The load path to the shotcrete from roof and floor diaphragms should not pass through the unreinforced masonry.

The masonry surface should be prepared to remove any paint or similar coating that reduces bond strength and should be wetted before application of shotcrete to increase bond strength. The shotcrete mix should have low shrinkage and should be wet-cured to minimize shrinkage.

Coatings and Near-Surface Mounted Reinforcement for URM Walls. A coated masonry wall may be considered a composite section as long as anchorage is provided at the interface between the coating and the masonry wall to transfer shear forces. Stresses in the masonry and coating should be determined considering the difference in elastic moduli for each material. If stresses exceed expected strengths of the coating material, then the coating should be considered ineffective.

Overlays and near-surface mounted bars of steel- or fiber-reinforced polymers bonded by adhesives can be used to alter the sequence of displacement-controlled and force-controlled actions of existing masonry walls (Moon et al. 2006, Ismail et al. 2011).

Stiffness assumptions, strength criteria, and acceptable deformations for coated masonry walls should be consistent with existing URM walls.

Reinforced Cores for URM Walls. A reinforced-cored masonry wall should be considered to behave as a reinforced masonry wall, provided that the bond between the new reinforcement and the grout and between the grout and the cored surface are capable of transferring reinforcement strain to the masonry. Vertical reinforcement should be embedded at the base of the wall to resist the full tensile strength of the reinforcement.

Grout in new reinforced cores should consist of cementitious materials whose hardened properties are compatible with those of the surrounding masonry.

Adequate shear strength must exist or should be provided, so that the strength of the new vertical reinforcement can be developed.

Stiffness assumptions, strength criteria, and acceptable deformations for URM walls with reinforced cores should be consistent with RM walls.

Prestressed Cores for URM Walls. A prestressed-cored masonry wall with unbonded tendons should be considered to behave as a URM wall with increased vertical compressive stress.

Losses in prestressing force caused by creep and shrinkage of the masonry should be accounted for in analyses conducted in accordance with Chapter 7.

Stiffness assumptions, strength criteria, and acceptable deformations for URM walls with unbonded prestressing tendons should be consistent with existing URM walls subjected to vertical compressive stress.

Grout Injections. Grout used for filling voids and cracks should have strength, modulus, and thermal properties compatible with the existing masonry.

Inspections should be conducted in accordance with Section 1.5.10 during the grouting process to ensure that voids are completely filled with grout.

Stiffness assumptions, strength criteria, and acceptable deformations for masonry walls with grout injections should be consistent with existing URM or RM walls.

Repointing. Bond strength of new mortar should be equal to or greater than that of the original mortar. Compressive strength of new mortar should be equal to or less than that of the original mortar.

Stiffness assumptions, strength criteria, and acceptable deformations for repointed masonry walls should be consistent with existing masonry walls. See also Section 11.2.2.4, "Condition Enhancement."

Braced Masonry Walls. Masonry walls with height-to-thickness ratios in excess of those permitted by Section 11.3.3.3, or out-of-plane bending stresses in excess of those permitted by Section 11.3.5, may be braced with external structural elements. Adequate strength and stiffness should be provided in the bracing element and connections to transfer forces from the masonry wall to the roof or floor diaphragm. The horizontal spacing of the vertical braces should not exceed one-half of the story height. Deflection of the bracing members sized in accordance with Chapter 7 or Chapter 13 should not exceed 10% of the wall thickness. Out-of-plane deflections of braced walls resulting from the transfer of vertical floor or roof loadings should be considered.

Stiffness assumptions, strength criteria, and acceptable deformations for braced masonry walls should be consistent with existing masonry walls. The reduced span of the masonry wall should be considered.

Stiffening Elements. Masonry walls with inadequate out-of-plane stiffness or strength may be stiffened with external structural members. The stiffening members should be proportioned to resist a tributary portion of seismic force applied normal to the plane of a masonry wall. Connections at the ends of the stiffening element should be provided to transfer the reaction force. Flexibility of the stiffening element should be considered where estimating seismic drift of a masonry wall panel.

Stiffness assumptions, strength criteria, and acceptable deformations for stiffened masonry walls should be consistent with existing masonry walls. The stiffening action that the new element provides shall be considered.

Cavity Walls, Multiwythe Walls with Inadequate Composite Action between Wythes, and Veneers and Their Attachment. For multiwythe walls that have adjacent wythes that are not effectively bonded by headers to develop composite action and that have collar joints filled with mortar or grout, TMS 402, Section 5.1.3.2.3, provides an alternative for installing wall ties to develop composite action.

Veneer, commonly a single wythe of unreinforced masonry units not tied to the core masonry wall by header courses, may be retrofitted as a part of the core wall by grouting the cavity between the veneer and core wall and installing ties from the veneer to the core wall. However, where cavities are filled, the effects of such alterations on the moisture and weathering resistance of the building should be considered. Spacing of the ties should conform to Section 13.2 of TMS 402.

C11.3.2 Unreinforced Masonry Walls and Wall Piers Subject to In-Plane Actions

C11.3.2.1 Stiffness of URM Walls and Wall Piers Subject to In-Plane Actions Laboratory tests of solid shear walls have shown that behavior can be depicted at low force levels using conventional principles of mechanics for homogeneous materials. In such cases, the lateral in-plane stiffness of a solid cantilevered shear wall, k, can be calculated using Equation (C11-1):

$$k = \frac{1}{\frac{h_{\text{eff}}^3}{3E_m I_g} + \frac{h_{\text{eff}}}{A_v G_m}} \qquad \text{(C11-1)}$$

where

h_{eff} = Wall height,
A_v = Shear area,
I_g = Moment of inertia for the gross section representing uncracked behavior,
E_m = Masonry elastic modulus, and
G_m = Masonry shear modulus.

Correspondingly, the lateral in-plane stiffness of a wall pier between openings with full restraint against rotation at its top and bottom can be calculated using Equation (C11-2):

$$k = \frac{1}{\frac{h_{\text{eff}}^3}{12E_m I_g} + \frac{h_{\text{eff}}}{A_v G_m}} \qquad \text{(C11-2)}$$

The design professional should be aware that a completely fixed condition is often not present in actual buildings.

Multiwythe solid brick walls with effective header courses typically have not less than 10% of the surface area of a wythe connected with bonded solid headers extending not less than 4 in. (102 mm) into the adjacent wythe(s). The clear distance between adjacent full-length headers shall not exceed 24 in. (610 mm) measured vertically or horizontally. Where the backing consists of two or more wythes, the headers shall extend not less than 4 in. (102 mm) into the most distant wythe, or the backing wythes should be bonded together with separate headers with their area and spacing conforming to the foregoing.

Wythes of walls not bonded as described earlier should be considered veneer. Veneer wythes should not be included in the effective thickness used in calculating height-to-thickness ratios, stiffnesses, and strengths of walls.

Linear Stiffness for In-Plane Spandrel Actions. The stiffness of URM spandrel beams subjected to seismic in-plane forces shall be determined by accounting for the spandrel shear and flexural flexibility. The spandrel stiffness depends on the modulus of elasticity of the masonry for loading parallel to the bed joints. Similar to URM piers, the initial stiffness of spandrels can be estimated using elastic beam theory (Beyer and Mangalathu 2014). For most spandrel configurations, the shear flexibility dominates the behavior, and reasonable estimates of the stiffness might be obtained if the shear flexibility only is considered. However, for more slender spandrel configurations, the flexural flexibility should be included. The total elastic stiffness, k_{el}, is calculated by Equation (C11-3):

$$k_{el} = \left(\frac{1}{k_s} + \frac{1}{k_{fl}}\right)^{-1} \qquad \text{(C11-3)}$$

where

k_s = Shear stiffness, and
k_{fl} = Flexural stiffness.

Because the section of a spandrel supported on an arch is not constant along its length, approximations concerning the effective height are required. The shear stiffness can be estimated using the height h_{sp} at midspan [Equation (C11-4)]:

$$k_s = \frac{5}{6} G \times \frac{h_{sp} \times t_{sp}}{l_{sp}} \qquad \text{(C11-4)}$$

where G is the shear modulus computed from the Poisson's ratio ν and the modulus of elasticity, E_{mh}, characterizing the stiffness of the masonry for loading parallel to the bed joints.

The flexural stiffness is computed for a beam subjected to double bending using Equation (C11-5):

$$k_{fl} = 12E_{mh} \times \frac{h_{sp}^3 \times t_{sp}}{12 l_{sp}^3} \qquad \text{(C11-5)}$$

For masonry spandrels with arches, the height varies along the length of the spandrel. Comparisons with numerical analyses have shown that this height can be approximated by the height of the spandrel, including the thickness of the arch at one-third of the span (Beyer and Mangalathu 2014).

The deformation of a spandrel shall be defined in terms of its rotation (Figure C11-2). The force–rotation relationship is shown in Figure C11-3.

If spandrel rotations are not obtained directly from finite-element calculations or similar, they can be computed from the rotations of the adjacent piers according to Equations (C11-6) and (C11-7):

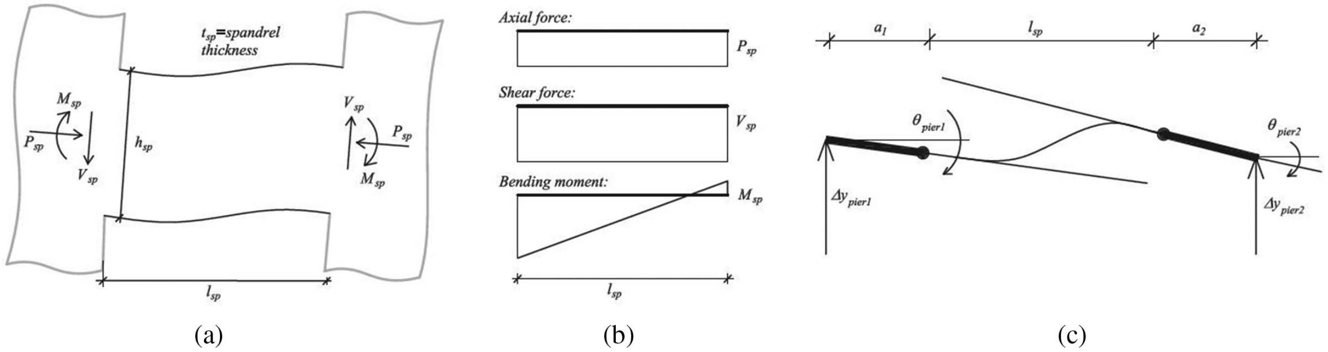


Figure C11-2. (a) Geometry of the spandrel, (b) internal force diagrams of the spandrel, and (c) spandrel deformation as a function of the vertical displacement and chord rotation of the adjacent piers.

Source: Beyer (2012); reproduced with permission from Elsevier.

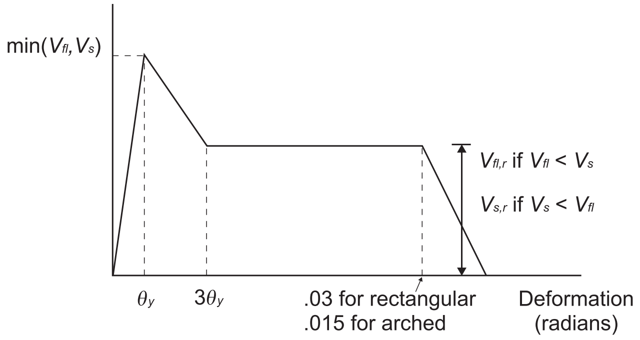


Figure C11-3. Generalized force–rotation relationship for masonry spandrels.

$$\Delta_{sp} = \Delta y_{\text{pier1}} - \Delta y_{\text{pier2}} + \theta_{\text{pier1}}\left(a_1 + \frac{l_{sp}}{2}\right) + \theta_{\text{pier2}}\left(a_2 + \frac{l_{sp}}{2}\right) \tag{C11-6}$$

$$\theta_{sp} = \frac{\Delta_{sp}}{l_{sp}} \tag{C11-7}$$

C11.3.2.2 Strength of URM Walls Subject to In-Plane Actions Typically for URM piers with low levels of vertical axial stress, rocking or sliding governs the response. These actions have been observed to exhibit large displacement capacities; however, they can result in significant residual displacements. At higher levels of vertical axial stress, diagonal tension and toe-crushing force-controlled actions are more common (Moon 2004). Mixed modes, or more accurately, sequences of different behavior modes, are common in URM wall pier experiments (FEMA 307) (FEMA1998b). For example, rocking piers can sequence into bed-joint sliding as a result of cracks propagating or toe crushing with increasing degrees of rotation. Mortar strength, aspect ratios, and vertical stresses are the most important factors determining the sequence of in-plane actions.

Wall spandrels that are stronger than piers can couple multiple piers and transmit overturning to adjacent piers, increasing axial forces in end piers and potentially changing their sequence of actions. Spandrels are typically not essential to the gravity load path, that is, to the stability of the structure under vertical loads. Studies on masonry wall configurations have, however, shown that spandrels can have a significant influence on the overall building behavior by increasing the stiffness and strength of the building when subjected to horizontal loads (Chen et al. 2008, Lagomarsino et al. 2013, Magenes 2000). Because of the higher axial force and steeper moment profile (smaller shear span of piers), the system deformation capacity is typically reduced by the presence of spandrels.

For engineering purposes, the effect of masonry spandrels on the global behavior of URM buildings is best assessed through equivalent frame or macroelement analysis (Chen et al. 2008, Lagomarsino et al. 2013, Magenes 2000). These analyses require as input the stiffness, strength, and deformation capacities of spandrels.

To determine whether perforated wall behavior is governed by weak spandrels versus strong piers or vice versa, consider calculating an index, S_i, that compares the demand–capacity ratios for the piers and spandrels at each joint i using Equation (C11-8):

$$S_i = \frac{\dfrac{\Sigma(Q_{UD}\text{ or }Q_{UF})_{\text{pier}}}{\Sigma\min\left(mkQ_{CE},\, kQ_{CL}\right)_{\text{pier}}}}{\dfrac{\Sigma(Q_{UD}\ or\ Q_{UF})_{\text{spandrel}}}{\Sigma\min\left(mkQ_{CE},\, kQ_{CL}\right)_{\text{spandrel}}}} \tag{C11-8}$$

where

$\Sigma(Q_{UD}\text{ or }Q_{UF})_{\text{pier}}$ = Sum of the applicable deformation-controlled shears or force-controlled shears acting on the piers above and below the joint,

$\Sigma\ \min(mkQ_{CE}, kQ_{CL})_{\text{pier}}$ = Sum of the minimum applicable modified deformation-controlled expected strengths or lower-bound force-controlled strengths of the piers above and below the joint,

$\Sigma\ \min(Q_{UD}, Q_{UF})_{\text{spandrel}}$ = Sum of the applicable deformation-controlled shears or force-controlled shears acting on the spandrels to the left and right of the joint, and

$\Sigma\ \min(mkQ_{CE}, kQ_{CL})_{\text{spandrel}}$ = Sum of the minimum applicable modified deformation-controlled expected strengths or lower-bound force-controlled strengths of the spandrels to the left and right of the joint.

When $S_i > 1.0$, a weak pier–strong spandrel mechanism can be expected to form.

When $S_i < 1.0$, a strong pier–weak spandrel mechanism can be expected to form. Alternatively, nonlinear analyses of URM piers and spandrels can help determine where hinges are more likely to form first in walls and have the advantage that the stresses and rotations developed in the URM components can be evaluated directly and deformation compatibility can be maintained.

Estimates of spandrel strengths, although not confirmed by component tests, can be used to determine if spandrels are likely to be weaker or stronger compared to piers (FEMA 306 1998a). The effects of global and component overturning and rocking of entire perforated walls depend on how effectively spandrels can transmit vertical shears and bending. Conversely, wall spandrels that are weak relative to adjacent piers may not provide fixity at the tops and bottoms of piers and may result in piers acting as cantilevers.

URM walls responding in-plane in an earthquake are often of nonrectangular section. Walls connected to and oriented perpendicular to in-plane walls are termed *flanges*, *return walls*, or *transverse walls*. Costley and Abrams (1996), Paquette and Bruneau (2003), Moon et al. (2006), Yi et al. (2008), and Russell and Ingham (2010) recognized through experimental research that flanges have the potential to influence the response of walls that resist seismic forces in plane. Flanges can influence in-plane wall failure modes, maximum strengths, and displacement capacities. Flanges can significantly increase sliding and rocking strength but may only contribute to minor increases in diagonal tension strength. Flanges were found to increase the limiting drift of walls failing in diagonal tension.

Flanges are defined by Moon et al. (2006) as the portions of the walls oriented out of plane that participate with the walls oriented in the plane of seismic loading. Yi et al. (2008) noted that previous experimental research on URM building systems (Costley and Abrams 1996, Paquette and Bruneau 2003, Moon et al. 2006, Yi et al. 2006b) highlighted the beneficial effects of flanges on the response of in-plane loaded walls and indicated the potential for flanges to influence maximum strength and pier failure modes. Paquette found that wall flanges increase overall wall stiffness for low-intensity ground motions compared with unflanged walls, but the influence of flanges on stiffness becomes significantly reduced after cracking in response to high-intensity ground motions. Following full-scale testing of a 2-story URM building (Moon 2004; Yi 2004; Yi et al. 2006a, b) where significant flange participation was observed, Yi et al. (2008) developed an analytical model to investigate the effects of flanges on the behavior of individual nonrectangular section URM piers. Yi et al. (2008) presumed an example wall and from a pushover analysis determined that the in-plane lateral strength of a wall with flanges could be expected to be greater compared with a similar wall with no flanges. It was also postulated by Yi et al. (2008) that the drift corresponding to lateral-force failure depends on the location of the flange in relation to the in-plane loaded wall. When the flange is at the toe of the wall (i.e., the flange is in compression), the flange reduces the compressive stress at the toe and delays toe-crushing failure. Conversely, when the flange is at the heel (i.e., the flange is in tension), the compressive stress in the toe increases because of the increased weight of the flange.

Russell and Ingham (2010) conducted further experimental analysis and also concluded that the effect of flanges on in-plane wall response can be significant. It was found that for URM walls with flanges, flexure is less likely as a behavior mode and shear is more likely to limit the lateral strength. It was also found that URM walls with flanges are able to sustain larger seismic forces than walls without flanges. Flanges were found to increase the displacement capacity of in-plane loaded walls when the flange is in compression, compared with similar walls without flanges. Moreover, a flange acting in tension increases the lateral strength of in-plane loaded walls. It was found that for walls with compression flanges and failing in a deformation-controlled action of stair-stepped cracking, the drift capacity at loss of seismic load capacity could be estimated to be 1.5 times greater than when no compression flange is present. This drift limit could be relaxed if a larger data set is available in the future.

One commonly used approach to model URM flanges is to assume that the lengths of flanges acting in compression are six times the thicknesses of the in-plane walls or the actual lengths of the flanges, whichever are less, consistent with TMS 402, and to assume that equivalent lengths of tension flanges to resist global or component overturning are based on likely crack patterns relating to uplift in flange walls (Yi et al. 2008). Other approaches that model or qualitatively consider different flange lengths may result in a variety of crack patterns and corresponding sequences of actions.

Axial stresses caused by the vertical component of seismic loading, including overturning and the interaction effects at wall intersections, can significantly alter the strengths and sequences of actions in URM wall piers, particularly those at ends of walls and with or without flanges. Explicit considerations of the effects of the vertical component of seismic loading and overturning are not recommended for linear procedures because realistic estimates of vertical load distributions are only feasible with nonlinear procedures. For nonlinear static and dynamic procedures, consider substituting $Q_G \pm Q_E$ for P_D in Equations (11-2), (11-5), (11-6), (11-7), (11-9), (11-11), and (11-12). For nonlinear static and dynamic procedures, consider substituting $(Q_G \pm Q_E)/A$ for f_a in Equations (11-12) and (11-13). Q_E is taken as the vertical component of the seismic loading.

C11.3.2.2.1 Expected In-Plane Rocking Strength of URM Walls and Wall Piers Different methods of modeling the effective height of masonry piers are found in the literature. The rocking equation for expected lateral strength is a revised equation from ASCE 41-06, Equation (11-8), that explicitly incorporates the weight of the wall or pier and its location. The factor 0.9 is an approximation that accounts for the difference in total pier length compared with the distance between the tension end of the pier and the location of the compression centroid. More accurate estimates of the location of the compression centroid can be used consistent with TMS 402 or can be considered explicitly within a nonlinear analysis building model or component-level moment-curvature analysis.

Assumptions of fixity or cantilever action depend on the stiffness and overall integrity of the spandrels above and below rocking piers. The potential for spandrel uplift along a line of resistance caused by pier rocking and effects of vertical seismic acceleration can also significantly affect pier response (Figure C11-4). The complete uplift of a spandrel from a pier can result in a loss of stability and shall not be permitted unless an alternate means of maintaining stability is provided.

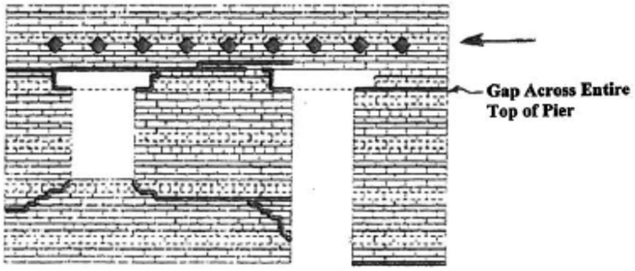
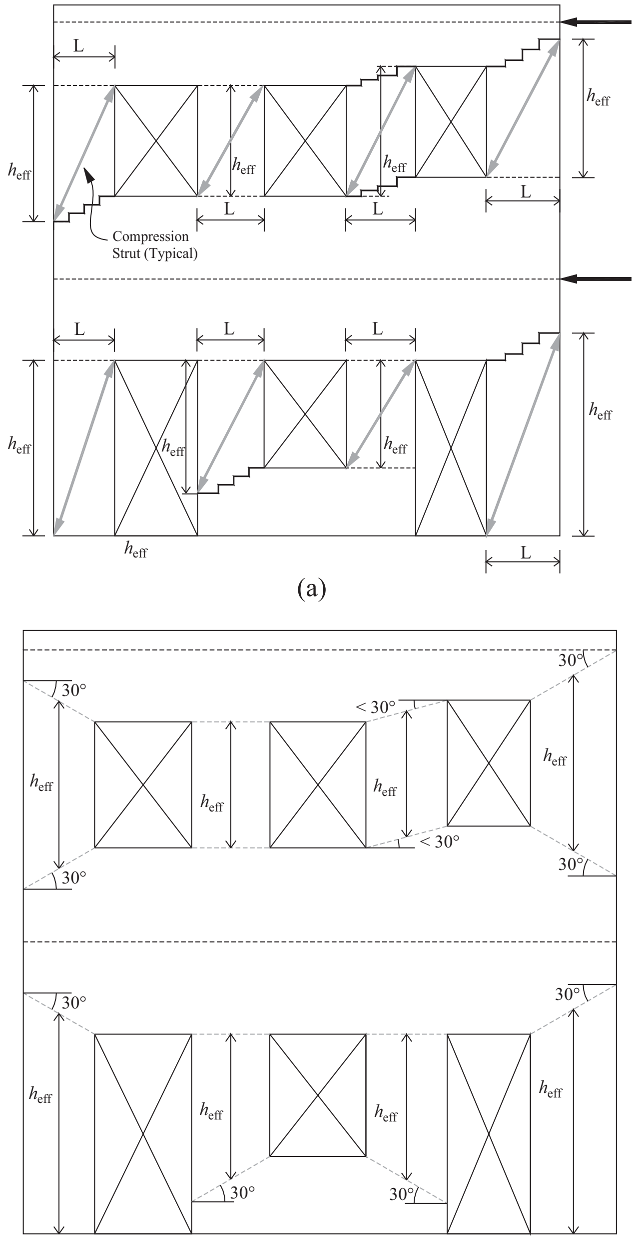


Figure C11-4. Perforated URM walls with rocking piers that have dissimilar aspect ratios and relatively strong spandrels can result in spandrel uplift and gaps forming across entire piers rendering relatively slender piers ineffective and potentially unstable.

Source: Paquette and Bruneau (2003), © ASCE.

For URM walls with openings of differing sizes and relatively weaker piers compared with stronger spandrels, Moon (2004) recommends that the effective height of each rocking pier be represented as the height over which a diagonal compression strut is most likely to develop in the pier at the steepest possible angle that would offer the least lateral resistance. As a result, effective heights for some rocking piers adjacent to unequal size openings vary depending on the direction of loading. The angles at pier hinges generally depend on bed and head joint dimensions and stair-step cracking along mortar joints (Figure C11-5a). Using Moon's approach, the locations of the effective heights vary depending on the direction of loading. Dolce (1989) proposed that the effective height be defined by the midpoints of lines representing maximum 30 degree inclinations of flexural cracks initiating from the corners of openings. This method does not depend on the direction of loading and is a simpler alternative for modeling rocking wall systems for loads in both directions (Figure C11-5b) compared to Moon's method (Figure C11-5a). Dolce also proposed further refinements to account for pier–spandrel joint flexibility, but for simplicity the refinements are not included in Figure C11-5b. The modeling approach based in part on Dolce in Figure C11-5b will be generally more conservative for perforated wall systems that have rocking piers as the most critical components if the assumed h_{eff} is greater than Moon's modeling approach depicted in Figure C11-5a. Most walls with rocking piers tend to respond asymmetrically to loads in different directions, so the analysis of rocking actions can benefit from modeling approaches that rely on incremental refinements and reanalysis.



(a)

(b)

Figure C11-5. URM rocking pier effective heights: (a) based on developing diagonal compression struts that vary with direction of seismic force, (b) based on flexural cracks starting at 30 degrees from opening corners per Dolce.
Source: (a) Adapted from Moon (2004), (b) adapted from Dolce (1989).

Test results of entire wall systems suggest that assumptions of boundary conditions can vary greatly from actual conditions. In addition, where estimated expected strengths for rocking are similar to expected strengths for toe crushing or bed-joint sliding, slight variations in actual conditions may substantially alter the strengths, drifts, and sequences of actions in piers and spandrels. Flanged walls can have considerably higher rocking strengths than those calculated by assuming that no flanges exist, and other actions, particularly force-controlled actions, may control rocking piers with flanges.

For rocking wall piers with relatively high axial loads, toe crushing can often onset as a secondary yield mechanism when the pier is subjected to a sufficiently large drift. For linear procedures, the m-factors for rocking are defined as a function of axial load with a force-controlled limit based on available testing described in C11.3.2.3.1 and Tremayne et al. (2012). If toe crushing can be demonstrated by analysis not to occur at higher axial loads, the pier can still be classified as deformation-controlled but the m-factor is capped at 1.0, consistent with the available testing. Moment-curvature analysis of a wall pier, considering material properties, geometry, and axial load, may be an acceptable means of demonstrating that the pier remains deformation controlled for f_a/f'_m ratios greater than 8%. For nonlinear procedures, the rocking provisions require the yield mechanism hierarchy to be explicitly considered. Given the potential for variation in response, users of this standard are encouraged to consider varying their assumptions about rocking wall and wall pier boundary conditions, effective pier heights, material properties, and yield hierarchy to determine the sensitivity of the expected performance.

C11.3.2.2.2 Expected In-Plane Bed-Joint Sliding Strength of URM Walls and Wall Piers Results from experimental testing undertaken by Abrams and Shah (1992), Magenes and Calvi (1992), Anthoine et al. (1995), Franklin et al. (2001), Paquette et al (2004), Moon et al. (2006), and Russell and Ingham (2010) have confirmed that URM elements that exhibit bed-joint sliding behavior have substantial deformation capacity past initial cracking.

The capacity for bed-joint sliding in masonry elements is a function of frictional resistance and bond. The bond component is progressively degraded as cracking occurs until only the frictional component remains. Equation (11-10) represents the initial uncracked bed-joint sliding strength, and Equation (11-11), the final frictional capacity, as detailed in FEMA 306 (1998a).

A second form of bed-joint sliding cracking exists with weak mortar, strong units, and low compressive stress, when the cracks propagate in a stair-stepped manner in head and bed joints. This mechanism occurs because of the principal tension stress in the applied stress state during earthquake loading exceeding the tension strength of the mortar joints, and there is considerable overlap between this behavior mode and diagonal tension cracking through masonry units. Drift limits for this failure mode were reported in a summary of experimental research in FEMA 307 (1998b) of up to 0.75%. More recently, Russell and Ingham (2010) found that the URM walls with flanges could sustain in-plane seismic forces to a drift of up to 1.0% before loss of lateral-force capacity when failing in this stair-stepped cracking mode, and for walls or piers without flanges, a similar drift limit of 0.4% was recommended. Priestley et al. (2007) suggest a drift limit of 0.4% for walls failing in a deformation-controlled, shear-dominated response.

C11.3.2.2.3 Lower-Bound In-Plane Toe-Crushing Strength of URM Walls and Wall Piers Equation (11-12) addresses toe crushing at the ends of walls and wall piers that can occur before other actions occur. In addition, toe crushing can occur after other actions initiate. See Section 11.3.2.2.3 for toe-crushing strength determination and acceptance criteria that occur after rocking initiates. The limit on L/h_{eff} to be taken not less than 0.67 was removed from Equation (11-12) in the 2013 edition of the standard to allow use of this equation for more slender rocking piers as one acceptable method to determine latent toe crushing in Table 11-4. Detailed moment-curvature analysis of slender piers will generally provide better estimates of the onset of latent toe crushing than Equation (11-12).

C11.3.2.2.4 Lower-Bound In-Plane Diagonal Tension Strength of URM Walls and Wall Piers In this behavior mode, diagonal cracking occurs in the masonry and involves complex mechanisms. This cracking is a result of the tension strength of the masonry being exceeded when subjected to the applied stress state during earthquake loading. The behavior mode is manifested by cracking directly through the masonry units. Cracking directly through the units—resulting from strong mortar, weak units, and high compressive stress—can be identified by diagonal cracks ("X" cracks) and occurs without significant ductile response. In many cases, the cracking is sudden and brittle, and vertical load capacity drops quickly. The cracks may then extend to the toe, and the triangles above and below the crack separate. In some cases, the load drop may be more gradual, with cracks increasing in size and extent with each cycle.

The lateral strength of walls or piers based on diagonal tension strength is determined using Equation (11-13), which is taken from Turnšek and Sheppard (1980). This equation was calibrated for the range of $0.67 \leq L/h_{\text{eff}} \leq 1.0$ and requires determination of masonry diagonal tension strength, f'_{dt} . For walls with L/h_{eff} above or below the caps, using the capped values is recommended; however, users should be aware that no substantiating research is available. In lieu of determining the diagonal tension strength, the lower-bound bed-joint shear strength, v_{mL}, as measured with the in-place shear test, may be substituted where it is assumed that the lower-bound diagonal tension strength is equal to the lower-bound value of the bed-joint strength. However, this strength value only applies to the mortar, not the masonry units. Thus, there is considerable uncertainty in diagonal tension strength estimates.

For conditions where axial stresses on walls or wall piers are relatively low and the mortar strengths are also low compared with the splitting strengths of the masonry units, diagonal tension actions may be judged not to occur before bed-joint sliding. However, there is no available research to help determine a specific threshold of axial stress and relative brick and mortar strengths that differentiates whether cracking occurs through the units or through the mortar joints.

C11.3.2.2.6 Expected Strengths of Rectangular URM Wall Spandrels Subject to In-Plane Actions Different configurations of spandrel elements exist: In old masonry buildings, the masonry spandrel is typically supported by a concrete, steel, or timber lintel or a masonry arch. In new buildings, steel lintels or reinforced concrete beams or slabs may support masonry spandrels.

Full-scale component tests on URM spandrels are available for reinforced concrete floor beams acting compositely with URM spandrels (Beyer and Dazio 2012b) and on URM spandrels supported by timber lintels and masonry arches (Amadio et al. 2011, Beyer and Dazio 2012a, Graziotti 2014). Furthermore, subassemblies of piers and spandrels (Foraboschi 2009, Knox 2012) and entire buildings were tested experimentally under horizontal loading (Costley 1996, Magenes et al. 1995, Paquette and Bruneau 2003). The tests on masonry spandrels showed that (1) the spandrel reaches its peak strength at relatively small chord rotations; if the rotations are increased, the force capacity of the spandrel drops to the residual strength, which remains rather stable for a large range of rotations (ATC 1998); (2) the peak strength depends on the quality of the masonry and the interlock of masonry units; (3) the residual strength is strongly dependent on the axial force that acts on the spandrel and the lintel or arch that supports the spandrel; and (4) the rotation capacity of the spandrel that is associated with the residual strength is often such that pier failure would occur before the spandrels lose their residual strength capacity.

A typical envelope of the shear force–rotation curve of spandrels has the following features (Figure C11-6): The shear force in the spandrel increases almost linearly up to V_{cr}, when the first cracks form. Thereafter, the stiffness diminishes until the peak strength, V_p, is reached. Up until the peak strength, the cracks in the spandrel remain rather small. The peak strength is followed by a significant drop in strength, and thereafter, the cracks grow significantly in width and number. The strength between the rotations θ_r and θ_{ult} is referred to as residual strength, that is, the strength of the spandrel after the formation of either a flexural or shear crack pattern in the spandrel. The residual

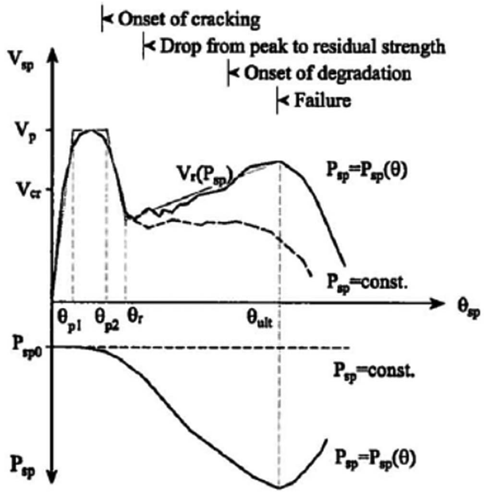


Figure C11-6. Schematic dependence of the spandrel's shear strength, V_{sp}, and axial force, P_{sp}, on the imposed deformation of the spandrel.

Source: Beyer (2012), reproduced with permission from Elsevier.

strength is closely related to the axial force in the spandrel. Because after cracking the axial force in the spandrel tends to increase, the shear force also increases. If the increase in axial force is large, the residual strength might eventually surpass the peak strength. The shear force–deformation envelope will flatten out, and the onset of degradation will eventually lead to failure. For URM piers, the ultimate deformation capacity of the piers is often defined as the deformation for which the shear strength drops to 80% of V_p. For most spandrel configurations, the drop in strength after attaining the peak shear strength will exceed 20% of V_p. Hence, if the same definition was applied to the spandrels, the deformation capacity of most spandrels would correspond to θ_{p2}. This would typically be overly conservative and lead to rather small deformation capacities of the entire URM wall. Seismic evaluations and retrofits therefore should explicitly consider the residual shear strength, V_r, of spandrels, which is generally lower than 80% of V_p.

The strength equations, including the explanatory text, are taken from the New Zealand Guidelines for the seismic assessment of URM buildings (NZSEE 2017). They are based on those in Beyer (2012), which build on those in FEMA 306 (ATC 1998). The strengths and shortcomings of the equations in FEMA 306 and a comparison with spandrel test data can be found in Beyer and Mangalathu (2014). The strength of spandrels based on diagonal tension is determined using Equations (11-19) and (11-26) for rectangular and arched spandrels, respectively. The strength equation is based on the model by Turnsek and Cacovic (1971). When compared with experimental data from URM spandrel subassembly tests (Beyer and Dazio 2012a, Graziotti et al. 2014) the Turnsek and Cacovic (1971) model has been shown to provide reasonable estimates of spandrel diagonal capacity. Limited experimental information is available on the performance of URM spandrels with lintels made from materials other than timber. It is anticipated that URM spandrels with concrete and steel lintels will perform in a similar manner to those with timber lintels. In unusual cases, where reinforced concrete lintels are present that are well embedded in the adjacent masonry walls, the capacity of the spandrel can be calculated neglecting the contribution of the URM. This calculation underestimates the actual strength of the spandrel, since the compression strut that forms in the masonry spandrel reduces the effective length of the reinforced concrete lintel (Beyer and Dazio 2012b). The embedment length needs to be sufficient to allow the formation of plastic hinge in the beam. If this is not the case, it is conservative to compute the spandrel strength using the preceding equations for concrete, steel, or timber lintels, which are only embedded over a short length, *sl*, in the adjacent masonry piers.

Axial stresses are generated in spandrel elements because of the restraint of geometric elongation and externally applied forces (e.g., pretensioned rods). Results from experimental research indicate that negligible geometric elongation can be expected when peak spandrel strengths are developed (Beyer 2012, Graziotti 2013). This is because peak spandrel strengths are developed at relatively small spandrel rotations. Consequently, little geometric elongation typically occurs. Unless the spandrel is prestressed, the axial stress in the spandrel can therefore be assumed to be negligible when determining the peak flexural and peak shear capacity.

Significant geometric elongation can occur once peak spandrel strengths have been exceeded, and significant spandrel cracking occurs within the spandrel as higher rotations are sustained in the element. An upper-bound estimate of the axial stress in a restrained spandrel, p_{sp}, can be determined using Equation (C11-9) (Beyer 2014):

$$p_{sp} = (1+\beta_s)f_{dt}\frac{l_{sp}}{2\sqrt{l_{sp}^2+h_{sp}^2}} \tag{C11-9}$$

where β_s is the spandrel aspect ratio (l_{sp}/h_{sp}).

