طراحی لرزه‌ای براساس عملکرد در ساختمان‌های بتن‌آرمه براساس ACI 318-19 و پیوست دوم استاندارد ۲۸۰۰

جواد قدرتی ینگجه

کارشناس ارشد زلزله از دانشگاه صنعتی شریف

عضو کارگروه طراحی و بهسازی سازمان نظام مهندسی ساختمان استان تهران

بخش اول (از دو بخش)

۷ دی ماه ۱۴۰۰

سازمان نظام مهندسی ساختمان خوزستان

موسسه آموزش عالی جهاد دانشگاهی خوزستان

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

برای دریافت اطلاعات بیشتر در زمینه طراحی عملکردی، تحلیل‌های غیرخطی و بهسازی لرزه‌ای می‌توانید به لینک‌های زیر مراجعه کنید:

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آیدی تلگرامی : @PBDEng

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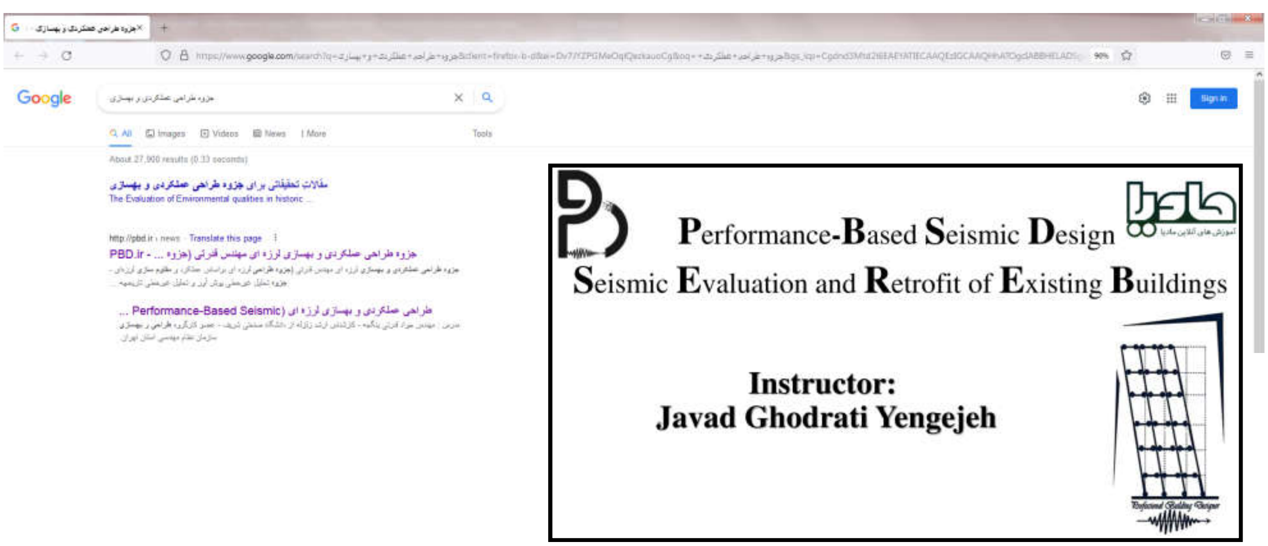
جزوه طراحی عملکردی و بهسازی لرزه‌ای



طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

رئوس مطالب برای وبینار اول:

۱-شرح مختصری بر فلسفه طراحی لرزه‌ای براساس عملکرد

۲-معرفی پروژه بزرگ ATC 114، راهنمای ACI 374، استاندارد ACI 369.1، استاندارد ASCE 41-17 و اپندیکس A از استاندارد ACI 318-19

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

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طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

پیوست ۲

راهنمای انجام تحلیل‌های غیرخطی

۱- کلیات

تحلیل غیرخطی سازه‌ها در برابر زلزله به دو روش تحلیل استاتیکی غیرخطی (بند ۳) و تحلیل تاریخچه زمانی غیرخطی (بند ۴) قابل انجام می‌باشند. روش‌های تحلیل غیرخطی برای تخمین سازوکارها (مکانیزم‌های) خمیری موردانتظار، توزیع آسیب‌های وارده و ارزیابی عملکرد سازه قابل استفاده می‌باشند.

سبک سازی با استفاده از طراحی لرزه‌ای براساس عملکرد مجاز است یا خیر؟

در صورت استفاده از طراحی لرزه‌ای براساس عملکرد می‌توان ضوابط تجویزی ACI 318-19 و ASCE 7-22 را گرفت؟

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

۳-۲-۳ روش‌های تحلیل غیرخطی

روش‌های تحلیل غیرخطی را می‌توان درکلیه ساختمان‌ها با هرتعداد طبقه به‌کاربرد، ولی برای استفاده از آنها ضروری است سازه علاوه بـر اقنـاع الزامـات آنهـا، ضـوابط تحلیـل و طراحی یکی از روش‌های خطی عنوان‌شده در بند (۳-۲-۲) را نیز اقنـاع نمایـد. الزامـات مربوط به روش‌های تحلیل غیرخطی در پیوست شماره (۲) ارائه شده است.

۳-۲-۲ روش‌های تحلیل خطی

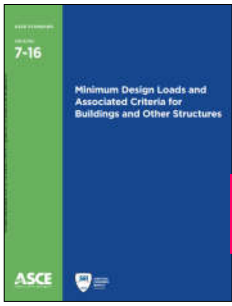
روش‌های تحلیل خطی را می‌توان درکلیه ساختمان‌ها با هرتعداد طبقه بـه‌کـاربرد. تنهـا، روش استاتیکی معادل را می‌توان در ساختمان‌های سه‌طبقه و کوتاه‌تر، از تراز پایـه و یـا ساختمان‌های زیر به‌کار گرفت:

الف- ساختمان‌های منظم با ارتفاع کمتر از ۵۰ متر از تراز پایه

ب- ساختمان‌های نامنظم با ارتفاع کمتر از ۵۰ متر از تراز پایه که دارای:

- نامنظمی زیاد و شدید پیچشی در پلان نباشد
- نامنظمی جرمی، نرم و خیلی نرم در ارتفاع نباشد

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)



ASCE 7-16

CHAPTER 16

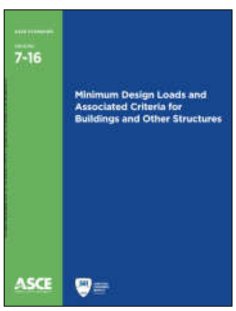
NONLINEAR RESPONSE HISTORY ANALYSIS

16.1 GENERAL REQUIREMENTS

16.1.1 Scope. It shall be permitted to use nonlinear response history analysis in accordance with the requirements of this chapter to demonstrate acceptable strength, stiffness, and ductility to resist maximum considered earthquake (MCE_R) shaking with acceptable performance. When nonlinear response history analysis is performed, the design shall also satisfy the requirements of Section 16.1.2. Nonlinear response history analysis shall include the effects of horizontal motion, and where required by Section 16.1.3, vertical motion. Documentation of the design and analysis shall be prepared in accordance with Section 16.1.4. Ground motion acceleration histories shall be selected and modified in accordance with the procedures of Section 16.2. The structure shall be modeled and analyzed in accordance with the criteria in Section 16.3. Analysis results shall meet the acceptance criteria of Section 16.4. Independent structural design review shall be performed in accordance with the requirements of Section 16.5.

16.1.2 Linear Analysis. In addition to nonlinear response history analysis, a linear analysis in accordance with one of the applicable procedures of Chapter 12 shall also be performed. The structure's design shall meet all applicable criteria of Chapter 12. Where soil–structure interaction in accordance with Chapter 19 is used in the nonlinear analysis, it shall be permitted to also use the corresponding spectral adjustment in the linear analysis.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)



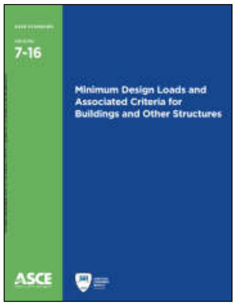
ASCE 7-16

16.1.2 Linear Analysis. In addition to nonlinear response history analysis, a linear analysis in accordance with one of the applicable procedures of Chapter 12 shall also be performed. The structure's design shall meet all applicable criteria of Chapter 12. Where soil–structure interaction in accordance with Chapter 19 is used in the nonlinear analysis, it shall be permitted to also use the corresponding spectral adjustment in the linear analysis.

EXCEPTIONS:

1. For Risk Category I, II, and III structures, Sections 12.12.1 and 12.12.5 do not apply to the linear analysis. Where mean computed drifts from the nonlinear analyses exceed 150% of the permissible story drifts per Section 12.12.1, deformation-sensitive nonstructural components shall be designed for 2/3 of these mean drifts.
2. The overstrength factor, Ω_0, is permitted to be taken as 1.0 for the seismic load effects of Section 12.4.3.
3. The redundancy factor, ρ, is permitted to be taken as 1.0.
4. Where accidental torsion is explicitly modeled in the nonlinear analysis, it shall be permitted to take the value of A_x as unity in the Chapter 12 analysis.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)



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CHAPTER 16

NONLINEAR RESPONSE HISTORY ANALYSIS

EXCEPTIONS:

1. For Risk Category I, II, and III structures, Sections 12.12.1 and 12.12.5 do not apply to the linear analysis. Where mean computed drifts from the nonlinear analyses exceed 150% of the permissible story drifts per Section 12.12.1, deformation-sensitive nonstructural components shall be designed for 2/3 of these mean drifts.
2. The overstrength factor, Ω_0, is permitted to be taken as 1.0 for the seismic load effects of Section 12.4.3.
3. The redundancy factor, ρ, is permitted to be taken as 1.0.
4. Where accidental torsion is explicitly modeled in the nonlinear analysis, it shall be permitted to take the value of A_x as unity in the Chapter 12 analysis.

12.12 DRIFT AND DEFORMATION

12.12.1 Story Drift Limit. The design story drift (Δ) as determined in Sections 12.8.6, 12.9.1, or 12.9.2 shall not exceed the allowable story drift (Δ_a) as obtained from Table 12.12-1 for any story.

12.12.1.1 Moment Frames in Structures Assigned to Seismic Design Categories D through F. For seismic force-resisting systems solely comprising moment frames in structures assigned to Seismic Design Categories D, E, or F, the design story drift (Δ) shall not exceed Δ_a/ρ for any story. ρ shall be determined in accordance with Section 12.3.4.2.

Risk Categories (ASCE 7-16, Table 1.5-1)

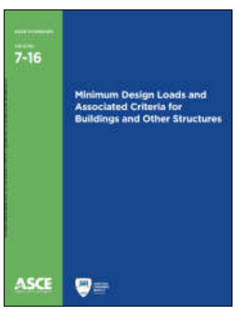
Risk Category	Description	Seismic Importance Factor, I_e
IV	**Essential facilities** (Hospitals, fire and police stations, emergency shelters, etc) **Structures containing extremely hazardous materials**	1.5
III	**Structures that pose a substantial hazard to human life in the event of failure** (buildings with 300 people in one area, day care facilities with capacity more than 150, schools with a capacity more than 250, etc)	1.25
II	**Buildings not in Occupancy Categories I, III, or IV** (most buildings)	1.0
I	**Buildings that represent a low hazard to human life in the event of failure** (agricultural facilities, temporary facilities, minor storage facilities)	1.0

Table 12.12-1 Allowable Story Drift, Δ_a[a,b]

Structure	Risk Category I or II	Risk Category III	Risk Category IV
Structures, other than masonry shear wall structures, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	$0.025h_{sx}$[c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below level x.
[b] For seismic force-resisting systems solely comprising moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.
[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support that are so constructed that moment transfer between shear walls (coupling) is negligible.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)



ASCE 7-16

EXCEPTIONS:

1. For Risk Category I, II, and III structures, Sections 12.12.1 and 12.12.5 do not apply to the linear analysis. Where mean computed drifts from the nonlinear analyses exceed 150% of the permissible story drifts per Section 12.12.1, deformation-sensitive nonstructural components shall be designed for 2/3 of these mean drifts.
2. The overstrength factor, Ω_0, is permitted to be taken as 1.0 for the seismic load effects of Section 12.4.3.
3. The redundancy factor, ρ, is permitted to be taken as 1.0.
4. Where accidental torsion is explicitly modeled in the nonlinear analysis, it shall be permitted to take the value of A_x as unity in the Chapter 12 analysis.

12.12.5 Deformation Compatibility for Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E, or F, every structural component not included in the seismic force-resisting system in the direction under consideration shall be designed to be adequate for the gravity load effects and the seismic forces resulting from displacement caused by the design story drift (Δ) as determined in accordance with Section 12.8.6 (see also Section 12.12.1).

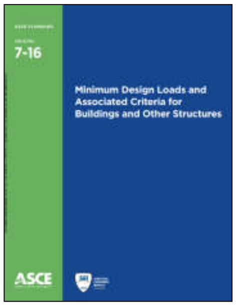
EXCEPTION: Reinforced concrete frame members not designed as part of the seismic force-resisting system shall comply with Section 18.14 of ACI 318.

Where determining the moments and shears induced in components that are not included in the seismic force-resisting system in the direction under consideration, the stiffening effects of adjoining rigid structural and nonstructural elements shall be considered, and a rational value of member and restraint stiffness shall be used.

Risk Categories (ASCE 7-16, Table 1.5-1)

Risk Category	Description	Seismic Importance Factor, I_e
IV	**Essential facilities** (Hospitals, fire and police stations, emergency shelters, etc) **Structures containing extremely hazardous materials**	1.5
III	**Structures that pose a substantial hazard to human life in the event of failure** (buildings with 300 people in one area, day care facilities with capacity more than 150, schools with a capacity more than 250, etc)	1.25
II	**Buildings not in Occupancy Categories I, III, or IV** (most buildings)	1.0
I	**Buildings that represent a low hazard to human life in the event of failure** (agricultural facilities, temporary facilities, minor storage facilities)	1.0

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)



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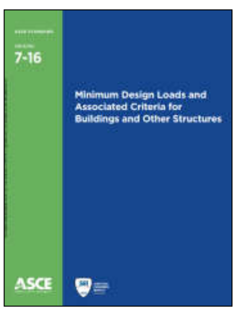
CHAPTER 16

NONLINEAR RESPONSE HISTORY ANALYSIS

C16.1.2. Linear Analysis. As a precondition to performing nonlinear response history analysis, a linear analysis in accordance with the requirements of Chapter 12 is required. Any of the linear procedures allowed in Chapter 12 may be used. The purpose of this requirement is to ensure that structures designed using nonlinear response history analyses meet the minimum strength and other criteria of Chapter 12, with a few exceptions.

In particular, when performing the Chapter 12 evaluations it is permitted to take the value of Ω_0 as 1.0 because it is felt that values of demand obtained from the nonlinear procedure is a more accurate representation of the maximum forces that will be delivered to critical elements, considering structural overstrength, than does the application of the judgmentally derived factors specified in Chapter 12.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)



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NONLINEAR RESPONSE HISTORY ANALYSIS

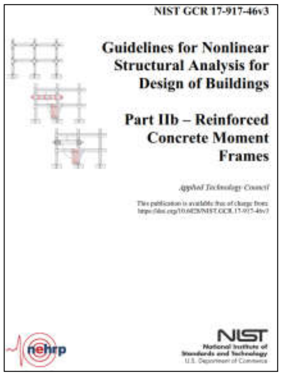
16.4.1.2 Story Drift. The mean story drift ratio shall not exceed two times the limits of Table 12.12-1. The story drift ratio shall be computed as the largest difference of the deflections of vertically aligned points at the top and bottom of the story under consideration along any of the edges of the structure. For masonry shear wall structures, the limits of Table 12.12-1 applicable to masonry structures shall not apply and these structures shall instead comply with the limits for other structures.

Table 12.12-1 Allowable Story Drift, Δ_a[a,b]

Structure	Risk Category: I or II	III	IV
Structures, other than masonry shear wall structures, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	$0.025h_{sx}$[c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below level x.
[b] For seismic force-resisting systems solely comprising moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.
[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support that are so constructed that moment transfer between shear walls (coupling) is negligible.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)



NIST GCR 17-917-46v3

B.2 Building Description

The example building is a five-story concrete office building located in San Francisco, California.

The site is located on Soil Type D and per ASCE/SEI 7-16 the building is designed for Seismic Design Category D. The site's location in San Francisco is outside the near-fault zone and within the deterministic cap.

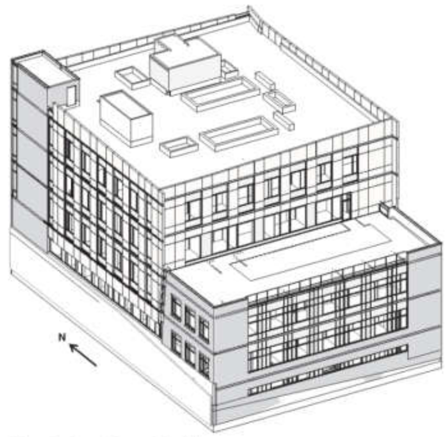
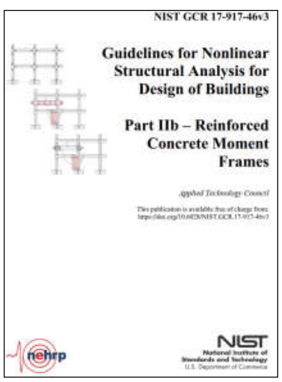


Figure B-1 Example building.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)



NIST GCR 17-917-46v3

B.3 Linear Structural Analysis and Design of Building

The example building is proportioned and designed under elastic analysis per prescriptive requirements of ASCE/SEI 7-16 and ACI 318-14, *Building Code Requirements for Reinforced Concrete* (ACI, 2014). The elastic analysis and design process is not documented in its entirety as the interest of this guideline is focused on the nonlinear analysis and design. However, some important elastic modeling assumptions, analysis results, and structural designs are provided below.

B.3.4 Drift Check

The Story drifts are computed in accordance with ASCE/SEI 7-16 Section 12.9.1.4.2 and then checked for acceptance based on Section 12.12.1. According to ASCE/SEI 7-16 Table 12.12-1, the story drift limit for this Occupancy Category II building is 2 percent of the story height. The story drifts are taken directly from the modal combinations in ETABS. A plot of the total deflection in both the N-S and E-W directions is shown in Figure B-5 and a plot of story drifts is in Figure B-6.

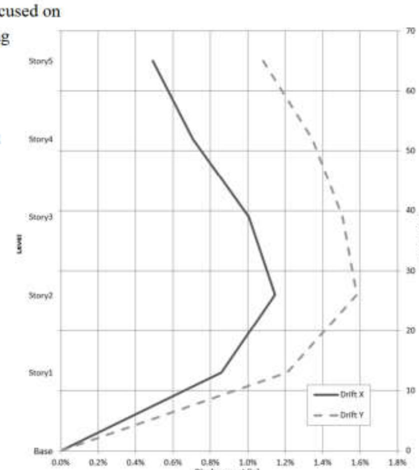


Figure B-6 Maximum story drift.

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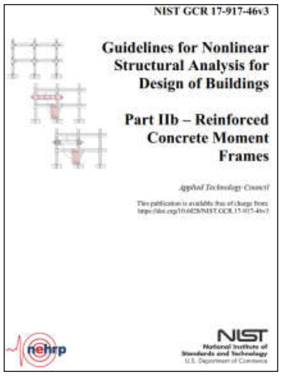


NIST GCR 17-917-46v3

B.5.2 Story Drift Check

In accordance with ASCE/SEI 7-16 Section 16.4.1.2, the mean story drift ratio is checked as a part of the global acceptance criteria. The allowable story drift limit is two times the limit employed for the elastic analysis. For the example building, which is Risk Category II and masonry-free, the allowable story drift ratio under nonlinear response history analysis is 4%. The mean story drift ratios in E-W and N-S directions are illustrated in Figures B-18 and B-19, respectively. It is observed that the building meets the drift criterion as the mean peak drifts at all levels and directions are less than 4%.

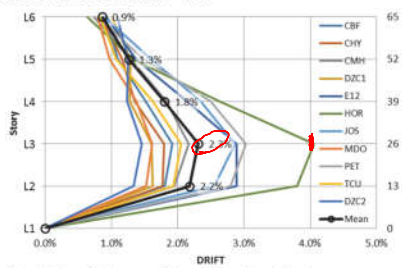


Figure B-18 Maximum peak story drift in east-west direction.

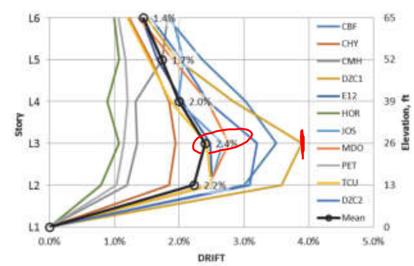
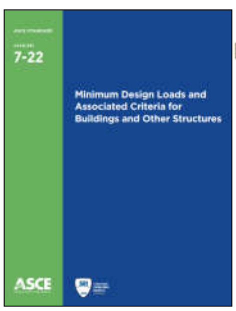


Figure B-19 Maximum peak story drift in north-south direction.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)



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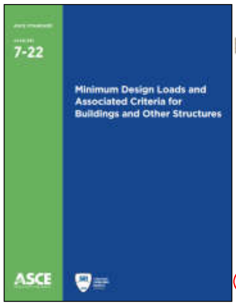
CHAPTER 16
NONLINEAR RESPONSE HISTORY ANALYSIS

16.1 GENERAL REQUIREMENTS

16.1.1 Scope It shall be permitted to use nonlinear response history analysis, in accordance with the requirements of this chapter, to demonstrate acceptable strength, stiffness, and ductility to resist maximum considered earthquake (MCE_R) shaking with acceptable performance. When nonlinear response history analysis is performed, the design shall also satisfy the requirements of Section 16.1.2. Nonlinear response history analysis shall include the effects of horizontal motion, and where required by Section 16.1.3, vertical motion. Documentation of the design and analysis shall be prepared in accordance with Section 16.1.4. Ground motion acceleration histories shall be selected and modified in accordance with the procedures of Section 16.2. The structure shall be modeled and analyzed in accordance with the criteria in Section 16.3. Analysis results shall meet the acceptance criteria of Section 16.4. Independent structural design review shall be performed in accordance with the requirements of Section 16.5.