Equation (C11-9) calculates the limiting axial stress generated in a spandrel associated with diagonal tension failure of the spandrel. The equation assumes that the spandrel has sufficient axial restraint to resist the axial forces generated by geometric elongation. In most typical situations, it can be assumed that spandrels that comprise the interior bays of multibay pierced URM walls will have sufficient axial restraint such that diagonal tension failure of the spandrels could occur.

Spandrels that comprise the outer bays of multibay pierced URM walls typically have significantly lower levels of axial restraint. In this case, the axial restraint may be insufficient to develop a diagonal tension failure in the spandrels. Sources of axial restraint that may be available include horizontal post-tensioning, diaphragm tie elements with sufficient anchorage into the outer pier, or substantial outer piers with sufficient strength and stiffness to resist the generated axial forces. For the latter to be effective, the pier would need to have adequate capacity to resist the applied loads as a cantilever.

It is anticipated that negligible axial restraint will be present in the outer bays of many typical unstrengthened URM buildings. In this case, the axial stress in the spandrel can be assumed to be zero when calculating the residual flexural strength.

Most tests on spandrels have been carried out with constant axial loads on the spandrels. This is not necessarily realistic but is convenient when modeling the spandrel behavior, but it does not reflect typical boundary conditions in spandrels. Once the spandrel cracks, it tends to elongate. However, the elongation of the spandrels is typically restrained by the piers. The restraint on spandrels of inner bays will be larger than on piers of outer bays. In addition, the restraint depends on the floor system, the story within the building, and the presence or absence of steel ties. The axial load can be estimated from finite-element analysis only if the model is capable of predicting the axial elongation of the spandrel due to cracking. Elastic beam element models and most plastic hinge models are not capable of predicting the axial elongation caused by cracking of the spandrel.

The contribution of the lintels to the peak flexural capacity of URM spandrels can be ignored. Lintels do not make a significant contribution to the peak shear capacity of URM spandrels and can be ignored. Lintels do not often make a significant contribution to the residual flexural capacity of URM spandrels and can be ignored. When no lintel is present, the residual shear capacity of URM spandrels shall be zero. To be considered as providing part of the strength and stiffness of the spandrels, lintels must be shown to be capable of sustaining the applied axial stress; otherwise, lintels must be neglected. Once shear cracking has occurred, the URM spandrel can no longer transfer in-plane shear demands. When present, lintels acting as beams (simply supported at one end and fixed at the other) can transfer the vertical component of the spandrel load, *F*, to the adjacent pier.

C11.3.2.2.7 Expected Strengths of URM Wall Spandrels with Shallow Arches Subject to In-Plane Actions The axial stress in the spandrel should be estimated in accordance with the previous section.

Equation (11-25) is the peak shear strength associated with the formation of cracks through the head and bed joints over almost the entire height of the spandrel and should apply when the mortar is weaker than the masonry units. For the case when the mortar is stronger than the masonry units and fracture of the masonry units will occur, Equation (11-26) should apply.

Once shear cracking has occurred, the URM spandrel itself can no longer transfer in-plane shear demands (refer to Figure 11-4b).

If the arch does not qualify as shallow, equations in Beyer and Mangalathu (2014) can be used to compute the strength of the spandrel.

C11.3.2.3 Acceptance Criteria for URM In-Plane Actions The sequence of in-plane actions is difficult to model reliably, particularly when actions have similar strengths or when combinations of actions can occur in one or more piers. Bidirectional effects are also difficult to quantify reliably. The most commonly observed seismic threat posed by URM walls is falling material caused by in-plane shear damage or out-of-plane collapse caused by instability. Stiffness degradation caused by in-plane shear failures adds to the probability of out-of-plane instability of the URM walls. Typically, out-of-plane failures initiate earlier than failures caused by in-plane actions.

C11.3.2.3.1 Linear Procedures for In-Plane URM Wall Actions m-Factors in Table 11-3 are generally based on response characteristics of wall subassemblages with lower-bound bed-joint shear strengths greater than or equal to 30 lb/in.^{2} (206.8 kPa). Walls with lower-strength mortars may exhibit less integrity and potentially different response characteristics and m-factors than are given in Table 11-3.

Rocking. The revisions to Table 11-3 compared with Table 7-3 of ASCE 41-06 are based on test results of individual URM piers that had rocking as primary modes of response and had sufficient information to estimate yield drifts, maximum tested drifts, and axial stress ratios. The maximum m-factors are based on approximately 0.75 times the ratios of maximum tested drift to observed yield drift, and they account for pier aspect ratios. The maximum m-factors are a proxy for limiting allowable drifts of rocking piers. Test results consistently indicate that m-factors are reduced with increased wall and pier axial forces. The m-factors in ASCE 41-06 were generally based on lightly axially loaded piers. The m-factors for primary elements remain the same as those in Table 7-3 of ASCE 41-06, but a new restriction has been added to cap the axial stress ratios in rocking walls and piers because test results indicate that the m-factors generated from test results on piers that have stress ratios beyond this cap are less than the tabulated values. The m-factors for secondary elements have been reduced from the values in Table 7-3 of ASCE 41-06 to correlate with test results and the axial stress ratio limit (Xu and Abrams 1992, Magenes and Calvi 1995, Anthoine et al. 1995, Costley and Abrams 1996, Franklin et al. 2001, Paquette and Bruneau 2003, Moon et al. 2006).

For guidance on evaluating the adequacy of solid bonded headers in multiwythe solid brick rocking walls and wall piers, see Section C11.3.2.1.

Sliding. The use of V_{bjs1} provides reasonable estimates of the deformation capacities of walls and wall piers undergoing sliding action when using linear procedures. Strengths eventually reduce to residual bed-joint sliding strengths, V_{bjs2}, after experiencing relatively large deformations, generally well beyond the limits imposed by linear procedures.

Redistribution of forces is an application of load sharing, as defined by Chapter 1, and can allow better utilization of the total strength of a line of resistance. This behavior is explicitly captured in nonlinear procedures and is approximated in the linear procedures by allowing limited redistribution of forces between wall piers on deformation-controlled lines of resistance. This approach is similar to that permitted by other guidelines for linear analysis of deformation-controlled lines of resistance with URM wall piers (NZSEE et al. 2017). Because Chapter 11 also has provisions for Collapse Prevention and secondary components, the maximum redistribution is capped at 20% and limited to primary components only. The 20% limit is based on case studies of archetypical wall pier configurations that indicate a reasonable benefit to this level of redistribution, while also noting that the standard permits the user to consider using secondary acceptance criteria or nonlinear procedures. The 15% limit for Life Safety is 0.75 times the Collapse Prevention value. The 0% limit for Immediate Occupancy is provided to allow users to interpolate for Damage Control. Alternatively, the user is permitted to consider whether components can be classified as secondary, in accordance with Chapters 7 and 11, and evaluated accordingly. The diaphragm, collector, and connection evaluation should be based on the forces that are required to be transferred to each wall pier, including effects of redistribution. Spandrels are required to be evaluated by Section 11.3.2.2.6, including consideration of any applied axial stresses.

For an individual line of resistance where redistribution is permitted to be applied, the provisions can be expressed algebraically using Equations (C11-10) and (C11-11) as follows.

For the wall line:

$$\sum_{i}^{n} V_{i,initial} = \sum_{i}^{n} V_{i,redistributed} \tag{C11-10}$$

For any individual wall pier in the wall line:

$$(V_{i,redistributed} - V_{i,initial})/V_{i,initial} \le R_{dist,max} \tag{C11-11}$$

where

$V_{i,initial}$ = Calculated shear force prior to redistribution in wall pier, i;

$V_{i,redistributed}$ = Shear force after redistribution in wall pier, i;

n = Total number of wall piers in the line of resistance; and

$R_{dist,max}$ = 20% for Collapse Prevention, 15% for Life Safety, and 0% for Immediate Occupancy.

C11.3.2.3.2 Nonlinear Procedures for In-Plane URM Wall Actions Nonlinear deformation capacities are generally based on response characteristics of wall subassemblages with lower-bound bed-joint shear strengths greater than or equal to 30 lb/in.^{2} (206.8 kPa). Walls with lower-strength mortars may exhibit less integrity and potentially different response characteristics, including different limiting behavior modes and acceptance criteria than given in Table 11-4.

Where the nonlinear static procedure (NSP) is used to analyze in-plane URM wall actions for three-dimensional building models with flexible diaphragms, consideration must be given to appropriate horizontal distributions of the static point loads at each floor. In such cases, the distribution of static point loads at each floor should be an approximation of the expected horizontal distribution of seismic inertial forces. For URM buildings with flexible diaphragms, see Section C7.2.11 and Equation (C7-1) for the distribution of inertial forces in the diaphragm.

Rocking. The revision to Figure C11-7 from Figure 7-4 of ASCE 41-06 is intended to provide the user of this standard with a generalized force–deformation relationship that is consistent with the engineering mechanics of a rocking system and test results of individual rocking URM piers.

The nonlinear response of rocking URM piers is generally characterized by a negative postyield slope caused by P-Δ effects and eventual toe crushing as the effective bearing area at the toe

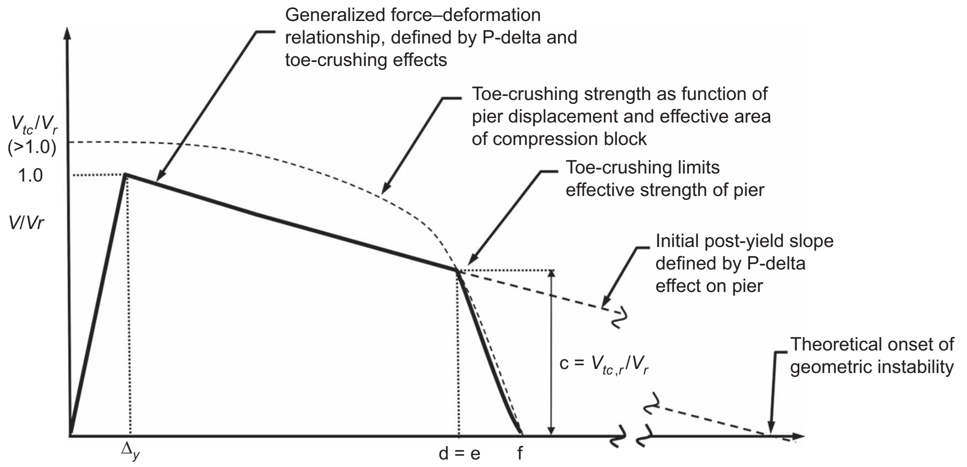


Figure C11-7. Generalized force–deformation relationship for rocking URM walls or wall piers.

of the rocking pier reduces to zero under increasing lateral displacement. This latent toe crushing differs from that specified in Section 11.3.2.2.3 because it typically occurs at larger rotations and lower shears. The lower (simplified evaluation) nonlinear rocking acceptance criteria in Table 11-4 are permitted to be used in lieu of a detailed moment-curvature analysis of each wall pier based on engineering mechanics, including nonlinear stress–strain response of constituent materials and boundary conditions. The deformation associated with the onset of toe crushing shall either be explicitly considered in the nonlinear analysis or established and checked (e.g., using expected deformation demands obtained from the analysis) using a moment-curvature or similar analytical approach. This approach also permits the use of alternative (comprehensive evaluation) acceptance criteria in accordance with the footnotes of Table 11-4 (moment-curvature analysis). Under rare conditions, geometric stability of the rocking pier caused by P-delta effects may govern the ultimate deformation capacity. The rocking systems exhibit very low levels of hysteretic damping. In the absence of substantiating test results, elastic unloading hysteretic characteristics shall be assumed for rocking URM in-plane walls and wall piers.

The revisions to the rocking modeling and acceptance criteria in Table 11-4 from Table 7-4 of ASCE 41-06 provides alignment with Figure C11-7. Furthermore, upper-bound limits on drift have been added based on test results of individual URM piers that had rocking as primary modes of response (Xu and Abrams 1992, Anthoine et al. 1995, Magenes and Calvi 1995, Costley and Abrams 1996, Franklin et al. 2001, Paquette and Bruneau 2003, Yi 2004, Moon et al. 2006). The test results indicate that for URM walls governed by an initial rocking response, drifts of at least 1.5% are sustainable for certain configurations of aspect ratio and axial load, with nominal strength degradation, provided that toe crushing is not found to control at lower drifts. For drifts greater than 1.5%, out-of-plane effects (e.g., twisting of piers at their bases) can influence wall performance. Users of this standard are cautioned as to the increased fragility of rocking piers subjected to the drift criteria for secondary elements, which are recommended only for use with piers with a minimum thickness of 12 in. (305 mm) to minimize the risk of bearing loss caused by out-of-plane effects.

For guidance on evaluating the adequacy of solid bonded headers in multiwythe (multileaf) solid brick rocking walls and wall piers, see Section C11.3.2.1.

Sliding. Research results indicate that secondary component deformation limits for Life Safety and Collapse Prevention can be increased (Magenes and Calvi 1992, Manzouri 1995, Russell and Ingham (2010). Moon et al. (2006) recommend a representative force–deformation curve, as shown in Figure C11-8. As bed joints slide, there is a gradual increase in axial stress and corresponding reduction in axial strength as the amount of wall or wall pier in bearing decreases. In the case of sliding, localized loss of bearing, particularly at the spring lines of arched lintels or at header courses at the ends of piers, occurs. Several test results are available out to 1% drift, but there are only two tests beyond that. Users should consider the layup of masonry at header courses, steel or concrete lintel bearing lengths, and spring lines of masonry arches when determining the potential for loss of vertical-load-carrying capacity. The one-half masonry unit width limit for Point f is based on judgment because no available tests currently extend to that drift level. Other values for limiting loss of vertical-load-carrying capacity may be appropriate, depending on the specific layup of each wall, pier, or lintel. See the plot of the reduction of vertical-load-carrying capacity versus demand in Figure C11-8. Vertical-load-carrying capacity beyond Point e is expected; however, very limited, unidirectional test data are available beyond Point e. No bidirectional tests are currently available that account for the potentially earlier loss of vertical load-carrying capacity before Point f that could be caused by out-of-plane actions compromising in-plane actions.

Pier heights, *h*, in Table 11-4 for sliding can be assumed to be consistent with effective heights for rocking, in accordance with Figure C11-5.

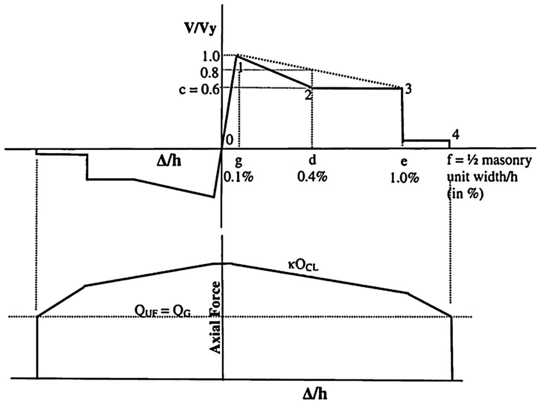


Figure C11-8. Bed-joint sliding action in URM walls.

Spandrels. Recommendations for the definition of force–rotation relationships of masonry spandrels were derived from experimental tests in Beyer (2013) and Knox (2012) and from numerical studies in Beyer and Mangalathu (2014) and condensed by Cattari and Beyer (2015).

C11.3.3.2 Strength of URM Walls Subject to Out-of-Plane Actions In situ tests have shown that timber diaphragms provide no arching action. Veneers, outer wythes of cavity walls, and wythes without adequate header courses or with effective collar-joint void ratios greater than 50% should not be considered as part of the effective thickness of URM walls for out-of-plane actions. However, for in-plane resistance, effective thickness can include the sum of all wythes, without necessarily considering the condition of the collar joints.

C11.3.3.3 Acceptance Criteria for URM Walls Subject to Out-of-Plane Actions For further information on evaluating the stability of URM walls out of plane, refer to *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Revised* (ABK 1984).

The suggested slenderness ratios assume that existing wall-to-diaphragm connections are sufficient to carry inertial forces from the wall into the diaphragm. Timber diaphragms shall be strengthened so that they can carry the forces transferred through connections to out-of-plane loaded walls. Wall-to-diaphragm connections are essential to achieve out-of-plane stability because walls that have inadequate connections to top diaphragms respond in cantilever modes and are much less stable. Research by Lam et al. (1995) and Doherty (2000) has shown that the seismic behavior of freestanding URM walls is analogous to that of a four times more slender simply supported URM wall.

More recent research indicated that *h/t* ratios in Table 11-5 of ASCE 41 (2017) may be conservative for undamaged URM walls responding to non-near-source ground motions with S_{a1} less than $0.45g$ (Simsir et al. 2004, Sharif et al. 2007, Dizhur et al. 2010, Derakhshan 2011). However, research on the influence of near-source ground motions with long pulses on out-of-plane actions (Derakhshan 2011) suggests that *h/t* ratios in high seismic regions can be unconservative. Research has also suggested that the behavior of walls that have the same slenderness ratio but different thickness is different (Sorrentino et al. 2008, Derakhshan 2011). Derakhshan (2011) suggests that of walls having the same slenderness ratio, thicker walls are generally more stable. These findings suggest that future research should be directed to study out-of-plane wall behavior by considering wall thickness.

Sorrentino et al. (2008) and Derakhshan (2011) have suggested that crack height substantially influences wall stability and that analytical models should consider an appropriate crack height. In addition, research to date has not captured all significant variables influencing the performance of out-of-plane URM walls. For example, research is currently under way to address the influence of in-plane demands on out-of-plane actions, overburden eccentricities, and dynamic characteristics of diaphragms (Penner and Elwood 2011).

Research to date has also focused on Collapse Prevention, so the margin between Life Safety and collapse is poorly understood because it has not been explicitly quantified. Localized loss of masonry units may still occur for URM walls that meet these criteria, potentially resulting in falling hazards that can cause serious injury.

Analytical studies that have attempted to capture the response of out-of-plane actions suggest that rigid-body rocking models that account for impact-based collision and restitution can be more reliable than oscillator-based models or displacement-based models (Doherty et al. 2000, Griffith et al. 2003, Lam et al. 2003, Makris and Konstantinidis 2003, Sharif et al. 2007).

ASCE 41 (2017) and earlier editions evaluated out-of-plane actions by *h/t* ratios based on research by ABK (Agbabian et al. 1981). However, out-of-plane evaluations have been revised in this edition using the assessment procedure from Penner and Elwood (2016), which was added in ASCE 41 (2017) for the Life Safety Structural Performance Level only. The procedure has been expanded to include factors for various structural performance levels. For these reasons, the evaluation results from this edition of the standard may differ from those obtained with ASCE 41 (2017).

The limit on S_{X1} in Equation (11-28a) is based on a proposed assessment procedure by Penner and Elwood (2016). Dynamic wall stability depends on the stiffnesses of the connected diaphragms and is governed by the more flexible of the two connected diaphragms at a given story. The assessment procedure was derived to provide a consistent probability of collapse based on results from a parametric study of URM wall dynamic stability. The rigid-body rocking model used in the parametric study was calibrated to shake table collapse tests of six full-scale three-wythe walls in one-way bending. S_{a1} was found to be the best indicator of collapse potential regardless of diaphragm period. For this reason, S_{X1} in Equation (11-28a) is permitted to be based solely on the spectral response acceleration at 1 s and need not consider the additional requirements per Section 21.4 of ASCE 7, as referenced via Section 2.3 of this standard. Penner and Elwood (2016) assumed 5% damping. Equation (11-28b) for flexible diaphragms has been adjusted from the Penner and Elwood model to reflect the higher damping levels allowed in ASCE 41 for wood diaphragms (see Section 7.2.4.6). In Equation (11-28b), the 1.8 value is a 20% increase over Penner and Elwood's (2016) value of 1.5 based on the difference in damping assumptions and the corresponding ratio of response for the suite of ground motions used in that study (Penner 2014). This 20% increase equates to approximately 10% damping using Equation (2-3).

One item not considered by the Penner and Elwood method is the influence of cross walls. However, the ABK method had included the cross wall effects and cross walls limit amplification of ground motions in higher seismic hazard zones (Kariotis et al. 1985). As noted in the ABK methodology (ABK 1984), the cross walls act as an inelastic damper for flexible, wood floor systems. Bruneau (1994) noted that cross walls reduced demands by a factor of approximately 1.7. Therefore, to account for the increased damping due to cross walls, a total damping of 20% was conservatively assumed, and the associated damping factor of approximately 1.5 was calculated using Equation (2-3). The 1.5 for 20% damping was divided by the 1.2 increase from the base 10% damping to achieve the cross wall factor of 1.25. This damping increase is only possible for cross walls that occur perpendicular to the masonry walls under consideration and when the diaphragm is constructed of wood. Concrete-framed diaphragms, steel deck with structural concrete fill, and bare steel deck diaphragms were specifically excluded from taking advantage of the cross walls in the ABK methodology (ABK 1984).

Probabilities of collapse achieved for different values of C_{pl} and diaphragm stiffnesses are summarized in Table C11-1 based on Penner and Elwood (2016). Values can be interpolated between the table's values for diaphragm stiffness between stiff and flexible. The 5% probability of collapse was chosen for the Life Safety Performance Level to greatly limit the chance of collapse. Per Penner and Elwood (2016), the 10% probability of collapse is "deemed to be a reasonable risk level for default high-risk conditions" and was chosen for Limited Safety. The 50% probably of collapse level was deemed too high a risk and was not used; Collapse Prevention used the 20% probability instead.

Table C11-1. Relation between Modification Factor C_{pl} and Probability of Collapse.

Probability of Out-of-Plane Collapse of URM Wall (%)	C_{pl} Factor	
	Stiff Diaphragms	Flexible Diaphragms
5	0.90	0.90
10	1.00	1.00
20	1.15	1.10
50	1.50	1.25

For the Damage Control value, per Section 2.2.1, it is between Immediate Occupancy (where flexural cracking out-of-plane is not permitted) and Life Safety and the acceptance criteria are halfway between the two. Therefore an approximate 2.5% probability of collapse would be appropriate, but this value was not calculated in the original research. The value of C_{pl} was therefore extrapolated downward; the relationship between C_{pl} and probability of collapse is not linear, but following the nonlinear pattern, halving the probability of collapse results in a step down of 0.1 in the C_{pl}, leading to a value of 0.8.

Penner and Elwood (2016) include no limits on *h*/*t* ratios, but some limits may be warranted based on empirical evidence. Empirical data do not exist for *h*/*t* values greater than 20. Observations of the acceptable performance of retrofitted URM walls with vertical wall bracing or intermediate wall bracing where *h*/*t* ratios are less than 8 in damaging earthquakes with moderate to strong shaking of short durations suggest that 8 is a reasonable lower limit for *h*/*t*. Research has not been conducted to determine the effectiveness, required stiffness, and deformation compatibility of wall bracing using vertical bracing members or intermediate wall bracing, but observations from past earthquakes suggest that the latter is less reliable than the former.

Equation (11-27) provides an estimate of the ground motion intensity causing collapse of a URM wall adequately attached to the floor diaphragms but without vertical bracing members (strong backs). If the ground motion intensity is exceeded, then vertical bracing members (or similar) should be provided. Spacing of vertical bracing members should be based on ensuring that the vertical bracing can support the inertial forces generated by the out-of-plane URM wall mass. *h*/*t* limits based on Equation (11-27) should not be used to determine vertical bracing member spacing.

The out-of-plane behavior of URM walls is very complex. A study of past tests, new shake table tests, and more than 220,000 parametric runs form the basis of these provisions (Penner 2014). These provisions account for diaphragm flexibility, wall overburden pressure, thin walls, cross walls, slenderness ratio, and performance level. However, there are additional variables that can also affect the behavior that have not yet been incorporated. Consideration of possible increases in the probabilities of collapse due to strength and stiffness degradation caused by in-plane actions in URM walls were not included in this research. In addition, the influence of velocity, rather than acceleration, on the wall movement has not been included, although velocity had been the primary parameter in the ABK method. The provision neglects the expected variations of diaphragm displacements and diaphragm response along the spans of the diaphragms, two-way bending action and wall geometry, arching action of walls, interactions with intersecting walls or partitions, and diaphragms present on both sides of the wall, all of which are expected to reduce the probabilities of collapse but to an unquantified degree. For example, the out-of-plane wall displacements and demands will be less away from the midspan of the diaphragm, thus reducing response of the wall, which would result in less out-of-plane rocking, allowing for potentially higher *h*/*t* ratios. [Australian Standard for design of unreinforced masonry buildings (AS 3700) (Standards Australia 2018) provides guidance on consideration of two-way bending, which may be of value to the assessment of walls bounded on three or more sides.] The provision also neglects amplifications of response up the building, but these amplifications are expected to primarily increase short-period spectral accelerations rather than S_{a1}. The effects of varying mortar or masonry unit strengths on collapse probability were beyond the scope of the research. The computer modeling accounted for moderate amounts of spalling at horizontal cracks that form before collapse. The effects of vertical acceleration were not included in this research, but it is expected that its influence on collapse probability is secondary because of the high-frequency, short-period nature of this effect.

Although mortar and masonry unit strengths are primary considerations for anchorage performance, other research (Meisl et al. 2007, Lumantarna 2012) suggests that mortar quality and the presence or absence of collar-joint mortar may have little effect on out-of-plane response as long as connection to diaphragms is maintained. However, failures of URM walls in past earthquakes suggest a strong correlation with lack of collar-joint mortar, low mortar strength, and masonry unit strength, or all three (Deppe 1988, Schmid 1994). Walls with lower strengths may exhibit less integrity and potentially different response characteristics; hence, dynamic stability of cracked walls is not considered reliable for mortar shear strengths less than 30 lb/in.^{2} (206.8 kPa).

Veneers are not explicitly addressed by the acceptance criteria; however, Section 11.3.3.2 requires that the veneer thickness in a cavity wall not be considered part of the effective wall thickness for out-of-plane strength. The presence of a veneer connected via adequate ties to a backing URM wall will tend to increase the effective out-of-plane demand on the backing wall in proportion to the ratio of total mass of the veneer plus backing wall, to the mass of the backing wall only.

C11.3.4 Reinforced Masonry Walls and Wall Piers In-Plane Actions In this standard, RM shear walls are considered to be either flexure governed or shear governed. Flexure-governed walls have the failure mechanism governed by masonry crushing, and the yielding, buckling, and possible fracture of the vertical reinforcement in the wall, whereas the failure of shear-governed walls is associated with diagonal cracking, and the yielding and possible fracture of the horizonal reinforcement. Flexure-governed walls normally exhibit more ductile behavior than shear-governed walls. Hence, this distinction has to be made in assessing the seismic performance of a wall. The shear strength equations given in TMS 402 for reinforced masonry are based on research by Shing et al. (1990a, b) on reinforced masonry walls constructed with hollow concrete masonry units and hollow clay masonry units. There is insufficient research to justify the same strength equations for walls constructed of solid units with a reinforced grout space between wythes without bed-joint reinforcing. The bond strength between the grout and the inside face of the brick may not be adequate to transfer the stresses between the reinforcing and the net section of the brick.

Shear sliding can also be a governing mechanism, especially for squat walls. Even though this is not addressed in this standard, it is prudent to check if the behavior of a wall could be governed by shear sliding and determine its actual contribution to the lateral resistance of a wall system. 2016 and later editions of TMS 402 have shear-friction provisions for calculating the shear sliding strength of an RM wall.

The criterion specified in this section to determine the governing mechanism takes into account the higher uncertainties and lower conservatism in determining the shear strength of an RM wall. When the expected shear strength is greater than 1.4 times the shear required to develop the expected flexural strength, the possibility of shear-governed behavior can be ruled out. The 1.4 factor is based on the product of the ratio of the actual to the calculated flexural strength, which is assumed to 1.2, and an uncertainty penalty factor of 1.125, which is assumed to be equal to the ratio of the respective ϕ factors in TMS 402. Furthermore, it is important that the effects of the coupling forces from horizontal diaphragms and masonry beams above openings on the lateral resistance of shear walls associated with the flexural mechanism be considered so that the flexural resistance of the walls will not be underestimated in determining the governing mechanism.

The generalized force–deformation relations shown in Figure 11-5 are based on the recommendations in NIST GCR 17-917-45 (NIST 2017). For calculating the initial stiffness and critical drift ratios, reduction factors are applied to the flexural and shear stiffness values to account for masonry cracking. Previous provisions in this standard recommended a reduction factor of 0.5 for I_g, with nothing for the shear stiffness, which could significantly overestimate the wall stiffness (NIST 2017). The values of the reduction factors recommended here are based on experimental data as discussed in NIST (2017) and Cheng and Shing (2022).

The effective height, h_{eff}, of a wall component or pier depends on its boundary conditions. In a coupled or perforated wall system, a wall component can be subjected to single or double curvatures depending on the stiffness and strength of the horizontal coupling elements. To apply the provisions in Section 11.3.4, one may follow the guidelines provided in Figure C11-9 to determine the value of h_{eff} in lieu of the recommendations in Figure C11-1. The effective height is the distance between the wall section at which moment is zero and the section at which the maximum moment is developed. The moment diagram can be obtained with an elastic analysis of the wall system.

The deformation capacities specified in this standard for flexure- and shear-governed walls are based on numerical and experimental data on individual wall components subjected to constant axial loads. The deformation capacity of a building system can be much higher than that shown by a single wall subjected to a constant axial compressive load because of the presence of alternative load paths for gravity loads in a building system. Experimental data of Cheng et al. (2020) and the numerical study in FEMA (2020c) have shown that the story-drift ratio of shear-governed fully grouted wall systems can be higher than 5% before collapse.

Although the provisions presented in this section are applicable to both fully grouted and partially grouted walls, they have not been extensively validated with experimental data for partially grouted walls. Furthermore, they are based on research conducted on RM walls constructed of hollow units. Therefore, they should not be applied to reinforced clay brick cavity walls, which may have very different behavior, unless justified by experimental data.

C11.3.4.3 Flexure-Governed In-Plane Actions of Reinforced Masonry Walls and Wall Piers RM walls can have rectangular or flanged sections. Flange action should be considered when the intersection between the flange and the web meets the conditions specified in Section 11.3.4.1, which are considered necessary for effective shear transfer.

A simple method is provided in this section to determine the moment capacities and construct the lateral force–drift ratio curves for flexure-governed walls with rectangular and flanged sections using values of nondimensionalized parameters calculated according to Table 11-6. Equations presented in the table are based on numerical data derived from fiber-section models that consider the crushing and cracking of masonry, and the yielding, buckling, and possible fracture of the reinforcement. The modeling method is presented in NIST GCR 17-917-45 (NIST 2017) considering fully grouted rectangular wall sections. The method has been extended to flanged walls sections by representing these sections with equivalent rectangular sections in an approximate manner, and to partially grouted wall sections, as discussed in Cheng and Shing (2022). The accuracy of the simplified analysis method was found to be satisfactory when compared to quasi-static test data on fully grouted walls with rectangular sections. As to flanged walls and partially grouted walls, experimental data available for validation are limited.

C11.3.4.4 Shear-Governed In-Plane Actions of Reinforced Masonry Walls and Wall Piers The lateral drift capacities specified in this section for shear-governed walls were empirically determined from wall test data as discussed in in NIST GCR 17-917-45 (NIST 2017). The influence of the axial load and reinforcement contents are ignored. The drift ratio values for partially grouted walls are based on the data of Bolhassani et al. (2016).

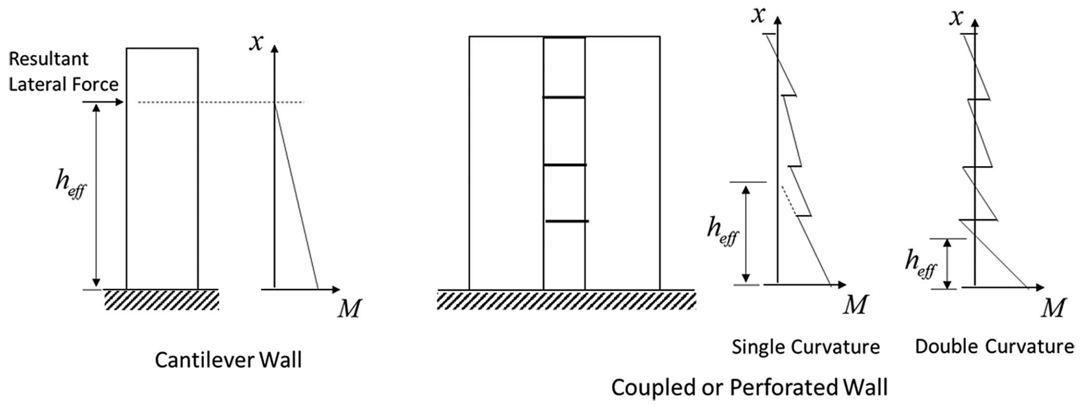


Figure C11-9. Effective height of RM wall components.

C11.3.4.6 Acceptance Criteria for In-Plane Actions of Reinforced Masonry Walls and Wall Piers

C11.3.4.6.1 Linear Procedures for In-Plane Actions of Reinforced Masonry Walls The *m*-factors for flexure-governed RM walls are based on the curvature ductility of a wall section with the assertion that the displacement ductility of a wall is proportional to the curvature ductility. The ratio $M_{\max}/(E_m I_g)$ in the equations in Table 11-8 represents the effective yield curvature used to define the ductility. The upper limits on the *m*-factors in Table 11-8 are based on the consideration that its value for the Life Safety (LS) level evaluation of a primary wall component should be slightly below the value of the *R* factor used for the design of new special load bearing RM walls according to ASCE 7.

The axial compressive stress limit for wall components governed by shear to be considered as deformation controlled is raised from 0.15 f_{me} to 0.3 f_{me} when the actual height-to-length ratio of the wall is less than or equal to 0.50. This is in consideration of the fact that sliding failure along diagonal shear cracks is unlikely for squat walls. Furthermore, in squat walls, the axial compressive stress could improve the aggregation-interlock resistance along diagonal and horizontal cracks as long as it is not too high to cause crushing.

The limit of $(3\sigma_a + \omega) > 0.6$ for wall components governed by flexure to be considered as deformation controlled is based on the consideration that a wall that has $(3\sigma_a + \omega) > 0.6$ will have a behavior that is in between that of an intermediate wall and an ordinary wall defined according to TMS 402.

C11.3.5 Reinforced Masonry Wall Out-of-Plane Actions

C11.3.5.3 Acceptance Criteria for Reinforced Masonry Wall Out-of-Plane Actions The limit states specified in this section are based on the masonry units that have significant cracking for Immediate Occupancy (IO), masonry units at a point of being dislodged and falling out of the wall for Life Safety (LS), and masonry units on the verge of collapse for Collapse Prevention (CP).

C11.4 MASONRY INFILLS

The design professional is referred to TMS 402, TMS 602, Angel et al. (1994), FEMA 306 (1998a), FEMA 307 (1998b), FEMA 308 (1998e), Flanagan and Bennett (1999), Stavridis (2009), Mosalam and Günay (2015), and Bose et. al (2016) for additional information regarding the engineering properties of masonry infills.

C11.4.1 Types of Masonry Infills

C11.4.1.1 Existing Masonry Infills It is well known that for undamaged infill panels, the arching action provides significant resistance to the out-of-plane forces. This action decreases when the infill is damaged because of in-plane forces.

C11.4.1.3 Retrofitted Masonry Infills Masonry infills may be retrofitted using the methods described in this section. Masonry infills enhanced in accordance with this section should be analyzed using the same procedures and performance criteria used for new infills.

Unless stated otherwise, methods applicable to unreinforced infills are intended to improve performance of masonry infills subjected to in-plane and/or out-of-plane forces.

Guidelines from the following sections pertaining to retrofit methods for reinforced masonry walls listed in Section C11.3.1.3 may also apply to URM infill panels: (1) infilled openings, (2) shotcrete, (3) coatings and near-surface-mounted reinforcement for URM walls, (4) grout injections, (5) repointing, and (6) stiffening elements. In addition, the following two retrofit methods may apply to masonry infill panels.

Boundary Restraints for Infill Panels. Infill panels not in tight contact with perimeter frame members cannot develop arching action and should be restrained for out-of-plane forces. This goal may be accomplished by installing structural steel angles or plates on each side of the infills and welding or bolting the angles or plates to the perimeter frame members.

Filling Gaps between Infill Panels and Bounding Frames. Gaps between an infill panel and the surrounding frame may be filled if integral infill–frame action is assumed for in-plane response. Testing of material used to fill gaps is recommended to document compressive stiffness, bonding to infill and frame, and fire resistance.

C11.4.2 Masonry Infill In-Plane Actions

Finite-element modeling schemes and calibration procedures have been proposed by Atkinson et al. (1989), Chiou et al. (1999), Al Chaar (2002), Al Chaar et al. (2003), Stavridis (2009), and Stavridis and Shing (2010), among others. Design professionals should note that the results of such models can be significantly affected by the selected strut locations, widths, strengths, and orientations. Therefore, a number of different configurations should be considered to ensure the objectivity of the model. A good practice is to adjust the properties of the equivalent struts so that the models can capture the likely range of the combined structural behavior of the infill and the bounding frame.

C11.4.2.1 Stiffness: Masonry Infill In-Plane Actions In-plane lateral stiffness of an infilled frame system is not the same as the sum of the frame and infill stiffnesses because of the interaction of the infill with the surrounding frame. Experiments have shown that, under seismic forces, the frame tends to separate from the infill at small lateral deformations. This separation causes the reduction of the lateral stiffness, which onsets the nonlinear behavior of the structure at Point 1 of Figure 11-1. The seismic force at this point has been noted to be up to 60% of the peak strength.

The infill panel is often considered to act as a diagonal compression strut. The location and orientation of the strut cannot be clearly defined, and different geometries have been proposed with struts along the diagonal of the frame located concentrically (Figure C11-10), eccentrically (Figure C11-11), at an angle of 45 degrees (Figure C11-12), or with a combination of struts to account for openings (Figure C11-13) in perforated infills. Because theoretical work and experimental data for determining the properties and placement of multiple struts are not sufficient to establish reliable guidelines for all possible infill configurations, the selection of the strut locations, widths, strengths, and orientations requires judgment on a case-by-case basis. The design professional should be aware that if analytical models with frame elements are constructed to simulate the behavior of infilled frames under seismic forces, the results can be significantly affected by the selected strut locations.

C11.4.2.2 Stiffness: Masonry Infill with Openings In-Plane Actions Experiments have shown that, under seismic forces, two sets of cracks develop at small lateral deformations and initiate the nonlinear behavior. The first set of cracks is along the frame–infill boundary, and the second set consists of cracks that initiate at the corners of openings and radiate in the infill at an angle close to 45 degrees. The stress field is clearly affected by the existence of the openings; however, the exact load transfer mechanism is still unknown. A possible representation of these stress fields with multiple compression struts, as shown in Figure C11-13, has been proposed by Hamburger (1993). Theoretical work and

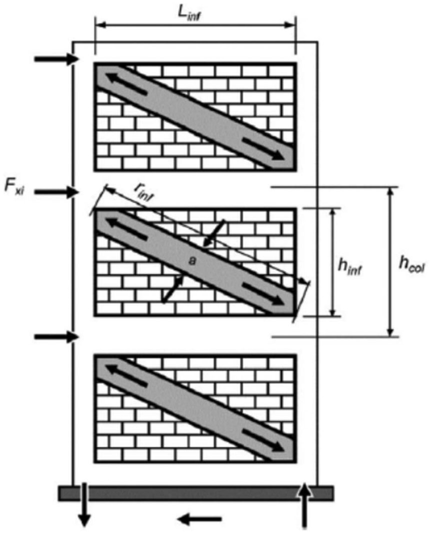


Figure C11-10. Compression strut analogy: concentric struts.

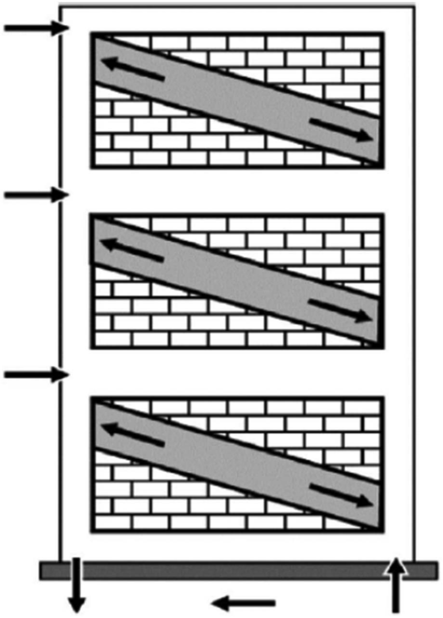


Figure C11-11. Compression strut analogy: eccentric struts.

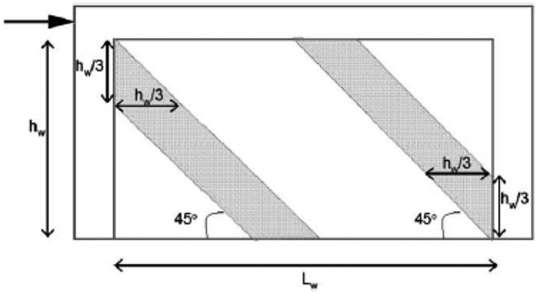


Figure C11-12. Compression strut analogy: Struts at 45 degrees acting at the top of the left (windward) column and the bottom of the right (leeward) column.

Source: Stavridis (2009), reproduced with permission.

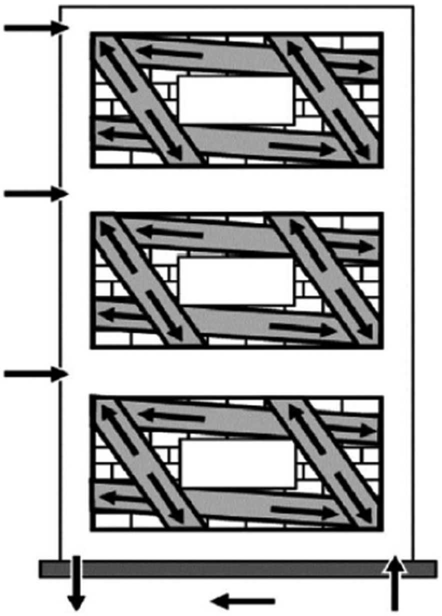


Figure C11-13. Compression strut analogy: Perforated infills.

experimental data for determining multiple strut placement and strut properties, however, are not sufficient to establish reliable guidelines.

C11.4.2.3 Strength: Infilled Reinforced Concrete Frames In-Plane Actions The load transfer and failure mechanisms of infilled frames depend on the relative strengths and stiffnesses of the infill and the surrounding frame. The classification of infilled concrete frames has been based on the parametric studies of Stavridis (2009) and Reese (2013), who investigated reinforced concrete frames with infills with height-to-length ratios between 0.33 and 2.85.The compressive force in the infill can be estimated assuming the development of one diagonal strut for *h*/*l* aspect ratios greater than 0.77 and two diagonal struts for smaller aspect ratios. In the latter case, the force is distributed between the diagonal struts along 45-degree angles that initiate near the top of the windward column and the bottom of the leeward column.

The bearing height of the strut on the columns and the bearing length of the strut acting on the beams can be assumed to be one-third of its height.

The bearing (compressive) strength of the infill for a bay with no openings in the infill is obtained from Equation (C11-12):

$$F_{mc} = f'_m \left(\frac{h_{inf}}{3}\right) t_{inf} \qquad \text{(C11-12)}$$

where

f'_m = Compressive strength of the masonry,
h_{inf} = Height of the infill wall, and
t_{inf} = Thickness of the infill wall.

The bearing strength of the infill can be considered as a cap for the force the infill can carry and shall be compared with the diagonal force carried by each strut. If the force is lower than the strength, the infill can transfer the estimated force. If the force is higher, the lateral resistance should be accordingly adjusted downward.

C11.4.2.4 Strength: Infilled Steel Frames In-Plane Actions The use of the provisions for ductile reinforced concrete frames for the estimation of the strength of continuous steel infilled frames with sufficiently strong connections is conservative as the steel frames tend to develop higher strength. However, this strength is developed at large lateral deformations at which the resistance of the infill may be compromised because of the in-plane cracks, which limit the effect of the arching action and can lead to out-of-plane collapses. If the inspection does not yield enough information on the steel frame continuity, the provisions for the nonductile reinforced concrete frames should be used.

The capacity of steel frames to effectively confine and develop full arching action in the infill wall panels may be limited by the capacity of the beam–column connections and column splices, if present. Such connections, including splices, are generally subjected to concurrent moments, shears, and axial load, which should be considered in the capacity evaluation. These connections are typically composed of riveted steel plates and angles, which cannot fully develop the section capacity of the connected steel beam or column sections. Section 11.4.2.4 requires that such limit states be considered in the evaluation of infill steel frames.

Any inspection should also determine whether the infill wall is concentric in plan with the centerline of the steel frame. Infill walls in steel frames are often eccentric, such that the infill is not fully confined by the frame, which tends to preclude full arching action from developing. In such instances, the arching action estimated from Section 11.4.2.3 should be reduced to account for the effective confinement.

The presence of gaps between infill walls and frames can significantly reduce the effective capacity (Dawe and Seah 1989). The presence of gaps should be verified by inspection of the infill frame.

C11.4.2.5 Drift: Infill Wall In-Plane Actions The envelope curve of the force-versus-deformation relation of an infilled frame can be defined considering the initial stiffness, shear forces, and corresponding drifts for the points of yield, peak, and onset of residual strength.

C11.4.2.6 Strut Model for Infill In-Plane Actions The width for the struts in all infilled bays shall be considered so that a model for the entire structure can be obtained. The width of the struts can be calibrated as described in Bose and Rai (2015) so that when added to the bare frame, they represent the envelope curve of the infilled frame. If compression-only struts are used in the model, the width of the strut in a bay with a solid infill panel α_{strut} can be obtained from Equation (C11-13):

$$\alpha_{\text{strut}} = \frac{K_{un}^{\text{solid}} - 2K_{\text{col}}}{d_w \cos^2\theta_{\text{strut}} E_m} L_{\text{strut}} \qquad \text{(C11-13)}$$

where

K_{un}^{solid} = Uncracked stiffness of the infilled frame obtained from Equation (11-28),
K_{col} = Flexural stiffness of each frame column,
d_w = Thickness of the masonry infill wall,
θ_{strut} = Angle of the strut with respect to the horizontal,
L_{strut} = Length of the strut, and
E_m = Modulus of elasticity of masonry.

If tension struts are used in the model, the strut width should be equal to that estimated from Equation (C11-13) divided by 2. In the case of an infill wall with an opening, K_{un}^{solid}, should be replaced with K_{un}^{op}, estimated from Equation (11-38).

For linear actions, the strut width estimated with Equation (C11-13) should be used when the total shear resistance of the infilled frame does not exceed the yield strength, V_y, determined from Equation (11-46).

C11.4.2.7 Acceptance Criteria for Infill Wall In-Plane Actions The Immediate Occupancy Structural Performance Level is assumed to be reached when significant visual cracking of an unreinforced masonry infill occurs. The Life Safety Structural Performance Level is assumed to be reached when substantial cracking of the masonry infill occurs. Collapse Prevention is assumed to be reached when the potential is high for the panel, or some portion of it, to drop out of the frame.

Failures in beams of infilled frames are not commonly observed because the beams are often protected by the confining effect caused by the in-plane stiffness of the upper-story masonry wall.

The residual strength, c, of infilled frames cannot be reliably estimated because of the brittle failure of infill systems. The in-plane cracks developing at drift ratio, d, can weaken the infill, increasing the potential for collapse because of the out-of-plane seismic forces. Hence, the residual strength should not be relied upon, and drift ratios d and e can be considered equal.

C11.4.3 Masonry Infill Wall Out-of-Plane Actions

C11.4.3.1 Stiffness: Infill Wall Out-of-Plane Actions Guidelines for estimating the thrusts on the frame components resulting from arching of an infill panel are detailed in Abrams et al. (1996). Frame beams and columns have not been observed to yield solely because of the effects of arching action of infill walls in past earthquakes, so a nonquantitative consideration of thrust resistance in Item 3 is generally adequate.

C11.4.3.2 Strength: Infill Wall Out-of-Plane Actions Equations (11-50) and (11-51) were developed by Abrams et al. (1996) and should be used to estimate the out-of-plane strength of an infill panel assuming arching in either direction (i.e., vertically or horizontally) provided that arching in either direction meets the criteria of Section 11.4.3.1. The stronger of the two directions should be assumed to govern, and the most flexible frame element in the direction of arching considered should be assumed to govern.

Equation (11-51) was developed by Mays et al. (1998) utilizing yield line theory and finite-element modeling for out-of-plane loads applied to concrete panels. Equation (11-51) has had limited experimental validation (Flanagan and Bennett 1999) and should be limited to panels with relatively small openings. Otherwise if large openings exist, assume no arching action and, if warranted, consider retrofitting by adding supplemental braces or other methods to enhance out-of-plane resistance.

C11.4.3.3 Strength: Infill Wall In-Plane and Out-of-Plane Interaction Equation (11-52) can be used as an acceptance criterion for the diagonal infill wall element in linear static procedure (LSP) or linear dynamic procedure (LDP). When using Equation (11-52), depending on the adopted modeling approach, attention should be paid to transforming the force and moment demands on the infill wall diagonal element to the indicated in-plane and out-of-plane strengths. The in-plane strength determined from Equation (11-53) can be used in NSP as the in-plane strength of the diagonal infill wall element after proper transformation. Equation (11-54) can be considered as a bending moment–axial force interaction diagram assigned to the diagonal infill wall element in an analysis conducted with nonlinear dynamic procedure (NDP). Similar to Equation (11-52), attention should be paid to transforming the force and moment demands on the infill wall diagonal element to the indicated in-plane and out-of-plane strengths in Equation (11-53).

Equations (11-52) through (11-54) are based on the experimental results of Flanagan and Bennett (1999) and the consecutive finite-element analyses of Hashemi and Mosalam (2007), the refinement introduced by the Kadysiewski and Mosalam model (2009), and the field evidence from recent earthquakes explained in Mosalam and Günay (2015). The value 1.5 in Equation (11-52) for reduction of the OOP force is derived from the out-of-plane infill wall tests reported in Walsh et al. (2015).

C11.5 ANCHORAGE TO MASONRY WALLS

C11.5.2 Analysis of Anchors Analysis provisions in Chapter 10, "Concrete," are considered appropriate for adhesive anchors in concrete masonry units because the failure modes and coefficients of variation in such masonry are similar to concrete. The International Code Council Evaluation Service's Acceptance Criteria AC 58 (2019) provides an example of modifications necessary for applying ACI 318 (2002) anchor provisions to concrete masonry units. The most current analysis and quality assurance approaches are summarized in AC 58 and, in principle, can be adapted to apply to anchors in reinforced and unreinforced masonry, in solid masonry, as well as ungrouted, partially grouted, and fully grouted masonry. Commentary on the analysis and design of anchors is also provided in FEMA P-750 (2009b).

C11.5.3 Quality Assurance for Anchors in Masonry Walls Poor quality in anchors and the existing masonry and mortar joints adjacent to anchors has been observed to be a significant contributing factor to catastrophic collapses and disproportionate damage of URM and RM buildings in past earthquakes (FEMA 2015d). Strict compliance with manufacturers' published installation instructions for proprietary anchors, specifications for generic anchors, and independent quality control by qualified inspectors are effective means of ensuring reliable performance of anchors.

Inspections for newly installed anchors should include verifying the locations of the anchors, any edge distance and spacing requirements, drill bit type and size, hole depth, hole cleaning technique, anchor type, size, embedment, and compliance with manufacturers' published installation procedures, including adhesive expiration date and dispensing, where applicable.

The quality assurance plan, testing procedures, and limits on the types of anchor installations should be developed considering that masonry walls are likely to be cracked before or during earthquakes and degrade, thus potentially compromising the integrity of load paths between the anchors and the walls.

Judgment should be exercised in the use of lower-bound material properties for anchors. Not all manufacturers of post-installed anchors publish information on the mean and the standard deviation of the ultimate anchor capacity. Older testing for existing postinstalled anchors is often reported at allowable stress design levels and may not be consistent with this standard. It is recommended that care and judgment be used to estimate pullout and shear strengths for anchors, particularly for those that are critical to satisfying the target performance level.

Guidance for developing quality assurance plans can be found in AC58 *Acceptance Criteria for Adhesive Anchors in Cracked and Uncracked Masonry Elements* (ICC-ES 2009), AC60 *Acceptance Criteria for Anchors in Unreinforced Masonry Elements* (ICC-ES), AC-10 *Acceptance Criteria for Quality Documentation* (ICC-ES 2009), ACI 355.2 *Qualification of Post-Installed Mechanical Anchors in Concrete* (ACI 2007b), ACI 355.4 *Qualification of Post-Installed Adhesive Anchors in Concrete* (ACI 2011a), ACI 318 *Building Code Requirements for Structural Concrete* (ACI 2002, 2014), AC308 *Acceptance Criteria for Post-Installed Adhesive Anchors in Concrete Elements* (ICC-ES 2016), AC193 *Acceptance Criteria for Mechanical Anchors in Concrete Elements* (ICC-ES 2015b), *Special Inspection Guidelines for Expansion and Adhesive Anchors* (CAMA 2011), *Adhesive Anchor Installer Certification Program* (ACI/CRSI 2017) and *International Existing Building Code*, Appendix Chapter A1 (ICC 2021b).

Testing provisions for adhesive concrete anchors in Chapter 10 are considered appropriate for adhesive anchors in concrete masonry units because the failure modes and coefficients of variation in such masonry are similar to concrete.

The exceptions that permit through bolts to be exempt from testing exist because through-bolt tension capacity is generally governed by the force-controlled punching shear capacity of the surrounding walls that can be estimated by calculations. The exceptions encourage the use of through bolts.

C11.6 MASONRY FOUNDATION ELEMENTS

C11.6.1 Types of Masonry Foundations Masonry foundations are common in older buildings and are still used for some modern construction. Such foundations may include footings and foundation walls constructed of stone, clay brick, or concrete block. In general, masonry footings are unreinforced; foundation walls may or may not be reinforced.

Spread footings transmit vertical column and wall loads to the soil by direct bearing. Seismic forces are transferred through friction between the soil and the masonry, as well as by passive pressure of the soil acting on the vertical face of the footing.

C11.6.3 Foundation Retrofit Measures Possible retrofit methods include the following:

1. Injection grouting of stone foundations;
2. Reinforcing of URM foundations;
3. Prestressing of masonry foundations;
4. Enlargement of footings by placement of reinforced shotcrete; and
5. Enlargement of footings with additional reinforced concrete sections.

Procedures for retrofit should follow provisions for enhancement of masonry walls where applicable, according to Sections 11.2.2.4 and 11.3.1.3.

C11.7 MASONRY DIAPHRAGMS

C11.7.1 General Masonry diaphragms are found in older steel buildings in conjunction with vertical systems of structural steel framing. The brick arches were typically covered with a very low strength concrete fill or a topping slab, usually unreinforced. In many instances, various masonry diaphragm systems were patented by contractors.

C11.7.2 Seismic Evaluation of Masonry Diaphragms Masonry diaphragms are typically a composite system of the steel framing, supporting a brick arch and concrete fill or a topping slab. The interaction of these components should be considered in the evaluation. The condition of the system, including integrity of the joints in the arched system, should be carefully investigated. Inelastic properties of masonry diaphragms should be chosen with caution for seismic analyses.

C11.7.3 Retrofit Measures for Masonry Diaphragms The following measures may be effective in retrofitting masonry diaphragms:

1. Adding diagonal members to form a horizontal truss,
2. Strengthening existing steel members by adding shear connectors to enhance composite action,
3. Removing weak concrete fill and replacing it with a structural concrete topping slab after verifying the effects of the added weight of concrete fill, and
4. Adding fiber-reinforced polymer (FRP) to strengthen the shear capacity of the masonry diaphragm and anchoring the FRP to the horizontal framing members.

CHAPTER C12
WOOD

C12.1 SCOPE

The Linear Static Procedure (LSP) presented in Chapter 7 is most often used for the analysis of wood buildings; however, properties of the idealized inelastic performance of various components and connections are included so that nonlinear procedures can be used if desired.

The evaluation and assessment of various structural components of wood buildings is found in Section 12.2. For a description and discussion of connections between the various components and elements, see Section 12.2.2.2.2. Properties of shear walls are described in Section 12.4, along with various retrofit or strengthening methods. Horizontal floor and roof diaphragms are discussed in Section 12.5, which also covers engineering properties and methods of upgrading or strengthening the elements. Wood foundations and pole structures are addressed in Section 12.6. For additional information regarding foundations, see Chapter 8.

As indicated in Chapter 1, great care should be exercised in selecting the appropriate retrofit approaches and techniques for application to historic buildings to preserve their unique characteristics.

C12.2 MATERIAL PROPERTIES AND CONDITION ASSESSMENT

C12.2.1 General Various grades and species of wood have been used in a cut dimension form, combined with other structural materials (e.g., steel and wood components), or in multiple layers of construction (e.g., glue-laminated wood components). Wood materials have also been manufactured into hardboard, wood structural panels [e.g., plywood, oriented strand board (OSB), and composite panels], waferboard, and particleboard products, which may have structural or nonstructural functions in construction. Early oriented strand board product was sometimes referred to regionally as waferboard, but these are different products. Caution should be used when classifying waferboard as OSB because waferboard does not have cross-aligned strands or similar structural properties and is typically weaker than rated OSB. Verification of the type of material used in a building can often be obtained by a label or markings indicating conformance to a given standard. Material properties should be verified with visual observations of markings on materials or by testing. The condition of the in-place wood materials greatly influences the future behavior of wood components in the building system.

Quantification of in-place material properties and verification of existing system configuration and condition are necessary to properly analyze the building. The focus of this effort shall be given to the primary components of vertical- and seismic-force-resisting systems. These primary components may be identified through initial analysis and application of loads to the building model.

The extent of in-place materials testing and condition assessment that must be accomplished is related to availability and accuracy of construction documents and as-built records, the quality of materials used and construction performed, and physical condition. A specific difficulty with wood construction is that structural wood components are often covered with other components, materials, or finishes; in addition, their behavior is influenced by past loading history. Knowledge of the properties and grades of material used in original component or connection fabrication is invaluable and may be effectively used to reduce the amount of in-place testing required. The design professional is encouraged to research and acquire all available records from the original construction, including design calculations.

Connection configuration also has a very important influence on response to applied loads and motions. A large number of connector types exist; the most prevalent are nails and through bolts. However, more recent construction has included metal straps and hangers, clip angles, and truss plates. An understanding of connector configuration and mechanical properties must be gained to properly analyze the anticipated performance of the building.

Wood construction has evolved over the years; wood is a common building material for residential and small commercial structures in the United States. It has often been used for the framing of roofs and floors and in combination with other materials. Establishing the age and recognizing the location of a building can be helpful in determining what types of seismic-force-resisting systems may be present.

Based on the approximate age of a building, various assumptions can be made about the design and features of construction. Older wood structures that predate building codes and standards usually do not have the types of elements considered essential for predictable seismic performance. In these conditions, new elements generally have to be added, or the existing elements have to be upgraded to obtain predictable performance.

If the age of a building is known, the code in effect at the time of construction and the general quality of the construction usual for the time can be helpful in evaluating an existing building. The level of maintenance of a building may be a useful guide in determining the structure's capacity to resist loads.

Users should be aware that wood material strengths presented in historical information are typically in allowable-stress format. Users should convert wood allowable stress values to expected-strength values in accordance with ASTM D5457.

The earliest wood buildings in the United States were built with post-and-beam or frame construction adopted from Europe and the British Isles. This method was followed by the development of balloon framing in about 1830 in the Midwest, which spread to the East Coast by the 1860s. This method, in turn, was followed by the development of western or platform framing shortly after the turn of the century. Platform framing is the system currently in use for multistory construction.

Gypsum wallboard (also known as drywall) was first introduced in about 1920; however, its use was not widespread until after World War II, when gypsum lath (button board) also came into extensive use as a replacement for wood lath.

With the exception of public schools in high seismic areas, modern wood structures detailed to resist seismic loads were generally not built before 1934. For most wood structures, either general seismic provisions were not provided or the codes that included them were not enforced until the mid-1950s or later, even in the most active seismic areas. This time frame varies somewhat, depending on local conditions and practice.

The design of buildings constructed after 1970 in high seismic areas usually included a well-defined seismic-force-resisting system. However, site inspections and code enforcement varied greatly. Thus, the inclusion of various features and details on the plans does not necessarily mean that they are in place or fully effective. Verification is needed to ensure that good construction practices were followed.

Until about 1950, wood residential buildings were frequently constructed on raised foundations and in some cases included a short stud wall, called a "cripple wall," between the foundation and the first-floor framing. Cripple wall conditions occur on both balloon-framed and platform-framed buildings. There may be an extra demand on these cripple walls because most interior partition walls do not continue to the foundation. Special attention is required in these situations. Adequate bracing must be provided for cripple walls and the attachment of the sill plate to the foundation.

C12.2.2.1.3 Nominal or Specified Properties Actions associated with wood shear wall assemblies generally are deformation controlled; thus, expected-strength material properties are used most often. Lower-bound values are used with components supporting discontinuous shear walls, bodies of connections, and axial compression of individual wood components, which are force controlled. Material properties listed in this chapter are expected-strength values. If lower-bound material properties are needed, they should be taken as mean minus one standard deviation values, or they can be adjusted from expected-strength values in accordance with Section 12.2.2.5.

C12.2.2.2 Component Properties

C12.2.2.2.1 Elements Structural elements of the seismic-force-resisting system consist of primary and secondary components, which collectively define element strength and resistance to deformation. Behavior of the components—including shear walls, beams, diaphragms, columns, and braces—is dictated by physical properties such as area; material grade; thickness, depth, and slenderness ratios; lateral-torsional buckling resistance; and connection details.

The actual physical dimensions should be measured; for example, in wood construction, the labeled 2 × 4 in. nominal stud dimensions are generally 1-1/2 × 3-1/2 in. (38 × 89 mm). Connected members include plywood, bracing, stiffeners, chords, sills, struts, and tie-down posts. Modifications to members include notching, holes, splits, and cracks. The presence of decay or deformation should be noted.

These primary component properties are needed to properly characterize building performance in the seismic analysis. The starting point for establishing component properties should be the available construction documents. Preliminary review of these documents should be performed to identify vertical-load (gravity-load) and seismic-force-resisting elements and systems, and their critical components and connections. Site inspections should be conducted to verify conditions and to ensure that remodeling has not changed the original design concept. In the absence of a complete set of construction documents, the design professional must thoroughly inspect the building to identify these elements, systems, and components, as indicated in Section 12.2.3.

C12.2.2.2.2 Connections The method of connecting the various components of the structural system is critical to its performance. The type and character of the connections must be determined by a review of the plans and a field verification of the conditions.

C12.2.2.3 Test Methods to Quantify Material Properties To obtain the desired in-place mechanical properties of materials and components, including expected strength, it is often necessary to use proven destructive and nondestructive testing methods.

Of greatest interest to wood building system performance are the expected orthotropic strengths of the installed materials for anticipated actions (e.g., flexure). Past research and accumulation of data by industry groups have led to published mechanical properties for most wood types and sizes [e.g., solid-sawn dimension lumber and glued laminated timber (glulam)]. Section 12.2.2.5 addresses these established default strengths and distortion properties. This information may be used, together with tests from recovered samples or observation, to establish the expected properties for use in component strength and deformation analyses. Where possible, the load history for the building shall be assessed for possible influence on component strength and deformation properties.

To quantify material properties and to analyze the performance of archaic wood construction, shear walls, and diaphragm action, more extensive sampling and testing may be necessary. This testing should include further evaluation of load history and moisture effects on properties and an examination of wall and diaphragm continuity and of the suitability of in-place connectors.

Where it is desired to use an existing assembly and little or no information about its performance is available, a cyclic load test of a mock-up of the existing structural elements can be used to determine the performance of various assemblies, connections, and load transfer conditions. See Section 7.6 for an explanation of the backbone curve and the establishment of alternative modeling parameters.

C12.2.2.4 Minimum Number of Tests To quantify expected strength and other in-place properties accurately, a minimum number of tests must be conducted on representative components. The minimum number of tests is dictated by available data from original construction, the type of structural system used, desired accuracy, and quality or condition of in-place materials. Visual access to the structural system also influences testing program definition. As an alternative, the design professional may elect to use the default strength properties in accordance with Section 12.2.2.5. However, using default values without testing is only permitted with the linear analysis procedures. It is strongly encouraged that the expected strengths be derived through testing of assemblies to model behavior accurately.

Removal of coverings, including stucco, fireproofing, and partition materials, is generally required to facilitate sampling and observations.

Component types include solid-sawn lumber, glulam, and plywood diaphragm. Element types include those that are part of gravity- and seismic-force-resisting systems. The observations shall consist of each connector type present in the building (e.g., nails, bolts, and straps), such that the composite strength of the connection can be estimated.

C12.2.2.5 Default Properties The results of any material testing performed should be compared with the default values for the particular era of building construction. If significantly reduced properties from testing are discovered, further evaluation should be undertaken.

Tables 12-1 and 12-2 contain default values for strength and stiffness of shear wall and diaphragm assemblies. The shear stiffness, G_d, for the assemblies should not be confused with the modulus of rigidity, G_v, for wood structural panels.

Actions associated with wood shear wall and diaphragm assemblies generally are deformation controlled, and expected-strength material properties are used most often. Lower-bound values are needed for actions that are force controlled. The 0.85 factor included in this standard to convert expected strength to lower-bound values is based on the results of shear wall testing. If more precise lower-bound material properties are desired, they should be taken as mean minus one standard deviation from test data for the components in question.

C12.2.2.5.1 Wood Construction Default Properties The load and resistance factor design (LRFD) methodology of AWC NDS is based on the concepts of limit-state design, similar to the provisions for strength design in steel or concrete. LRFD resistance values are based on ASTM D5457, which provides methodologies for calculation directly from data or by format conversion from reference allowable stress values. Use of a format conversion (i.e., the LRFD equivalent of allowable stresses) for computing expected strengths of wood materials comprising individual wood components and for wood connectors (nails, screws, lags, bolts, split rings, and so forth) is permitted. This format conversion methodology is not applicable for adjustment of tabulated expected-strength values for wood shear wall and diaphragm assemblies covered in Tables 12-1 and 12-2. For use with this chapter, capacities for shear wall and diaphragm assemblies are to be taken directly from the tables or as indicated by the table footnotes.

LRFD Manual for Engineered Wood Construction (AF&PA 1996) contains a guideline for calculating resistance values for connection hardware for which published report values are in allowable-stress format. Where computing the expected strength of connections, all limit states, including that of the connection hardware, must be considered (e.g., in addition to the published strength of a tie-down device inclusive of the connectors net section strength of the end post is considered in the design).

The connector deformation at yield may be calculated by dividing the load by the load/slip modulus. The load/slip modulus for dowel-type connections (bolts, lag screws, screws, and nails) is calculated as $180D^{1.5}$ kip/in. for wood-to-wood connections and $270D^{1.5}$ kip/in. for wood-to-steel side plate connections.

C12.2.3 Condition Assessment

C12.2.3.1 General The physical condition of existing components and elements and their connections must be examined for degradation. Degradation may include environmental effects (e.g., decay; splitting; fire damage; and biological, termite, corrosion, and chemical attack) or past or current loading effects (e.g., overload, damage from past earthquakes, buckling, crushing, and twisting). Natural wood also has inherent discontinuities, such as knots, checks, and splits, which must be noted. Configuration problems observed in recent earthquakes, including effects of discontinuous components; improper nailing, screwing, welding, or bolting; poor fit-up; and connection problems at the foundation level, should also be evaluated. Often, unfinished areas, such as attic spaces, basements, and crawl spaces, provide suitable access to structural components and can give a general indication of the condition of the rest of the structure. Invasive inspection of critical components and connections is typically required.

Connections require special consideration and evaluation. The load path for the system must be determined, and each connection in the load path(s) must be evaluated. This path includes diaphragm-to-component and component-to-component connections. The strength and deformation capacity of connections must be checked where the connection is attached to one or more components that are expected to experience significant inelastic response. Anchorage of exterior walls to roof and floors in concrete and masonry buildings, for which wood diaphragms are used for out-of-plane loading, requires detailed inspection. Bolt holes in relatively narrow straps sometimes preclude the ductile behavior of the steel strap. Twists and kinks in the strap can also have a serious effect on its anticipated behavior. Cross ties, which are part of the wall anchorage system, need to be inspected to confirm their presence, along with the connection of each piece, to ensure that a positive load path exists to tie the building walls together.

The condition assessment also affords an opportunity to review other conditions that may influence wood elements and systems and overall building performance. Of particular importance is the identification of other elements and components that may contribute to or impair the performance of the wood system in question, including infills, neighboring buildings, and equipment attachments. Limitations posed by existing coverings, wall and ceiling space insulation, and other material should also be defined such that prudent retrofit measures can be planned.

C12.2.3.2 Scope and Procedures for Condition Assessment Accessibility constraints may necessitate the use of instruments such as a fiberscope or video probe to reduce the amount of damage to covering materials and fabrics. The knowledge and insight gained from the condition assessment is invaluable to understanding load paths and the ability of components to resist and transfer loads. The degree of assessment performed also affects the knowledge factor, which is discussed in Section 12.2.4.

Direct visual inspection provides the most valuable information because it can be used to identify any configuration issues, it allows measurement of component dimensions, and it identifies the presence of degradation. The continuity of load paths may be established by viewing components and connection condition. From visual inspection, the need for other test methods to quantify the presence and degree of degradation may be established.

The scope of the removal effort is dictated by the component and element design. For example, in a braced frame, exposure of several key connections may suffice if the physical condition is acceptable, and the configuration matches the construction documents. However, for shear walls and diaphragms, it may be necessary to expose more connection points because of varying designs and the critical nature of the connections. For encased walls and frames for which no construction documents exist, it is necessary to indirectly view or expose all primary end connections for verification.

The physical condition of components and connectors may also support the need to use certain destructive and nondestructive test methods. Devices normally used for the detection of reinforcing steel in concrete or masonry may be used to verify the diagonal braced straps and hardware located beneath finish surfaces.

C12.2.3.3 Basis for the Mathematical Building Model The acceptance criteria for existing components depend on the design professional's knowledge of the condition of the structural system and material properties, as previously noted. Certain damage—such as water staining, evidence of prior leakage, corrosion, splitting, cracking, checking, buckling, warping, and twisting—may be acceptable. The design professional must establish a case-by-case acceptance for such damage on the basis of capacity loss or deformation constraints. Degradation at connection points should be carefully examined; significant capacity reductions may be involved, as well as a loss of ductility.

C12.3 GENERAL ASSUMPTIONS AND REQUIREMENTS

C12.3.2.2 Deformation-Controlled Actions The relative magnitude of the m-factors alone should not be interpreted as a direct indicator of performance. The stiffness of a component and its expected strength, Q_{CE}, must be considered where evaluating expected performance. For example, whereas the m-factors for gypsum plaster are higher than those for wood structural panels, the stiffness assigned to gypsum plaster is relatively high and the expected-strength values are much lower than those for wood structural panels. As a result, worse performance for a given displacement is predicted.

C12.3.2.3 Force-Controlled Actions The maximum forces developed in yielding shear walls and diaphragms are consistently 1.5 to 2 times the yield force. Other components and connectors exhibit similar overstrength.

C12.3.3 Connection Requirements In considering connections in this standard, connectors are distinguished from bodies of connected wood elements and bodies of connection hardware. Connectors, which consist of the nails, screws, welds, lags, bolts, split rings, and shear plates used to link pieces of a connection assembly together, are considered to have the ability to deform in a ductile manner, provided that the bodies of the connected wood elements or bodies of connection hardware do not prematurely fracture. Much of the ductility in a wood shear wall or diaphragm assembly comes from the connectors, such as bending in the nails before the point where nails pull through the sheathing or withdraw from framing. In bolted connections, the connectors, including bolt bending or crushing of the wood around the bolt hole, are ductile sources of deformation in an assembly. Brittle failure can occur in the bodies of connected wood elements, such as net section tension failure, and row and group tearout in an end post, or in the bodies of connection hardware, such as tension rupture in the metal tie-down. For this reason, connectors are considered deformation controlled, and bodies of connected wood elements and bodies of connection hardware are considered force controlled. Where determining the demand on force-controlled portions of the connection assembly, use of a limit-state analysis to determine the maximum force that can be delivered to the connection is recommended.

Where computing the strength of connections, all potential limit states should be considered, including those associated with the bodies of connected wood elements (e.g., wood member strength limit states), the bodies of metal connection hardware, and connectors with which the assembly may be composed. For example, in addition to the strength of a metal tie-down device, strength of the bolts attaching the device to the wood end post, and net section of the end post should be considered. The controlling condition determines the expected or lower-bound strength of the connection.

C12.3.5 Retrofit Measures Wood structural panels are used to provide lateral strength and stiffness to most modern wood buildings and are generally recommended for the retrofit of horizontal diaphragms and shear walls of existing buildings. The system relies on the in-plane strength and stiffness of the panels and their connection to the framing. Panels are connected together by nailing into the same structural member to create, in effect, one continuous panel. The various panels are described in Sections 12.4 and 12.5. The performance of the structural panels is dependent to a great degree on the attachment to the framing. The attachment spacing and effectiveness should be investigated if the existing panels are expected to withstand significant loads. If fasteners are to be added to existing panels, they should be the same size as the existing fasteners.

C12.4 WOOD SHEAR WALLS

C12.4.1 General The behavior of wood shear walls is complex and influenced by many factors; the primary factor is the wall sheathing. Provisions for combination of dissimilar materials on opposite sides of the wall require coordination of m-factors and modeling parameters for default shear wall types. Where test data are available, there is no restriction on consideration of strength and stiffness of the wall assembly sheathed on opposite sides with dissimilar materials. Further information on approaches for consideration of effect of dissimilar materials on strength and deformation response of a shear wall can be obtained from FEMA P-807 (2013).

Wall sheathings can be divided into many categories (e.g., brittle, elastic, strong, weak, good at dissipating energy, or poor at dissipating energy). In many existing buildings, the walls were not expected to act as shear walls (e.g., a wall sheathed with wood lath and plaster). Most shear walls are designed based on values from monotonic load tests and historically accepted values. The allowable shear per unit length used for design was assumed to be the same for long walls, narrow walls, walls with stiff tie-downs, and walls with flexible tie-downs. Only recently have shear wall assemblies—framing, covering, and anchorage—been tested using cyclic loading. A summary of fully reversed cyclic testing results and comparison of the test-based response capacities to Chapter 12 recommendations for select wood frame shear walls is provided in NIST GCR 17-917-45 (2017).