16.1.2 Linear Analysis In addition to nonlinear response history analysis, a linear analysis in accordance with one of the applicable procedures of Chapter 12 shall also be performed. The structure's design shall meet all applicable criteria of Chapter 12.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)



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CHAPTER 16

NONLINEAR RESPONSE HISTORY ANALYSIS

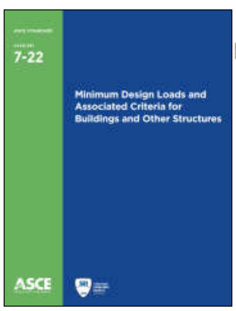
16.1.2 Linear Analysis In addition to nonlinear response history analysis, a linear analysis in accordance with one of the applicable procedures of Chapter 12 shall also be performed. The structure's design shall meet all applicable criteria of Chapter 12.

12.12.4 Deformation Compatibility for Seismic Design Categories D through F

EXCEPTIONS:

1. For Risk Category I, II, and III structures, Sections 12.12.1 and 12.12.5 do not apply to the linear analysis. Where mean computed drifts from the nonlinear analyses exceed 150% of the permissible story drifts per Section 12.12.1, deformation-sensitive nonstructural components shall be designed for 2/3 of these mean drifts.
2. The overstrength factor, Ω_0, is permitted to be taken as 1.0 for the seismic load effects of Section 12.4.3.
3. The redundancy factor, ρ, is permitted to be taken as 1.0.
4. Where accidental torsion is explicitly modeled in the nonlinear analysis, it shall be permitted to take the value of A_x as unity in the Chapter 12 analysis.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)



ASCE 7-22

CHAPTER 16

NONLINEAR RESPONSE HISTORY ANALYSIS

C16.1.2 Linear Analysis As a precondition to performing nonlinear response history analysis, a linear analysis, in accordance with the requirements of Chapter 12, is required. Any of the linear procedures allowed in Chapter 12 may be used. The purpose of this requirement is to ensure that structures designed using nonlinear response history analyses meet the minimum strength and other criteria of Chapter 12, with a few exceptions.

In particular, when performing the Chapter 12 evaluations, it is permitted to take the value of Ω_0 as 1.0 because it is felt that values of demand obtained from the nonlinear procedure are a more accurate representation of the maximum forces that will be delivered to critical elements, considering structural overstrength, than does the application of the judgmentally derived factors specified in Chapter 12. Similarly, it is permitted to use a value of 1.0 for the redundancy factor, ρ, because it is felt that the inherent nonlinear evaluation of response to MCE_R shaking required by this chapter provides improved reliability relative to the linear procedures of Chapter 12. For Risk Category I, II, or III structures, it is permitted to neglect the evaluation of story drift when using the linear procedure because it is felt that the drift evaluation performed using the nonlinear procedure provides a more accurate assessment of the structure's tolerance to earthquake-induced drift. However, linear drift evaluation is required for Risk Category IV structures because it is felt that this level of drift control is important to attaining the enhanced performance desired for such structures.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)


APPENDIX A—DESIGN VERIFICATION USING NONLINEAR RESPONSE HISTORY ANALYSIS

ACI 318-19 (Appendix A)

A.2—Scope

A.2.2 The provisions of Appendix A shall be in addition to the provisions of Chapters 1 through 26.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

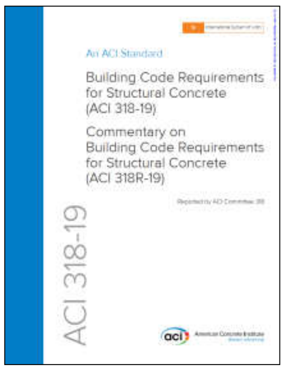
چرا استاندارد ۲۸۰۰ و استاندارد ASCE 7 در طراحی لرزه‌ای براساس عملکرد، تحلیل‌های خطی تجویزی را اساسِ حداقل مقاومت سازه لحاظ می‌کنند؟

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Design Strength & Required Strength

1-Lower Bound Material Strength **2-Expected Material Strength**

کاربرد مقاومت مصالح کرانه پایین و مقاومت مصالح مورد انتظار در موارد زیر چه تفاوتی با یکدیگر دارد؟



- **طراحی عادی براساس ACI 318-19**
- **طراحی لرزه‌ای براساس ACI 318-19**
- **طراحی لرزه‌ای براساس عملکرد ACI 318-19**

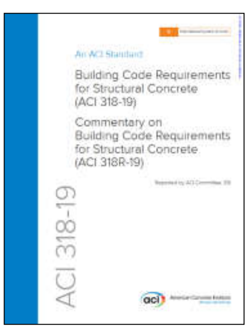
seismic reliability targets specified in ASCE/SEI 7

The bias factor, ***B***

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Design Strength & Required Strength

ACI 318-19



4.6—Strength

4.6.1 Design strength of a member and its joints and connections, in terms of moment, shear, torsional, axial, and bearing strength, shall be taken as the nominal strength S_n multiplied by the applicable strength reduction factor ϕ.

4.6.2 Structures and structural members shall have design strength at all sections, ϕS_n, greater than or equal to the required strength U calculated for the factored loads and forces in such combinations as required by this Code or the general building code.

design strength ≥ required strength

$$\phi S_n \geq U$$

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Non-seismic Design According to ACI 318-19

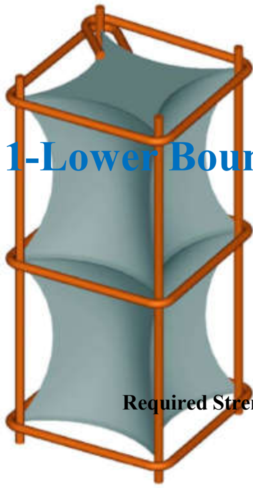
- Analysis (Static Loading)
  - Equivalent Static
    - DC Actions
      - Demand → Load Combinations
      - Capacity → Lower Bound Material Strength → Design Strength (ϕS_n)
    - FC Actions
      - Demand → Load Combinations
      - Capacity → Lower Bound Material Strength → Design Strength (ϕS_n)
  - Response-Spectrum
    - DC Actions
      - Demand → Load Combinations
      - Capacity → Lower Bound Material Strength → Design Strength (ϕS_n)
    - FC Actions
      - Demand → Load Combinations
      - Capacity → Lower Bound Material Strength → Design Strength (ϕS_n)

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Seismic Design According to ACI 318-19

- Analysis (Static Loading)
  - Equivalent Static
    - DC Actions
      - Demand → Seismic Provisions (e.g. Weak Beam-Strong Column)
      - Capacity → Lower Bound Material Strength → Design Strength (ϕS_n)
    - FC Actions
      - Demand → Yielding of DC Actions with Expected Strength $(\Omega_0 E \ , \ tensil \ \ stress = 1.25 f_y)$
      - Capacity → Lower Bound Material Strength → Design Strength (ϕS_n)
  - Response-Spectrum
    - DC Actions
      - Demand → Seismic Provisions (e.g. Weak Beam-Strong Column)
      - Capacity → Lower Bound Material Strength → Design Strength (ϕS_n)
    - FC Actions
      - Demand → Yielding of DC Actions with Expected Strength $(\Omega_0 E \ , \ tensil \ \ stress = 1.25 f_y)$
      - Capacity → Lower Bound Material Strength → Design Strength (ϕS_n)

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)



Design Strength & Required Strength

1-Lower Bound Material Strength **2-Expected Material Strength**

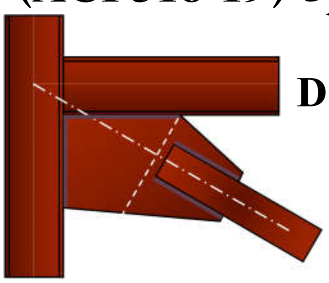
تمامی مفروضات محتمل در واقعیت (اجرای ساختمان)، که منجر به **افزایش نیاز لرزه‌ای** می‌گردد، بایستی به نحو مناسبی در طراحی لرزه‌ای وارد شود.

- برای محاسبه مقاومت خمشی مقطع از مقاومت کران پایین آرماتورهای طولی استفاده می‌شود. **Design Strength**
- برای محاسبه مقاومت برشی مقطع از مقاومت کران پایین مصالح استفاده می‌شود. **Design Strength**
- برای محاسبه نیاز برشی لرزه‌ای مقطع، فرض می‌شود آرماتورهای طولی با مقاومت تسلیم مورد انتظار، در خمش جاری می‌شوند. **Required Strength**
- بنابراین در روند طراحی تجویزی، برای طراحی خمشی مقطع، **مقاومت کرانه پایین آرماتورهای طولی**، و برای طراحی برشی، **مقاومت مورد انتظار آرماتورهای طولی** ملاک محاسبات خواهد بود. لذا در روند طراحی براساس ACI 318-19، یک آرماتور طولی مشخص هم با مقاومت کرانه پایین وارد محاسبات شده و هم با مورد انتظار ! ...

Longitudinal Reinforcement Yield Stress

$$\begin{cases} P-M_2-M_3\ (DC) \Rightarrow \text{Capacity} \Rightarrow f_s = f_{yL} \\ V_2\ or\ V_3 (FC) \Rightarrow \text{Demand} \Rightarrow f_s = 1.25 f_y \end{cases}$$

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)



Design Strength & Required Strength

تمامی مفروضات محتمل در واقعیت (اجرای ساختمان)، که منجر به **افزایش نیاز لرزه‌ای** می‌گردد، بایستی به نحو مناسبی در طراحی لرزه‌ای وارد شود.

1-Lower Bound Material Strength

2-Expected Material Strength

- برای محاسبه مقاومت تسلیم کششی مقطع مهاربند، از مقاومت کران پایین مصالح پروفیل فولادی استفاده می‌شود. **Design Strength**
- برای محاسبه مقاومت محوری فشاری ستون کنار مهاربند از مقاومت کران پایین مصالح پروفیل فولادی استفاده می‌شود. **Design Strength**
- برای محاسبه نیاز لرزه‌ای ستون کنار مهاربند (نیروی محوری فشاری لرزه‌ای)، فرض می‌شود مهاربندها با مقاومت تسلیم مورد انتظار، در کشش جاری می‌شوند. **Required Strength**
- بنابراین در روند طراحی تجویزی، برای طراحی کششی مهاربند، مقاومت کرانه پایین تسلیم کششی مهاربند و برای محوری فشاری ستون، مقاومت مورد انتظار تسلیم کششی مهاربند ملاک محاسبات خواهد بود. لذا در روند طراحی براساس AISC 360-16 و AISC 341-16، یک مهاربند مشخص هم با مقاومت کرانه پایین وارد محاسبات شده و هم با مورد انتظار ! ...