Another major factor influencing the behavior of shear walls is the aspect ratio of the wall. The AWC SDPWS limits the aspect ratio (height-to-width) for wood structural panel shear walls to 2:1 for full-design shear capacity and permits reduced-design shear capacities for walls with aspect ratios up to 3.5:1. The interaction of the floor and roof with the wall, the end conditions of the wall, and the redundancy or number of walls along any wall line would affect the wall behavior for walls with the same aspect ratio. In addition, the rigidity of the tie-downs at the wall ends has an important effect in the behavior of narrow walls.

The presence of any but small openings in shear walls causes a reduction in the stiffness and strength because of a reduced length of wall available to resist seismic forces. Special analysis techniques and detailing are required at the openings. The presence or addition of chord members around the openings reduces the loss in overall stiffness and limits damage in the area of openings. AWC SDPWS covers design of shear walls with openings.

For wood shear walls, the important limit states are sheathing failure, connection failure, tie-down failure, and excessive deflection. Limit states define the point of Life Safety and, often, of structural stability. To reduce damage or retain usability immediately after an earthquake, deflection must be limited (see Section 1.5.6). The ultimate capacity is the maximum capacity of the assembly, regardless of the deflection.

C12.4.2 Types of Wood Shear Walls

C12.4.2.1 Existing Wood Shear Walls

C12.4.2.1.1 Single-Layer Horizontal Lumber Sheathing or Siding Typically, nominal 1 in. (25.4 mm) wide horizontal lumber sheathing or siding is applied directly to studs. Forces are resisted by nail couples. Horizontal boards, from nominal 1 × 4 in. (25.4 × 102.6) to 1 × 12 in., typically are nailed to 2 in. (50.8 mm) nominal or wider studs with two or more nails (typically 8d or 10d) per stud.

C12.4.2.1.2 Diagonal Lumber Sheathing Typically, nominal 1 × 6 in. (25.4 × 152.4 mm) to 1 × 8 in. diagonal lumber sheathing, applied directly to the studs, resists lateral forces primarily by triangulation (i.e., direct tension and compression). A second layer of diagonal lumber sheathing is sometimes added on top of the first layer, at 90 degrees to the first layer (called double-layer diagonal lumber sheathing), for increased load capacity and stiffness.

C12.4.2.1.3 Vertical Wood Siding Only Typically, nominal 1 × 8 in. (25.4 × 203.2 mm), 1 × 10 in. (25.4 × 254 mm) 1 × 12 in. (25.4 × 304.8 mm) vertical boards are nailed directly to 2 in. (50.8 mm) nominal or wider studs and blocking with 8d or 10d galvanized nails. The lateral forces are resisted by nail couples, similarly to horizontal siding.

C12.4.2.1.4 Wood Siding over Horizontal Lumber Sheathing Typically, siding is nailed with 8d or 10d galvanized nails through the lumber sheathing to the studs. Lateral forces are resisted by nail couples for both layers.

C12.4.2.1.5 Wood Siding over Diagonal Lumber Sheathing Typically, siding is nailed with 8d or 10d galvanized nails to and through the lumber sheathing into the studs. Diagonal lumber sheathing provides most of the lateral resistance by triangulation (see Section 12.4.2.1.2).

C12.4.2.1.6 Wood Structural Panel Sheathing or Siding Typically, 4 × 8 ft (101.6 × 203.2 mm) panels are applied vertically or horizontally to 2 in. (50.8 mm) nominal or wider studs and nailed with 6d to 10d nails. These panels resist lateral forces by panel diaphragm action.

C12.4.2.1.7 Stucco on Studs Typically, 7/8 in. (22.2 mm) portland cement plaster is applied to wire lath or expanded metal lath. Wire lath or expanded metal lath is nailed to the studs with 11-gauge nails or 16-gauge staples at 6 in. (152.4 mm) on center. This assembly resists lateral forces by panel diaphragm action.

C12.4.2.1.8 Gypsum Plaster on Wood Lath Typically, 1 in. (25.4 mm) gypsum plaster is keyed onto spaced 1-1/4 in. (34.7 mm) nominal wood lath that is nailed to studs with 13-gauge nails. Gypsum plaster on wood lath resists lateral forces by panel diaphragm-shear action.

C12.4.2.1.9 Gypsum Plaster on Gypsum Lath Typically, 1/2 in. (12.7 mm) plaster is glued or keyed to 16 × 48 in. (406.4 mm × 1.2 m) gypsum lath, which is nailed to studs with 13-gauge nails. Gypsum plaster on gypsum lath resists lateral forces by panel diaphragm action.

C12.4.2.1.10 Gypsum Wallboard Typically, 4 × 8 ft (1.2 × 2.4 m) to 4 × 12 ft (1.2 × 3.6 m) panels are laid up horizontally or vertically and nailed to studs or blocking with 5d to 8d cooler nails at 4 to 7 in. (101 to 177 mm) on center. Multiple layers are used in some situations. The assembly resists lateral forces by panel diaphragm action.

C12.4.2.1.11 Gypsum Sheathing Typically, 4 × 8 ft (1.2 × 2.4 m) to 4 × 12 ft (1.2 × 3.6 m) panels are laid up horizontally or vertically and nailed to studs or blocking with galvanized 11-gauge 7/16 in. (11 mm) diameter head nails at 4 to 7 in. (101 to 177 mm) on center. Gypsum sheathing is usually installed on the exterior of structures with siding over it to improve fire resistance. Lateral forces are resisted by panel diaphragm action.

C12.4.2.1.12 Plaster on Metal Lath Typically, 1 in. (25.4 mm) gypsum plaster is applied on expanded wire lath that is nailed to the studs. Lateral forces are resisted by panel diaphragm action.

C12.4.2.1.13 Horizontal Lumber Sheathing with Cut-In Braces or Diagonal Blocking Horizontal lumber sheathing with cut-in braces or diagonal blocking is installed in the same manner as horizontal lumber sheathing, except that the wall is braced with cut-in (or let-in) braces or blocking. The bracing is usually installed at a 45 degree angle and nailed with 8d or 10d nails at each stud and at the top and bottom plates. Bracing provides only nominal increase in resistance.

C12.4.2.1.14 Fiberboard or Particleboard Sheathing Typically, 4 × 8 ft (1.2 × 2.4 m) panels are applied directly to the studs with nails. Fiberboard requires nails (typically 8d) with large heads, such as roofing nails. Lateral forces are resisted by panel diaphragm action.

C12.4.2.2 Enhanced Wood Shear Walls Possible retrofit methods for wood shear walls include the following:

Wood Structural Panel Sheathing Added to Unfinished Stud Walls: Wood structural panel sheathing may be added to one side of unfinished stud walls to increase the wall shear capacity and stiffness.

Examples of unfinished stud walls are cripple walls and attic end walls.

Wood Structural Panel Sheathing Overlay of Existing Shear Walls: The following types of existing shear walls may be overlaid with wood structural panel sheathing:

1. Single-layer horizontal lumber sheathing or siding,
2. Single-layer diagonal lumber sheathing,
3. Vertical wood siding only,
4. Gypsum plaster or wallboard on studs (also on gypsum lath and gypsum wallboard),
5. Gypsum sheathing,
6. Horizontal lumber sheathing with cut-in braces or diagonal blocking, and
7. Fiberboard or particleboard sheathing.

This method results in a moderate increase in shear capacity and stiffness and can be applied in most places in most structures. For example, plywood sheathing can be applied over an interior wall finish. For exterior applications, the wood structural panel can be nailed directly through the exterior finish to the studs.

Where existing shear walls are overlaid with wood structural panels, the connections of the overlay to the existing framing must be considered. Splitting can occur in both the wood sheathing and the framing. The length of nails needed to achieve full-capacity attachment in the existing framing must be determined. This length varies with the thickness of the existing wall covering. Sometimes staples are used instead of nails to prevent splitting. The overlay is stapled to the wood sheathing instead of the framing. Nails are recommended for overlay attachment to the underlying framing. In some cases, new blocking at wood structural panel joints may also be needed.

Wood Structural Panel Sheathing Added under Existing Wall Covering: The existing wall covering may be removed; wood structural panel sheathing, connections, and tie-downs may be added; and the wall covering may be replaced.

This method results in a significant increase in shear capacity. In some cases, where seismic forces are large, this may be the best method of retrofit. This retrofit procedure can be used on any of the existing shear wall assemblies. Additional framing members can be added if necessary, and the wood structural panels must be cut to fit existing stud spacings.

Increased Attachment: Additional nailing, collector straps, splice straps, tie-downs, or other collectors may be added to existing wood structural panel-sheathed walls to increase their rigidity and capacity.

For existing structural panel-sheathed walls, additional nailing results in higher capacity and increased stiffness. Other connectors—collector straps, splice straps, or tie-downs—are often necessary to increase the rigidity and capacity of existing structural panel shear walls. Increased ductility does not necessarily result from the additional nailing. Access to these shear walls often requires the removal and replacement of existing finishes.

Enhanced Connections: Where absent, new connections between shear walls and diaphragms and foundations may be added. Where needed, blocking between floor and roof joists at shear walls may be added. Blocking should be connected to the shear wall and the diaphragm to provide a load path for seismic forces. Wood for framing members or blocking should be kiln dried or well-seasoned to prevent it from shrinking away from the existing framing or from splitting.

Most shear wall retrofit procedures require a check of all existing connections, especially to diaphragms and foundations. Sheet metal framing clips can be used to provide a verifiable connection between the wall framing, the blocking, and the diaphragm. Framing clips are also often used for connecting blocking or rim joists to sill plates. Frequently, bolting between sill plates and foundations must be added.

The framing in existing buildings is usually very dry, hard, and easily split. Care must be taken not to split the existing framing when adding connectors. Predrilling holes for nails reduces splitting, and framing clips that use small nails are less likely to split the existing framing.

C12.4.2.3 New Wood Shear Walls New shear walls using the existing framing generally are sheathed with wood structural panels (i.e., plywood or oriented strand board). The thickness and grade of these panels can vary. In most cases, the panels are placed vertically and are fastened directly to the studs and plates. This method reduces the need for added blocking at the panel edges. All edges of panels must be attached to framing or blocking to obtain full capacity. The thickness and grade of panels, size and number of fasteners, framing spacing and specific gravity, and the shear wall aspect ratio are among factors that determine the capacity of the new walls. Additional information on requirements for shear walls can be found in AWC SDPWS and documents from the APA—The Engineered Wood Association (APA) such as Tissell (1993), *Plywood Design Specification* (APA 1997), and *Panel Design Specification* (APA 2012).

C12.4.3 Stiffness, Strength, Acceptance Criteria, and Connection Design for Wood Shear Walls

C12.4.3.1 Single-Layer Horizontal Lumber Sheathing or Siding Shear Walls

C12.4.3.1.1 Stiffness of Single-Layer Horizontal Lumber Sheathing or Siding Shear Walls Horizontal lumber sheathed shear walls are weak and very flexible and have long periods of vibration. The strength and stiffness degrade with cyclic loading. These shear walls are suitable only where seismic forces are low and deflection control is not required.

C12.4.3.1.2 Strength of Single-Layer Horizontal Lumber Sheathing or Siding Shear Walls This capacity is dependent on the width of the boards; spacing of the studs; and the size, number, and spacing of the nails. Allowable capacities are provided for various configurations in AWC SDPWS. Allowable capacities are listed for various configurations together with a description of the nail couple method, in the *Western Woods Use Book* (WWPA 1996). See also *Guidelines for the Design of Horizontal Wood Diaphragms*, ATC-7 (1981), for a discussion of the nail couple method.

C12.4.3.1.3 Acceptance Criteria for Single-Layer Horizontal Lumber Sheathing or Siding Shear Walls Deformation acceptance criteria are determined by the capacity and gravity-load and seismic-force-resisting components and elements to deform with limited damage or without failure. Excessive deflection could result in major damage to the structure and/or its contents.

C12.4.3.1.4 Connections of Single-Layer Horizontal Lumber Sheathing or Siding Shear Walls The capacity and ductility of these connections often determines the failure mode and the capacity of the assembly. Ductile connections with sufficient capacity give acceptable and expected performance (see Section 12.2.2.2.2).

C12.4.3.2 Diagonal Lumber Sheathing Shear Walls

C12.4.3.2.1 Stiffness of Diagonal Lumber Sheathing Shear Walls Diagonal lumber sheathed shear walls are stiffer and stronger than horizontal sheathed shear walls. They also provide greater stiffness for deflection control, and thereby greater damage control.

C12.4.3.2.2 Strength of Diagonal Lumber Sheathing Shear Walls The strength of diagonal lumber sheathing is dependent on the width of the boards, the spacing of the studs, the size of nails, the number of nails per board, and the boundary conditions. Allowable capacities are listed for various configurations in AWC SDPWS and *Western Woods Use Book* (WWPA 1996).

C12.4.3.3 Vertical Wood Siding Shear Walls

C12.4.3.3.1 Stiffness of Vertical Wood Siding Shear Walls Vertical wood siding has a very low seismic-force-resistance capacity and is very flexible. The strength and stiffness degrade with cyclic loading. These shear walls are suitable only where seismic forces are very low and deflection control is not needed.

C12.4.3.3.2 Strength of Vertical Wood Siding Shear Walls The strength of vertical wood siding is dependent on the width of the boards; the spacing of the studs; the spacing of blocking; and the size, number, and spacing of the nails. The nail couple method described in the *Western Woods Use Book* (WWPA 1996) can be used to calculate the capacity of vertical wood siding in a manner similar to the method used for horizontal siding.

C12.4.3.3.4 Connections of Vertical Wood Siding Shear Walls The load capacity of the vertical siding is low, which makes the capacity of connections between the shear wall and the other elements of less concern (see Section 12.2.2.2.2).

C12.4.3.4 Wood Siding over Horizontal Lumber Sheathing Shear Walls

C12.4.3.4.1 Stiffness of Wood Siding over Horizontal Lumber Sheathing Shear Walls Double-layer horizontal lumber sheathed shear walls are stiffer and stronger than single-layer horizontal lumber sheathed shear walls. These shear walls are often suitable for resisting seismic forces that are low to moderate in magnitude. They also provide greater stiffness for deflection control and, thereby, greater damage control.

C12.4.3.4.2 Strength of Wood Siding over Horizontal Lumber Sheathing Shear Walls This capacity is dependent on the width of the boards; the spacing of the studs; the size, number, and spacing of the nails; and the location of joints.

C12.4.3.5 Wood Siding over Diagonal Lumber Sheathing Shear Walls

C12.4.3.5.1 Stiffness of Wood Siding over Diagonal Lumber Sheathing Shear Walls Horizontal wood siding over diagonal lumber sheathing provides stiff, strong shear walls. These shear walls are often suitable for resisting seismic forces that are moderate in magnitude. They also provide good stiffness for deflection control and damage control.

C12.4.3.5.2 Strength of Wood Siding over Diagonal Lumber Sheathing Shear Walls The capacity of wood siding over diagonal lumber sheathing is dependent on the width of the boards; the spacing of the studs; the size, number, and spacing of the nails; the location of joints; and the boundary conditions.

C12.4.3.6 Wood Structural Panel Sheathing or Siding Shear Walls

C12.4.3.6.1 Stiffness of Wood Structural Panel Sheathing or Siding Shear Walls The response of wood structural panel shear walls is dependent on the thickness of the wood structural panels, the height-to-width (h/b) ratio, the nailing pattern, and other factors. Values for modulus of rigidity, G, and effective thickness, t, for various sheathing materials are contained in *Panel Design Specification* (APA 2012), *Plywood Design Specification* (APA 1997), and AWC SDPWS Commentary.

C12.4.3.6.2 Strength of Wood Structural Panel Sheathing or Siding Shear Walls Shear capacities of wood structural panel shear walls are primarily dependent on the nailing at the wood structural panel edges and the thickness and grade of the wood structural panel. Nominal unit shear capacities are tabulated for various configurations of shear wall construction in AWC SDPWS, Tables 4.3A and B. AWC SDPWS tabulated nominal unit shear capacities for design of shear walls (i.e., LRFD unit shear value associated with $\phi = 1.0$) are 2.8 times the associated allowable stress design unit shear values for seismic design. Expected strengths of wood structural panel shear walls are 3.0 times the associated allowable stress design unit shear values for seismic design. A method for calculating the unit shear capacity of wood structural panel shear walls based on accepted nail values is provided in Tissell (1993). For this method, use LRFD-based fastener strengths.

The presence of 2 in. (50.8 mm) nominal framing at adjoining panel edges is common in older wood structural panel shear walls constructed before the 1982 Uniform Building Code (UBC) (ICBO 1982) required use of minimum 3 in. (76.2 mm) nominal width framing where nails are closely spaced. The 0.90 factor is based on the 10% strength reduction recognized in the 1979 UBC (ICBO 1979) for such shear walls. Further strength reductions should be imposed based on assessment of quality of nailing, presence of excessive splitting such as indicated by visible fracture of framing receiving the nail, and any other conditions adversely affecting strength of the panel-to-framing nailing. Use of an effective nail spacing based on exclusion of ineffective nailing due to excessive splitting, improper or missing nails, or other conditions adverse to developing the full strength of the framing-to-panel nailing is one approach to account for strength reductions beyond those associated with the 0.90 factor.

C12.4.3.7 Stucco on Studs, Sheathing, or Fiberboard Shear Walls

C12.4.3.7.1 Stiffness of Stucco on Studs, Sheathing, or Fiberboard Shear Walls Stucco is brittle, and the seismic-force-resisting capacity of stucco shear walls is low. The walls are stiff until cracking occurs, but the strength and stiffness degrade under cyclic loading. These shear walls are suitable only where seismic forces are low.

C12.4.3.7.2 Strength of Stucco on Studs, Sheathing, or Fiberboard Shear Walls This capacity is dependent on the attachment of the stucco netting to the studs and the embedment of the netting in the stucco.

C12.4.3.7.4 Connections of Stucco on Studs, Sheathing, or Fiberboard Shear Walls Of less concern is the connection of the stucco to the netting. Unlike plywood, the tensile capacity of the stucco material (portland cement), rather than the connections, often governs failure. See Section 12.2.2.2.2.

C12.4.3.8 Gypsum Plaster on Wood Lath Shear Walls

C12.4.3.8.1 Stiffness of Gypsum Plaster on Wood Lath Shear Walls Gypsum plaster shear walls are similar to stucco, except their strength is lower. As is the case for stucco, the walls are stiff until failure, but the strength and stiffness degrade under cyclic loading. These shear walls are suitable only where seismic forces are very low.

C12.4.3.8.4 Connections of Gypsum Plaster on Wood Lath Shear Walls The tensile and bearing capacity of the plaster, rather than the connections, often govern failure. The relatively low strength of this material makes connections between parts of the shear wall assembly and the other elements of the seismic-force-resisting system of less concern.

C12.4.3.9 Gypsum Plaster on Gypsum Lath Shear Walls

C12.4.3.9.1 Stiffness of Gypsum Plaster on Gypsum Lath Shear Walls Gypsum plaster on gypsum lath is similar to gypsum wallboard (see Section 12.4.3.11).

C12.4.3.9.4 Connections of Gypsum Plaster on Gypsum Lath Shear Walls The tensile and bearing capacity of the plaster, rather than the connections, often govern failure. The relatively low strength of this material makes connections between parts of the shear wall assembly and the other elements of the seismic-force-resisting system of less concern.

C12.4.3.10 Gypsum Wallboard Shear Walls

C12.4.3.10.1 Stiffness of Gypsum Wallboard Shear Walls Gypsum wallboard has a very low seismic-force-resisting capacity but is relatively stiff until cracking occurs. The strength and stiffness degrade under cyclic loading. These shear walls are suitable only where seismic forces are very low.

C12.4.3.10.2 Strength of Gypsum Wallboard Shear Walls The default capacity listed in Table 12-1 is for typical 7 in. (177.8 mm) nail spacing of 1/2 in. (12.7 mm) or 5/8 in. (15.8 mm) nominal thick panels with 4d or 5d nails. Higher capacities can be used if closer nail spacing, multilayers of gypsum board, and/or the presence of blocking at all panel edges is verified.

C12.4.3.11 Gypsum Sheathing Shear Walls

C12.4.3.11.1 Stiffness of Gypsum Sheathing Shear Walls Gypsum sheathing is similar to gypsum wallboard (see Section 12.4.3.10.1).

C12.4.3.11.2 Strength of Gypsum Sheathing Shear Walls The default capacity listed in Table 12-1 is based on typical 7 in. (177 mm) nail spacing of 1/2 in. (12.7 mm) or 5/8 in. (15.8 mm) nominal thick panels with 4d or 5d nails. Higher capacities can be used if closer nail spacing, multilayers of gypsum board, and/or the presence of blocking at all panel edges is verified.

C12.4.3.12 Plaster on Metal Lath Shear Walls

C12.4.3.12.1 Stiffness of Plaster on Metal Lath Shear Walls Plaster on metal lath is similar to plaster on wood lath, and the seismic-force-resisting capacity of these shear walls is low. The walls are stiff until cracking occurs, but the strength and stiffness degrade under cyclic loading. These shear walls are suitable only where seismic forces are low.

C12.4.3.12.4 Connections of Plaster on Metal Lath Shear Walls The tensile and bearing capacity of the plaster, rather than the connections, often govern failure. The relatively low strength of this material makes connections between parts of the shear wall assembly and the other elements of the seismic-force-resisting system of less concern.

C12.4.3.13 Horizontal Lumber Sheathing with Cut-In Braces or Diagonal Blocking Shear Walls

C12.4.3.13.1 Stiffness of Horizontal Lumber Sheathing with Cut-In Braces or Diagonal Blocking Shear Walls This assembly is similar to horizontal lumber sheathing without braces, except that the cut-in braces or diagonal blocking provide higher stiffness at initial loads. After the braces or blocking fail (at low loads), the behavior of the wall is the same as with horizontal lumber sheathing without braces. The strength and stiffness degrade under cyclic loading.

C12.4.3.13.4 Connections of Horizontal Lumber Sheathing with Cut-In Braces or Diagonal Blocking Shear Walls The capacity and ductility of these connections often determine the failure mode and the capacity of the assembly. Ductile connections with sufficient capacity give acceptable performance (see Section 12.2.2.2.2).

C12.4.3.14 Fiberboard or Particleboard Sheathing Shear Walls

C12.4.3.14.1 Stiffness of Fiberboard or Particleboard Sheathing Shear Walls Fiberboard sheathing is very weak, lacks stiffness, and is unable to resist lateral forces. Particleboard comes in two varieties: one is similar to structural panels, and the other (nonstructural) is slightly stronger than gypsum board but more brittle. Nonstructural particleboard should only be used where seismic forces are very low.

C12.4.3.14.2 Strength of Fiberboard or Particleboard Sheathing Shear Walls Fiberboard has very low strength and is therefore not considered a structural element for resisting seismic loads.

C12.4.3.14.4 Connections of Fiberboard or Particleboard Sheathing Shear Walls The capacity and ductility of the connections in structural particleboard shear walls often determine the failure mode and the capacity of the assembly. Ductile connections with sufficient capacity give acceptable performance. The tensile and bearing capacity of the nonstructural particleboard, rather than the connections, often govern failure. The relatively low strength of this material makes connections between parts of the shear wall assembly and the other elements of the seismic-force-resisting system of less concern.

C12.5 WOOD DIAPHRAGMS

C12.5.1 General The behavior of horizontal wood diaphragms is influenced by the type of sheathing, size and amount of fasteners, existence of perimeter chord or flange members, and the ratio of span length to width of the diaphragm.

The presence of any but small openings in wood diaphragms causes a reduction in the stiffness and strength of the diaphragm because of a reduced length of diaphragm available to resist seismic forces. Special analysis techniques and detailing are required at the openings. The presence or addition of chord members around the openings reduces the loss in stiffness of the diaphragm and limits damage in the area of the openings. See *Guidelines for the Design of Horizontal Wood Diaphragms*, ATC-7 (1981) and Tissell and Elliott (1997) for a discussion of the effects of openings in wood diaphragms.

The presence of chords at the perimeter of a diaphragm significantly reduces the diaphragm deflection caused by bending and increases the stiffness of the diaphragm over that of an unchorded diaphragm. However, the increase in stiffness caused by chords in a single straight-sheathed diaphragm is minimal because of the flexible nature of these diaphragms.

C12.5.2 Types of Wood Diaphragms

C12.5.2.1 Existing Wood Diaphragms

C12.5.2.1.1 Single-Layer Straight Lumber Sheathing Typically, single-layer straight lumber sheathed diaphragms consist of 1 in. (25.4 mm) nominal lumber sheathing laid perpendicular to the framing members; 2 in. (50.8 mm) or 3 in. (76.2 mm) nominal lumber sheathing may also be present. The sheathing serves the dual purpose of supporting gravity loads and resisting shear forces in the diaphragm. Most often, 1 in. (25.4 mm) nominal lumber sheathing is nailed with 8d or 10d nails, with two or more nails per sheathing board at each support. Shear forces perpendicular to the direction of the sheathing are resisted by the nail couple. Shear forces parallel to the direction of the sheathing are transferred through the nails in the supporting joists or framing members below the sheathing joints.

C12.5.2.1.2 Double-Layer Straight Lumber Sheathing Construction of double-layer straight lumber sheathed diaphragms is the same as that for single-layer straight lumber sheathed diaphragms, except that an upper layer of straight lumber sheathing is laid over the lower layer of lumber sheathing. The upper sheathing can be placed either perpendicular or parallel to the lower layer of sheathing. If the upper layer of sheathing is parallel to the lower layer, the board joints are usually offset sufficiently that nails at joints in the upper layer of sheathing are driven into a common sheathing board below, with sufficient edge distance. The upper layer of sheathing is nailed to the framing members through the lower layer of sheathing.

C12.5.2.1.3 Single-Layer Diagonal Lumber Sheathing Typically, 1 in. (25.4 mm) nominal lumber sheathing is laid at an approximate 45-degree angle to the framing members. In some cases, 2 in. (50.8 mm) nominal lumber sheathing may also be used. The sheathing supports gravity loads and resists shear forces in the diaphragm. Commonly, 1 in. (25.4 mm) nominal lumber sheathing is nailed with 8d nails, with two or more nails per board at each support. The recommended nailing for diagonal lumber sheathing diaphragms is published in *Western Woods Use Book* (WWPA 1996) and AWC SDPWS. The shear capacity of the diaphragm is dependent on the size and quantity of the nails at each sheathing board.

C12.5.2.1.4 Diagonal Lumber Sheathing with Straight Lumber Sheathing or Flooring Above Typically, these constructions consist of a lower layer of 1 in. (25.4 mm) nominal diagonal lumber sheathing laid at a 45-degree angle to the framing members, with a second layer of straight lumber sheathing or wood flooring laid on top of the diagonal sheathing at a 90-degree angle to the framing members. Both layers of sheathing support gravity loads and resist shear forces in the diaphragm. Sheathing boards are commonly connected with two or more 8d nails per board at each support.

C12.5.2.1.5 Double-Layer Diagonal Lumber Sheathing Typically, double-layer diagonal lumber sheathed diaphragms consist of a lower layer of 1 in. (25.4 mm) nominal diagonal lumber sheathing with a second layer of 1 in. (25.4 mm) nominal diagonal lumber sheathing laid at a 90-degree angle to the lower layer. The sheathing supports gravity loads and resists shear forces in the diaphragm. The sheathing is commonly nailed with 8d nails, with two or more nails per board at each support. The recommended nailing for double-layer diagonal lumber sheathed diaphragms is published in *Western Woods Use Book* (WWPA 1996) and AWC SDPWS.

C12.5.2.1.6 Wood Structural Panel Sheathing Typically, these constructions consist of wood structural panels, such as plywood or oriented strand board, placed on framing members and nailed in place. Different grades and thicknesses of wood structural panels are commonly used, depending on requirements for gravity load support and shear capacity. Edges at the ends of the wood structural panels are usually supported by the framing members. Edges at the sides of the panels can be blocked or unblocked. In some cases, tongue-and-groove wood structural panels are used. Nailing patterns and nail size can vary greatly. Nail spacing is commonly in the range of 3 to 6 in. (76 to 152 mm) on center at the supported and blocked edges of the panels, and 10 to 12 in. (254 to 305 mm) on center at the panel infield. Staples are sometimes used to attach the wood structural panels.

C12.5.2.1.7 Braced Horizontal Diaphragms Typically, these constructions consist of “X” rod bracing and wood struts forming a horizontal truss system at the floor or roof levels of the building. The “X” bracing usually consists of steel rods drawn taut by turnbuckles or nuts. The struts usually consist of wood members, which may or may not be part of the gravity-load-bearing system of the floor or roof. The steel rods function as tension members in the horizontal truss, and the struts function as compression members. Truss chords (similar to diaphragm chords) are needed to resist bending in the horizontal truss system.

C12.5.2.2 Enhanced Wood Diaphragms Possible retrofit methods for wood diaphragms include the following:

Wood Structural Panel Overlays on Straight or Diagonal Lumber Sheathing: Existing lumber sheathed diaphragms may be overlaid with new wood structural panels. Nails or staples may be used to connect the new structural panels to the existing diaphragms. Nails should be of sufficient length to provide the required embedment into framing members below the sheathing.

These diaphragms typically consist of new wood structural panels placed over existing straight or diagonal lumber sheathing and nailed or stapled to the existing framing members through the existing lumber sheathing. If the new overlay is nailed to the existing framing members only—without nailing at the panel edges perpendicular to the framing—the response of the new overlay is similar to that of an unblocked wood structural panel diaphragm.

If a stronger and stiffer diaphragm is desired, the joints of the new wood structural panel overlay should be placed parallel to the joints of the existing lumber sheathing, with the overlay nailed or stapled to the existing lumber sheathing. The edges of the new wood structural panels should be offset from the joints in the existing lumber sheathing below by a sufficient distance that the new nails may be driven into the existing lumber sheathing without splitting the lumber sheathing. If the new panels are nailed at all edges as previously described, the response of the new overlay is similar to that of a blocked wood structural panel diaphragm. As an alternative, new blocking may be installed below all panel joints perpendicular to the existing framing members.

Because the joints of the overlay and the joints of the existing sheathing may not be offset consistently without cutting the panels, it may be advantageous to place the wood structural panel overlay at a 45-degree angle to the existing lumber sheathing. If the existing diaphragm is straight lumber sheathed, the new overlay should be placed at a 45-degree angle to the existing lumber sheathing and joists. If the existing diaphragm is diagonal lumber sheathed, the new wood structural panel overlay should be placed perpendicular to the existing joists at a 45-degree angle to the diagonal lumber sheathing. Nails should be driven into the existing lumber sheathing with sufficient edge distance to prevent splitting of the existing lumber sheathing. At boundaries, nails should be of sufficient length to penetrate through the lumber sheathing into the framing below. New structural panel overlays shall be connected to shear walls or vertical bracing elements to ensure the effectiveness of the added panel.

Care should be exercised where placing new wood structural panel overlays on existing lumber diaphragms. The changes in stiffness and dynamic characteristics of the diaphragm may have negative effects by causing increased forces in other components or elements. The increased stiffness and the associated increase in dynamic forces may not be desirable in some diaphragms for certain Performance Levels.

Wood Structural Panel Overlays on Existing Wood Structural Panels: Existing wood structural panel diaphragms may be overlaid with new wood structural panels. Panel joints should be offset, or the overlay should be placed at a 45-degree angle to the existing wood structural panels.

The placement of a new overlay over an existing diaphragm should follow the same construction methods and procedures as those used for straight lumber sheathed and diagonal lumber sheathed diaphragms (see Section 12.5.3.7).

Increased Attachment: The nailing or attachment of the existing sheathing to the supporting framing may be increased. Nailing or attachment to the supporting framing should be increased, and blocking for the diaphragm at the wood structural panel joints should be added.

For straight lumber sheathed diaphragms, the increase in shear capacity is minimal. Double-layer straight lumber sheathed diaphragms with minimal nailing in the upper or both layers of sheathing may be enhanced significantly by adding new nails or staples to the existing diaphragm. The same is true for diaphragms that are single-layer diagonal lumber sheathed, double-layer diagonal lumber sheathed, or single-layer diagonal lumber sheathed with straight lumber sheathing or flooring.

In some cases, increased nailing at the wood structural panel infield may also be required. If the required shear capacity or stiffness is greater than that which can be provided by increased attachment, a new overlay on the existing diaphragm may be required to provide the desired enhancement.

C12.5.2.3 New Wood Diaphragms

C12.5.2.3.1 New Wood Structural Panel Sheathing Typically, these constructions consist of wood structural panels—such as wood structural panel or oriented strand board—nailed or stapled to existing framing members after existing sheathing has been removed. Different grades and thicknesses of wood structural panels can be used, depending on the requirements for gravity load support and diaphragm-shear capacity. In most cases, the panels are placed with the long dimension perpendicular to the framing members, and panel edges at the ends of the panels are supported by, and nailed to, the framing members. Edges at the sides of the panels can be blocked or unblocked, depending on the shear capacity and stiffness required in the new diaphragm. Wood structural panels can be placed in various patterns, as shown in AWC SDPWS.

C12.5.2.3.4 New Braced Horizontal Diaphragms Because new horizontal truss systems induce new forces on existing framing members, it may be more economical to design floor or roof sheathing as a diaphragm. This method eliminates the potential need to strengthen wood members at the compression struts. Braced horizontal diaphragms are more feasible where sheathing cannot provide sufficient shear capacity or where diaphragm openings reduce the shear capacity of the diaphragm and additional shear capacity is needed.

C12.5.3 Stiffness, Strength, Acceptance Criteria, and Connection Design for Wood Diaphragms

C12.5.3.1 Single-Layer Straight Lumber Sheathing Diaphragms

C12.5.3.1.1 Stiffness of Single-Layer Straight Lumber Sheathing Diaphragms Single-layer straight lumber sheathed diaphragms are characterized by high flexibility with a long period of vibration. These diaphragms are suitable for low shear conditions where control of diaphragm deflections is not needed to attain the desired Performance Level. See Section C12.5.3.6.1 for discussion of calculation of deflection caused by diaphragm chords and diaphragm chord-splice slip.

C12.5.3.1.2 Strength of Single-Layer Straight Lumber Sheathing Diaphragms The expected capacity of single-layer straight lumber sheathed diaphragms is dependent on the size, number, and spacing between the nails at each sheathing board, and the spacing of the supporting framing members. The shear capacity of single-layer straight lumber sheathed diaphragms can be calculated using the nail couple method. See *Guidelines for the Design of Horizontal Wood Diaphragms*, ATC-7 (1981) for a discussion of calculating the shear capacity of straight lumber sheathed diaphragms.

C12.5.3.1.3 Acceptance Criteria for Single-Layer Straight Lumber Sheathing Diaphragms Deformation acceptance criteria largely depend on the allowable deformations for other structural and nonstructural components and elements that are laterally supported by the diaphragm. Allowable deformations must also be consistent with the permissible damage state of the diaphragm.

C12.5.3.1.4 Connections of Single-Layer Straight Lumber Sheathing Diaphragms The load capacity of connections between diaphragms and shear walls or other vertical elements, as well as diaphragm chords and shear collectors, is critical.

C12.5.3.2 Double-Layer Straight Lumber Sheathing Diaphragms

C12.5.3.2.1 Stiffness of Double-Layer Straight Lumber Sheathing Diaphragms The double-layer straight lumber sheathed system provides a significant increase in stiffness over a single-layer straight lumber sheathed diaphragm, but very little test data are available on the stiffness and strength of these diaphragms. Both layers of straight lumber sheathing must have sufficient nailing, and the joints of the top layer must be either offset or perpendicular to the bottom layer.

C12.5.3.2.2 Strength of Double-Layer Straight Lumber Sheathing Diaphragms The strength and stiffness of double-layer straight lumber sheathed diaphragms is highly dependent on the nailing of the upper layer of lumber sheathing. If the upper layer has minimal nailing, the increase in strength and stiffness over a single-layer straight lumber sheathed diaphragm may be slight. If the upper layer of lumber sheathing has nailing similar to that of the lower layer of lumber sheathing, the increase in strength and stiffness is significant.

C12.5.3.3 Single-Layer Diagonal Lumber Sheathing Diaphragms

C12.5.3.3.1 Stiffness of Single-Layer Diagonal Lumber Sheathing Diaphragms Single-layer diagonal lumber sheathed diaphragms are significantly stiffer than single-layer straight lumber sheathed diaphragms but are still quite flexible.

C12.5.3.3.2 Strength of Single-Layer Diagonal Lumber Sheathing Diaphragms Single-layer diagonal lumber sheathed diaphragms are usually capable of resisting moderate shear loads.

Because the diagonal lumber sheathing boards function in tension and compression to resist shear forces in the diaphragm and the boards are placed at a 45 degree angle to the chords at the ends of the diaphragm, the component of the force in the sheathing boards that is perpendicular to the axis of the end chords creates a bending force in the end chords. If the shear in diagonal lumber sheathed diaphragms is limited to approximately 300 lb/ft (4.38 kN/m) or less, bending forces in the end chords are usually neglected. If shear forces exceed 300 lb/ft (4.38 kN/m), the end chords should be designed or reinforced to resist bending forces from the sheathing. See *Guidelines for the Design of Horizontal Wood Diaphragms*, ATC-7 (1981) for methods of calculating the shear capacity of diagonal lumber sheathed diaphragms.

C12.5.3.4 Diagonal Lumber Sheathing with Straight Lumber Sheathing or Flooring above Diaphragms

C12.5.3.4.1 Stiffness of Diagonal Lumber Sheathing with Straight Lumber Sheathing or Flooring above Diaphragms Straight lumber sheathing or flooring over diagonal lumber sheathing provides a significant increase in stiffness over single-layer lumber sheathed diaphragms.

C12.5.3.4.2 Strength of Diagonal Lumber Sheathing with Straight Lumber Sheathing or Flooring above Diaphragms Shear capacity is dependent on the nailing of the diaphragm. The strength and stiffness of diagonal lumber sheathed diaphragms with straight lumber sheathing above is highly dependent on the nailing of both layers of sheathing. Both layers of sheathing should have at least two 8d common nails per board at each support.

C12.5.3.5 Double-Layer Diagonal Lumber Sheathing Diaphragms

C12.5.3.5.1 Stiffness of Double-Layer Diagonal Lumber Sheathing Diaphragms Double-layer diagonal lumber sheathed diaphragms have greater stiffness than diaphragms with a single layer of diagonal lumber sheathing. The response of these diaphragms is similar to the response of diagonal lumber sheathed diaphragms with straight lumber sheathing overlays.

C12.5.3.5.2 Strength of Double-Layer Diagonal Lumber Sheathing Diaphragms Shear capacity is dependent on the nailing of the diaphragm, but these diaphragms are usually suitable for moderate to high shear loads.

Shear capacities are similar to those of diagonal lumber sheathed diaphragms with straight lumber sheathing overlays.

The sheathing boards in both layers of sheathing should be nailed with at least two 8d common nails at each support. The presence of a double layer of diagonal lumber sheathing eliminates the bending forces that single-layer diagonal lumber sheathed diaphragms impose on the chords at the ends of the diaphragm. As a result, the bending capacity of the end chords does not have an effect on the shear capacity and stiffness of the diaphragm.

C12.5.3.6 Wood Structural Panel Sheathing Diaphragm

C12.5.3.6.1 Stiffness of Wood Structural Panel Sheathing Diaphragms The response of wood structural panel diaphragms is dependent on the thickness of the wood structural panels, the length-to-width (L/b) ratio, nailing pattern, and presence of chords in the diaphragm, as well as other factors. Values for shear stiffness ($G_v t_v$) for various sheathing materials are contained in *Plywood Design Specification* (APA 1997), AWC SDPWS commentary, and *Panel Design Specification* (APA 2008).

In most cases, the area of the diaphragm chord equals the area of the continuous wood (or steel) member to which the sheathing is attached. For buildings with wood diaphragms and concrete or masonry walls, however, the area of the diaphragm chord is more difficult to identify, and engineering judgment is required. The tension area of the diaphragm chord on both edges of the diaphragm should be used for deflection calculations. In general, this result is conservative because it results in a larger calculated deflection. Use of the tension area of the diaphragm chord may not yield conservative results, however, where calculating the period of the building using Equation (7-20).

The term $\Delta_c X$ is determined by multiplying the assumed diaphragm chord slip at a single chord splice, Δ_c, by the distance, X, from the diaphragm chord splice to the nearest support (shear wall).

An alternate constant that can be used in the nail slip contribution term where panel nailing is not uniform is provided in Appendix C of *Diaphragms and Shear Walls Design/Construction Guide* (APA 2007).

Example calculations of diaphragm deflection are provided in *Design of Wood Structures* (Breyer et al. 2014) and AWC SDPWS commentary.

C12.5.3.6.2 Strength of Wood Structural Panel Sheathing Diaphragms Shear capacities of wood structural panel diaphragms are primarily dependent on the nailing at the wood structural panel edges and the thickness and grade of the wood structural panel in the diaphragm. Nominal unit shear capacities are tabulated for various configurations of diaphragm construction in AWC SDPWS, Tables 4.2A through C. AWC SDPWS tabulated nominal unit shear capacities for diaphragms (i.e., LRFD unit shear value associated with $\phi = 1.0$) are 2.8 times the associated allowable stress design unit shear values for seismic design. Expected strengths of wood structural panel shear walls are 3.0 times the associated allowable stress design unit shear values for seismic design. A method for calculating the unit shear capacity of wood structural panel diaphragms based on accepted nail values and panel shear strength is provided in Tissell and Elliott (1997). For this method, use LRFD-based fastener strengths.

The presence of 2 in. (50.8 mm) nominal framing at adjoining panel edges is common in older diaphragms constructed before the 1982 UBC (ICBO 1982) requirement for minimum 3 in. (76.2 mm) nominal width framing where nails are closely spaced. The 0.80 factor is based on the combination of the 0.89 factor in APA Report 138 (Tissell and Elliott 1997) for use of 2 in. (50.8 mm) nominal width framing in lieu of 3 in. (76.2 mm) nominal width framing and a 0.90 factor associated with a 10% strength reduction recognized in the 1979 UBC (ICBO 1979). Further strength reductions should be imposed based on assessment of quality of nailing, presence of excessive splitting such as indicated by visible fracture of framing receiving the nail, and any other conditions adversely affecting strength of the panel-to-framing nailing. Use of an effective nail spacing based on exclusion of ineffective nailing due to excessive splitting, improper or missing nails, or other conditions adverse to developing the full strength of the framing-to-panel nailing is one approach to account for strength reductions beyond those associated with the 0.80 factor.

C12.5.3.7 Wood Structural Panel Overlays on Straight or Diagonal Lumber Sheathing Diaphragms

C12.5.3.7.1 Stiffness of Wood Structural Panel Overlays on Straight or Diagonal Lumber Sheathing Diaphragms The stiffness of existing straight lumber sheathed diaphragms can be increased significantly by placing a new wood structural panel overlay over the existing diaphragm. The stiffness of existing diagonal lumber sheathed diaphragms and wood structural panel diaphragms is increased but not in proportion to the stiffness increase for straight lumber sheathed diaphragms.

Depending on the nailing of the new overlay, the response of the diaphragm may be similar to that of a blocked or an unblocked diaphragm.

C12.5.3.8 Wood Structural Panel Overlays on Existing Wood Structural Panel Sheathing Diaphragms

C12.5.3.8.1 Stiffness of Wood Structural Panel Overlays on Existing Wood Structural Panel Sheathing Diaphragms According to Tissell and Elliott (1997), Equation (12-5) is not applicable to two-layer diaphragms, presumably because of the difficulty in estimating the combined nail slip. Diaphragm deflection may be estimated using principles of mechanics that include consideration of nail slip, blocking, and the embedment of nails into the framing.

C12.6 WOOD FOUNDATIONS

C12.6.1 Types of Wood Foundations

Wood Piling: Wood piles are generally used with a concrete pile cap and are usually keyed into the base of the concrete cap. The piles are usually treated with preservatives.

Piles are classified as either friction- or end-bearing piles. Piles are generally not able to resist uplift loads because of the manner in which they are attached to the pile cap. The piles may be subjected to lateral forces from seismic loading, which are resisted by bending of the piles. The analysis of pile bending is generally based on a pinned connection at the top of the pile and fixity of the pile at some depth established by the geotechnical engineer. However, it should be evaluated with consideration for the approximate nature of the original assumption of the depth to point of fixity. Where battered piles are present, the lateral forces can be resisted by the horizontal component of the axial load.

Wood Footings: Wood grillage footings, sleepers, skids, and pressure-treated all-wood foundations can be encountered in existing structures. These foundations are highly susceptible to deterioration. The seismic resistance of wood footings is generally very low; they are essentially dependent on friction between the wood and soil for their performance.

Pole Structures: Pole structures resist seismic forces by acting as cantilevers fixed in the ground, with the seismic forces considered to be applied perpendicular to the pole axis. It is possible to design pole structures to have moment-resisting capacity at floor and roof levels by the use of knee braces or trusses. Pole structures are frequently found on sloping sites. The varying unbraced lengths of the poles generally affect the stiffness and performance of the structure and can result in unbalanced loads to the various poles, along with significant torsional distortion, which must be investigated and evaluated. Additional horizontal and diagonal braces can be used to reduce the flexibility of tall poles or reduce the torsional eccentricity of the structure.

C12.6.2 Analysis, Strength, and Acceptance Criteria for Wood Foundations The strength of the components, elements, and connections of a pole structure are the same as for a conventional structure.

C12.6.3 Retrofit Measures for Wood Foundations Wood footings showing signs of deterioration may be replaced with reinforced concrete footings. Wood pole structures can be retrofitted with the installation of diagonal braces or other supplemental seismic-force-resisting elements. Structures supported on wood piles may be retrofitted by the installation of additional piles.

C12.7 OTHER WOOD ELEMENTS AND COMPONENTS

C12.7.1 General Other wood elements include knee-braced frames, rod-braced frames, and braced horizontal diaphragms, among other systems.

Knee-braced frames produce moment-resisting joints by the addition of diagonal members between columns and beams. The resulting "semirigid" frame resists lateral forces. The moment-resisting capacity of knee-braced frames varies widely. The controlling part of the assembly is usually the connection; however, bending of members can be the controlling feature of some frames. Once the capacity of the connection is determined, members can be checked and the capacity of the frame can be determined by statics. Particular attention should be given to the beam–column connection. Additional tensile forces may be developed in this connection because of knee-brace action under vertical loads.

Similar to knee-braced frames, the connections of rods to timber framing usually govern the capacity of the rod-braced frame. Typically, the rods act only in tension. Once the capacity of the connection is determined, the capacity of the frame can be determined by statics.

Braced horizontal diaphragms are described in Section 12.5.2.1.7.

C12.7.1.2 Strength of Other Wood Elements and Components The strength of wood elements is dependent on the strength of the individual components that compose the assembly. In many cases, the capacity of the connections between components is the limiting factor in the strength of the assembly.

C12.7.1.3 Acceptance Criteria for Other Wood Elements and Components Deformation acceptance criteria largely depend on the allowable deformations for other structural and nonstructural components that are supported by the element. Allowable deformations must also be consistent with the desired performance level. Actions on connection types that do not appear in Table 12-3 (e.g., truss plates) are force controlled.

CHAPTER C13
ARCHITECTURAL, MECHANICAL, AND ELECTRICAL COMPONENTS

C13.1 SCOPE

The core of this chapter is contained in Table 13-1, which provides the following:

1. A list of nonstructural components subject to the Hazards Reduced, Life Safety, and Position Retention requirements of this standard.
2. Evaluation and retrofit requirements related to the Level of Seismicity and Hazards Reduced, Life Safety, and Position Retention Nonstructural Performance Levels. Requirements for Operational Nonstructural Performance are not included in this standard. References that may be used to seismically qualify equipment and systems to achieve Operational Nonstructural Performance for some nonstructural components are provided in Section C2.3.2.1.
3. Identification of the required evaluation procedure (analytical or prescriptive).

Section 2.2 provides general requirements and discussion of Performance Objectives and Performance Levels as they pertain to nonstructural components. Criteria for means of egress are not specifically included in this standard.

Section 13.4 provides sets of equations for a simple, default, force analysis, as well as an extended analysis method that considers additional factors. This section defines the analytical procedure for determining drift ratios and relative displacement and outlines general requirements for the prescriptive procedure.

Section 13.5 notes the general ways in which nonstructural evaluation and retrofit are carried out.

Sections 13.6, 13.7, and 13.8 provide the evaluation and retrofit criteria for each component category identified in Table 13-1. For each component, the following information is given:

1. Definition and scope,
2. Component behavior and retrofit methods,
3. Acceptance criteria, and
4. Evaluation requirements.

C13.2 EVALUATION AND RETROFIT PROCEDURE FOR NONSTRUCTURAL COMPONENTS

The Authority Having Jurisdiction should be consulted to establish the areas of the building for which nonstructural hazards shall be considered. Other nonstructural components, such as those designated by the owner, also should be included in those that are evaluated.

The architectural, mechanical, and electrical components and systems of a historic building may be historically significant, especially if they are original to the building, very old, or innovative. Historic buildings may also contain hazardous materials, such as lead pipes and asbestos, which may or may not pose a hazard, depending on their location, condition, use or abandonment, containment, and/or disturbance during the retrofit.

C13.2.1 Classification of Components Classification of acceleration-sensitive or deformation-sensitive components is discussed, where necessary, in each component section: Sections 13.6, 13.7, and 13.8. The guiding principle for deciding whether a component requires a force analysis, as defined in Section 13.4, is that analysis of inertial loads generated within the component is necessary to properly consider the component's seismic behavior. The guiding principle for deciding whether a component requires a drift analysis, as defined in Section 13.4, is that analysis of drift is necessary to properly consider the component's seismic behavior. Some components may be classified as acceleration sensitive in one direction and drift sensitive in the other direction. An example is a nonstructural partition wall that is sensitive to drift in plane and acceleration out of plane.

Glazing or other components that can hazardously fail at a drift ratio less than 0.01 (depending on installation details) or components that can undergo greater distortion without hazardous failure resulting—for example, typical gypsum board partitions—should be considered.

Use of Drift Ratio Values as Acceptance Criteria: The data on drift ratio values related to damage states are limited, and the use of single median drift ratio values as acceptance criteria must cover a broad range of actual conditions. It is therefore suggested that the limiting drift values shown in this chapter be used as a guide for evaluating the probability of a given damage state for a subject building, but they should not be used as absolute acceptance criteria. At higher nonstructural performance levels, it is likely that the criteria for nonstructural deformation-sensitive components may control the structural retrofit design. These criteria should be regarded as a flag for the careful evaluation of structural and/or nonstructural interaction and consequent damage states, rather than the required imposition of absolute acceptance criteria that might require costly redesign of the structural retrofit.

C13.3 COMPONENT ASSESSMENT AND ANCHORAGE TESTING

The provisions in ASCE 7 for components that are required to be designed with a component importance factor, I_p, of 1.5 are the most comprehensive criteria for the Operational Nonstructural Performance Level. In addition to requirements for anchorage and bracing, there are requirements for the design, evaluation, and testing of the components to certify that they can function immediately after the design seismic scenario. Evaluation, retrofit, and acceptance criteria for the Position Retention Nonstructural Performance Level may be used for the Operational Nonstructural Performance Level if more appropriate data are not available.

Forces on nonstructural components calculated in accordance with Section 13.4 are at a strength design level. Where allowable stress values are available for proprietary products used as bracing for nonstructural components, these values shall be factored up to strength design levels. In the absence of manufacturers' data on strength values, allowable stress values can be increased by a factor of 1.4 to obtain strength design values.

In cases where the Basic Performance Objective for Existing Buildings (BPOE) or Basic Performance Objective Equivalent to New Building Standards (BPON) is not required—such as where the Reduced Performance Objective is selected—there may be more latitude in the selection of components or criteria for nonstructural retrofit.

C13.3.1 Condition Assessment For the purpose of visual observation, nonstructural component types should be based on the general types listed in Table 13-1. Further distinction can be made where difference in structural configuration of the component or its bracing exists.

Seismic interactions between nonstructural components and systems can have a profound influence on the performance of these systems. Where appropriate, the condition assessment should include an interaction review. A seismic interaction involves two components: a source and a target. An interaction source is the component or structure that could fail or displace and interact with another component. An interaction target is a component that is being impacted, sprayed, or spuriously activated. For an interaction to affect a component, it must be credible and significant. A credible interaction is one that can take place. For example, the fall of a ceiling panel located overhead from a motor control center is a credible interaction because the falling panel can reach and impact the motor control center. The target (the motor control center) is said to be within the zone of influence of the source (the ceiling panel). A significant interaction is one that can result in damage to the target. For example, the fall of a light fixture on a 20 ft (6.1 m) steel pipe may be credible (the light fixture being above the pipe) but may not be significant (the light fixture would not damage the steel pipe). An important aspect of the interaction review is engineering judgment because only credible and significant sources of interaction should be considered in the condition assessment.

C13.3.2 Testing Requirements for Evaluating the Performance of Existing Attachments for Nonstructural Components The requirements in the section are not to determine the tensile strength of the anchors using a statistical approach but rather to verify if the existing installation provides the adequate level of protection for a desired Performance Objective. Anchor testing is necessary to achieve a target Performance Objective even if existing anchor details are available because the anchors may not have been properly installed, tested, or inspected to the requirements of the code under which they were installed. If documentation exists for the installation of the anchors, including size, type, embedment depth, and manufacturers' design data, the testing requirements for these anchors may be reduced or eliminated depending on the reliability of the data. Because the specified testing of the anchors is not intended to result in failure of the anchor, the results cannot be used to establish a statistical basis for anchor capacity.

Anchorage testing requirements for cladding panels should be tested similar to the requirements for out-of-plane wall anchorage specified in Chapter 10.

Shear strength is deemed to comply when anchors are tension tested in accordance with this section. Where force demands result in no tension on the anchor or group of anchors, torque testing is permitted in accordance with Section 13.3.2.6 to estimate shear capacity of the anchor.

Although Chapter 13 of ASCE 7 exempts some components from seismic anchorage design requirements based on the assumption that because of either their inherent strength and stability or the low level of seismic demand on the brace, the components are considered adequate to satisfy the desired Performance Objective. This exemption is not intended to allow components to lack a positive anchorage or bracing or to rely on frictional resistance produced by the effects of gravity. Evaluation of anchorage to steel or wood structures may be based on calculated capacities without the need for testing where the size and condition of the anchors can be observed.

C13.3.2.1 Components Evaluated to the Operational Performance Level

C13.3.2.1.2 Concrete or Masonry Anchors Used in the Attachment of Equipment and Other Components Anchors of diameter 1/4 in. (6.4 mm) are considered lightly loaded either because of the small size of the component that they are anchoring or because the calculated seismic overturning demands are predominantly resisted by self-weight. If the calculated demands are low, the consequence of the anchor being installed deficiently would be minimal and would not justify the effort required for testing.

C13.3.2.2 Components Evaluated to the Position Retention or Life Safety Performance Level

C13.3.2.2.1 Concrete or Masonry Anchors Used in the Seismic Bracing of Distributed Systems Anchors of diameter 1/4 in. (6.4 mm) are considered lightly loaded either because of the small size of the component that they are anchoring or because the calculated seismic overturning demands are predominantly resisted by self-weight. If the calculated demands are low, the consequence of the anchor being installed deficiently would be minimal and would not justify the effort required for testing.

C13.3.2.3 Tension Testing Procedure The tension test apparatus should be suitable for the in situ conditions. Tension testing equipment specified in ASTM E488 for determination of strength of anchors in concrete may be used, but caution should be exercised in use of such equipment because the test apparatus may not be accommodated by the on-site conditions.

C13.3.2.5 Alternate Test Criteria These alternate test criteria are similar to the test acceptance criteria in Sections 13.3.2.1 and 13.3.2.2, except that it may be necessary to spot-check adequacy of individual anchors or to establish testing criteria in excess of the requirements in the referenced sections. Testing frequency lower than the specified sample frequency for the desired Performance Objective does not qualify as being adequate to achieve the target Performance Objective.

C13.4 EVALUATION PROCEDURES

C13.4.2 Analytical Procedure For nonstructural components, the analytical procedure, which consists of the default equation and general equation approaches, is applicable to any case. The prescriptive procedure is limited by Table 13-1 to specified combinations of seismicity and component type for compliance with the Life Safety Nonstructural Performance Level.

C13.4.3 Prescriptive Procedure A prescriptive procedure consists of published standards and references that describe the design concepts and construction features that must be present for a given nonstructural component to be seismically protected. No engineering calculations are required in a prescriptive procedure, although in some cases an engineering review of the design and installation is required.

Suggested references for prescriptive requirements are listed in the commentary of the "Component Behavior and Retrofit Methods" subsections of Sections 13.6 through 13.8 for each component type.

C13.4.4.1 Horizontal Seismic Forces The nonstructural force in Equation (13-1) assumes that the story accelerations vary as a triangular distribution over the height of the building. For mid-rise and low-rise buildings, this assumption is generally adequate. For buildings taller than about six stories with periods greater than 1 s, the story accelerations are more uniform over the height of the building, except at the roof level.

The value of x to use in Equation (13-1) can vary depending on the direction of load being considered. An exterior wall panel, for example, may have rigid connections at the base and push–pull connections at the top. For in-plane loading, the point of attachment would be at the bottom of the panel. For out-of-plane loading, the average point of attachment would be halfway between the top connection and the bottom connection.

Seismic forces for nonstructural components are generated based on three effects: the ground acceleration at the base of the building, the ratio of the floor acceleration at the location of the nonstructural component to the ground acceleration, and the dynamic amplification caused by resonance between the nonstructural component and the building response. Equation (13-1) provides an estimate of the horizontal acceleration of a nonstructural component. The peak ground acceleration is calculated as 0.4 times the short-period response acceleration (S_{XS}). The value of S_{XS} should be consistent with the seismic hazard used for the evaluation or retrofit of the structure, however, the value of S_{XS} does not need to be larger than the value used for designing a similar new nonstructural component in the building, because Section 13.1 allows new components in an existing building to be designed using the requirements for similar components for new buildings.

The ratio of the floor acceleration at the location of the nonstructural component is based on a linearly increasing variation of acceleration over the height of the building. The term $(1 + 2x/h)$ is used to calculate this variation based on a linear variation of floor accelerations over the height of the building and is based on an assumed first-mode response of a building with uniform stiffness and mass. For buildings that have significant higher mode response, this linearly increasing assumption may overestimate the acceleration at floors below the roof. A linear dynamic analysis using a response spectrum can be used as an alternate method of estimating the variation of floor accelerations.

The a_p factor provides an estimate of the dynamic amplification caused by the resonance of response of the nonstructural component with one of the modes of vibration of the building. Tables 13-2 and 13-3 have been reproduced from ASCE 7, Chapter 13. These tables provide estimates of this amplification for most nonstructural components. In the referenced tables, components assumed to be rigid are assigned an a_p value of 1, and components assumed to be flexible are assigned an a_p value of 2.5. A period of vibration of 0.06 s, which may be calculated in accordance with Equation (13-4), is used to distinguish between rigid and flexible components. The engineer should verify that the a_p value used is appropriate for the actual component and its support system.

For many buildings, the primary mode of vibration in each direction has the most influence on the dynamic amplification of nonstructural components. For buildings with primary mode periods greater than 1 s, the second or third mode of vibration may also cause some dynamic amplification.

For the Operational Performance Level, where greater accuracy in prediction of floor accelerations can be important, nonlinear dynamic analysis may be preferred.

C13.4.4.3 Load Combination The force to be applied for anchors to concrete and masonry is amplified by the factor Ω_0, similar to the requirements in ASCE 7. This factor is used to provide a factor of safety for anchors, as required in ACI 318, by amplifying the seismic force without a commensurate amplification of the resisting dead load.

C13.4.4.4 Nonstructural Support Capacity

C13.4.4.4.1 Existing Components The design of nonstructural components, bracing for the nonstructural components, and the anchorage of the nonstructural components can require the use of several different design standards. This standard allows the use of applicable design standards for determining the capacity of the nonstructural components, the bracing, and the anchorage. For consistency with the material chapters in this standard, the strength reduction factor, φ, can be taken as 1.0.

C13.4.4.4.2 New Components Where new nonstructural components, bracing, and anchorage are being designed for an existing building, the design should be based on the applicable design standards. This includes designing new anchorage or bracing for an existing nonstructural component. To be consistent with ASCE 7, which is commonly used for the design of new nonstructural components, the strength reduction factors should be based on the applicable design standards and should not be taken as the value of 1.0 that can be used for evaluating existing components.

C13.4.5 Deformation Analysis Where nonstructural components and their attachment are required to accommodate building drifts, either between separate buildings or between different portions of the same building, these components and their attachments should accommodate the calculated displacements without brittle failure or dislodging of the component that would create a falling hazard. The components and their anchorages need not remain elastic for the calculated drifts. An analysis should compare the calculated displacement demands on the component and its attachment with the displacements that can be accommodated by elastic or inelastic deformation, sliding, or another reliable method. The analysis or testing used to demonstrate the acceptable displacement limits should be approved by the Authority Having Jurisdiction.

Where nonstructural components are supported between, rather than at, structural levels, as frequently occurs for glazing systems, partitions, stairs, veneers, and mechanical and electrical distributed systems, the height over which the displacement demand, D_p, must be accommodated may be less than the story height and should be considered carefully. Refer to ASCE 7 commentary for additional description of relative seismic displacement considerations.

C13.5 RETROFIT APPROACHES

A general set of alternate methods for the retrofit of nonstructural components includes replacement, strengthening, repair, bracing, and attachment, as described below. However, the choice of retrofit technique and its design is the responsibility of the design professional, and use of alternative approaches to those noted below or otherwise customarily in use is acceptable, provided that it can be shown to the satisfaction of the building official that the acceptance criteria are met.

For the Life Safety Performance Level, most nonstructural components that are acceleration sensitive should be retrofit considering Position Retention. Nonstructural components that are drift sensitive should be retrofitted to allow for imposed deformation. Nonstructural components that are drift sensitive need not be designed to prevent damage to the nonstructural component or its attachments, provided that stability of the component is maintained. Components that are acceleration sensitive in one direction and drift sensitive in the other direction should be retrofitted considering both effects.

Replacement: Replacement involves the complete removal of the component and its connections and its replacement by new components, for example, the removal of exterior cladding panels, the installation of new connections, and installation of new panels. As with structural components, the installation of new nonstructural components as part of a seismic retrofit project should be the same as for new construction.

Strengthening: Strengthening involves additions to the component to improve its strength to meet the required force levels; for example, additional members might be welded to a support to prevent buckling.

Repair: Repair involves the repair of any damaged parts or members of the component to enable the component to meet its acceptance criteria; for example, some corroded attachments for a precast concrete cladding system might be repaired and replaced without removing or replacing the entire panel system.

Bracing: Bracing involves the addition of members and attachments that brace the component internally or to the building structure. A suspended ceiling system might be retrofitted by the addition of diagonal wire bracing and vertical compression struts.

Attachment: Attachment refers to methods that are primarily mechanical, such as bolting, by which nonstructural components are attached to the structure or other supporting components. Typical attachments are the bolting of items of mechanical equipment to a reinforced concrete floor or base. Supports and attachments for mechanical and electrical equipment should be designed according to accepted engineering principles. The following guidelines are recommended:

1. Attachments and supports transferring seismic loads should be constructed of materials suitable for the application and should be designed and constructed in accordance with a nationally recognized standard.
2. Attachments embedded in concrete should be suitable for cyclic loads.
3. Rod hangers may be considered seismic supports if the length of the hanger from the supporting structure is 12 in. (304.8 mm) or less. Rod hangers should not be constructed in a manner that would subject the rod to bending moments.
4. Seismic supports should be constructed so that support engagement is maintained.
5. Friction clips should not be used for anchorage attachment.
6. Expansion anchors should not be used for mechanical equipment rated over 10 hp, unless undercut expansion anchors are used.
7. Drilled and grouted-in-place anchors for tensile load applications should use either expansive cement or expansive epoxy grout.
8. Supports should be specifically evaluated if weak-axis bending of cold-formed support steel is relied on for the seismic load path.
9. Components mounted on vibration isolation systems should have a bumper restraint or snubber in each horizontal direction.
10. Oversized washers should be used at bolted connections through the base sheet metal if the base is not reinforced with stiffeners.
11. Lighting fixtures resting in a suspended ceiling grid may be retrofitted by adding wires that directly attach the fixtures to the floor above, or to the roof structure, to prevent their falling.

C13.6 ARCHITECTURAL COMPONENTS: DEFINITION, BEHAVIOR, AND ACCEPTANCE CRITERIA

C13.6.1 Exterior Wall Components

C13.6.1.1 Adhered Veneer

C13.6.1.1.1 Definition and Scope Adhered veneers are generally thinner materials, although thicker veneers, especially masonry, stone, and terra-cotta, may be encountered. The behavior of these systems is dominated by the backup system to which the veneer is adhered. Although the behavior of the thicker veneers is still dominated by the behavior of the backup systems, the threat to life safety caused by failure may rise significantly for thicker, heavier veneers because of failures of the substrate bonding the veneer to the backup systems. The height of the veneer and the likely size of falling fragments should be considered.

Tile, masonry, stone, terra-cotta, and similar materials are typically less than 1 in. (24.5 mm) thick. Glass mosaic blocks are typically $2 \times 2 \times 3/8$ in. ($50.8 \times 50.8 \times 9.5$ mm) thick and are a type of adhered veneer. Veneer larger than these blocks likely would require direct attachment to the backup system, as opposed to simply being adhered to it, and thus should be considered anchored veneer and evaluated per Section 13.6.1.2.

C13.6.1.1.2 Component Behavior and Retrofit Methods Adhered veneers are predominantly deformation sensitive. Deformation of the substrate leads to cracking or separation of the veneer from its backing. Poorly adhered veneers may be dislodged by direct acceleration.

Nonconformance requires limiting drift, special detailing to isolate substrate from structure to permit drift, or replacement with drift-tolerant material. Poorly adhered veneers should be replaced.

C13.6.1.1.4 Evaluation Requirements Tapping may indicate either defective bonding to the substrate or excessive flexibility of the supporting structure.

C13.6.1.2 Anchored Veneer

C13.6.1.2.1 Definition and Scope Masonry units are typically 5 in. (127 mm) or less thick. Stone slab units are typically 2 in. (50.8 mm) or less thick.

C13.6.1.2.2 Component Behavior and Retrofit Methods Anchored veneer is both acceleration and deformation sensitive. Heavy units can be dislodged by direct out-of-plane acceleration, which distorts or fractures the mechanical connections. Special attention should be paid to corners and around openings, which are likely to experience large deformations. In-plane or out-of-plane deformations of the supporting structure, particularly if it is a frame, may similarly affect the connections, and the units may be displaced or dislodged by racking. Thick anchored veneer may possess significant in-plane stiffness, which can greatly amplify the demands placed on the connections if the supporting structure racks.

Drift analysis is necessary to establish conformance with drift acceptance criteria related to Performance Level. The drift analysis should consider the construction and behavior of the veneer and its backing to assess the individual parts of the nonstructural component that are required to deform to accommodate the required drift. These parts of the nonstructural component should be checked for their capability of allowing for the calculated deformation of the structure. Nonconformance requires limiting structural drift, or special detailing to isolate substrate from structure to permit drift. Defective connections must be replaced.

C13.6.1.2.3 Acceptance Criteria As an alternative to the drift limits in Section 13.6.1.2.3, the nonstructural component and its backing can be shown by approved testing or analysis to meet the intended Performance Level for the calculated drift.

C13.6.1.3 Glass Block Units and Other Nonstructural Masonry

C13.6.1.3.2 Component Behavior and Retrofit Methods Glass block and nonstructural masonry are both acceleration and deformation sensitive. Failure in plane generally occurs by deformation in the surrounding structure that results in unit cracking and displacement along the cracks. Failure out of plane takes the form of dislodgment or collapse caused by direct acceleration.

Nonconformance with deformation criteria requires limiting structural drift or special detailing to isolate the glass block wall from the surrounding structure to permit the required drift. The drift analysis should consider the construction and behavior of the veneer and its backing to assess the individual parts of the nonstructural component that are required to deform to accommodate the required drift. These parts of the nonstructural component should be checked for their capability of allowing for the calculated deformation of the structure. Sufficient reinforcing must be provided to deal with out-of-plane forces. Large walls may need to be subdivided by additional structural supports into smaller areas that can meet the drift or force criteria.

C13.6.1.4 Prefabricated Panels

C13.6.1.4.1 Definition and Scope Prefabricated panels are generally attached at discrete locations around their perimeters to the structural framing with mechanical connections.

C13.6.1.4.2 Component Behavior and Retrofit Methods Lightweight panels may be damaged by racking; heavy panels may be dislodged by direct acceleration, which distorts or fractures the mechanical connections. The imposed in-plane and out-of-plane deformations are generally accommodated by the connections and not by the prefabricated panels. These connections need to be checked for the detailing to accommodate the required drift. This check is generally accomplished by a connection detailed to allow sliding with a slotted or oversize hole. Drift can also be accommodated by deformation of the connections.

Excessive deformation of the supporting structure—most likely if it is a frame—may result in the panels imposing external racking forces on one another and distorting or fracturing their connections, with consequent displacement or dislodgment.

Drift analysis is necessary to establish conformance with drift acceptance criteria related to the Nonstructural Performance Level. The drift analysis should consider the construction and behavior of the panel and its connections to assess the individual parts of the nonstructural component that are required to deform to accommodate the required drift.

Nonconformance requires limiting structural drift, or special detailing to isolate panels from the structure to permit the required drift; this method generally requires panel removal. Defective connections must be replaced.

C13.6.1.5 Glazed Exterior Wall Systems

C13.6.1.5.1 Definition and Scope The following types of glass are used within each of the glazed exterior wall systems:

1. Annealed glass,
2. Heat-strengthened glass,
3. Fully tempered glass,
4. Laminated glass, and
5. Sealed insulating glass units.

The use of some of these glass types is regulated in building codes.

There are two glazing methods for installing glass in glazed curtain wall and glazed storefront systems:

1. Wet glazing, which can use three types of materials:
   1.1. Preformed tape;
   1.2. Gunable elastomeric sealants
      (a) Noncuring and
      (b) Curing; and
   1.3. Putty and glazing compounds.
2. Dry glazing, which uses extruded rubber gaskets as one or both of the glazing seals.

C13.6.1.5.2 Component Behavior and Retrofit Methods Glazed exterior wall systems are predominantly deformation sensitive but may also become displaced or detached by large acceleration forces. Glass components within glazed exterior wall systems are deformation sensitive. Glass performance during earthquakes, which is a function of the wall system type, glazing type, and glass type, falls into one of four categories:

1. Glass remains unbroken in its frame or anchorage;
2. Glass shatters but remains in its frame or anchorage while continuing to provide a weather barrier and remains otherwise serviceable;
3. Glass shatters and remains in its frame or anchorage in a precarious condition, liable to fall out at any time; or
4. Glass falls out of its frame or anchorage, either in fragments, shards, or whole panels.

Drift analysis and testing or compliance with prescriptive procedures are necessary to establish conformance with drift acceptance criteria related to Performance Level. Nonconformance requires limiting structural drift, or special detailing to isolate the glazing system from the structure to accommodate drift, or selection of a glass type that shatters safely or remains in the frame when shattered. This option would require removal of the glass or glazed wall system and replacement with an alternative design.

C13.6.1.5.3 Acceptance Criteria D_{clear} in Equation (13-10) is derived from a similar equation in Bouwkamp and Meehan (1960) that permits calculation of the story drift required to cause glass-to-frame contact in a given rectangular window frame. Both equations are based on the principle that a rectangular window frame (specifically one that is anchored mechanically to adjacent stories of the primary structural system of the building) becomes a parallelogram as a result of story drift, and that glass-to-frame contact occurs when the length of the shorter diagonal of the parallelogram is equal to the diagonal of the glass panel itself.

The 1.25 factor in Equations (13-12) and (13-13) reflects uncertainties associated with calculated inelastic seismic displacements in building structures. Wright (1989) stated that "post-elastic deformations calculated using the structural analysis process may well underestimate the actual building deformation by up to 30%. It would therefore be reasonable to require the curtain wall glazing system to withstand 1.25 times the computed maximum story displacement to verify adequate performance." Wright's comments form the basis for using the 1.25 factor.

C13.6.1.5.4 Evaluation Requirements Alternatively, to establish compliance with Criterion 1.4 or 2.3, glazed exterior wall systems may be tested in accordance with AAMA 501.4 (AAMA 2015).

C13.6.2 Partitions

C13.6.2.1 Definition and Scope Definitions such as light and heavy partitions are somewhat subjective, which is why examples are given such as masonry for heavy partitions and wood with lath and plaster for light. However, the user should make the determination of whether the partition is actually light or heavy. For example, a hollow-clay tile wall can weigh about 25 lb/ft^2 (1.2 kN/m^2) and would be considered heavy. A stud wall with cement plaster on both sides can weigh about 22 lb/ft^2 (1.05 kN/m^2). For the latter case, the user should assess the consequence of failure of the partition and whether it constitutes a Life Safety Hazard and if so, treat it as a heavy partition.

Heavy partitions include hollow-clay tile or concrete block. Only non-load-bearing partitions are considered in this section. Structural partitions, including heavy masonry partitions, shall be retrofitted in accordance with Chapter 11.

Partitions may span laterally from floor to underside of the floor or roof above, with connections at the top that may or may not allow for isolation from in-plane drift. Other partitions extend only up to a hung ceiling and may or may not have lateral bracing above that level to structural support or may be freestanding.

Modular office furnishings that include movable partitions are considered as contents rather than partitions, and as such are not within the scope of this standard.

C13.6.2.2 Component Behavior and Retrofit Methods Partitions attached to the structural floors both above and below, and loaded in plane, can experience shear cracking, distortion, and fracture of the partition framing and detachment of the surface finish because of structural deformations. Similar partitions loaded out of plane can experience flexural cracking, failure of connections to structure, and collapse. The high incidence of unsupported block partitions in low and moderate seismic levels represents a significant collapse threat.

Partitions subject to deformations from the structure can be protected by providing a continuous gap between the partition and the surrounding structure, combined with attachment that provides for in-plane movement but out-of-plane restraint. Lightweight partitions that are not part of a fire-resistive system are regarded as replaceable.

C13.6.2.4 Evaluation Requirements For concrete block partitions, presence of reinforcing and connection conditions at edges is important. For light partitions, bracing or anchoring of the top of the partitions is important.

C13.6.3 Interior Veneers

C13.6.3.2 Component Behavior and Retrofit Methods Interior veneers typically experience in-plane cracking and detachment but may also be displaced or detached out of plane by direct acceleration. Interior partitions loaded out of plane and supported on flexible backup support systems can experience cracking and detachment.