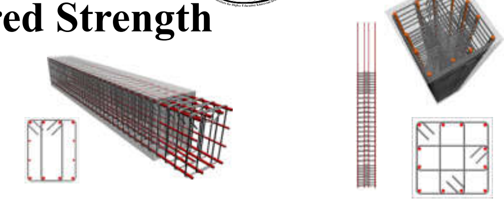
CBF Elements Yield Stress

$$\begin{cases} \text{Brace: Axial P (DC)} \Rightarrow \text{Capacity} \Rightarrow F_s = F_{yL} \\ \text{Column: Axial P\_Compression\_(FC)} \Rightarrow \text{Demand} \Rightarrow F_s = F_{yE} = R_y F_y \end{cases}$$

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Design Strength & Required Strength

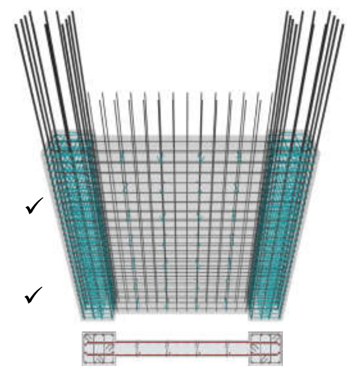
DC Actions
- Demand — Load combination & Seismic Provisions
- Capacity — Lower Bound Material Strength



FC Actions
- Demand — Yielding of DC Actions with Expected Strength $(\Omega_0 E\ ,\ f_{ye} = 1.25 f_y\ ,\ F_{ye} = R_y F_y)$

$$V_e = \Omega_v \omega_v V_u \le 3V_u$$

- Capacity — Lower Bound Material Strength



✓ در آنالیز خطی و طراحی تجویزی اثرات مقاومت مورد انتظار تلاش‌های DC برای محاسبه Demand این تلاش‌ها مغفول مانده است.

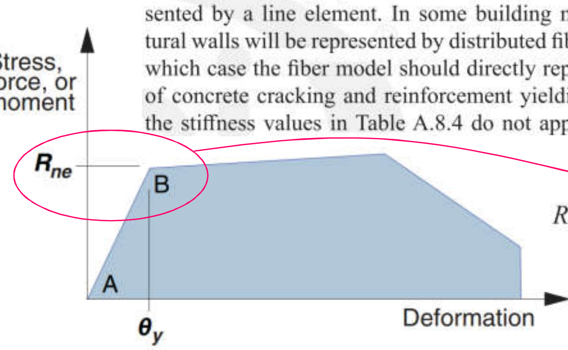
✓ برای لحاظ کردن مقاومت مورد انتظار تلاش‌های DC در روند تحلیل و طراحی چیست؟

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Performan-Based Seismic Design According to ACI 318-19

ACI 318-19 (Appendix A)

RA.8.4 The effective stiffness values are intended to represent the slope from A to B in Fig. RA.8.3, where B corresponds to expected yield strength. Effective stiffness values for beams and columns are based on Elwood et al. (2007), and incorporate the effects of reinforcement slip along the development length. Tabulated values for structural walls are appropriate to use where the wall is represented by a line element. In some building models, structural walls will be represented by distributed fiber models, in which case the fiber model should directly represent effects of concrete cracking and reinforcement yielding, such that the stiffness values in Table A.8.4 do not apply.



R_{ne} = expected yield strength

Fig. RA.8.3—Generalized force-deformation relations.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Performan-Based Seismic Design According to ACI 318-19

ACI 318-19 (Appendix A)

A.11—Expected strength for force-controlled actions

A.11.1 Force-controlled actions shall be evaluated in accordance with the general building code, with expected strength taken as $\phi_s B R_n$.

A.11.2 ϕ_s shall be in accordance with Table A.11.2, with ϕ determined in accordance with Chapter 21, except that 21.2.4.1 shall not apply.

Table A.11.2—Seismic resistance factor

Force-controlled action	ϕ_s
Critical	ϕ
Ordinary	$\phi/0.9 \leq 1.0$
Noncritical	$\phi/0.85 \leq 1.0$

A.11.3 Bias factor, B, shall be taken as 1.0. Alternatively, it shall be permitted to calculate B using Eq. (A.11.3):

$$B = 0.9R_{ne}/R_n \geq 1.0 \qquad \text{(A.11.3)}$$

A.11.3.1 Nominal strength, R_n, shall be in accordance with Chapter 18, 22, or 23.

A.11.3.2 The expected strength, R_{ne}, is permitted to be defined in accordance with the nominal strength provisions of Chapters 18, 22, or 23, with f_{ce}' substituted for f_c' and f_{ye} substituted for f_y or f_{yt}, except as provided in A.11.3.2.1 and A.11.3.2.2.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

با توجه به ضوابط فصل ۱۶ از استاندارد ASCE 7-22، علاوه بر ضوابط تحلیل غیرخطی تاریخچه زمانی بایستی ضوابط فصل ۱۲ این استاندارد نیز اغنا گردد (با چند استثنا). بنابراین حداقل ضوابط فصل ۱۲ اجباری بوده و کمتر از آنها مجاز نخواهد بود. لذا استفاده از طراحی لرزه‌ای براساس عملکرد به قصد سبک سازی مجاز نمی‌باشد.

طراحی لرزه‌ای براساس عملکرد در ساختمان‌های بتن‌آرمه براساس ACI 318-19 و پیوست دوم استاندارد ۲۸۰۰

جواد قدرتی ینگجه
کارشناس ارشد زلزله از دانشگاه صنعتی شریف
عضو کارگروه طراحی و بهسازی سازمان نظام مهندسی ساختمان استان تهران

بخش دوم (از دو بخش)

۱۴ دی ماه ۱۴۰۰

موسسه آموزش عالی جهاد دانشگاهی خوزستان
سازمان نظام مهندسی ساختمان خوزستان
انجمن علمی مهندسی عمران

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

برای دریافت اطلاعات بیشتر در زمینه طراحی عملکردی، تحلیل‌های غیرخطی و بهسازی لرزه‌ای می‌توانید به لینک‌های زیر مراجعه کنید:

وبسایت PBD : www.PBD.ir

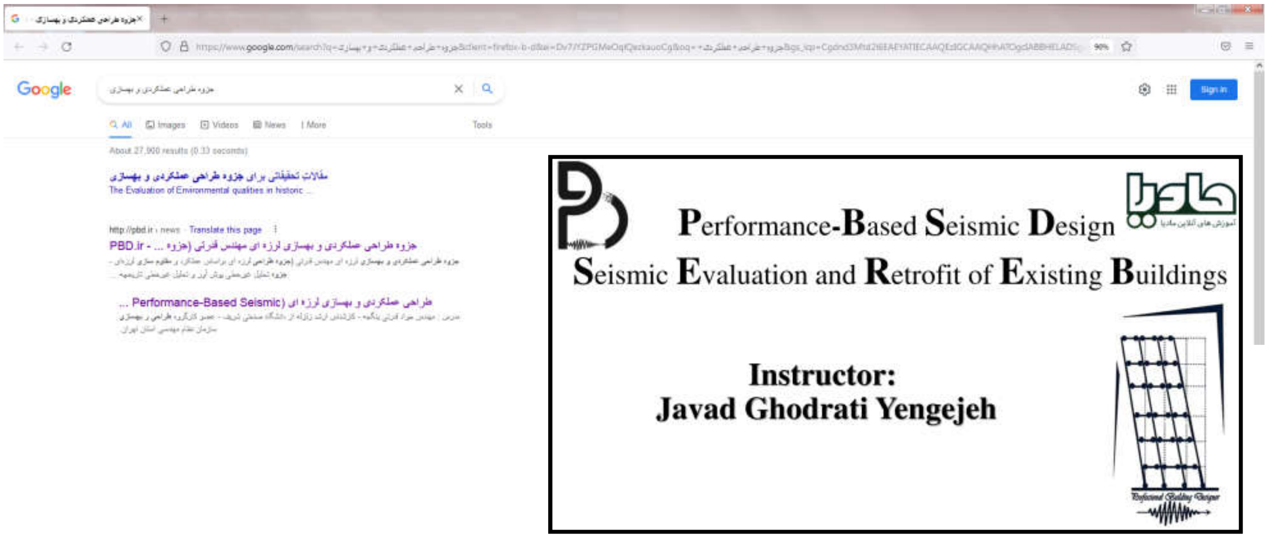
صفحه اینستاگرام PBD : www.instagram.com/PBD_ir

کانال تلگرام PBD : https://t.me/PBD_ir

آیدی تلگرامی : @PBDEng

ایمیل: J.yengejeh@yahoo.com

جزوه طراحی عملکردی و بهسازی لرزه‌ای



طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

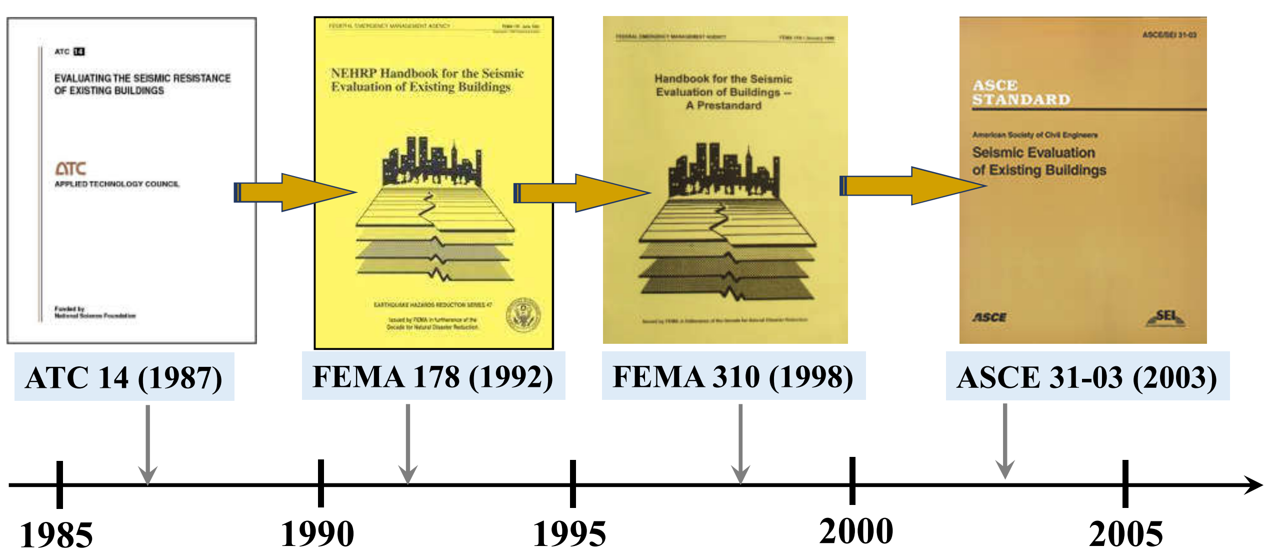
رئوس مطالب برای وبینار دوم:

۱-شرح مختصری بر فلسفه طراحی لرزه‌ای براساس عملکرد (در وبینار اول بحث شد)