Drift analysis is necessary to establish conformance with drift acceptance criteria related to the Nonstructural Performance Level. Nonconformance requires limiting structural drift or special detailing to isolate the veneer support system from the structure to permit drift; this isolation generally requires disassembly of the support system and veneer replacement. Inadequately adhered veneer must be replaced.

C13.6.4 Ceilings

C13.6.4.1 Definition and Scope Furring materials include wood or metal furring, acoustical tile, gypsum board, plaster, or metal panel ceiling materials.

Some older buildings have heavy decorative ceilings of molded plaster, which may be directly attached to the structure or suspended; these are typically Category a or Category c ceilings.

C13.6.4.2 Component Behavior and Retrofit Methods Surface-applied or furred ceilings are primarily influenced by the performance of their supports. Retrofit of the ceiling takes the form of ensuring good attachment and adhesion. Metal lath and plaster ceilings depend on their attachment and bracing for large ceiling areas. Analysis is necessary to establish the acceleration forces and deformations that must be accommodated. Suspended integrated ceilings are highly susceptible to damage if not braced, causing distortion of grid and loss of panels; however, this is not regarded as a Life Safety threat with lightweight panels [less than 2 lb/ft^2 (0.1 kN/m^2)].

Retrofit takes the form of bracing, attachment, and edge details designed to prescriptive design standards such as CISCA (1991) for seismic hazard levels 0 to 2 and CISCA (1990) for seismic hazard levels 3 and 4.

C13.6.5 Parapets and Cornices

C13.6.5.1 Definition and Scope Other appendages, such as flagpoles and signs that are similar to the aforementioned in size, weight, or potential consequence of failure, may be retrofitted in accordance with this section.

C13.6.5.2 Component Behavior and Retrofit Methods Materials or components that are not properly braced may become disengaged and topple; the results are among the most seismically serious consequences of any nonstructural components.

Prescriptive design strategies for masonry parapets not exceeding 4 ft (1.2 m) high consist of bracing in accordance with the concepts shown in FEMA 172 (NEHRP 1996) and FEMA E-74 (FEMA 2011), with detailing to conform to accepted engineering practice. Braces for parapets should be spaced at a maximum of 8 ft (2.4 m) on center and, where the parapet construction is discontinuous, a continuous backing component should be provided. Where there is no adequate connection, roof construction should be tied to parapet walls at the roof level. Other parapets and appendages should be analyzed for acceleration forces and should be braced and connected according to accepted engineering principles.

C13.6.6 Architectural Appendages and Marquees

C13.6.6.1 Definition and Scope Canopies and marquees are generally used to provide weather protection.

Marquees are often constructed of metal or glass.

C13.6.6.2 Component Behavior and Retrofit Methods The variety of design of canopies and marquees is so great that they must be independently analyzed and evaluated for their ability to withstand seismic forces. Retrofit may take the form of improving attachment to the building structure, strengthening, bracing, or a combination of measures.

C13.6.9 Chimneys and Stacks

C13.6.9.2 Component Behavior and Retrofit Methods Chimneys and stacks may fail through flexure, shear, or overturning. They may also disengage from adjoining floor or roof structures and damage them, and their collapse or overturning may also damage adjoining structures. Retrofit may take the form of strengthening and/or bracing and material repair. Residential chimneys may be braced in accordance with the concepts shown in FEMA E-74 (FEMA 2011).

C13.6.10 Stairs and Ramps

C13.6.10.1 Definition and Scope Where stairs or ramps are provided with sliding or ductile connections, they can be considered nonstructural components, and their effects on the overall response of the structure can be ignored; however, the connections should be checked for the capability of accommodating the imposed displacements without failure of the connections. Where the stairs or ramps are rigidly connected to the structure, they may provide lateral stiffness to the structure; therefore, the effects of these elements should be considered in the lateral force analysis. Refer to Section 7.2.3.3 for requirements for modeling of nonstructural components as structural components.

C13.6.10.2 Component Behavior and Retrofit Methods The stairs themselves may be independent of the structure or integral with the structure. If integral, they should form part of the overall structural evaluation and analysis, with particular attention paid to the possibility of response modification caused by localized stiffness. If independent, the stairs must be evaluated for normal stair loads and their ability to withstand direct acceleration or loads transmitted from the structure through connections.

Stair enclosure materials may fall and render the stairs unusable because of debris.

Retrofit of integral or independent stairs may take the form of necessary structural strengthening or bracing or the introduction of connection details to eliminate or reduce interaction between stairs and the building structure.

Retrofit of enclosing walls or glazing should follow the requirements of the relevant sections of this document.

C13.6.11 Doors Required for Emergency Services Egress in Essential Facilities

C13.6.11.1 Definition and Scope Door systems in essential facilities, such as fire stations or other structures necessary for emergency operations, can become jammed or otherwise inoperable because of building movements and racking of door openings and can subsequently delay emergency response after an earthquake. Recent reports (Bello et al. 2006) have documented the vulnerability of fire station garage doors in past earthquakes and have made recommendations for how this risk should be addressed.

C13.6.12 Computer Access Floors

C13.6.12.1 Definition and Scope Access floors vary in height but generally are less than 3 ft (0.9 m) above the supporting structural floor. The systems include structural legs, horizontal panel supports, and panels.

C13.6.12.2 Component Behavior and Retrofit Methods Computer access floors may displace laterally or buckle vertically under seismic loads. Retrofit of access floors usually includes a combination of improved attachment of computer and communication racks through the access floor panels to the supporting steel structure or to the underlying floor system, while improving the seismic-force-resisting capacity of the steel stanchion system by installing braces or improving the connection of the stanchion base to the supporting floor, or both.

Retrofit should be designed in accordance with concepts described in FEMA E-74 (FEMA 2011). The weight of the floor system and supported equipment should be included in the analysis.

C13.6.12.4 Evaluation Requirements Possible future equipment should also be considered in the evaluation.

C13.7 MECHANICAL, ELECTRICAL, AND PLUMBING COMPONENTS: DEFINITION, BEHAVIOR, AND ACCEPTANCE CRITERIA

C13.7.1 Mechanical Equipment

C13.7.1.1 Definition and Scope Equipment such as manufacturing or processing equipment related to the occupant's business should be evaluated separately for the effects that failure caused by a seismic event could have on the operation of the building.

C13.7.1.2 Component Behavior and Retrofit Methods The provisions of Section 13.7 focus on Position Retention, which is a primary consideration for the Life Safety Performance Level.

At the Operational Performance Level, Position Retention alone may be insufficient to ensure conformance with the stated goals of that performance level. The expectation is that whereas some nonstructural damage is expected, the building is expected to function after the earthquake, provided that utilities are available. To achieve this level of functionality, the designer must consider the essential post-earthquake functions of the building and then identify those mechanical, electrical, and plumbing components that must operate for the building to function. Components may be identified as critical (components that must be functional) and noncritical (those components for which function after an earthquake is desirable but not essential to the continued occupancy of the building). For critical components for which operability is vital, the commentary of Section 13.2.2 of ASCE 7 provides guidance for seismically qualifying the component.

Position Retention failure of components consists of sliding, tilting, or overturning of floor- or roof-mounted equipment off its base, possible loss of attachment (with consequent falling) for equipment attached to a vertical structure or suspended, and failure of piping or electrical wiring connected to the equipment.

Construction of mechanical equipment to nationally recognized codes and standards, such as those approved by the American National Standards Institute, provides adequate strength to accommodate all normal and upset operating loads.

For Position Retention, basic retrofit consists of securely anchoring floor-mounted equipment by bolting, with detailing appropriate to the base construction of the equipment. ASHRAE RP-812 (ASRAE 1999) provides more information on designing and detailing seismic anchorage.

Function and operability of mechanical and electrical components is affected only indirectly by increasing design forces. However, on the basis of past earthquake experience, it may be reasonable to conclude that if structural integrity and stability are maintained, function and operability after an earthquake will be provided for many types of equipment components. For complex components, testing or experience may be the only reasonable way to improve the assurance of function and operability. Testing is a well-established alternative method of seismic qualification for small to medium-size equipment. Several national standards have testing requirements adaptable for seismic qualification.

Existing attachments for attached or suspended equipment must be evaluated for seismic load capacity and must be strengthened or braced as necessary. Attachments that provide secure anchoring eliminate or reduce the likelihood of piping or electrical distribution failure.

C13.7.1.4 Evaluation Requirements Existing concrete anchors may have to be tested by applying torque to the nuts to confirm that adequate strength is present.

C13.7.2 Storage Vessels and Water Heaters

C13.7.2.1 Definition and Scope The vessel may be fabricated of materials such as steel, other metals, or fiberglass, or it may be a glass-lined tank. These requirements may also be applied, with judgment, to vessels that contain solids that act as a fluid, and vessels containing fluids not involved in the operation of the building.

C13.7.2.2 Component Behavior and Retrofit Methods Category 1 vessels fail by stretching of anchor bolts, buckling and disconnection of supports, and consequent tilting or overturning of the vessel. A Category 2 vessel may be displaced from its foundation, or its shell may fail by yielding near the bottom, creating a visible bulge or possible leakage. Displacement of both types of vessel may cause rupturing of connecting piping and leakage.

Category 1 residential water heaters with a capacity no greater than 100 gal. (379.5 L), may be retrofitted by prescriptive design methods, such as concepts described in FEMA 172 (1992a) or FEMA E-74 (2011). Category 1 vessels with a capacity less than 1,000 gal. (3,785.4 L) should be designed to meet the force provisions of Section 13.4.3 and may be brace-strengthened or added as necessary. Other Category 1 and 2 vessels should be evaluated against a recognized standard, such as API 650 (API 1998), for vessels containing petroleum products or other chemicals, or AWWA D100-96 (AWWA 1996) for water vessels. ASHRAE RP-812 (ASHRAE 1999) provides more information on designing and detailing seismic anchorage and bracing.

C13.7.2.4 Evaluation Requirements Existing concrete anchors may have to be tested by applying torque to the nuts to confirm that adequate strength is present.

C13.7.3 Pressure Piping

C13.7.3.2 Component Behavior and Retrofit Methods Appendix Chapter 6 of the 2003 NEHRP Provisions FEMA 450 (FEMA 2004) provides preliminary criteria for the establishment of such performance criteria and their use in the assessment and design of piping systems. The performance criteria, from least restrictive to most severe, are Position Retention, leak tightness, and operability. In particular, the interaction of systems and interface with the relevant piping design standards is addressed. For the Life Safety Performance Level, the focus is on Position Retention, which is defined as the condition of a piping system characterized by the absence of collapse or fall of any part of the system.

For the Position Retention Nonstructural Performance Level, leak tightness, the condition of a piping system characterized by containment of contents or maintenance of a vacuum with no discernible leakage, is required. Operability, the condition of a piping system characterized by leak tightness and continued delivery and shutoff or throttle of pipe contents flow by means of unimpaired operation of equipment and components such as pumps, compressors, and valves, is desirable, but it requires a significantly higher level of effort to achieve.

The most common failure of piping is joint failure, caused by inadequate support or bracing.

Retrofit is accomplished by prescriptive design approaches to support and bracing. Piping systems should be evaluated for compliance with consensus standards, such as ASME B31, B31.1 (2001a), B31.3 (2002b), B31.4 (2002a), B31.5 (2001b), B31.8 (2000b), B31.9 (2000c), and B31.11 (2002c), and ASHRAE RP-812 (ASHRAE 1999) where applicable. For large critical piping systems, the building official or responsible engineer must establish forces and evaluate supports. ASHRAE RP-812 provides more information on designing and detailing seismic bracing.

C13.7.3.4 Evaluation Requirements High-pressure piping may be tested in accordance with ASME B31.9 (2000a).

C13.7.4 Fire Suppression Piping

C13.7.4.2 Component Behavior and Retrofit Methods The most common failure of fire suppression piping is joint failure, caused by inadequate support or bracing, or by sprinkler heads impacting adjoining materials.

Retrofit is accomplished by prescriptive design approaches to support and bracing. The prescriptive requirements of NFPA 13 should be used.

C13.7.4.3 Acceptance Criteria Past performance of fire suppression piping in essential facilities has shown inadequate performance when these systems are prescriptively designed and installed. The Olive View Hospital, for example, experienced broken piping and water leakage during the 1994 Northridge Earthquake that led to the evacuation of patients. The prescriptive requirements of NFPA 13 (NFPA 2019) are not permitted to be used to establish that fire suppression piping meets Operational Nonstructural Performance; the analytical procedure should be used.

C13.7.4.4 Evaluation Requirements The support and bracing of bends of the main risers and laterals, and maintenance of adequate flexibility to prevent buckling, are especially important.

C13.7.5 Fluid Piping Other Than Fire Suppression

C13.7.5.1 Definition and Scope Hazardous materials and flammable liquids that would pose an immediate Life Safety danger if exposed are defined in NFPA 49 (1994b), NFPA 325 (1994a), NFPA 491 (1997), and NFPA 704 (2012).

C13.7.5.2 Component Behavior and Retrofit Methods The most common failure is joint failure, caused by inadequate support or bracing.

Category 1 piping retrofit is accomplished by strengthening support and bracing, using the prescriptive methods of MSS SP-58 (ANSI/MSS 2018). The piping systems themselves should be designed to meet the force provisions of Section 13.4.3 and the relative displacement provisions of Section 13.4.4. The effects of temperature differences, dynamic fluid forces, and piping contents should be taken into account.

Category 2 piping retrofit is accomplished by strengthening support and bracing using the prescriptive methods of MSS SP-58 (ANSI/MSS 2018) as long as the piping falls within the size limitations of those guidelines. Piping that exceeds the limitations of those guidelines shall be designed to meet the force provisions of Section 13.4.3 and the relative displacement provisions of Section 13.4.4.

More information on designing and detailing seismic bracing can be found in ASHRAE RP-812 (ASHRAE 1999).

C13.7.5.4 Evaluation Requirements The support and bracing of bends in the main risers and laterals, and maintenance of adequate flexibility to prevent buckling, are especially important.

C13.7.6 Ductwork

C13.7.6.2 Component Behavior and Retrofit Methods Damage to ductwork is caused by failure of supports or lack of bracing that causes deformation or rupture of the ducts at joints, leading to leakage from the system.

Retrofit consists of strengthening supports and strengthening or adding bracing. Prescriptive design methods may be used in accordance with ANSI/SMACNA 001 (ANSI/SMACNA 2008). More information on designing and detailing seismic bracing can be found in ASHRAE RP-812 (ASHRAE 1999).

Retrofit may be accomplished by strengthening support and bracing using the prescriptive methods contained in SMACNA's *Rectangular Industrial Duct Construction Standards* (2004) and *HVAC Duct Construction Standards—Metal and Flexible* (2005).

C13.7.7 Electrical and Communications Equipment

C13.7.7.2 Component Behavior and Retrofit Methods Failure of these components consists of sliding, tilting, or overturning of floor- or roof-mounted equipment off its base; possible loss of attachment (with consequent falling) for equipment attached to a vertical structure or suspended; and failure of electrical wiring connected to the equipment.

Construction of electrical equipment to nationally recognized codes and standards, such as those approved by the American National Standards Institute (ANSI), provides adequate strength to accommodate all normal and upset operating loads.

Basic retrofit consists of securely anchoring floor-mounted equipment by bolting, with detailing appropriate to the base construction of the equipment.

C13.7.7.4 Evaluation Requirements Larger equipment requiring the analytical procedure must be analyzed to determine forces and must be visually evaluated. Concrete anchors may have to be tested by applying torque to the nuts to confirm that adequate strength is present.

C13.7.8 Electrical and Communications Distribution Components

C13.7.8.2 Component Behavior and Retrofit Methods Failure occurs most commonly by inadequate support or bracing, deformation of the attached structure, or impact from adjoining materials.

C13.7.9 Light Fixtures

C13.7.9.2 Component Behavior and Retrofit Methods Failure of Categories 1 and 2 components occurs through failure of attachment of the light fixture and/or failure of the supporting ceiling or wall. Failure of Category 3 components occurs through loss of support from the T-bar system and by distortion caused by deformation of the supporting structure or deformation of the ceiling grid system, allowing the fixture to fall. Failure of Category 4 components is caused by excessive swinging, which results in the pendant or chain support breaking on impact with adjacent materials or the support being pulled out of the ceiling.

Retrofit of Categories 1 and 2 components involves attachment upgrade or fixture replacement in association with necessary retrofit of the supporting ceiling or wall. Retrofit of Category 3 components involves the addition of independent support for the fixture from the structure or substructure in accordance with FEMA E-74 (FEMA 2011) design concepts. Retrofit of Category 4 components involves strengthening of attachment and ensuring freedom to swing without impacting adjoining materials.

C13.7.10 Rooftop Solar Photovoltaic Arrays

C13.7.10.1 Definition and Scope The evaluation of PV arrays in this section applies only to the seismic evaluation of an existing PV array and its anchorage. New installation should be designed in accordance with the governing code of the jurisdiction, or if no code exists, ASCE 7. A separate evaluation for wind loads may also be necessary.

C13.7.10.2 Component Behavior and Retrofit Methods A methodology for the evaluation of ballasted PV arrays was introduced in ASCE 7, based on work by the Structural Engineers Association of California (SEAOC 2012). The ballasted systems are allowed to slide on the roof and are therefore treated as deformation sensitive. A PV array that is anchored to the roof framing is considered acceleration sensitive since the anchorage is required to resist the seismic forces generated by the PV array.

C13.7.10.4 Evaluation Requirements The evaluation of the PV array should include the entire system, including anchors and support framing.

C13.7.11 Elevators

C13.7.11.2 Component Behavior and Retrofit Methods Components of elevators may become dislodged or derailed. Shaft walls and the construction of machinery room walls are often not engineered and must be considered in a way similar to that for other partitions. Shaft walls that are of unreinforced masonry or hollow tile must be considered with special care because failure of these components violates Life Safety Nonstructural Performance Level criteria.

Elevator machinery may be subject to the same damage as other heavy floor-mounted equipment. Electrical power loss renders elevators inoperable.

Retrofit measures include a variety of techniques taken from specific component sections for partitions, controllers, and machinery. Retrofit specific to elevator operation can include seismic shutoffs, cable restrainers, and counterweight retainers; such measures should be in accordance with ASME A17.1 (ASME 2000c).

C13.7.11.4 Evaluation Requirements The possibility of displacement or derailment of hoistway counterweights and cables should be considered, as should the anchorage of elevator machinery.

C13.7.12 Conveyors

C13.7.12.2 Component Behavior and Retrofit Methods Conveyor machinery may be subject to the same damage as other heavy floor-mounted equipment. In addition, deformation of adjoining building materials may render the conveyor inoperable. Electrical power loss renders the conveyor inoperable.

Retrofit of the conveyor involves prescriptive procedures using special skills provided by the conveyor manufacturer.

C13.8 FURNISHINGS AND CONTENTS: DEFINITION, BEHAVIOR, AND ACCEPTANCE CRITERIA

C13.8.1 Steel Storage Racks

C13.8.1.1 Definition and Scope Storage racks are usually constructed of cold-formed or hot-rolled steel with one or more levels of framing to support contents, including pallet storage racks, movable-shelf racks, rack-supported systems, automated storage and retrieval systems (stacker racks), push-back racks, pallet-flow racks, pick modules, and rack-supported platforms. Other types of racks, such as drive-in or drive-through racks, cantilever racks, portable racks, or racks made of materials other than steel are not considered storage racks for the purpose of this standard. Steel storage racks are generally purchased as proprietary systems installed by a tenant and are often not under the direct control of the building owner. Thus, they are usually not part of the construction contract and often have no foundation or foundation attachment. However, they are often permanently installed, and their size and loaded weight make them an important hazard to life, property, or the surrounding structure. Although typically supported at the ground level, steel storage racks may also be located on elevated floors. Steel storage racks located in occupied locations shall be considered where the Life Safety Nonstructural Performance Level is selected. Steel storage racks less than 8 ft (2.4 m) tall may be considered to be contents and evaluated using section 13.8.2.

C13.8.1.2 Component Behavior and Retrofit Methods Steel storage racks may fail internally, through inadequate bracing or moment-resisting capacity, or externally, by overturning caused by absence or failure of foundation attachments.

Retrofit is usually accomplished by the addition of bracing to the rear and side panels of racks and/or by improving the connection of the rack columns to the supporting slab. In rare instances, foundation improvements may be required to remedy insufficient bearing or uplift load capacity.

Seismic forces can be established by analysis in accordance with Section 13.4.3. However, special attention should be paid to the evaluation and analysis of large, heavily loaded rack systems because of their heavy loading and lightweight structural members.

C13.8.2 Contents

C13.8.2.1 Definition and Scope Contents that are taller than 4 ft (1.2 m) can be a hazard if they overturn during an earthquake. Contents that have a height-to-width ratio less than 2 are less prone to overturning and need not be evaluated unless located in an area that may result in injury to occupants, such as in areas with small children, or in areas where the contents may impact other nonstructural components.

C13.8.2.2 Component Behavior and Retrofit Methods Bookcases may deform or overturn because of inadequate bracing or attachment to floors or adjacent walls, columns, or other structural members. Retrofit is usually accomplished by adding metal cross bracing to the rear of the bookcase, to improve its internal resistance to racking forces, and by bracing the bookcase both in and out of plane to the adjacent structure or walls to prevent overturning and racking.

C13.8.3 Hazardous Material Storage

C13.8.3.2 Component Behavior and Retrofit Methods Upset of the storage container may release the hazardous material. Failure occurs because of buckling and overturning of supports and/or inadequate bracing. Retrofit consists of strengthening and increasing supports or adding bracing designed according to concepts described in FEMA 172 (FEMA 1992b) and FEMA E-74 (FEMA 2011).

C13.8.4 Computer and Communication Racks

C13.8.4.1 Definition and Scope Racks may be supported on either structural or access floors and may or may not be attached directly to these supports.

C13.8.4.2 Component Behavior and Retrofit Methods Computer and communication racks may fail internally, through inadequate bracing or moment-resisting capacityor externally by overturning caused by absence or failure of floor attachments.

Retrofit is usually accomplished by the addition of bracing to the rear and side panels of the racks and/or by improving the connection of the rack to the supporting floor using concepts shown in FEMA 172 (FEMA 1992b) or FEMA E-74 (FEMA 2011).

CHAPTER C14
SEISMIC ISOLATION

C14.1 SCOPE

The basic form and formulation of requirements for seismic isolation systems have been established and coordinated with the Performance Objectives, target Building Performance Levels, and Seismic Hazard Level criteria of Chapter 2 and the linear and nonlinear procedures of Chapter 7.

Seismic isolation systems may be used for buildings that have only a Limited Performance Objective or Partial Retrofit. However, an additional requirement of this chapter is evaluation and retrofit to at least the BSE-2E hazard level for select provisions. This is required because introduction of a seismic isolation system can fundamentally change the collapse mode of an existing building when the displacement capacity of the seismic isolation system is exceeded. Evaluation and retrofit to at least the BSE-2E hazard level for select provisions provides assurance that the retrofitted building with the seismic isolation system performs no worse than the existing, fixed-base building for hazard levels beyond that considered for the Limited Performance Objective or Partial Retrofit and up to the BSE-2E.

In most cases, seismic isolation systems are implemented with additional conventional strengthening of the building; in all cases, they require evaluation of existing structural and nonstructural components. As such, this chapter supplements and amends the requirements of other chapters with additional criteria and methods of analysis appropriate for buildings retrofitted with a seismic isolation system.

C14.2 GENERAL REQUIREMENTS

C14.2.2 Seismic Hazard

C14.2.2.1 Ground Motion Acceleration Histories Development of ground motion acceleration histories for the evaluation and retrofit of seismically isolated buildings using the nonlinear dynamic procedure generally follows the requirements for fixed-base buildings. However, the period range of interest differs for seismically isolated versus fixed-base buildings. The lower-bound period range of interest defined in these provisions, while suitable for estimating isolation system displacement and building base shear, may not be sufficiently low to capture higher modes of response, which have a strong effect on floor spectra. Where calculation of floor spectra from the nonlinear response history analysis is an objective (e.g., for determining nonstructural component demands in Chapter 13), it may be necessary to reduce the lower-bound period range of interest to include these higher modes.

C14.2.3 Isolation System

C14.2.3.1 Environmental Conditions Evaluation and design for environmental conditions is required by this section to ensure that the isolation system can remain effective for the design life of the building. Where environmental conditions affect the properties of isolators, Section 14.3.3.3 can be used to quantify their effect.

C14.2.3.2 Wind Displacement Although existing buildings are typically not required to be evaluated for wind forces by this standard, when a fixed-base building is retrofitted with a seismic isolation system, it is necessary to perform an evaluation of wind displacements at the isolation interface. Wind displacement across the isolation interface is primarily a serviceability concern. While the limit provided in these provisions has some historical precedence, the design professional may consider evaluating the wind serviceability movements for an isolated building if such displacements are expected to be important in the operation or to the occupants of the building. Wind serviceability may be evaluated in accordance with the commentary to Appendix C of ASCE 7.

C14.2.3.3 Fire Resistance Where fire may adversely affect the lateral performance of the isolation system, the system must be protected to maintain the gravity load resistance and stability required for the other elements of the superstructure supported by the isolation system.

C14.2.3.4 Lateral Restoring Force The restoring force requirement is intended to limit residual displacements in the isolation system resulting from any earthquake event so that the isolated building will adequately withstand aftershocks and future earthquakes. The potential for residual displacements is discussed in Section C14.2.5.3.

C14.2.3.5 Displacement Restraint The use of a displacement restraint to limit displacements beyond the total displacement for the largest hazard level considered is discouraged. Where a displacement restraint system is used, the nonlinear dynamic procedure is required to account for the effects of engaging the displacement restraint.

C14.2.3.6 Vertical Load Stability The vertical loads used to assess the stability of a given isolator should be calculated using bounding values of dead load, live load, and the peak earthquake demand. Because earthquake loads are reversible in nature, peak earthquake load should be combined with bounding values of dead and live load in a manner that produces both the maximum downward force and the maximum upward force on any isolator. Stability of each isolator should be verified for these two extreme values of vertical load at D_{TX} of the isolation system. In addition, all elements of the isolation system require testing or equivalent measures that demonstrate their stability for the BSE-2X ground motion levels.

C14.2.3.7 Overturning The intent of this requirement is to prevent both global structural overturning and overstress of elements caused by localized uplift. Isolator uplift is acceptable as long as the isolation system does not disengage from its horizontal-resisting connection details. The connection details used in certain isolation systems do not develop tension resistance, a condition that should be accounted for in the analysis and design. Where the tension capacity of an isolator is used to resist uplift forces, design and testing in accordance with Section 14.6.3.4 can be used to demonstrate the adequacy of the system to resist tension forces under the demands from the largest hazard level considered.

C14.2.3.8 Inspection and Replacement Although most isolation systems do not require replacement following an earthquake event, access for inspection, repair, and replacement are required by these provisions. In some cases (Section 14.2.3.4), recentering may be required. The isolation system should be inspected periodically as well as after significant earthquake events, and any damaged elements should be repaired or replaced.

C14.2.4 Structural System

C14.2.4.2 Minimum Separations A minimum separation between the isolated building and rigid obstructions is needed to allow unrestricted horizontal translation of the isolation system in all directions during an earthquake event. A minimum separation between the isolated building and other structures is needed to prevent building pounding.

C14.2.5 Elements of Structures and Nonstructural Components

C14.2.5.2 Components at or above the Isolation Interface Where floor response spectra are available from the nonlinear dynamic procedure, they can be used to design components at or above the isolation interface. However, an exception exists, which allows provisions that neglect the isolating effect to be adopted from Chapter 13. This exception is available regardless of the analysis procedure used in this chapter.

C14.2.5.3 Components Crossing the Isolation Interface To accommodate the differential horizontal and vertical movement between the isolated building and the ground, flexible utility connections are needed. In addition, other elements crossing the isolation interface (such as stairs, elevator shafts, and walls) must be detailed to accommodate the total maximum displacement. This is typically accomplished by hanging stairs and elevators from the level above the isolation interface, thus eliminating their connection below the isolation interface, or by providing small isolators specifically at the stair and elevator connections below the isolation interface. Where non-negligible permanent residual displacement across the isolation plane is expected (e.g., isolation systems exhibiting a high yield force and low post-yield stiffness subjected to large displacements), certain elements crossing the isolation interface (e.g., sewer, water and fire suppression pipes, electrical conduit, etc.) need to perform following an earthquake where those elements are necessary to maintain building functionality and where building functionality prior to significant repair is an objective.

Recent full-scale shake table tests (Ryan et al. 2012) and analytical studies (Katsaras et al. 2006) have shown that isolation systems that possess longer periods, relatively high yield and friction levels, and small yield displacements will result in post-earthquake residual displacements. In these studies, residual displacements ranging from 2 to 6 in. (51 to 152 mm) were measured and computed for isolated building structures with a period of 4 s or more and a yield level in the range of 8% to 15% of the structure's weight. This permanent offset may affect the serviceability of the structure and possibly jeopardize the functionality of elements crossing the isolation plane (e.g., fire-protection and weatherproofing elements, egress and entrance details, elevators, and joints of primary piping systems). Because it may not be possible to recenter some isolation systems, isolated structures with such characteristics should be detailed to accommodate these permanent offsets.

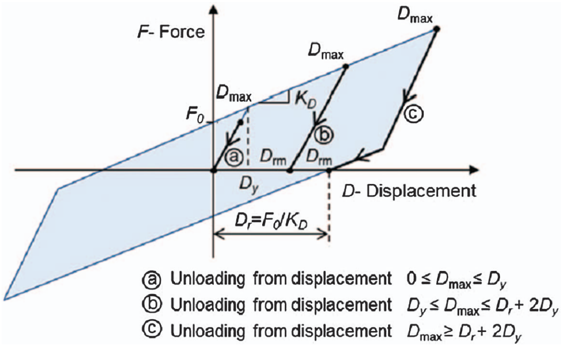


Figure C14-1. Definitions of static residual displacement D_{rm} for a bilinear hysteretic system.

Katsaras et al. (2006) provides recommendations for estimating the permanent residual displacement in any isolation system based on an extensive analytical and parametric study. The residual displacements measured in full-scale tests (Ryan et al. 2012) are reasonably predicted by this procedure, which uses an idealized bilinear isolation system shown in Figure C14-1. The three variables that affect the residual displacement are the isolated period (based on the second slope stiffness, K_D), the yield/friction level (F_o), and the yield displacement D_y.

The procedure for estimating the permanent residual displacement, D_{rd} [Equation (C14-1)] is a function of the system yield displacement, D_y; the static residual displacement, $D_r = F_o/K_D$; and D_{rm}, which is a function of D_m, the maximum earthquake displacement shown in Table C14-1 and Figure C14-1. For most applications D_{rm} is typically equal to D_r:

$$D_{rd} = \frac{0.87 D_{rm}}{\left(1 + 4.3\dfrac{D_{rm}}{D_r}\right)\left(1 + 31.7\dfrac{D_y}{D_r}\right)} \qquad \text{(C14-1)}$$

Thus, there is a simple two-step process to estimate the permanent residual displacement, D_{rd}:

- Calculate the static residual displacement, D_r, based on the isolated period (using the second slope stiffness, K_D) and the yield or friction levels; and
- Using the value of D_r calculated for the isolation system and the yield displacement, D_y, of the system, the residual displacement D_{rd} can be calculated from Equation (C14-1).

C14.2.6 Seismic Load Effects and Load Combinations

Although the load combinations of Chapter 7 are required to be considered for the entire building, additional load

Table C14-1. Values of D_{rm}.

Range of Maximum Displacement, D_{max}	Static Residual Displacement, D_{rm}
$0 \le D_{max} \le D_y$	0
$D_y \le D_{max} < D_r + 2D_y$	$D_r\ (D_{max} - D_y)\ /\ (D_r + D_y)$
$D_r + 2D_y \le D_{max}$	D_r

combinations exist for the isolation system design and for isolator testing. The extent of these additional load combinations include, for example, the design of the isolators themselves and their connections to the superstructure above and substructure below, and isolator testing.

C14.3 SEISMIC ISOLATION SYSTEM DEVICE PROPERTIES

The effectiveness and performance of different isolation devices in building structures under a wide range of ground motion excitations have been assessed through numerous experimental and analytical studies (Kelly et al. 1980, 1981, 1990; Zayas et al. 1987; Constantinou et al. 1999; Buckle et al. 2002; Mosqueda et al. 2004; Warn and Whittaker 2006). The experimental programs included in these studies have typically consisted of reduced-scale test specimens, constructed with relatively high precision under laboratory conditions. These studies initially focused on elastomeric bearing devices, although in recent years the attention has shifted to the single- and multiple-concave friction pendulum bearings. The latest knowledge of lifetime behavior of isolators and methodology for establishing lower- and upper-bound values for isolator basic mechanical properties based on property modification factors is presented in Constantinou et al. (2007). The methodology presented uses property modification factors to adjust isolator design properties.

Examples of application in the analysis and design of bridges may be found in Constantinou et al. (2011), and for buildings, in McVitty and Constantinou (2015). These examples may serve as guidance in the application of the methodology in this standard. Constantinou et al. (2011) also presents procedures for estimating the nominal properties of lead–rubber and friction pendulum isolators, again based on the assumption that prototype test data are not available. Data used in the estimation of the range of properties were based on available test data, all of which were selected to heighten heating effects. Such data would be appropriate for cases of high-velocity motion and large lead core size or high friction values.

C14.3.1 Isolation System Device Types The type and size of an isolator refer to its dimensions, configuration, and design properties. In addition to type and size, vertical load may also be considered for grouping isolators, although this is usually already reflected indirectly in isolator size. It is not expected that all isolators within a common group will have identical properties (e.g., geometrically identical friction isolators with moderately different vertical loads will have different friction coefficients) but rather that the variation within a group is adequately quantified through the property modification factors.

Elastomeric isolators include any one of the following: high-damping rubber isolators, low-damping rubber isolators, or low-damping rubber isolators with a lead core. Sliding isolators include flat assemblies or have a curved surface, such as the friction pendulum system. Rolling isolators are a subset of sliding isolators, and they can be flat assemblies or can have a curved or conical surface, such as the ball and cone system.

C14.3.2 Nominal Design Properties of Isolation System Devices In the early applications of base isolation technology, the design properties were obtained from prototype tests, which generally led to an extended design process. As the number of applications has increased, the prototype test data that are now available from manufacturers of the more widely used systems have increased significantly, and it is now possible to get reasonably accurate nominal design properties from the manufacturers early in design. These nominal design properties can either be confirmed by project-specific prototype tests later in the design phase of the project, or similarity may be used to accept the previous prototype tests on which the nominal design properties are based.

C14.3.3 Bounding Properties of Isolation System Devices

C14.3.3.1 Specification Tolerance on Design Properties As part of the design process, it is important to recognize that there are variations in the design properties caused by manufacturing tolerances. Results from testing of a small number of prototype isolators may not necessarily provide the best estimate of the design properties and the associated upper- and lower-bound specification limits. This potential discrepancy occurs because the average of two prototype test results may be at the upper or lower end of the range of a larger population.

Recommended values for the specification tolerance on the average properties of all isolators of a given isolator type and size are typically in the ±15% range. For a ±15% specification tolerance, the corresponding specification property modification factors would be $\lambda_{\text{spec max}} = 1.15$ and $\lambda_{\text{spec min}} = 0.85$. Variations in individual isolator properties may be greater than the tolerance on the average properties of all isolators of a given type and size (e.g., $\lambda_{\text{spec max}} = 1.2$ and $\lambda_{\text{spec min}} = 0.8$). It is recommended that the isolator manufacturer be consulted when establishing these values. The wider specification tolerance for individual isolators is not used for analysis of the isolation system but is used in determining acceptance of isolator production testing. Where the specification property modification factor for individual isolators significantly exceeds that for the average across all isolators of a common type and size, the designer may consider amplifying analysis forces locally at isolator locations (e.g., amplifying analysis forces for isolator connection design).

C14.3.3.2 Testing Variations on Design Properties The force–displacement models of isolators that are used in analysis typically assume constant properties during the earthquake, whereas, in reality, the properties are instantaneously changing because of the isolator's velocity and vertical load dependency and because of scragging and heating effects. The purpose of the testing property modification factors ($\lambda_{\text{test max}}$ and $\lambda_{\text{test min}}$) is to account for this behavior where it is not directly accounted for in the analytical model of the isolator.

Equivalent energy results from Warn and Whittaker (2004) show that the equivalent number of cycles (fully reversed at the design displacement) experienced in an earthquake depend on the isolation system properties and type of earthquake excitation, as distinguished by proximity to fault and soil properties. For near-fault applications, the representative number of cycles is small, and bounding the heating effects to the third cycle may not be warranted for high-speed testing. Soft-soil sites and far-field ground motions typically have more equivalent cycles. In any case, the design professional must decide and substantiate the representative number of cycles.

The following comments are provided in the approach to be followed for the determination of the bounding values of mechanical properties of isolators:

1. Heat effects for some systems may become significant, and misleading, if insufficient cooling time is not included between adjacent tests. The first-cycle or scragging effects observed in some isolators may recover with time, so back-to-back testing may result in an underestimation of these effects. Refer to Constantinou et al. (2007) and Kalpakidis and Constantinou (2008) for additional information. The impact of this behavior may be mitigated by basing the λ_{test} factors on tests performed relatively early in the test regime before these effects become significant.