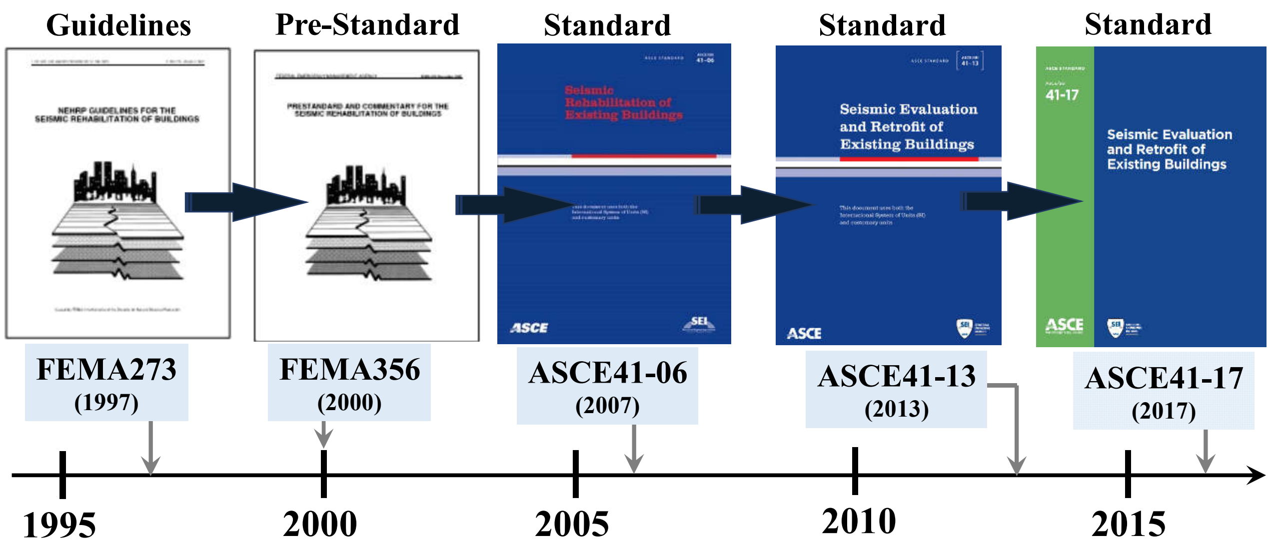
۲-معرفی پروژه بزرگ ATC 114، راهنمای ACI 374، استاندارد ACI 369.1، استاندارد ASCE 41-17 و اپندیکس A از استاندارد ACI 318-19

۳-ضرائب کاهش سختی در المان‌های بتن‌آرمه براساس استاندارد ASCE 41-17 و اپندیکس A از استاندارد ACI 318-19 و ضوابط احتمالی در ACI 369.1-23

Evolution of the development of ASCE/SEI 31



Evolution of the development of ASCE/SEI 41

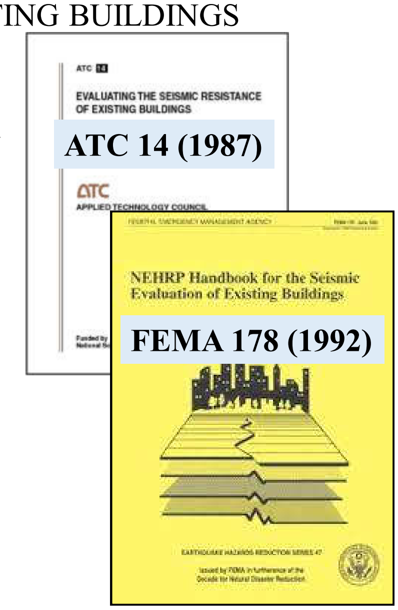


طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

ATC 14 EVALUATING THE SEISMIC RESISTANCE OF EXISTING BUILDINGS

- Applied Technology Council (ATC) published one of the first set of *Guidelines for the "Seismic* **Evaluation** *of Existing Buildings"*, ATC-14 (1987)
  - First Attempt creating Seismic Evaluation Tool
  - Later modified and published as ATC-22 (1989)

- FEMA 178 NEHRP Handbook For The Seismic **Evaluation** Of Existing Buildings (1992)
  - Development of Checklists



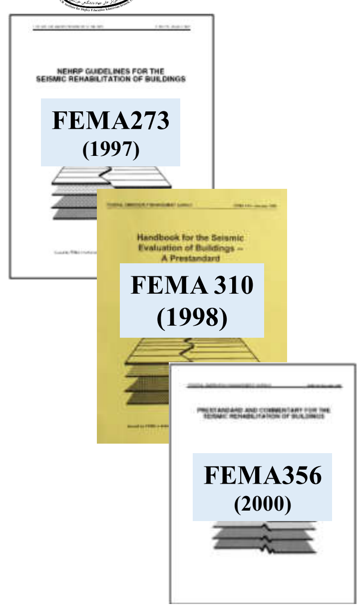
طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

- **1989 Loma Prieta, CA**
  - Magnitude estimated 6.9
  - 63 Deaths and nearly 4,000 Injuries
  - ~$12B in damages (in 2017 money)

- **1994 Northridge, CA**
  - Magnitude estimated 6.7
  - 57 Deaths and nearly 8,700 Injuries
  - Up to ~$70B (in 2017 money)

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

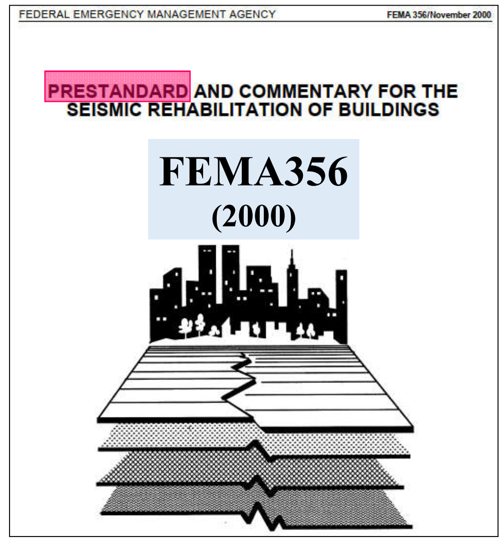
- In the 1990's, FEMA sought to update FEMA 178 methodology from recent events such as the 1989 Loma Prieta and the 1994 Northridge earthquakes (as well as from the development of performance based design procedures)
- FEMA 273 and 274 NEHRP Guidelines and Commentary For The Seismic **Rehabilitation** Of Buildings (1997)
- FEMA published a revised guideline, designated FEMA 310 (January 1998) entitled Handbook for the Seismic **Evaluation** of Buildings – A Prestandard.
- FEMA 356 Prestandard and Commentary for the Seismic **Rehabilitation** of Buildings (2000)



طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

➢ FEMA 356–Seismic Rehabilitation of Existing Buildings

The title of this document, FEMA 356 **Prestandard** and Commentary for the Seismic Rehabilitation of Buildings, incorporates a word that not all users may be familiar with. That word—**prestandard**—has a special meaning within the ASCE Standards Program in that it signifies **the document has been accepted for use as the start of the formal standard development process**, however, the document has yet to be fully processed as a voluntary consensus standard.



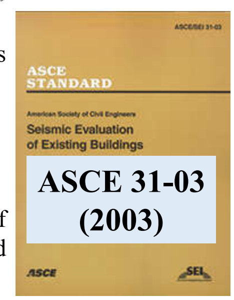
طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

➢ ASCE 31-03 – Seismic **Evaluation** of Existing Buildings

- ASCE/SEI 31-03 is intended to replace **FEMA 310**, Handbook for Seismic Evaluation of Buildings — A Prestandard (1998). This Standard was written to:

1- Reflect advancements in technology;
2- Incorporate the experience of design professionals;
3- Incorporate lessons learned during recent earthquakes;

be compatible with FEMA 356, Prestandard and Commentary for the Seismic Rehabilitation of Buildings (2000); be suitable for adoption in building codes and contracts; be nationally applicable; and provide evaluation techniques.



➢ Life Safety and Immediate Occupancy Performance Levels.

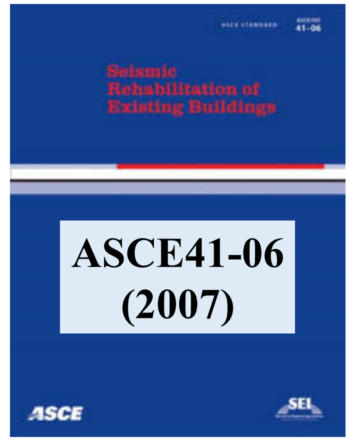
➢ Three-tier approach for screening buildings with checklist requirements for each tier.

- ✓ Tier 1 - Screening Phase
- ✓ Tier 2 - Evaluation Phase
- ✓ Tier 3 - Detailed Evaluation

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

- ASCE 41-06–Seismic Rehabilitation of Existing Buildings

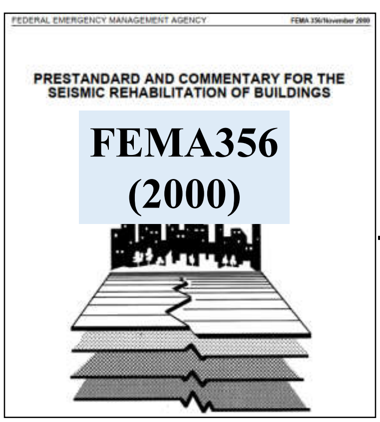
Seismic Rehabilitation of Existing Buildings presents the latest generation of performance-based seismic rehabilitation methodology. This new national consensus standard was developed from the FEMA 356, Prestandard and Commentary for the Seismic Rehabilitation of Buildings. This Standard represents state-of-the-art knowledge in earthquake engineering and is a valuable tool for the structural engineering profession to improve building performance in future earthquakes. It includes significant improvements in current understanding of building behavior in earthquakes, such as: **improved C-coefficients for calculation of the pseudo-lateral force and target displacement based on recommendations in FEMA 440;**



طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

- ASCE 41-06–Seismic Rehabilitation of Existing Buildings

ASCE/SEI Standard 41-06 is a valuable tool for structural engineers and the public for improving seismic performance of existing buildings.



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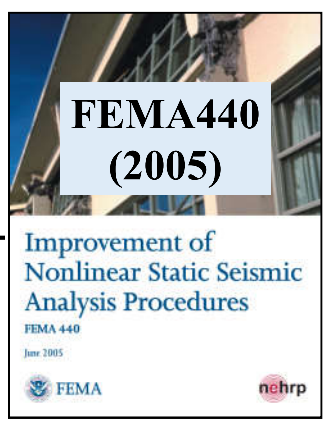

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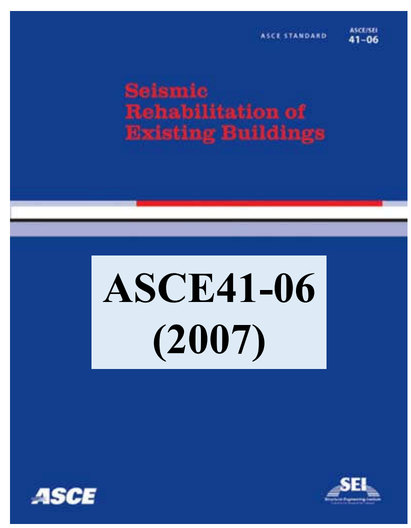
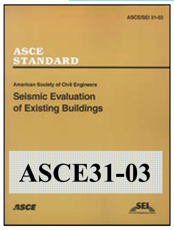


طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

- **ASCE 41-13: Seismic Evaluation and Retrofit ~~Rehabilitation~~ of Existing Buildings**

Seismic Evaluation



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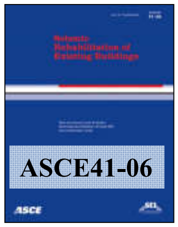
Seismic Retrofit



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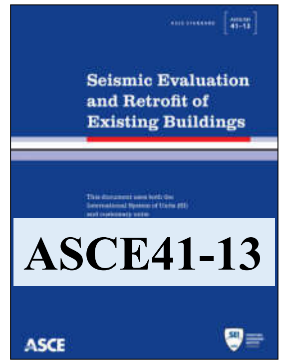
Update to ASCE/SEI 41 Concrete Provisions

⇒



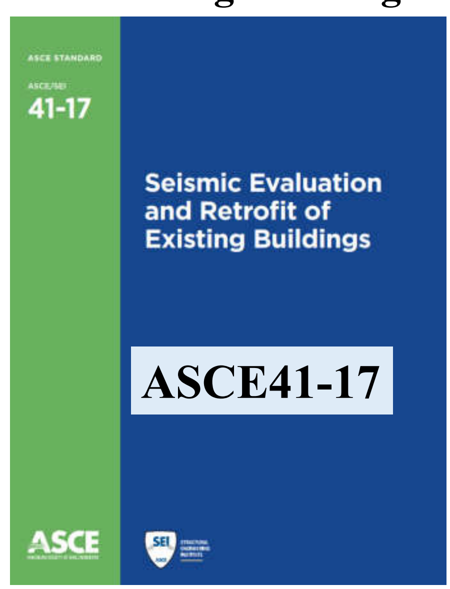
- Three-tiered process for seismic evaluation
  - Tier 1 Screening
  - Tier 2 Deficiency-Based Evaluation and Retrofit
  - Tier 3 Systematic Evaluation and Retrofit

- First time Evaluation and Rehabilitation codes of Existing Buildings were combined.
- Utilizes ASCE 7-10.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

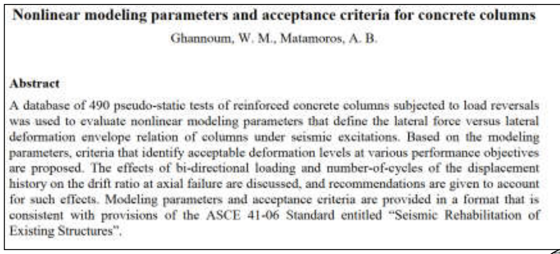
ASCE41-17: Seismic Evaluation and Retrofit of Existing Buildings

Standard ASCE/SEI 41-17, describes deficiency-based and systematic procedures that use performance-based principles to evaluate and retrofit existing buildings to withstand the effects of earthquakes. The standard presents a three-tiered process for seismic evaluation according to a range of building performance levels by connecting targeted structural performance and the performance of nonstructural components with seismic hazard levels. The deficiency-based procedures allow evaluation and retrofit efforts to focus on specific potential deficiencies deemed to be of concern for a specified set of building types and heights. The systematic procedure, applicable to any building, sets forth a methodology to evaluate the entire building in a rigorous manner. This standard establishes analysis procedures and acceptance criteria, and specifies requirements for foundations and geologic site hazards; components made of steel, concrete, masonry, wood, and cold-formed steel; architectural, mechanical, and electrical components and systems; and seismic isolation and energy dissipation systems.



طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

ASCE41-17: Seismic Evaluation and Retrofit of Existing Buildings

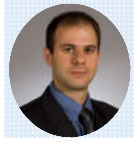


Nonlinear modeling parameters and acceptance criteria for concrete columns

Ghannoum, W. M., Matamoros, A. B.

Abstract

A database of 490 pseudo-static tests of reinforced concrete columns subjected to load reversals was used to evaluate nonlinear modeling parameters that define the lateral force versus lateral deformation envelope relation of columns under seismic excitations. Based on the modeling parameters, criteria that identify acceptable deformation levels at various performance objectives are proposed. The effects of bi-directional loading and number-of-cycles of the displacement history on the drift ratio at axial failure are discussed, and recommendations are given to account for such effects. Modeling parameters and acceptance criteria are provided in a format that is consistent with provisions of the ASCE 41-06 Standard entitled "Seismic Rehabilitation of Existing Structures".

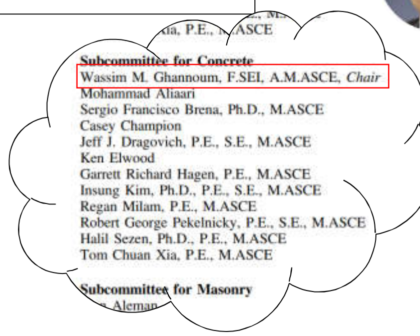


Wassim M. Ghannoum

University of Texas at Austin

Adolfo Benjamin Matamoros

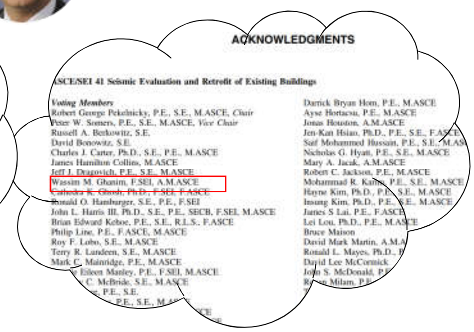
University of Texas at San Antonio



Subcommittee for Concrete

Wassim M. Ghannoum, F.SEI, A.M.ASCE, *Chair*
Mohammad Aliaari
Sergio Francisco Brena, Ph.D., M.ASCE
Casey Champion
Jeff J. Dragovich, P.E., S.E., M.ASCE
Ken Elwood
Garrett Richard Hagen, P.E., M.ASCE
Insung Kim, Ph.D., P.E., S.E., M.ASCE
Regan Milam, P.E., M.ASCE
Robert George Pekelnicky, P.E., S.E., M.ASCE
Halil Sezen, Ph.D., P.E., M.ASCE
Tom Chuan Xia, P.E., M.ASCE

Subcommittee for Masonry



ACKNOWLEDGMENTS

ASCE/SEI 41 Seismic Evaluation and Retrofit of Existing Buildings

Voting Members
Robert George Pekelnicky, P.E., S.E., M.ASCE, *Chair*
Peter W. Somers, P.E., S.E., M.ASCE, *Vice Chair*
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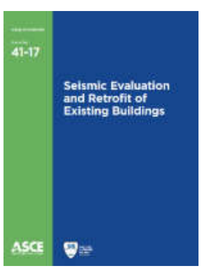
Bruce Maison
David Lee McCormick

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)



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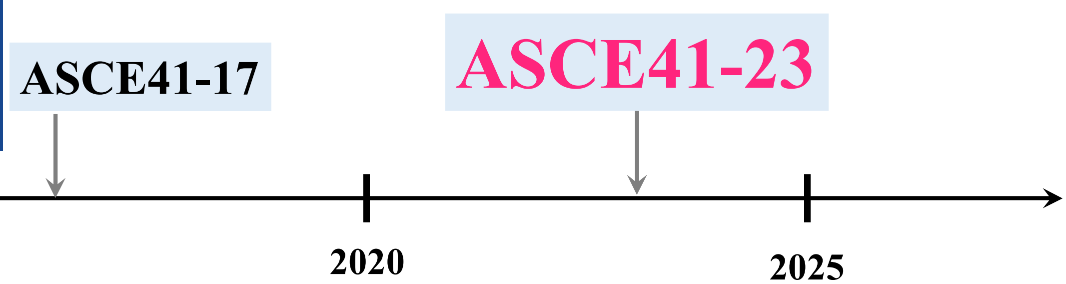


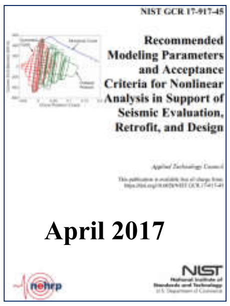
The ASCE/SEI 41 Committee is in the process of creating the
ASCE 41-23
version of the standard
"Seismic **Evaluation** and **Retrofit** of Existing Buildings."

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

ATC Project No. : ATC - 114

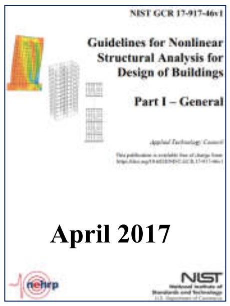
Project Title : Development of Accurate Models and Efficient Simulation Capabilities for Collapse Analysis to Support Implementation of Performance Based Seismic Engineering. This task has been completed and produced four reports:

NIST.GCR.17-917-46v1, NIST.GCR.17-917-46v2, NIST.GCR.17-917-46v3, NIST.GCR.17-917-45


NIST GCR 17-917-45

Recommended Modeling Parameters and Acceptance Criteria for Nonlinear Analysis in Support of Seismic Evaluation, Retrofit, and Design

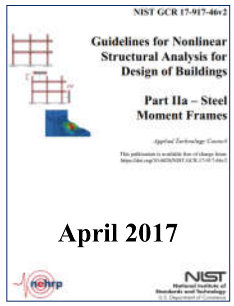
April 2017


NIST GCR 17-917-46v1

Guidelines for Nonlinear Structural Analysis for Design of Buildings

Part I – General

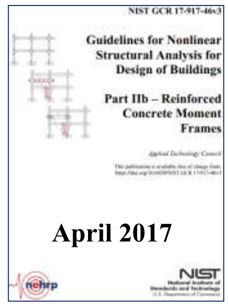
April 2017


NIST GCR 17-917-46v2

Guidelines for Nonlinear Structural Analysis for Design of Buildings

Part IIa – Steel Moment Frames

April 2017


NIST GCR 17-917-46v3

Guidelines for Nonlinear Structural Analysis for Design of Buildings

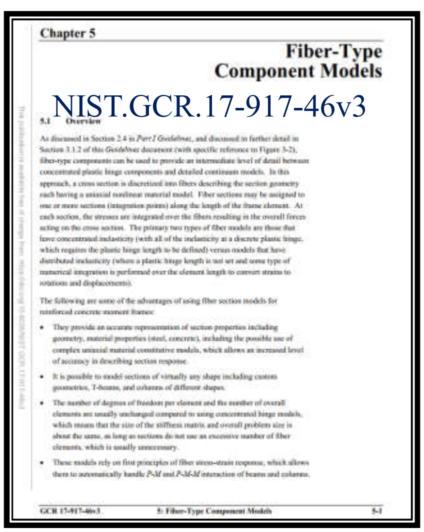
Part IIb – Reinforced Concrete Moment Frames

April 2017

Funding Agency :