2. Heating effects (hysteretic or frictional) may be accounted for on the basis of a rational theory (e.g., Kalpakidis and Constantinou 2009, Kalpakidis et al. 2010, Constantinou et al. 2007). This is true for lead–rubber isolators, where lead of high purity and of known thermomechanical properties is used. For sliding isolators, the composition of the sliding interface affects the relation of friction to temperature and therefore cannot be predicted by theory alone. Moreover, heating generated during high-speed motion may affect the bond strength of liners. Given that there are numerous sliding interfaces (and typically proprietary ones), that heating effects in sliding isolators are directly dependent on pressure and velocity, and that size is important in the heating effects (Constantinou et al. 2007), full-scale dynamic prototype testing is very important for sliding isolators.
3. Heating effects are important for sliding isolators and the lead core in lead–rubber isolators. They are less important and can often be neglected for elastomeric isolators of either low or high damping. The reason for this is described in Constantinou et al. (2007), where it has been shown, based on theory and experimental evidence, that the rise in temperature of elastomeric isolators during cyclic motion (about 1 °C per cycle) is too small to significantly affect their mechanical properties. Prototype and production testing of full-size specimens at the expected loads and displacements should be sufficient to detect poor material quality and poor material bonding in plain elastomeric isolators, even if done quasi-statically.
4. Scragging and recovery to the virgin rubber properties (see Constantinou et al. 2007 for details) is dependent on the rubber compound, size of the isolator, the vulcanization process, and the experience of the manufacturer. Also, it has been observed that scragging effects are more pronounced for rubber of low shear modulus and that the damping capacity of the rubber has a small effect. Furthermore, some manufacturers are capable of producing low-modulus rubber without significant scragging effects, whereas others cannot. It is therefore recommended that the manufacturer present data on the behavior of the rubber under virgin conditions (not previously tested and immediately after vulcanization) so that scragging property modification factors can be determined. The scragging factor is defined as the ratio of the effective stiffness in the first cycle to the effective stiffness in the third cycle, typically obtained at a representative rubber shear strain (e.g., 100%). It has been observed that this factor can be as high as or can exceed a value of 2.0 for shear modulus rubber less than or equal to 65 lb/in.^{2} (0.45 MPa). Also, it has been observed that some manufacturers can produce rubber with a shear modulus of 65 lb/in.^{2} (0.45 MPa) and a scragging factor of approximately 1.2 or less. Accordingly, it is preferred to establish this factor by testing for each project or to use materials qualified in past projects. Note that the property modification factor associated with scragging and used in analysis is less than the scragging factor because it is the ratio of the first cycle to nominal (close to second-cycle) properties.

C14.3.3.3 Aging and Environmental Effects on Design Properties Aging in elastomeric isolators generally has small effects (typically increases in stiffness and strength of the order of 10%, $\lambda_{ae\ max} = 1.1$, to 30%, $\lambda_{ae\ max} = 1.3$, over the lifetime of the building), provided that scragging is also minor. It is believed that scragging is mostly the result of incomplete vulcanization, which is thus associated with aging as chemical processes in the rubber continue over time. Inexperienced manufacturers may produce low shear modulus elastomers by incomplete vulcanization, which then results in significant aging.

Aging in sliding isolators depends on the composition of the sliding interface. Bimetallic interfaces are discouraged, even in the absence of corrosion, or should be penalized by using large aging property modification factors. Lubricated interfaces also warrant high aging and contamination property modification factors. The designer can refer to Constantinou et al. (2007) for concerns with bimetallic interfaces and for modification factors depending on the conditions of operation and the environment of exposure. Lubrication is meant to be *liquid* lubrication typically applied either directly at the interface or within dimples. Solid lubrication in the form of graphite or similar materials that are integrated in the fabric of liners and used in contact with stainless steel for the sliding interface does not have the problems experienced by liquid lubrication.

In general, ambient temperature effects can be ignored for most isolation systems if they are in a space where the expected temperature varies between 30 °F (−1.1 °C) and 100 °F (37.8 °C).

C14.3.4 Property Modification Factors This section combines sources of variability in isolation system mechanical properties measured by prototype testing, permitted by manufacturing specification tolerances, and occurring over the life span of the building because of aging and environmental effects.

The 0.75 factor reflects the fact that the full impacts of all aging and environmental effects do not occur simultaneously. This concept originated with a report by Constantinou et al. (1999), which was then incorporated into the AASHTO *Guide Specifications for Seismic Isolation Design* (AASHTO 1999, 2010) and was also included in the recommended AASHTO *LRFD Bridge Design Specifications* (AASHTO 2011). The design professional may opt to use a higher value based on the significance of the building (e.g., health care facilities) or based on the number of extreme events considered in the establishment of the property modification factor.

The limits of Table 14-1 are based on the default property modification factors table for unknown manufacturers in ASCE 7, Chapter C17. These default property modification factors presume incomplete test data and unknown manufacturers.

Accordingly, there is a considerable range in the upper and lower values of the property modification factors for unknown manufacturers. Yet these values should be used with caution, because low-quality fabricators could use materials and vulcanization and manufacturing processes that result in even greater property modifications. The preferred approach for establishing property modification factors is through rigorous qualification testing of materials and manufacturing methods by a high-quality manufacturer, by dynamic prototype testing of full-size specimens, and by quality control testing at project-specific loads and displacements. These test data on similar-sized isolators take precedence over the default values. Property modification factors for quality manufacturers are also provided in a separate table in ASCE 7, Chapter C17.

C14.3.5 Upper- and Lower-Bound Properties Upper-bound and lower-bound values of isolation system device behavior (e.g., for use in the nonlinear dynamic procedure) and maximum and minimum values of isolation system effective stiffness and damping based on these bounding properties (e.g., for use in the linear static procedure) are established in this section.

C14.4 MODELING

C14.4.1 Isolation System Device Modeling An upper- and lower-bound representation of each type of isolation system

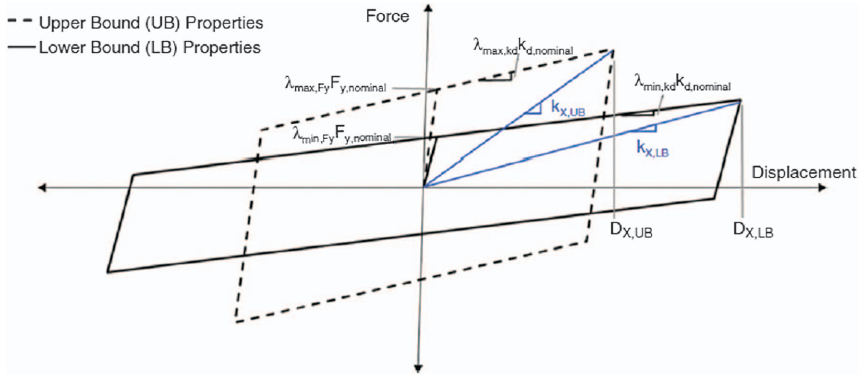


Figure C14-2. Example of the upper-bound and lower-bound properties of a bilinear force–deflection system.

device is required to be developed using the property modification factors developed in Section 14.3.5. An example of a bilinear force–deflection loop is shown in Figure C14-2. In this example, the upper-bound and lower-bound property modification factors (λ) are applied to the nominal properties of the yield/friction level (F_y) and the post-yield stiffness (k_d) of the lateral force–displacement curve to determine the upper- and lower-bound representation of the isolation system device. The nomenclature shown in Figure C14-2 is important to note. The effective stiffness and effective damping are calculated for both the upper- and lower-bound properties at the corresponding D_X. The maximum and minimum effective stiffness and effective damping are then developed from these upper- and lower-bound lateral force–displacement relationships in Section 14.4.1.2.

C14.4.2 Isolation System and Superstructure Modeling

C14.4.2.3 Superstructure Model This section permits the structure above the base level to be modeled, evaluated, and designed as linear when the conditions of the exception are met. The structure at or below the base level is always required to be modeled as linear and evaluated and designed as force-controlled per other requirements of this chapter. This exception reflects that, when low levels of ductility are expected in the superstructure, results from linear modeling of the superstructure would not differ significantly from nonlinear modeling of the superstructure. This philosophy is also reflected in the modeling, evaluation, and design of fixed-base structures as evidenced by the limitation on maximum demand-capacity ratios for use of linear procedures in Chapter 7. However, the user should be aware that as the superstructure ductility in individual components and the number of components experiencing significant ductility increases, the accuracy of linear analysis in matching nonlinear analysis results decreases. The exception applies regardless of analysis procedure selected (e.g., linear static, linear dynamic, nonlinear static, or nonlinear dynamic).

C14.5 ANALYSIS PROCEDURES

C14.5.1 Selection of Analysis Procedure

C14.5.1.1 Linear Static Procedure The requirements in this section need only be met for nominal isolation properties. This limitation helps alleviate the need to check all requirements with both upper- and lower-bound properties for two hazard levels. If, however, the requirements can be shown to be met for both upper- and lower-bound properties, the nominal case need not be checked. Although only nominal design properties are required to be checked for determining the use of the analysis procedures, both upper-bound and lower-bound analyses are required when actually implementing the procedure.

The checks performed in Items 1 and 2 only refer to the portion of the structure above the isolation plane and do not include checks of the isolation system itself. There may be rare cases where a building meets all of the checks in these provisions and the linear static procedure is still not suitable (e.g., if extreme torsion exists in the isolation plane itself). Such cases require engineering judgment to determine whether the linear static procedure will provide sufficiently accurate design actions.

C14.5.2 Linear Static Procedure

C14.5.2.2 Minimum Lateral Displacements

C14.5.2.2.1 Isolation System Displacement The lateral displacements given in this section approximate peak earthquake displacements of a single-degree-of-freedom, linear-elastic system of period, T_X, and effective damping, β_X. The equation for calculating D_X is used to compute the peak displacement in the isolation system at the center of mass for each hazard level considered. A damping term, β_X, is used to decrease (or increase) the computed displacement demand where the effective damping coefficient of the isolation system is greater (or smaller) than 5% of critical damping. In the 2017 and previous editions of the standard, the equation in this section used S_{X1}/T_X to represent the spectral acceleration at the period of the isolation system. In the 2023 edition of the standard, S_{X1}/T_X was replaced with $S_a(T_X)$ to reflect the change to multi-period response spectra.

A comparison of values obtained from the equation in this section and those obtained from nonlinear time history analyses is given in Kircher et al. (1988) and Constantinou et al. (1993).

The calculations are performed separately for upper-bound and lower-bound isolation system properties, and the governing case is considered for design. Upper-bound properties will typically, but not always, result in a lower D_X, higher damping (β_X), and higher lateral forces (V_b and V_{st}).

C14.5.2.2.2 Effective Period at the Displacement D_X. The effective period T_X is determined separately for the upper-bound and lower-bound isolation properties and for each hazard level considered.

C14.5.2.2.3 Total Isolation System Displacement The equation for calculating total (translational and torsional) displacement, which only affects the design of the isolation system, need only be applied for the largest hazard level considered. This equation for calculating D_{TX} includes a term and corresponding equations that reward isolation systems configured to resist torsion (Wolff et al. 2014).

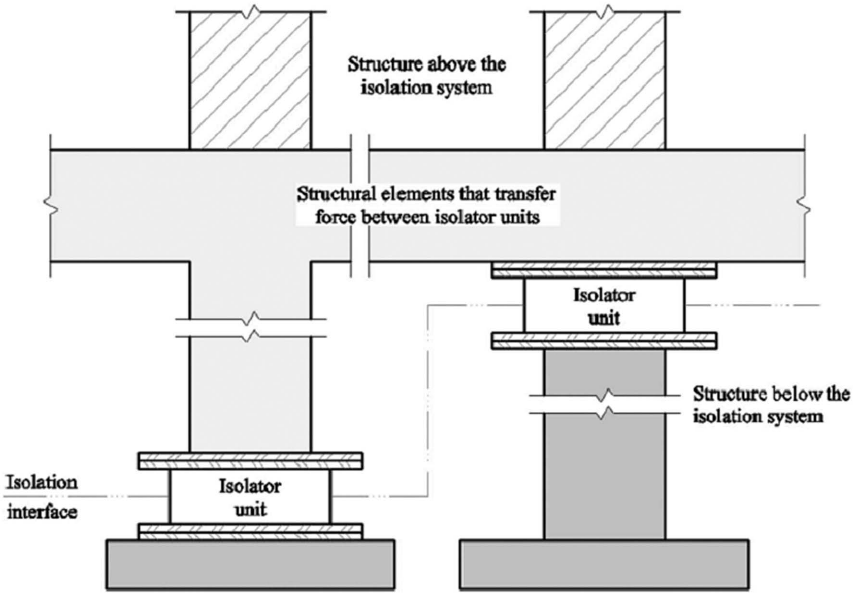


Figure C14-3. Isolation system terminology.

The isolation system for a seismically isolated building should be configured to minimize eccentricity between the center of mass of the superstructure and the center of rigidity of the isolation system, thus reducing the effects of torsion on the displacement of isolation elements. For conventional buildings, allowance must be made for accidental eccentricity in both horizontal directions. The equation for calculating D_{TX} provides a simplified formula for estimating the response caused by torsion in lieu of a more refined analysis. The additional component of displacement caused by torsion increases the design displacement at the corner of a building by about 15% (for one perfectly square in plan) to about 30% (for one long and rectangular in plan) if the eccentricity is 5% of the maximum plan dimension. These calculated torsional displacements correspond to buildings with an isolation system whose stiffness is uniformly distributed in plan. Isolation systems that have stiffness concentrated toward the perimeter of the building, or certain sliding systems that minimize the effects of mass eccentricity, result in smaller torsional displacements.

C14.5.2.3 Minimum Lateral Forces Figure C14-3 illustrates the terminology for elements at, below, and above the isolation base level. The concept of a *base level* was introduced in the 2017 edition of the standard as the first floor immediately above the isolation system.

C14.5.2.3.1 Isolation System and Structural Elements at or below the Base Level The equation for V_b specifies the peak elastic seismic shear for design of all structural elements at or below the base level. The structural elements at and below the base level are intended to remain elastic and are therefore required to be evaluated and designed as force-controlled.

In the 2013 and earlier editions of the standard, the elastic design base shear forces for a given hazard level were determined using a mixture of the upper-bound effective stiffness and the displacement obtained using the lower-bound properties of the isolation system. This was known to be conservative. The elastic design base shear is now calculated with a consistent set of upper-bound and lower-bound stiffness properties. For additional discussion of how this change affects the design base shear, refer to Section C17.5.4 of ASCE 7.

C14.5.2.3.2 Structural Elements above the Base Level Equations (14-13) for V_{st} defines lateral force on elements above the base level in terms of reduced seismic weight (seismic weight excluding the base level), and the effective damping of the isolation system, based on York and Ryan (2008). For a lightly damped isolation system, theory suggests that the lateral forces at the level immediately above the isolation system (i.e., the base level) are reduced by the inertia force associated with the base diaphragm. However, damping diminishes the reduction; thus the equation for V_{st} depends on damping. An exception reflects that the lateral forces are also affected by the hysteresis properties of the isolation system, and it makes an adjustment when the isolation system is characterized by an abrupt transition from pre-yield to post-yield behavior or pre-slip to post-slip behavior. For additional discussion of how isolation system characteristics affect the distribution of lateral forces over the height of the building, refer to Section C17.5.5 of ASCE 7.

In this formulation, it is assumed that the base level is located immediately above the isolation interface. When the base level is not located immediately above the isolation interface, the flexibility occurring between the isolation system and the base level becomes important and the full (unreduced) seismic weight of the building above the isolation interface is used in calculating V_{st} to define lateral forces on elements above the base level.

C14.5.2.3.3 Limits on V_{st} The limits given on V_{st} are needed so that the superstructure does not yield prematurely before the isolation system has been activated and significantly displaced.

The requirement that V_{st} exceed the factored wind design load ensures that, for areas of low seismicity, the structural elements above the base level are evaluated and designed to at least withstand wind loads.

C14.5.2.4 Vertical Distribution of Force The provisions of this section were revised in the 2017 edition of the standard to incorporate a more accurate distribution of shear over height considering the period of the superstructure and the effective damping of the isolation system. The specified method for vertical distribution of forces calculates the force at the base level immediately above the base isolation plane, then distributes the remainder of the base shear among the levels above.

The vertical force distribution in the provisions is based on analytical studies, that is, York and Ryan (2008), in collaboration with SEAONC Protective Systems Subcommittee. Linear theory of base isolation predicts that base shear is uniformly distributed over the height of the building. The uniform distribution is consistent with the first-mode shape of an isolated building, whereas a linear distribution is consistent with the first-mode shape of a fixed-base building. However, a linear distribution may be overly conservative for an isolated building, especially for 1- or 2-story buildings with heavy base mass relative to the roof. The principle established in York and Ryan (2008) was to develop two independent equations: one to predict the superstructure base shear V_{st} relative to the base shear across the isolators V_b, and a second to distribute V_{st} over the height of the building. Considering a reduction in V_{st} relative to V_b allowed for the often-significant inertial forces at the base level, which can be amplified because of disproportionate mass at the base level, to be accounted for in design. The study also assumed that the superstructure base shear was distributed over the height using a k distribution (i.e., lateral force proportional to $w_x h_x^k$ where w_x is the weight and h_x the height to level x), where $k = 0$ is a uniform distribution and $k = 1$ is a linear distribution. For additional discussion of how isolation system characteristics affect the distribution of lateral forces over the height of the building, refer to Section C17.5.5 of ASCE 7.

The exception permits the distribution of story shear based on nonlinear response history analysis of a simplified model (i.e., a column representation of the superstructure and a single force–displacement relationship of the full isolation system). This exception allows a project-specific determination of story shear distribution without requiring the nonlinear dynamic procedure on a complete building model to be used for the entirety of evaluation and design.

C14.5.5 Nonlinear Dynamic Procedure

C14.5.5.2 Accidental Mass Eccentricity The exception of this section avoids the need to perform a large number of nonlinear response history analyses that include suites of ground motion acceleration histories for both BSE-1X and BSE-2X events, the upper and lower isolator properties, and five or more locations of the center of mass.

The following procedure is one acceptable method of developing appropriate amplification factors for deformations and forces for use with center-of-mass nonlinear dynamic procedure analyses to account for the effects of accidental torsion. The use of other rationally based amplification factors is permitted.

The most critical directions for moving the calculated center of mass are such that the accidental eccentricity adds to the inherent eccentricity in each principal direction at each level. For each of these two eccentric mass cases, and with lower-bound isolator properties, the suite of nonlinear response history analyses should be run, and the results should be processed. The analysis cases are defined in Table C14-2.

The results from Cases IIa and IIb are then compared with those from Case I. The following amplification factors (ratio of Case IIa or IIb response to Case I response) are computed:

1. The amplification of story drift in the building at the plan location with the highest drift, enveloped over all stories; and
2. The amplification of frame-line shear forces at each story for the frame subjected to the maximum drift.

The larger of the two resulting scalars on drift should be used as the deformation amplifier, and the larger of the two resulting scalars on force should be used as the force amplifier. The effects of accidental eccentricity should then be considered as follows: Nonlinear dynamic procedure analyses for the inherent mass eccentricity case should be run, considering the variation of isolator properties. Response quantities should be computed. For each isolator property modification, all deformation response quantities should be increased by the deformation amplifier and all force quantities should be increased by the force amplifier, before being used for evaluation or design.

Table C14-2. Analysis Cases for Evaluation of Effect of Accidental Eccentricity.

Case	Isolator Properties	Accidental Eccentricity
I	Lower-bound	No
IIa	Lower-bound	Yes, X-direction
IIb	Lower-bound	Yes, Y-direction

C14.6 ISOLATION SYSTEM TESTING AND DESIGN PROPERTIES

C14.6.3 Prototype Tests

C14.6.3.5 Dynamic Testing This section clarifies when dynamic testing is required. Many common isolator types exhibit velocity dependence; however, this testing can be expensive and can only be performed by a limited number of test facilities. The intent is not that dynamic testing of isolators be performed for every project. Sufficient dynamic test data must be available to characterize the cyclic performance of the isolator, in particular the change in isolator properties during the test (i.e., with respect to the test average value). Dynamic testing must therefore be used to establish the high-speed nominal design properties and corresponding $\lambda_{\text{test min}}$ and $\lambda_{\text{test max}}$ values, because the ranges set by these values are typically underestimated from slow-speed test data. If project prototype testing is to be performed at slow speeds, this testing would also be used to establish factors that account for the effect of velocity and heating on the test average values of k_X, k_d, and E_{loop}. These factors either can be thought of as a separate set of velocity-correction factors to be applied on slow-speed test average (nominal) values, or they can be incorporated into the $\lambda_{\text{test min}}$ and $\lambda_{\text{test max}}$ values themselves.

Although reduced-scale prototype specimens are permitted to quantify the rate-dependent properties of isolators in accordance with this section, it is recommended that full-scale specimens be used whenever possible. Section 14.6.3.1 also requires that full-scale specimens be used for prototype testing. Therefore, if reduced-scale prototype specimens are used to quantify the rate-dependent properties of isolators in accordance with this section, they would be in addition to the full-scale specimens of Section 14.6.3.1.

C14.6.3.9 Testing Similar Isolation System Devices This section provides specific limits related to the acceptability of data from testing of similar isolators. A wider range of acceptability is permitted for dynamic test data in Section 14.6.3.5. Further commentary on the similarity requirements is provided in ASCE 7, Chapter 17.

C14.6.4 Production Testing The testing of 100% of the isolators serves to verify the quality of the product and to verify the manufacturing tolerance. Quasi-static testing is acceptable for production testing. The design professional responsible for the structure must define the scope of the manufacturing quality control test program, as well as allowable variations in the measured properties of the production isolation system devices.

The combined compression and shear testing reveals the most relevant characteristics of the completed isolator and permits the designer to verify that the production isolators provide force–deflection behavior that is consistent with the structural evaluation and design assumptions. Quasi-static production testing requires a relationship to be established between properties determined under dynamic conditions (used for analysis and design) to the behavior under quasi-static loading. This relationship requires that the prototype isolators that are tested under dynamic conditions (for obtaining nominal design properties and related property modification factors) are also tested under the same conditions as the production isolators to establish criteria for acceptance.

The quality control program should also include testing of isolator component materials in a similar fashion to other construction materials for the project. The objective of this material testing is to ensure consistency throughout the entire run of production isolators for the project with a previously tested prototype isolator. The design professional should coordinate with the isolator manufacturer to establish the details of the material testing program.

C14.6.5 Determination of Force–Deflection Characteristics The exception to this section permits alternate methods of determining k_d (e.g., straight line fit of k_d directly to the hysteresis curve and then determining k_1 to match E_{loop}, defining D_y and F_y by visual fit and then determining k_d to match E_{loop}, etc.) when such methods are the subject of design review.

C14.6.6 Test Specimen Adequacy The test specimen adequacy criteria of 2013 and earlier editions of this standard can be traced back to historical documents, where testing was performed quasi-statically and where the displacement and force demands were not as significant as more recent seismic isolation applications. With the systematic approach of using property modification factors in bounding analysis beginning in the 2013 edition of this standard, the design professional explicitly accounts for the change in isolation system properties in analysis and design. Therefore, the test specimen adequacy criteria has been explicitly linked to the nominal design properties as well as the specification and testing property modification factors.

The test specimen adequacy section in the 2023 edition of this standard was edited to align closely with the the companion test specimen adequacy section for new seismically isolated structures in ASCE 7-22, Section 17.8.4, which are essentially unchanged from those in ASCE 7-16. However, a few requirements in ASCE 7-22, Section 17.8.4, were adjusted to better reflect the ASCE 41 Seismic Isolation and Energy Dissipation Subcommittee's understanding of the intent of ASCE 7, especially where confusion has arisen on past projects using ASCE 7. Notably, these include the following clarifications:

- "Positive incremental force-resisting capacity" in ASCE 7-22 Section 17.8.4, Item 1 was not intended to preclude systems that have negligible post-yield stiffness (e.g., flat sliding isolators). Instead, "non-negative incremental force-resisting capacity" was used.
- Reference to "including the effects of heating and rate of loading" in ASCE 7-22 Section 17.8.4, Item 2 was inappropriate. This is because heating and rate of loading are effects contained within λ_{test}, whereas this requirement is a check using λ_{spec}.
- The term "nominal value of post-yield stiffness" in ASCE 7-22 Section 17.8.4, Item 3 was really intended to read "average tested value of post-yield stiffness." This is because comparison of individual cycle results to the nominal design value would need to capture the combination of λ_{test} and λ_{spec}, whereas this requirement is a check on λ_{test} only.
- The term "nominal design value" in ASCE 7-22 Section 17.8.4, Item 5 was really intended to read "average tested value." This is because comparison of individual cycle results to the nominal design value would need to capture the combination of λ_{test} and λ_{spec}, whereas this requirement is a check on λ_{test} only.
- Reference to Test 4a in ASCE 7-22 Section 17.8.4, Item 5 was really intended to also include Test 4b. Instead, Test 4 was referenced in its entirety (i.e., either of Test 4a or Test 4b depending on which was pursued as part of the prototype testing).
- An exception to Item 5 was retained from the 2017 edition of this standard, which does not occur in ASCE 7-22, Section 17.8.4 giving the design professional more flexibility to determine the representative or equivalent number of cycles. This is because the total energy dissipated by an isolation system will vary between projects as it depends on the properties of the isolation system (strength and stiffness), the site conditions, and the ground motion characteristics, including proximity to the fault (Warn and Whittaker 2007).

For a site on soft soils or subjected to subduction zone shaking, a minimum of four cycles in the exception to Item 5 may not be sufficient. The design professional may still want to consider the performance of the isolator for additional cycles, above those considered for bounding analysis, to evaluate the durability of the isolator subject to multiple earthquake events.

C14.7 DESIGN REVIEW

The provisions allow for a single peer reviewer to evaluate the isolation system design. The reviewer should be a registered design professional, and if the engineer of record is required to be a licensed structural engineer, the owner may consider requiring that there is at least one licensed structural engineer on the peer review team. On more significant buildings, it is likely that the design review panel may be more than one individual but, for many isolated buildings, a single well-qualified peer reviewer is sufficient. If a manufacturer with unknown experience in the United States is selected as the supplier, the building owner may consider requiring the reviewer to attend prototype tests.

This standard requires peer review to be performed by design professionals who are independent of the design team and other project contractors. The reviewer or review panel should include individuals with special expertise in one or more aspects of the design, analysis, and implementation of seismic isolation systems.

The peer reviewer or review panel should be formed before the development of design criteria (including site-specific ground-shaking criteria) and isolation system design options. Furthermore, the review panel should have full access to all pertinent information and the cooperation of the general design team and regulatory agencies involved in the project.

CHAPTER C15

DESIGN REQUIREMENTS FOR STRUCTURES WITH SUPPLEMENTAL ENERGY DISSIPATION

C15.1 SCOPE

The basic form and formulation of requirements for supplemental energy dissipation systems have been established and coordinated with the Performance Objectives, target building performance levels, and Seismic Hazard Level criteria of Chapter 2 and the linear and nonlinear procedures of Chapter 7.

In the 2017 edition of the standard (ASCE 2017b), supplemental energy dissipation provisions were removed from Chapter 14, and this separate chapter for supplemental energy dissipation was created. The chapter is based on similar provisions in ASCE 7 because the subcommittee responsible for this chapter did not believe there to be a reason that the provisions of ASCE 41 differ from those in ASCE 7, as the theory and application of supplemental energy dissipation is the same regardless of whether it is used in a retrofit of an existing building or the design of a new building.

Energy dissipation systems include a wide variety of concepts and devices. In some cases, these systems and devices are implemented with some additional conventional strengthening of the structure; in all cases, they require evaluation of existing building components. Criteria for modeling the stiffness, strength, and deformation properties of conventional structural components of buildings are given in Chapters 9 through 12. This chapter supplements the requirements of these other chapters with additional criteria and methods of analysis that are appropriate for buildings retrofitted with energy dissipation devices.

Energy dissipation devices dampen earthquake excitation of the structure that would otherwise cause higher levels of response and damage to components of the building. Energy dissipation systems have a wide range of building height applications. Other criteria may also influence the decision to use energy dissipation devices, because these devices can also be useful for control of building response caused by wind or mechanical loads.

Energy dissipation systems should be considered early in the design process and should be based on the Performance Objectives established for the building. In general, energy dissipation systems are more attractive as a retrofit strategy for buildings that have higher Performance Objectives than for ordinary buildings (i.e., higher building performance levels and/or more severe Seismic Hazard Levels). The costs associated with the design, fabrication, and installation of energy dissipation devices are typically offset by the reduced need for stiffening and strengthening measures that would otherwise be required to meet Performance Objectives.

Whenever either the Limited Performance Objective of Section 2.4.3, or a Partial Retrofit of Section 2.4.5 is selected, the structural design requirements are less than those required for the potential seismic event. There is concern that response to this potential earthquake could exceed the design limits of the energy dissipation devices, leading to device failure. Therefore, the displacement and force design of these devices for these two lower Performance Objectives require a conservative multiplier.

The damping system (DS) is defined separately from the seismic-force-resisting system (SFRS), although the two systems may have common elements. As illustrated in Figure C15-1, the DS may be external or internal to the structure and may have no shared elements, some shared elements, or all elements in common with the SFRS. Elements common to the DS and the SFRS must be designed for the loads resulting from the interaction of both systems. When the DS and SFRS have no common elements, the damper forces must be collected and transferred to members of the SFRS.

C15.2 GENERAL DESIGN REQUIREMENTS

C15.2.2.1 Ground Motion Acceleration Histories Development of ground motion acceleration histories for the evaluation and retrofit of buildings with supplemental energy dissipation systems using the nonlinear dynamic procedure generally follows the corresponding requirements for nondamped buildings. The maximum and minimum device properties, required elsewhere in this chapter, need not be considered to determine the period range of interest because the variation in stiffness of typical supplemental energy dissipation systems is captured by the factors on calculated periods provided in Section 2.3.4. For supplemental energy dissipation systems with large variation in stiffness, an extended period range of interest may be required by design review.

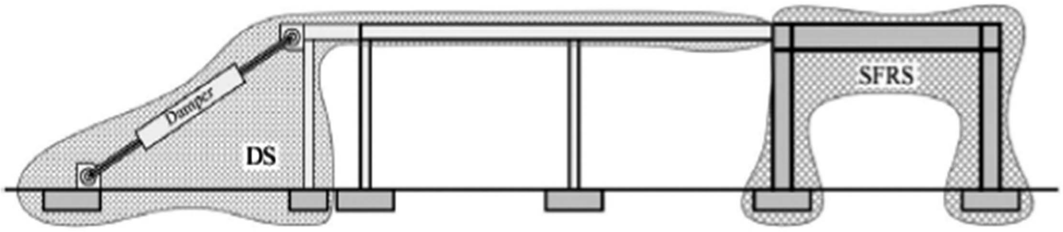
C15.2.3 Damping Device Requirements

C15.2.3.1 Device Classification Energy dissipation devices add damping (and sometimes stiffness) to the building. A wide variety of energy dissipation devices are available, including fluid viscous dampers, viscoelastic materials, and hysteretic devices. Damping devices that have found applications or have potential for application may be classified as follows:

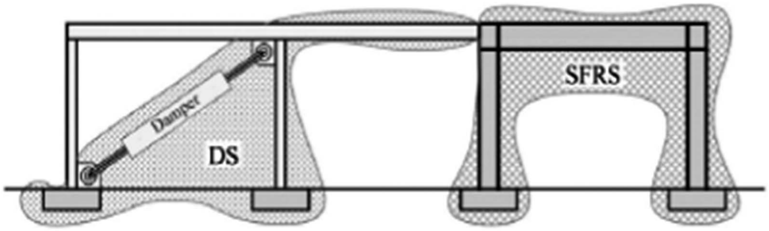
Fluid viscous dampers (or oil dampers). These dampers are devices that operate on the principle of forcing a viscous fluid, typically some form of oil, though an orifice. These devices require substantial engineering and precision machining such that properties are known within a narrow range.

Viscoelastic fluid or solid devices. These devices operate on the principle of shearing of highly viscous fluids or viscoelastic solids. Properties of these devices are strongly dependent on frequency and temperature.

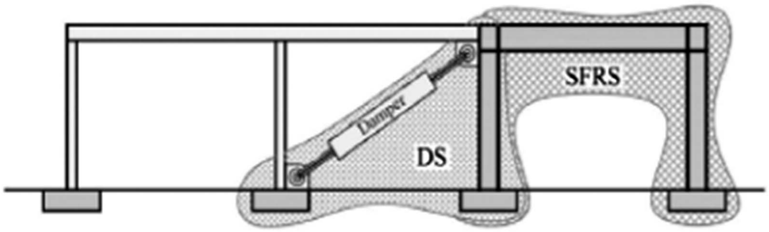
Metallic yielding devices. These devices dissipate energy through yielding of steel elements. Typically, these devices are



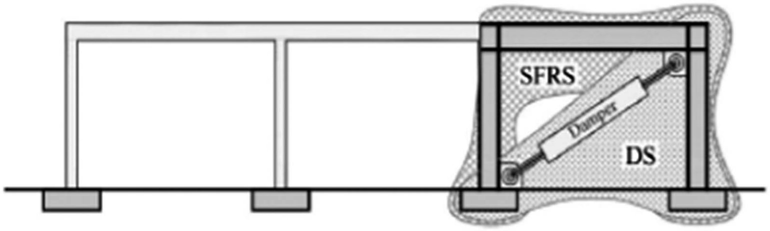
External damping devices



Internal damping devices—no shared elements



Internal damping devices—some shared elements



Internal damping devices—common elements

Figure C15-1. Damping system and seismic-force-resisting system configurations.

manufactured from steel with carefully controlled yield properties. The range of values of the yield strength can be determined with simple material rests.

Friction devices. These devices operate on the principle of preloaded sliding interfaces. Properties are dependent on the materials used and may be highly sensitive to thermal, environmental, and duration effects.

Other devices. Examples include shape-memory alloys (superelastic effect); friction–spring assemblies with recentering capability; and fluid-restoring, force-damping devices.

C15.2.3.4 Performance Objectives and System Redundancy The increase in displacement and velocity capacity is dependent on the level of redundancy in the supplemental damping system.

Research has shown that including a factor of 130% over the values calculated by analysis at BSE-2X can provide a greater margin of safety at large earthquakes. Accordingly, this standard requires that energy dissipation devices be capable of sustaining larger displacements (and velocities for velocity-dependent devices) than the maxima calculated by analysis in the BSE-2X. The response of a building frame incorporating four or more devices in each principal direction in each story is more reliable than a frame with fewer devices in each principal direction, because the increase in displacement and velocity capacity is dependent on the level of redundancy in the supplemental damping system. The increased force caused by the additional displacement and velocity capacity in the devices shall be used to design the framing that supports the energy dissipation devices.

The damping system (DS) must be designed for the actual (unreduced) forces and deflections. For certain elements of the DS (such as the connections or the members into which the damping devices frame), other than damping devices, limited yielding is permitted provided that such behavior does not affect DS function or exceed the amount permitted for elements of conventional structures by the standard.

C15.3 PROPERTIES OF ENERGY DISSIPATION DEVICES

C15.3.1 Nominal Design Properties The nominal design properties can be obtained from prototype tests. Alternatively, existing prototype test data, available from manufacturers of the more widely used devices can be used to get reasonably accurate nominal design properties. These nominal design properties can be confirmed by prototype tests, if desired.

C15.3.2 Maximum and Minimum Damper Properties As part of the design process, it is important to recognize that there will be variations in the production damper properties from the nominal properties. This difference is caused by manufacturing variation. Recommended values for the specification tolerance on the average properties of all devices of a given type and size are typically in the ±10% to ±15% range. For a ±10% specification tolerance, the corresponding λ factors would be $\lambda_{\text{spec max}} = 1.1$ and $\lambda_{\text{spec min}} = 0.9$. Variations for individual device properties may be greater than the tolerance on the average properties of all devices of a given type and size. It is recommended that the device manufacturer be consulted when establishing these tolerance values.

The specification (λ_{spec}), environmental (λ_{ae}), and testing (λ_{test}) factors are used to establish maximum ($\lambda_{\max}$) and minimum ($\lambda_{\min}$) damper properties for each device type and size for use in mathematical models of the damped structure. These factors are typically applied to whatever parameters govern the mathematical representation of the device. For fluid viscous devices, these factors typically apply to the damper constant and not the velocity exponent.

The system property adjustment factor (SPAF) was designed to recognize that a full and simultaneous increase in each parameter is unlikely to occur at the same time.

C15.4 ANALYSIS PROCEDURE SELECTION

C15.4.1 General Limitations for the Linear Analysis Procedures For buildings that have dampers in all stories, procedures other than the nonlinear dynamic procedure (NDP) have been shown to provide a reasonable estimate of the global performance of the building. However, the studies conducted to date have been limited in scope and have focused on the cases where dampers have been provided in all stories. Because damping devices introduce concentrated damping at their point of attachment, the authors of the standard recognize that such damping cannot be represented by a global damping ratio. As such, when dampers are not present in all stories, use of procedures other than NDP analysis can lead to inaccuracies in calculating the demand on structural components. An exception allowing the use of linear procedures is permitted when the structural components are subjected to limited ductility demands, in addition to satisfying other configuration limitations, analysis, and acceptance criteria, including the provisions of Chapter 7.

C15.5 NONLINEAR DYNAMIC PROCEDURES

C15.5.1 General Requirements If energy dissipation devices are dependent on loading frequency, operating temperature (including temperature rise caused by excitation), deformation (or strain), velocity, sustained loads, or bilateral loads, such dependence should be accounted for in the nonlinear time-history analysis. One way to account for variations in the force–deformation response of energy dissipation devices is to perform multiple analyses of the rehabilitated building using the likely bounding response characteristics of the energy dissipation devices. The design of the retrofitted building, including the energy dissipation devices, should be based on the maximum responses computed from the multiple analyses.