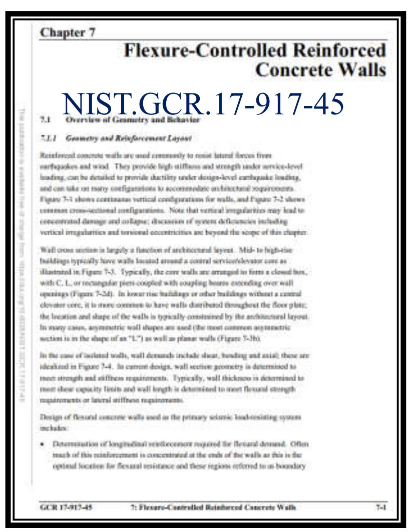
National Institute of Standards and Technology

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)


Chapter 5

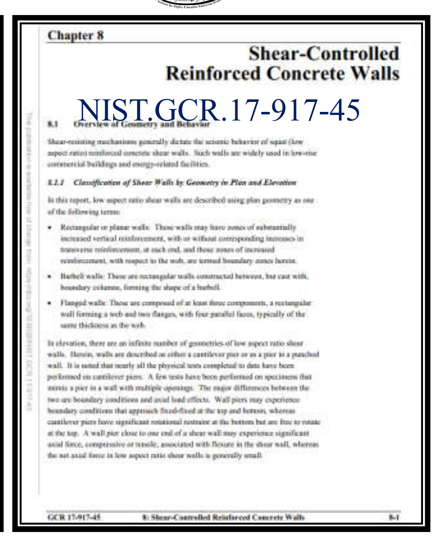
Fiber-Type Component Models

NIST.GCR.17-917-46v3


Chapter 7

Flexure-Controlled Reinforced Concrete Walls

NIST.GCR.17-917-45


Chapter 8

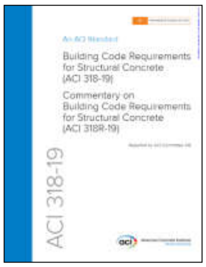
Shear-Controlled Reinforced Concrete Walls

NIST.GCR.17-917-45

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

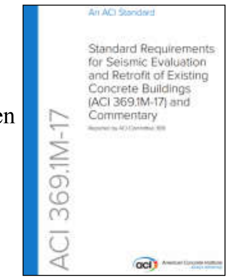
Ongoing Activities for Future Updates of the ACI PBD Provisions

ACI 318-19	ACI 374.3-16	ACI 369.1-17



ACI 318-0H
Seismic Provisions
Chair: John Wallace

Performance-Based Seismic Design of Concrete Buildings
Chair: Garrett Hagen



Seismic Repair and Rehabilitation
Chair:
Adolfo Matamoros

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Ongoing Activities for Future Updates of the ACI PBD Provisions

300 DESIGN AND CONSTRUCTION

American Concrete Institute
Always advancing

Committee: ACI 318 - Structural Concrete Building Code

ACI 318 Subcommittee H Meeting – Virtual Fall Convention Tuesday, 19 October 2021

1. Introductions
2. Review and approval of agenda
3. Approval of minutes from virtual meeting in March 2021
4. Review of 318 schedule
5. ACI 318-22
   a. Reference update
   b. Commentary review (Chapter 12, 18, Appendix A)

The main idea of the commentary review is to identify commentary that might be outdated, redundant, or simply not needed. Also, where commentary provides a lengthy discussion that is covered elsewhere in published reports or papers, the commentary might be deleted and a reference provided to a separate publication.

ACI 374
Performance-Based Seismic Design of Concrete Buildings

Committee Mission: Develop and report information on performance-based seismic analysis and design of concrete buildings.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Ongoing Activities for Future Updates of the ACI PBD Provisions

300 DESIGN AND CONSTRUCTION

American Concrete Institute
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Committee: ACI 318 - Structural Concrete Building Code

ACI 318-0H - Seismic Provisions
Chair: John Wallace

10. Change proposals under development – Not discussed during March 2021 meeting, updates expected at October 19, 2021 meeting

a. CH020 – Column Shear Strength – Matamoros/Abdullah
b. CH030 – Column Shear Amplification – Abdullah/Matamoros
c. CH170 – Splices at the Base of Walls – Lehman/McCabe
d. CH260 – Web Cross Ties – Lehman/Abdullah
e. CH210 – Load Combinations in Shear Wall Design – Moehle
f. CH260 – Wall web detailing I – Abdullah/Fields/Lehman
g. CH300 – Special Walls R-factor – Fields/McFarlane
h. CH310 – Redistribution of demands in coupling beams – Fields/McFarlane
i. CH320 – Wall web detailing II Splices – Abdullah/Fields
j. CH330 – Phi factor for shear amplification slender walls (clarification) – Fields/McFarlane

Dawn E Lehman

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Ongoing Activities for Future Updates of the ACI PBD Provisions

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Dawn E Lehman

Meeting Minutes

ACI Committee 374
Performance-Based Seismic Design of Concrete Buildings

Tuesday, March 16, 1999
2:00 - 5:00 PM
Buckingham Room
Hyatt Regency, Chicago, IL

ATTENDANCE:

Members Present: Sergio Alcocer, Mark Aschheim, John Bonacci, Joe Bracci, JoAnn Browning, Duane Castenada, Juan Carlos Esquivel, Jack Hayes, Mary Beth Hueste, Brian Kehoe, Dominic Kelly, Ron Klemencic (Committee Chairman), Richard Klingner, Mervyn Kowalsky, Mike Kreger, Vilas Mujumdar, Javeed Munshi, Murat Saatcioglu, Shamim Sheikh, Andy Taylor, Raj Valluvan, John Wallace, Fernando Yanez

Associate Members Present: No associate members have yet been appointed to this new committee.

Visitors Present: Patricio Bonelli, Marc Eberhard, Angel Herrera, Jim LaFave, Dawn Lehman, Andres LePage, Steve McCabe, Conrad Paulson, Victor Pavon, David Sanders, Art Schultz, Bahram Shahrooz, Frank Vecchio, Eric Williamson

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Ongoing Activities for Future Updates of the ACI PBD Provisions

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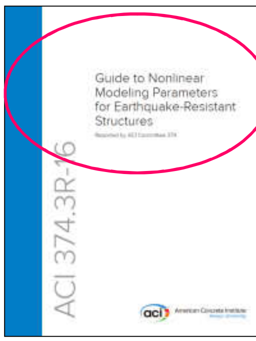
Committee: ACI 374 - Performance-Based Seismic Design of Concrete Buildings



This guide provides information regarding nonlinear modeling of components in special moment frame and structural wall systems resisting earthquake loads. The reported modeling parameters provide a modeling option for licensed design professionals (LDPs) performing nonlinear analysis for performance-based seismic design of reinforced concrete building structures designed and detailed in accordance with ACI 318.

ACI 374 ACI 374-23 ?

Performance-Based Seismic Design of Concrete Buildings

Committee Mission: Develop and report information on performance-based seismic analysis and design of concrete buildings.

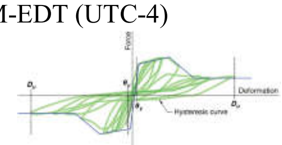
Goals: 1) Coordinate efforts within ACI on performance-based seismic design; 2) Draft guides and technical notes on performance based seismic design topics; 3) Develop technical sessions.

Chair: Garrett Hagen

ACI 374B Committee Meeting

Guide to Nonlinear Modeling
374B - 10/17/2021 3:00 PM-5:00 PM-EDT (UTC-4)

- ACI 318 Appendix A:
  - C015a ballot: `Ultimate deformation capacity` **(Kim)**



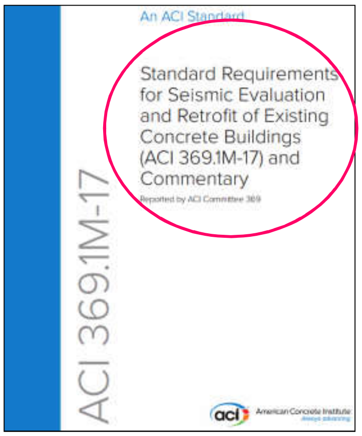
5. Other New Business:
   - Slab-wall / Slab-column connection
   - Energy Dissipation and Hysteresis Modeling (**Guidelines for Performance-Based Seismic Design of RC Buildings, KCI)**

Ongoing Activities for Future Updates of the ACI Standard 369-17

300 DESIGN AND CONSTRUCTION

American Concrete Institute
Always advancing

Committee: ACI 369 - Seismic Repair and Rehabilitation



PREFACE

This standard provides retrofit and rehabilitation criteria for reinforced concrete buildings based on results from the most recent research on the seismic performance of existing concrete buildings. The intent of this standard is to provide a continuously updated resource document for modifications to Chapter 10 of ASCE 41-17, similar to how the National Earthquake Hazards Reduction Program (NEHRP) Recommended Seismic Provisions produced by the Federal Emergency Management Agency (FEMA) (FEMA 450) have served as source documents for the International Building Code (IBC) and its predecessor building codes. Specifically, this version of ACI 369.1M serves as the basis for Chapter 10, "Concrete," of ASCE 41-17.

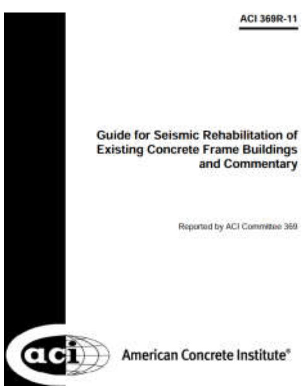
ACI 369.1M-17 was adopted September 22, 2017, and published February 2018.

Ongoing Activities for Future Updates of the ACI Standard 369-17

300 DESIGN AND CONSTRUCTION

American Concrete Institute
Always advancing

Committee: ACI 369 - Seismic Repair and Rehabilitation



This guide, which was developed based on the format and content of ASCE/SEI 41-06, Chapter 6.0, "Concrete," describes methods for estimating the seismic performance of both existing and new concrete components in an existing building. The guide is intended to be used with the analysis procedures and Rehabilitation Objectives established in ASCE/SEI 41-06 for the Systematic Rehabilitation Method. The guide provides recommendations for modeling parameters and acceptance criteria for linear and nonlinear analysis of beams, columns, joints, and slab-column connections of concrete buildings and the procedures for obtaining material properties necessary for seismic rehabilitation design.

ACI 369R-11 was adopted and published February 2011.

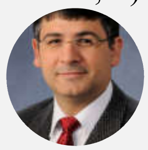
ACI 369 ACI 369-23

Seismic Repair and Rehabilitation

Committee Mission: Develop and report information on the repair or rehabilitation of building structures to improve earthquake resistance.