The viscous forces (if any) developed in the seismic framing system should be accounted for in the analysis and design of the seismic framing system. Evaluation of component action histories should be based on nodal displacements (operating on member stiffness matrices) and nodal velocities (operating on member damping matrices). In addition, framing system components should be modeled and evaluated in accordance with the requirements of Chapter 7 for deformation-controlled and force-controlled actions.

The analysis procedures and acceptance criteria adopted by this section are consistent with the requirements of Chapter 7 of this standard for the NDP. In comparison, Chapter 18 of ASCE 7 permits calculated component forces to exceed 1.5 times the expected strengths when the components are modeled as linear; however, the overall seismic-force-resisting system is required to be evaluated at the MCE_R hazard and is also required to satisfy the prescriptive requirements of Chapter 12 of ASCE 7.

Key to the acceptable response of a retrofitted building incorporating energy dissipation devices is the stable response of the energy dissipation devices. The forces and deformations in the energy dissipation devices that develop during the design earthquake should be demonstrated to be adequate by prototype testing in accordance with Section 15.8.

C15.5.2.2 Velocity-Dependent Devices

C15.5.2.2.1 Solid Viscoelastic Devices The cyclic response of viscoelastic solids is generally dependent on the frequency and amplitude of the motion and the operating temperature (including temperature rise caused by excitation).

C15.5.2.2.2 Fluid Viscoelastic Devices The cyclic response of fluid viscoelastic devices is generally dependent on the frequency and amplitude of the motion and the operating temperature (including temperature rise caused by excitation).

C15.5.2.3 Other Types of Devices Other energy dissipating devices, such as those having hysteresis of the type having recentering capabilities as shown in Section 15.5, require modeling techniques different from those previously described. Nims et al. (1993), Tsopelas and Constantinou (1994), and Pekcan et al. (1995) describe analytical models for some of these devices.

C15.5.3 Accidental Eccentricity The following procedure is one acceptable method of developing appropriate amplification factors for deformations and forces for use with center-of-mass NDP analyses, to account for the effects of accidental torsion. The use of other rationally based amplification factors is permitted.

The most critical directions for moving the calculated center of mass are such that the accidental eccentricity adds to the inherent eccentricity in each principal direction at each level. For each of these two eccentric mass cases, and with lower-bound damper properties, the suite of NDP analyses should be run, and the results should be processed. The analysis cases are defined in Table C15-1.

The results from Cases IIa and IIb are then compared with those from Case I. The following amplification factors (ratio of Case IIa or IIb response to Case I response) are computed:

1. The amplification of story drift in the structure at the plan location with the highest drift, enveloped over all stories; and
2. The amplification of frame-line shear forces at each story for the frame subjected to the maximum drift.

Table C15-1. Acceptable Analysis Cases for Accidental Eccentricity.

Case	Damper Properties	Accidental Eccentricity
I	Lower-bound	No
IIa	Lower-bound	Yes, *X*-direction
IIb	Lower-bound	Yes, *Y*-direction

The larger of the two resulting scalars on drift should be used as the deformation amplifier, and the larger of the two resulting scalars on force should be used as the force amplifier. If both of these scalars are less than 1.1, the effects of accidental torsion need not be considered. If either scalar is greater than or equal to 1.1, the effects of accidental eccentricity should be considered as follows.

NDP analyses of record need consider only the model reflecting the inherent mass eccentricity. Damper property variation need only be considered for this model. Response quantities should be computed per Section 7.4.4.3. All deformation response quantities should be increased by the deformation amplifier, and all force quantities should be increased by the force amplifier, before being used for evaluation or design.

C15.7 DESIGN REVIEW

Review of the seismic and other dynamic input is required because this review should be a part of the project design criteria. Although review of the prototype test program is mandated, the design reviewer is no longer required to witness the prototype tests. The independent design review of many structures incorporating supplemental damping may be performed adequately by one registered and appropriately experienced design professional. However, for projects involving significant or critical structures, it is recommended that a design review panel consisting of two or three registered and appropriately experienced design professionals be used.

C15.8 REQUIRED TESTS OF ENERGY DISSIPATION DEVICES

C15.8.2 Production Tests The registered design professional is responsible for defining in the project specifications the scope of the production damper test program, including the allowable variation in the average measured properties of the production damping devices. The registered design professional must decide on the acceptable variation of damper properties on a project-by-project basis. This range must agree with the specification tolerance from Section 15.3.2. The standard requires that all production devices of a given type and size are tested.

Individual devices may be permitted a wider variation (typically ±15% or ±20%) from the nominal design properties. For example, in a device characterized by $F = C_0|\dot{D}|^{\alpha} \times \operatorname{sgn}(\dot{D})$, the mean of the force at a specified velocity for all tested devices might be permitted to vary no more than ±10% from the specified value of force, but the force at a specified velocity for any individual device might be permitted to vary no more than ±15% from the specified force.

The production dynamic cyclic test is identical (except for three versus five cycles) to one of the prototype tests of Section 15.8.1.2, so that direct comparison of production and prototype damper properties is possible.

The exception is intended to cover those devices that would undergo yielding or be otherwise damaged under the production test regime.

C15.10 NONLINEAR STATIC PROCEDURE

C15.10.2 Velocity-Dependent Devices The use of Equation (15-25) generally captures the maximum displacement of the building.

CHAPTER C16

SYSTEM-SPECIFIC PERFORMANCE PROCEDURES

C16.1 SCOPE

The intent of this chapter is to permit the use of well-established procedures for evaluating and retrofitting buildings that are different from the analysis procedures for Tier 2 and Tier 3 of this standard. This standard includes only the special procedure for unreinforced masonry from ASCE 31-03 (ASCE 2003), but the intent is that in future editions of the standard this chapter will include additional alternate procedures for specific building systems as they are developed and evaluated. The individual procedures are only valid for the Performance Objectives specified in the respective sections.

Currently, many special procedures are in use as parts of model building codes and individual jurisdictions' enforced building code provisions that have not been officially adopted and incorporated into this chapter. These other special procedures have not been evaluated to determine the seismic Performance Objectives achieved by their application. That is not to say that those provisions should not be used for seismic evaluation or retrofit but that if they are used and the user wishes to declare equivalence to a Performance Objective in this standard, it shall be the responsibility of the Authority Having Jurisdiction and potentially an independent reviewer, if the jurisdictional authority feels it necessary, to confirm that declaration.

An ASCE 41 Seismic Performance Level must be determined by use of the procedures of this standard with a specified Seismic Hazard Level. Tier 1, 2, or 3 evaluations may be used for this purpose. The limitations and conditions stated in the referenced regulations for their application should be followed.

The *International Existing Building Code* (IEBC) (ICC 2021b) provides five special procedures in its Appendix A, "Guidelines for the Seismic Retrofit of Existing Buildings," that can be considered as candidate additional special procedures:

1. Chapter A1. Seismic Strengthening Provisions for Unreinforced Masonry Bearing Wall Buildings;
2. Chapter A2. Earthquake Hazard Reduction in Existing Reinforced Concrete and Reinforced Masonry Flexible Diaphragm;
3. Chapter A3. Prescriptive Provisions for Seismic Strengthening of Cripple Walls and Sill Plate Anchorage of Light, Wood-Framed Residential Buildings;
4. Chapter A4. Earthquake Hazard Reduction in Existing Wood-Frame Residential Buildings with Soft, Weak, or Open-Front Walls; and
5. Chapter A5. Earthquake Hazard Reduction in Existing Concrete Buildings.

As a note, the unreinforced masonry provisions of Section 16.2 are similar to those of Chapter A1 of the IEBC for buildings with flexible diaphragms. The committee for this standard considered the requirements of Chapter A1 in revisions of the special procedure of Section 16.2. IEBC, Chapter A5, was based on and has several references to portions of ASCE 41-06 and ASCE 31-03.

Many building departments have other special procedures that can be considered. Among these are those of the Los Angeles Building Code (City of Los Angeles 2017):

1. Chapter 88. Earthquake Hazard Reduction in Existing Buildings (in unreinforced masonry buildings constructed before 1934);
2. Chapter 91. Earthquake Hazard Reduction in Existing Tilt-Up Concrete Wall Buildings;
3. Chapter 95. Voluntary Earthquake Hazard Reduction in Existing Reinforced Concrete Buildings and Concrete Frame Buildings with Masonry Fill; and
4. Chapter 96. Voluntary Earthquake Hazard Reduction in Existing Reinforced Concrete and Masonry Wall Buildings with Flexible Diaphragms.

In some cases, these chapters are on comparable topics to the IEBC, but they contain different requirements. This standard takes no position as to which is preferred. There are many other examples that may be applied from other jurisdictions.

The use of such a special procedure may address only some of the deficiencies of an existing building that may be identified in an evaluation using this standard. It is advisable that when these procedures are applied voluntarily, one should assess whether other major deficiencies exist that are not addressed by the procedure and that could be important to achieving the client's objectives. The review of the application of any special procedure should always consider whether the modification of some elements increases the hazard to other elements of the building, thereby increasing the seismic hazard posed by the building.

C16.2 SPECIAL PROCEDURE FOR UNREINFORCED MASONRY

C16.2.1 Scope The intent of this chapter is to permit the use of special procedures for unreinforced masonry bearing wall buildings. As stated in previous building codes (ICBO 1997, ICC 2018), "the purpose of the special procedure is to promote public safety and welfare by reducing the risk of death or injury that may result from the effects of earthquakes on existing reinforced masonry bearing wall buildings ... compliance will not necessarily prevent loss of life or injury, or prevent earthquake damage to retrofitted buildings."

This procedure was developed in the 1980s (ABK 1984) and has been included in various codes and standards, including FEMA 178 (FEMA 1992b), *Uniform Code for Building Conservation* (ICBO 1997), ASCE 31-03 (ASCE 2003), and *International Existing Building Code* (ICC 2021b). The procedure has received widespread use in the United States and has been a valuable tool in the evaluation and upgrade of unreinforced masonry structures, particularly historic buildings. Limited experimental testing, analyses, and experience have shown that structures upgraded using the special procedure or its predecessor procedures have generally met the Collapse Prevention level for at least ground motions with a 20% probability of exceedance.

The expected performance is limited to risk reduction for extreme ground motions or alternatively limited to Collapse Prevention for moderate ground motions for the following reasons:

- The original testing and analysis in the ABK (1984) program was based on the ground motion information available at that time. Time-history records were scaled to ATC-3 (1978) spectra for various regions of the country. The spectra had peak velocities of 30 in./s (0.76 m/s) in coastal California and of 15 in./s (0.38 m/s) in the Puget Sound and Wasatch areas. Soil amplification effects were not included in the scaled time histories. The ATC-3 (ATC 1978) and ABK (1984) velocities are close to the current peak velocities in these respective regions for 20% in 50-year ground motions and Site Class C. These velocities are much lower than those for the extreme events that are considered elsewhere in this standard. For buildings on Site Class D or softer soils, peak velocities could significantly exceed those considered in the ABK (1984) program.
- In addition to the limitations on amplitude, the time histories used in the ABK (1984) program did not include either the near-fault pulse effects or the long-duration subduction zone effects that have been recorded in numerous earthquakes since the 1980s.
- Many unreinforced masonry buildings evaluated or upgraded using the special procedure have experienced moderate ground motions (e.g., the Loma Prieta, Northridge, and Nisqually earthquakes) and have met the Collapse Prevention Performance Objective. In fact, many of these buildings met the Immediate Occupancy Performance Objective in areas of lower ground motions. However, at this time, few upgraded buildings have been subjected to extreme ground motions, pulse effects, or long-duration effects. Finally, it would be reasonable to assume higher performance for unreinforced masonry buildings in regions of lower seismicity that are not subject to these types of ground motion effects.
- The special procedure has not been analytically calibrated to the acceptance criteria in Section 7.5. This subject is a potential area for future research.

The limiting building characteristics for the procedure are based on assumptions used in the original ABK research and testing (1984). Although the original ABK research was limited to unreinforced masonry wall systems, the special procedure can be used for buildings that include predominantly masonry walls with some minimal amount of concrete walls. For guidance on evaluation and retrofit of unreinforced masonry buildings with stiff diaphragms, refer to Section C3.2.1, and Building Type URMa in Table 3-1.

C16.2.2 Condition of Existing Materials Refer to Sections C11.2.2.4 and C11.1, for precautions about crack repairs.

C16.2.2.1.3 Walls with Other Layups When justified, layup patterns such as English, Flemish, Flemish Cross, Dutch, and Dutch Cross are generally appropriate alternatives to common bond layups described in Sections 16.2.2.1 and 16.2.2.1.1. These provisions are not considered appropriate for walls with stack bond layup.

C16.2.2.2 Testing In choosing test locations, one should consider factors such as work quality at different building height levels, weathering of exterior surfaces, condition of interior surfaces, and deterioration caused by water or other substances contained within the building.

Pointing: All deteriorated mortar joints in URM walls should be pointed. Pointing should be performed under a permit and with special inspection. Any raking of mortar joints or drilling in URM structures should be done using nonimpact tools.

C16.2.2.2.1 In-Place Mortar Tests The available standard for masonry shear strength test is ASTM C1531, *Standard Test Methods for In Situ Measurement of Masonry Mortar Joint Shear Strength Index* (ASTM 2019). Multiwythe masonry laid with headers should use the in-place shear push test. The bed joints of the outer wythe of the masonry should be tested in shear by laterally displacing a single brick relative to the adjacent bricks in the same wythe. The head joint opposite the loaded end of the test brick should be excavated and cleared. The brick adjacent to the loaded end of the test brick should be removed and excavated to provide space for a hydraulic ram and steel loading blocks. Steel blocks, the size of the end of the brick, should be used on each end of the ram to distribute the load to the brick. The blocks should not contact the mortar joints. The load should be applied horizontally, in the plane of the wythe. Load should be recorded at the first sign of movement of the test brick as indicated by spalling of the face of the mortar bed joints. The strength of the mortar should be calculated by dividing the load at the first movement of the test brick by the nominal gross area of the sum of the two bed joints.

C16.2.2.2.2 Masonry Different types of masonry require different tests to determine the shear strength. As a general guide for selecting the correct test method for modern masonry, the design professional should consider using a core tested as prescribed in ASTM C496 to determine the tensile-splitting stress, although this test is intended for concrete, not masonry. The tensile-splitting stress is the same as the horizontal shear stress. Wythes of solid masonry units should be tested by sampling the masonry by drilled cores of not less than 8 in. (203.2 mm) in diameter. A bed-joint intersection with a head joint should be in the center of the core. The core shall be placed in the test apparatus with the bed joint 45 degrees from the horizontal.

C16.2.2.2.3 Wall Anchors

C16.2.2.2.3.3 Prequalification Tests for Nonconforming Anchors The reduced distance between the tested anchor and the test apparatus support in Section 16.2.2.2.3.1 is recommended to be used only where obstructions occur. Special inspection of generic proprietary anchors should be required as part of a quality assurance plan consistent with Section 11.5.3, "Quality Assurance for Anchors in Masonry Walls." All new embedded anchors should be subject to periodic special inspection before placement of the grout or adhesive in the drilled hole in accordance with a quality assurance plan.

C16.2.2.3 Masonry Strength

C16.2.2.3.1 Shear Strength The correlation of v_{tL} and v_{mL} was obtained by physical testing made by the ABK joint venture. Equation (16-3) is an empirical formula. Equation (16-4) is a theoretical formula consistent with this standard in Section 11.2.3.6.4.

C16.2.2.3.2 Masonry Compression There is no specific check for axial loads in this procedure. However, axial loads are used in determining the shear strength values [Equations (16-1), (16-3), and (16-4)]. Also, loss of masonry capacity caused by seismic forces also may result in a loss of gravity-carrying support. Therefore, the design professional should be aware of any heavily loaded walls during the evaluation.

C16.2.2.3.3 Masonry Tension Masonry is assumed to have no tensile strength. The overturning forces should therefore be resisted by the gravity-resisting moment.

C16.2.3 Analysis A general statement regarding material strengths for new elements was not included in previous editions of the standard. A requirement to use the material standards referenced in Chapter 9 through 12 has been added. A strength reduction factor, φ, equal to 1.0 is included to be consistent with the 2021 IEBC Appendix Section A108.1.2 (ICC 2021b). A clarification was added indicating that specified values, not expected values, are to be used in the strength determination because the concept of lower-bound and expected strengths is typically not used in Section 16.2.

C16.2.3.2 Diaphragms

C16.2.3.2.3 Acceptability Criteria It is conservative to assume only shear walls are stiff enough to divide the diaphragm span. In ASCE 41-17 (ASCE 2017b), the typical drift limit for new elements was 0.015. Case studies documented in FEMA P-2208 (FEMA 2023) showed that vertical elements, such as a moment frame, designed to meet the 0.0075 drift limit prescribed by this section reduce the diaphragm deflection significantly, effectively dividing the span. Therefore, the provision has been expanded beyond shear walls to include other, more flexible systems, provided they meet the 0.0075 drift limit. Diaphragm span lengths, *L*, for cases with single spans and double spans are shown in Figure C16-1.

C16.2.3.5 New Vertical Elements

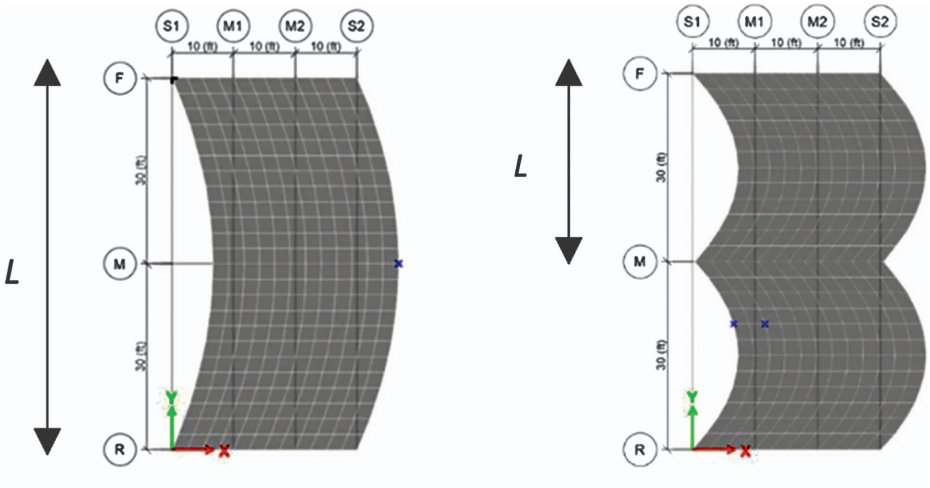
C16.2.3.5.2 Combinations of Vertical Elements

C16.2.3.5.2.1 Lateral Force Distribution Given the potential differences between the strength and stiffness of new vertical elements and existing masonry wall elements, it is possible that a new wall could be designed to carry 100% of the required forces for the wall line but have insufficient stiffness to attract significant loads away from the masonry elements, leading to substantial damage in the masonry before the new elements provided effective resistance. To address this, the standard requires that loads be shared by relative rigidity between the new and existing elements. To emphasize this, there is an explicit requirement that, after the loads have been distributed, the masonry must be evaluated using the provisions of Section 16.2.3.3. Clarity has been added to differentiate lateral load distributions for when vertical elements are in line with one another and when they are not in line. For stiffness assumptions to use in relative rigidity calculations, language from Chapter 11 is utilized; specifically, Section C11.3.2.1 provides guidance on veneer that should be used to determine pier stiffness.

In some cases, the existing masonry elements, such as in heavily punctured shear walls, may have insufficient strength to resist the loads they attract, and adding stiff enough new elements is impractical, particularly in the context of the Limited Performance Objective of the special procedure. In these situations, the new elements are designed for 100% of the required forces on the wall line; and, at that wall line, supports are added to rafters, girders, and joists per Section 16.2.4.4 and vertical bracing is added per Section 16.2.4.2.2, even where not required based on wall slenderness, in order to improve the stability of the existing wall.

C16.2.3.5.4 Forces on New Vertical Elements The forces to be used for the design of new elements are based on the forces and diaphragm capacities developed as part of the seismic evaluation using the special procedure. This approach is consistent with the traditional standard of practice for retrofitting existing unreinforced masonry wall buildings. It removes the provision that was added in ASCE 41-17 to require new elements to be evaluated per the provisions of Chapter 7, which have a different underlying design basis than was used to develop the special procedure demands.

C16.2.3.5.5 Acceptance Criteria for New Vertical Elements Capacities of new vertical elements are based on the referenced material standards, except that as indicated in Section C16.2.3, a strength reduction factor, φ, of 1.0 is used to be consistent with the 2021 IEBC Appendix Section A108.1.2. Existing and new footings supporting new vertical elements are required to be evaluated and designed to transfer loads into the supporting soil.



Single Span

Double Span

Figure C16-1. Diaphragm span lengths.

C16.2.3.5.6 Drift Limits The drift limit of 0.0075, together with the force levels determined in accordance with Section 16.2.3.5.4, was determined as part of three-dimensional 2-story and 6-story case study analysis models that included in-plane and out-of-plane masonry walls, semirigid wood diaphragms, and midspan steel moment frames. Moment frames designed to meet that stiffness were found not to attract the force demands assumed by the tributary area method assumed by the special procedure, but they were nonetheless found to be effective in significantly reducing midspan diaphragm displacements and in attracting substantial force away from parallel in-plane masonry walls. It should be recognized that moment frames, braced frames, or shear walls with lower drifts will further reduce midspan diaphragm deflections and attract more loads. Details on the case studies that were performed are described in FEMA P-2208 (FEMA 2023).

C16.2.4 Other Components and Systems of URM Buildings

C16.2.4.2 Out-of-Plane Demands Slender unreinforced masonry bearing walls with large height-to-thickness (*h/t*) ratios have a potential for damage caused by out-of-plane forces that may result in falling hazards and potential localized collapse of the structure.

The original table limiting *h/t* ratios was based on research by Agbabian et al. (1981) and has been used since the late 1980s to assess the stability of URM walls. ABK found that input velocity was well correlated with the out-of-plane response of URM walls. More recent research has led to the development of an equation-based check for URM out-of-plane stability, which is the method used in Chapter 11 and discussed in Chapter 11commentary. More recent research by Penner and Elwood (2016) has led to factors for load on the wall, wall thickness, and diaphragm type as well as *h/t*. For the special procedure, Table 16-6 has been expanded by using this newer procedure because the values in the existing table are unconservative for higher levels of shaking based on the results of recent research. The values in Table 16-6 are typically conservative—using no additional dead load—so the use of Chapter 11 equations is also permitted for those looking to refine their results. $C_{pl} = 1.1$ was chosen as most closely corresponding with the Collapse Prevention level and indicating a Probability of Out-of-Plane Collapse of 20% (see Table C11-1). Because of the limited research, *h/t* ratios of less than 8 are deemed acceptable regardless of seismicity.

For $S_{X1} \geq 0.4$ s, the ABK method (Agbabian et al. 1984) allowed buildings with cross walls and diaphragms meeting minimum demand/capacity ratios and maximum spans to use higher *h/t* ratios. These values were based on testing that showed the velocity amplification was reduced owing to nonlinear diaphragm behavior under those specific conditions. Bruneau (1994) summarizes the masonry testing and analysis that had been performed up to that time and used for the development of the table. When expanding Table 16-6, the Penner and Elwood equations for determining *h/t* limits did not include variables for cross walls or diaphragm behavior, although these variables do have an influence. Therefore, engineering judgment was used to extrapolate between the ABK method and the Penner and Elwood equations to determine values for Column A at $S_{X1} \geq$ 0.5 s. This extrapolation allows the special procedure to maintain past methods that are still valid, while updating for newer methods. S_{a1} was found by Penner and Elwood to be the best indicator of collapse potential regardless of diaphragm period. For this reason, S_{X1} in Table 16-6 and Section 16.2.4.2 is permitted to be based solely on the spectral response acceleration at 1 s and need not consider the additional requirements per Section 21.4 of ASCE 7, as referenced via Section 2.4 of this standard.

C16.2.4.3 Wall Anchorage Masonry walls that are not positively anchored to the diaphragms may separate from the structure, leading to a significant falling risk. If the walls are bearing walls, this separation may also lead to partial collapse of the floors and roof. Adding an anchor from the wall to the diaphragm corrects a portion of the load path, but if the anchor-to-diaphragm connection is not developed far enough into the diaphragm, the diaphragm may fail just beyond the end of the anchor-to-diaphragm connection. Out-of-plane wall failures have occurred where the retrofitted wall anchor and a joist running parallel to the wall pulled away from the diaphragm with the masonry wall because there was insufficient tension capacity in the diaphragm. To show that the out-of-plane force is developed into the diaphragm, the engineer must be able to draw a free-body diagram demonstrating the transfer of anchorage forces from one element to another and demonstrate that each load path element has adequate capacity. One example is transfer from the anchor bolt to a strap, into blocking, and from there into the diaphragm as shown in Figure C16-2. A common assumption is a linear force transfer from the blocking into the diaphragm when checking the blocking length is adequate for the diaphragm capacity.

Section 7.2.13.1 requires the anchorage force to be fully developed into the diaphragm, using subdiaphragms if necessary. As stated in Section C16.2.1, the unreinforced masonry special procedure is intended to reduce risk but at a Limited Performance Objective. Crossties and chords are not typically warranted in the special procedure because they can result in significant retrofit costs and interior disruptions that are often disproportionate to the benefits that they may provide. Reasonable development lengths into the diaphragm were developed based on parametric studies described in FEMA P-2208 (FEMA 2023). Single, straight-sheathed diaphragms and single, diagonal-sheathed diaphragms and systems other than those specified were not exempted from the subdiaphragm check because they lack the redundancy and extra strength of the specified wood diaphragms, particularly in the direction perpendicular to the board layup. Nail-laminated timber diaphragms consist of dimensional lumber on edge with boards flush next to each other and nails connecting the laminations. Nail-laminated timber diaphragms were historically built in place, with long, heavy nails passing through multiple boards and typically with sheathing, finish wood floor, or plywood overlay. Nail-laminated timber diaphragms have performed similar to diagonal sheathing with an overlay during seismic events.

Although crossties are not required in the special procedure, adding them will improve the building performance. If adding crossties is not possible, attaching the walls to longer elements that extend into the diaphragm, such as girders or cross-wall top plates that are perpendicular to the walls, will increase the building performance.

Only buildings where S_{X1} is greater than 0.2 are required to develop the anchorage to exempt low-risk locations. The 0.2 limit was chosen to match the diaphragm evaluation limit in Section 16.2.3.2.2.

C16.2.5 Detailing for New Elements Limiting systems to those permitted only in Seismic Design Category C or higher eliminates systems with poor ductility such as plain (unreinforced) concrete and masonry shear wall systems and ordinary concrete moment frames. Other ordinary, systems are

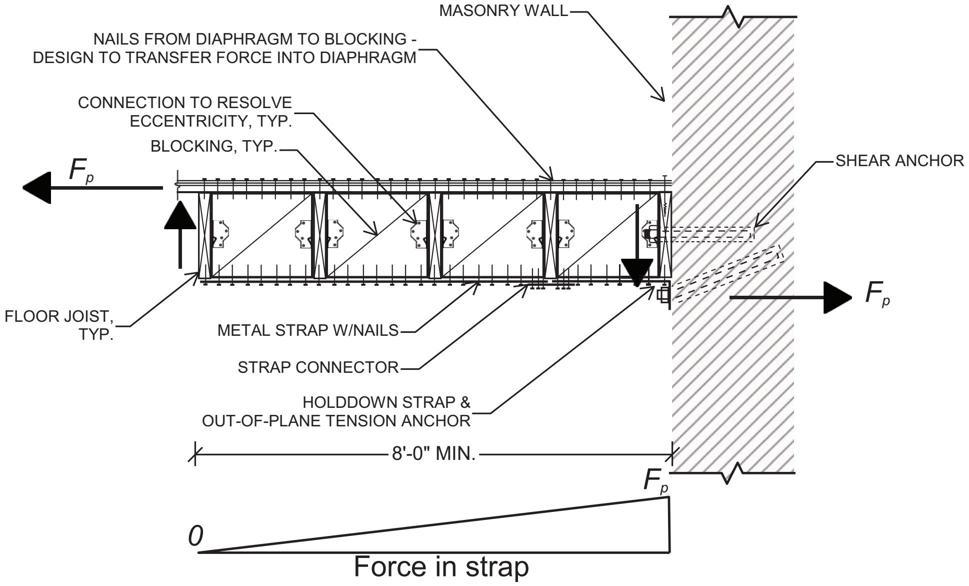


Figure C16-2. Out-of-plane force transfer from wall to diaphragm, joist parallel to wall.

typically not eliminated by this requirement. However, as described in FEMA P-2208 (2023), case studies have shown that the demands used to design elements in Section 16.2.3 have sufficient strength to place relatively limited ductility demands on new elements. Thus, ordinary detailing requirements, where prescribed by the applicable material standards, are considered sufficient to meet the Limited Performance Objective of Section 16.2.1. Strength-based design procedures should be used to be compatible with the force demands derived per Chapter 16. The Authority Having Jurisdiction is permitted to require additional detailing that is compliant with current building code requirements for new construction.

Regarding the exception for reinforced concrete shear walls, for ordinary reinforced concrete shear walls, ACI 318-19 has no additional seismic provisions in Chapter 18, "Earthquake Resistant Structures." The regular provisions in Chapter 11, "Walls," in ACI 318-19 require a minimum reinforcing ratio of 0.0025; spacing of a maximum of 18 in. (457.2 mm) or three times the wall thickness; and two curtains of reinforcing when the wall is over 10 in. (254 mm), but the provisions do not require hooks in horizontal bars at the ends of horizontal bars. To provide a reasonable improvement in ductility for a nominal effort in the field, hooks at the ends of walls are required for horizontal bars.

Section C11.3 provides recommended guidance for different retrofit methods for enhancing masonry walls. Quantitative provisions for capacities of selected enhancement procedures such as reinforced cores or fiber-reinforced polymer overlays are not available and are permitted to be determined through rational analysis or testing.

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CHAPTER C17

TIER 1 CHECKLISTS

C17.1 BASIC CHECKLISTS

The commentary reference section numbers after each evaluation statement (the last column in each table) refer to the sections in Appendix A regarding the statement's purpose and the corresponding Tier 2 evaluation procedures. If additional information on the evaluation statement is required, refer to the commentary in the Tier 2 procedure for that evaluation statement.

Refer to Table 3-1, and Table C3-1, for a general description of the building type associated with each individual checklist. The evaluation statements in the Collapse Prevention and Immediate Occupancy checklists are based on common seismic deficiencies and observed earthquake structural damage during actual earthquakes for this specific building type and are intended to provide the design professional with a general sense of the structure's potential deficiencies and behavior during an earthquake relative to the selected performance level.

C17.1.1 Very Low Seismicity Checklist The evaluation statements provided represent all of the required statements for buildings in Very Low Seismicity being evaluated for Collapse Prevention, including structural and nonstructural. The statements in the Very Low Seismicity Checklist need not be completed for buildings in Low, Moderate, and High Seismicity and for buildings in Very Low Seismicity being evaluated for Immediate Occupancy Performance Level because those statements are repeated where appropriate in the Basic Configuration Checklist and the building type checklists. Certain statements may not apply to the building being evaluated.

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

C17.1.2 Basic Configuration Checklist The evaluation statements in the Collapse Prevention and Immediate Occupancy checklists represent general configuration issues applicable for most buildings based on observed structural damage during actual earthquakes.

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

C17.2 STRUCTURAL CHECKLISTS FOR BUILDING TYPES W1: WOOD LIGHT FRAMES, SMALL RESIDENTIAL

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

C17.3 STRUCTURAL CHECKLISTS FOR BUILDING TYPE W2: WOOD FRAMES, LARGE RESIDENTIAL, COMMERCIAL, INDUSTRIAL, AND INSTITUTIONAL

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

C17.4 STRUCTURAL CHECKLISTS FOR BUILDING TYPES S1: STEEL MOMENT FRAMES WITH STIFF DIAPHRAGMS, AND S1A: STEEL MOMENT FRAMES WITH FLEXIBLE DIAPHRAGMS

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

C17.5 STRUCTURAL CHECKLISTS FOR BUILDING TYPES S2: STEEL BRACED FRAMES WITH STIFF DIAPHRAGMS, AND S2A: STEEL BRACED FRAMES WITH FLEXIBLE DIAPHRAGMS

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

C17.6 STRUCTURAL CHECKLISTS FOR BUILDING TYPE S3: METAL BUILDING FRAMES

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

C17.7 STRUCTURAL CHECKLISTS FOR BUILDING TYPE S4: DUAL SYSTEMS WITH BACKUP STEEL MOMENT FRAMES AND STIFF DIAPHRAGMS

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

Refer to Appendix A, Section A.3.1.3, for additional commentary on steel moment frames, Section A.3.2.2 for concrete shear walls, and Section A.3.3 for steel braced frames.

C17.8 STRUCTURAL CHECKLISTS FOR BUILDING TYPES S5: STEEL FRAMES WITH INFILL MASONRY SHEAR WALLS AND STIFF DIAPHRAGMS, AND S5A: STEEL FRAMES WITH INFILL MASONRY SHEAR WALLS AND FLEXIBLE DIAPHRAGMS

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

C17.9 STRUCTURAL CHECKLISTS FOR BUILDING TYPE CFS1: COLD-FORMED STEEL LIGHT-FRAME BEARING WALL CONSTRUCTION, SHEAR WALL LATERAL SYSTEM

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

Buildings of this type that have diaphragms of precast concrete planks are not permitted to be classified as this common building type and are not permitted to be evaluated using Tier 1 procedures.

C17.10 STRUCTURAL CHECKLISTS FOR BUILDING TYPE CFS2: COLD-FORMED STEEL LIGHT-FRAME BEARING WALL CONSTRUCTION, STRAP-BRACED LATERAL WALL SYSTEM

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

Buildings of this type that have diaphragms of precast concrete planks are not permitted to be classified as this common building type and are not permitted to be evaluated using Tier 1 procedures.

C17.11 STRUCTURAL CHECKLISTS FOR BUILDING TYPE C1: CONCRETE MOMENT FRAMES

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

Refer to Appendix A, Sections A.3.1, for additional commentary related to moment frames in general and Section A.3.1.4 for additional commentary related to concrete moment frames.

C17.12 STRUCTURAL CHECKLISTS FOR BUILDING TYPES C2: CONCRETE SHEAR WALLS WITH STIFF DIAPHRAGMS, AND C2A: CONCRETE SHEAR WALLS WITH FLEXIBLE DIAPHRAGMS

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

Refer to Appendix A, Sections A.3.2.1 and A.3.2.2 for additional commentary related to concrete shear walls.

C17.13 STRUCTURAL CHECKLISTS FOR BUILDING TYPES C3: CONCRETE FRAMES WITH INFILL MASONRY SHEAR WALLS, AND C3A: CONCRETE FRAMES WITH INFILL MASONRY SHEAR WALLS AND FLEXIBLE DIAPHRAGMS

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

C17.14 STRUCTURAL CHECKLISTS FOR BUILDING TYPES PC1: PRECAST OR TILT-UP CONCRETE SHEAR WALLS WITH FLEXIBLE DIAPHRAGMS, AND PC1A: PRECAST OR TILT-UP CONCRETE SHEAR WALLS WITH STIFF DIAPHRAGMS

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

Refer to Appendix A, Section A.3.2, for additional commentary related to shear walls in general, and Section A.3.2.3 for commentary related to precast shear walls.

C17.15 STRUCTURAL CHECKLISTS FOR BUILDING TYPE PC2: PRECAST CONCRETE FRAMES WITH SHEAR WALLS

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

C17.16 STRUCTURAL CHECKLISTS FOR BUILDING TYPE PC2A: PRECAST CONCRETE FRAMES WITHOUT SHEAR WALLS

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

C17.17 STRUCTURAL CHECKLISTS FOR BUILDING TYPES RM1: REINFORCED MASONRY BEARING WALLS WITH FLEXIBLE DIAPHRAGMS, AND RM2: REINFORCED MASONRY BEARING WALLS WITH STIFF DIAPHRAGMS

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

C17.18 STRUCTURAL CHECKLISTS FOR BUILDING TYPES URM: UNREINFORCED MASONRY BEARING WALLS WITH FLEXIBLE DIAPHRAGMS, AND URMA: UNREINFORCED MASONRY BEARING WALLS WITH STIFF DIAPHRAGMS

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

C17.19 NONSTRUCTURAL CHECKLIST

Refer to Section C17.1 for general commentary related to the Tier 1 checklists.

Checklist items are grouped by system or component type. Each item is preceded by an annotation indicating the Level(s) of Seismicity for which it is required, given a desired performance level. The performance level is designated by HR for Hazards Reduced, LS for Life Safety, or PR for Position Retention. The Level of Seismicity is designated by L, M, or H, for Low, Moderate, and High, respectively. For example, the annotation "HR—not required; LS—H; PR—LMH" indicates that the checklist item is not required when the performance level is Hazards Reduced regardless of Level of Seismicity, is required in High Seismicity when the performance level is Life Safety, and is required in Low, Moderate, or High Seismicity when the performance level is Position Retention.

The commentary reference section numbers listed after each evaluation statement refer to the section in Appendix A regarding the statement's purpose and the corresponding Tier 2 evaluation procedures using Chapter 13. Refer to the pertinent sections in Chapter 13 for additional information for the nonstructural components.

CHAPTER C18
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