Goals: 1) Maintain the standard "Seismic Evaluation and Retrofit of Existing Concrete Buildings" that is the source document for the concrete provisions of ASCE 41; 2) Provide extended commentary for the standard; 3) **Develop guides and other documents on seismic retrofit techniques.**

Chair: Adolfo Matamoros



طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

ACI committee 369 now updates the concrete provisions of ASCE/SEI 41

- ✓ Published under ACI 369.1 Standard
- ✓ ACI 369.1 is on 3 year cycle
- ✓ ASCE/SEI 41 is on a 6 year cycle
- ✓ ACI 369.1-20 will have substantial updates
- ✓ Standards will merge again in 2023
- ✓ ACI 369 "Seismic Repair and Rehabilitation"
  - – One Main committee and five sub-committees
  - – Main committee: **23 voting member** (balanced), **9 consulting members**, **49 associate members**
  - – 369-0A: General Provisions, **11 voting members**
  - – 369-0C: Frames, **7 voting members**
  - – 369-0D: Walls, **8 voting members**
  - – 369-0E: Diaphragms and Foundations, **2 voting members**
  - – 369-0F: Retrofit, **9 voting members**

Ongoing Activities for Future Updates of the ACI Standard 369.1-17

- ❑ 369 Committee Chair: Adolfo Matamoros
- ❑ 369-0A: General Provisions Chair: Insung Kim

- ❑ 369-0C: Frames Chair: Siamak Sattar

- ❑ 369-0D: Walls Chair: Garrett Hagen

- ❑ 369-0E: Diaphragms and Foundations Chair: Arne Halterman

- ❑ 369-0F: Retrofit Chair: Sergio Breňa

FEMA Support for ACI 369/ASCE 41

FEMA providing funding through ATC Project 140 to help ASC/SEI 41 and ACI 369 update provisions of their standards

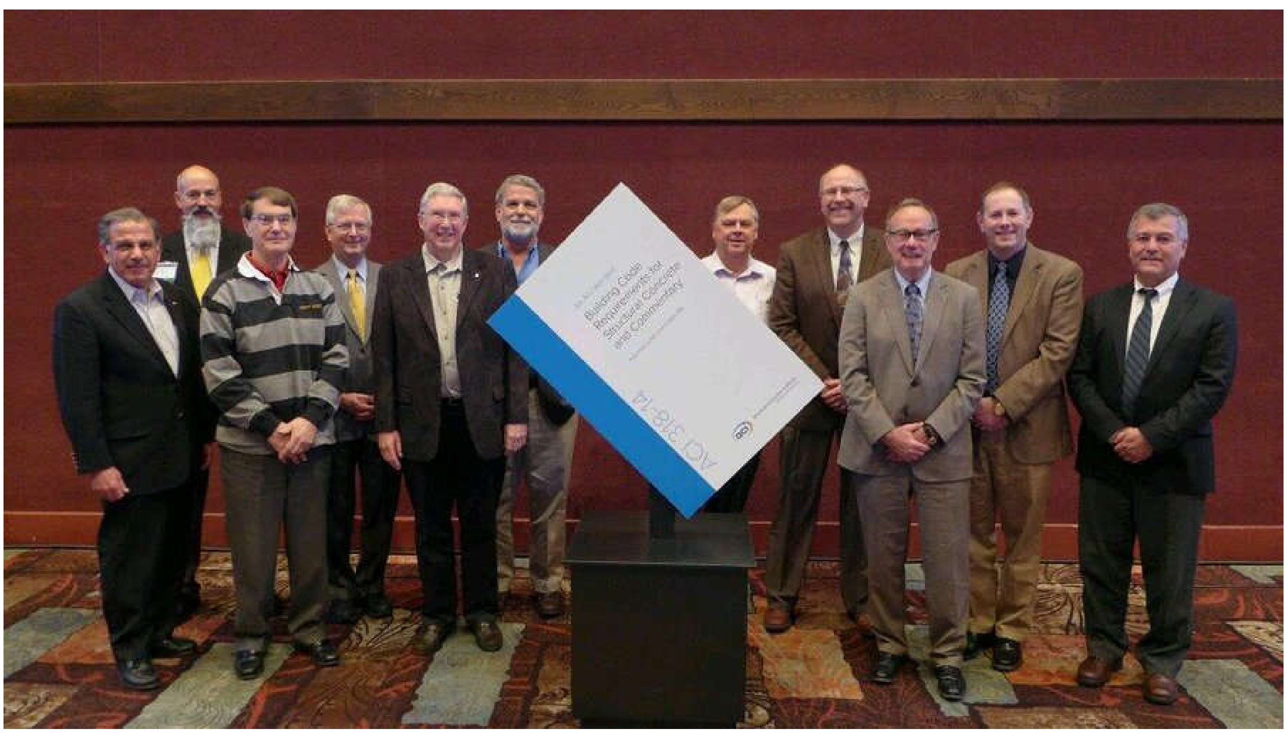
ATC **ATC Project No. : ATC 140-1**

Project Title: Update of Seismic Evaluation and Retrofit of Existing Buildings Guidance

Client: Federal Emergency Management Agency

Purpose: To investigate and address technical issues regarding the evaluation and retrofit of existing buildings and develop material for the expanded FEMA design applications document that will replace the current FEMA 275 Design Examples document.

ACI 318 Committee



طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

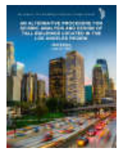
<u>ACI 318-19 Appendix A</u>

Design Verification Using Nonlinear Response History Analysis

ACI 318-19 (Appendix A)

Why was Appendix A added to the Code?

ASCE 7-16 ✓



2017 LATBSDC Guidelines ✓

PEER TBI 2017 ✓

ACI 318-14 ✗

ACI 318-19 ✓

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

رئوس مطالب برای وبینار دوم:

۱-شرح مختصری بر فلسفه طراحی لرزه‌ای براساس عملکرد (در وبینار اول بحث شد)

۲-معرفی پروژه بزرگ ATC 114، راهنمای ACI 374، استاندارد ACI 369.1، استاندارد ASCE 41-17 و اپندیکس A از استاندارد ACI 318-19

۳-ضرائب کاهش سختی در المان‌های بتن‌آرمه براساس استاندارد ASCE 41-17 و اپندیکس A از استاندارد ACI 318-19 و ضوابط احتمالی در ACI 369.1-23

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

ACI 318-19 Appendix A

Design Verification Using Nonlinear Response History Analysis

برای تعریف تک تک پارامترهای زیر (یا هر پارامتر دیگری که مربوط به المان‌های لرزه‌بر باشد) در روند تحلیل‌های غیرخطی، سه سوال کلیدی وجود دارد:

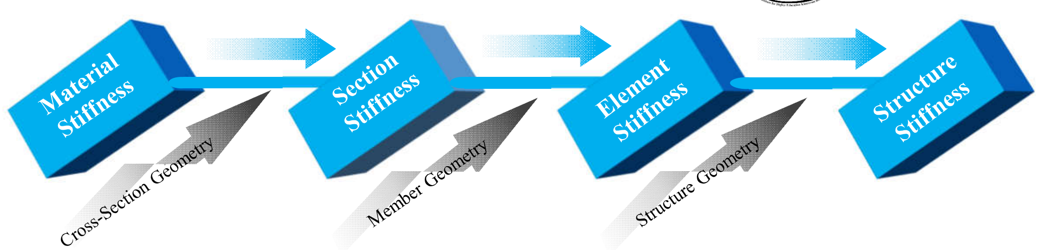
۱-پارامتر مذکور در سلسه مراتب مصالح، مقطع، المان و کل سازه در چه موقعیتی قرار دارد؟

۲-کدام یک از تحلیل‌های استاتیکی غیرخطی (پوش‌آور) یا دینامیکی تاریخچه زمانی غیرخطی انجام خواهد شد؟

۳-کدام یک از مدل‌های غیرخطی مفصل پلاستیک یا فایبر استفاده می‌شود؟

- **1-Stiffness** $(E_{cE} \text{ or } E_{cL}, E_{steel})$?
- **2-Strength** (Lower Bound or Expected) ?
- **3-Nonlinearity** (Monotonic or Cyclic) ?
- **4-Ductility** (Monotonic or Cyclic) ?
- **5-Deterioration** (in-cycle or out-cycle (cyclic)) ?
- **6-Hysteresis Behavior** ?
- **7-Acceptance Criteria** (Robustness (progressive collapse)) ?

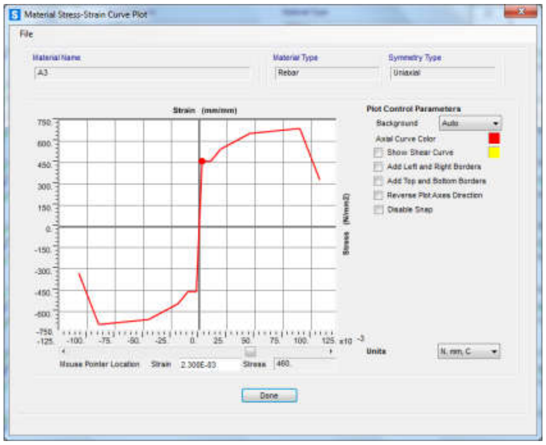
طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

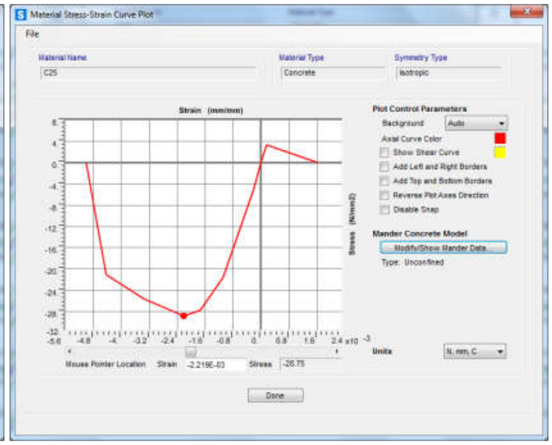


- سختی مقاطع وابسته به سختی مصالح و هندسه کلی مقطع می‌باشد.
- سختی المان‌ها وابسته به سختی مقطع و هندسه کلی المان می‌باشد.
- سختی کلی سازه، وابسته به سختی المان‌ها، هندسه کلی سازه و نحوه اتصال المان‌ها به یکدیگر است.
- تمامی این سختی‌ها می‌توانند در محدوده خطی یا غیرخطی مصالح باشند.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

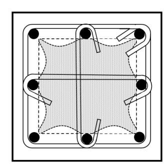
Material Stiffness — Section Stiffness — Element Stiffness — Structure Stiffness

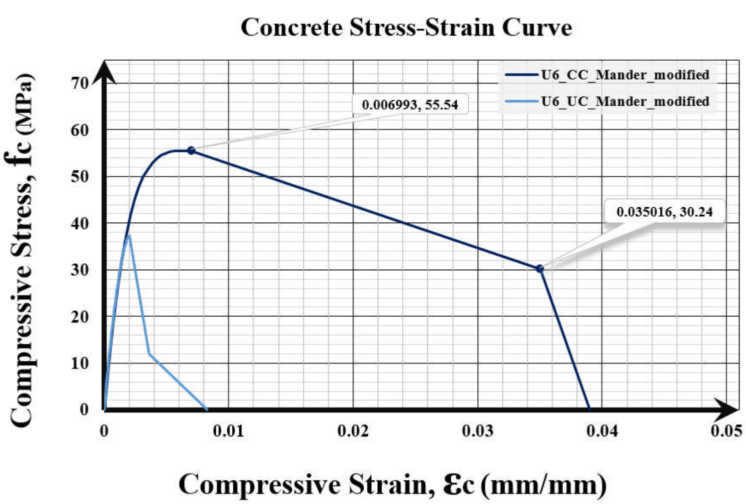


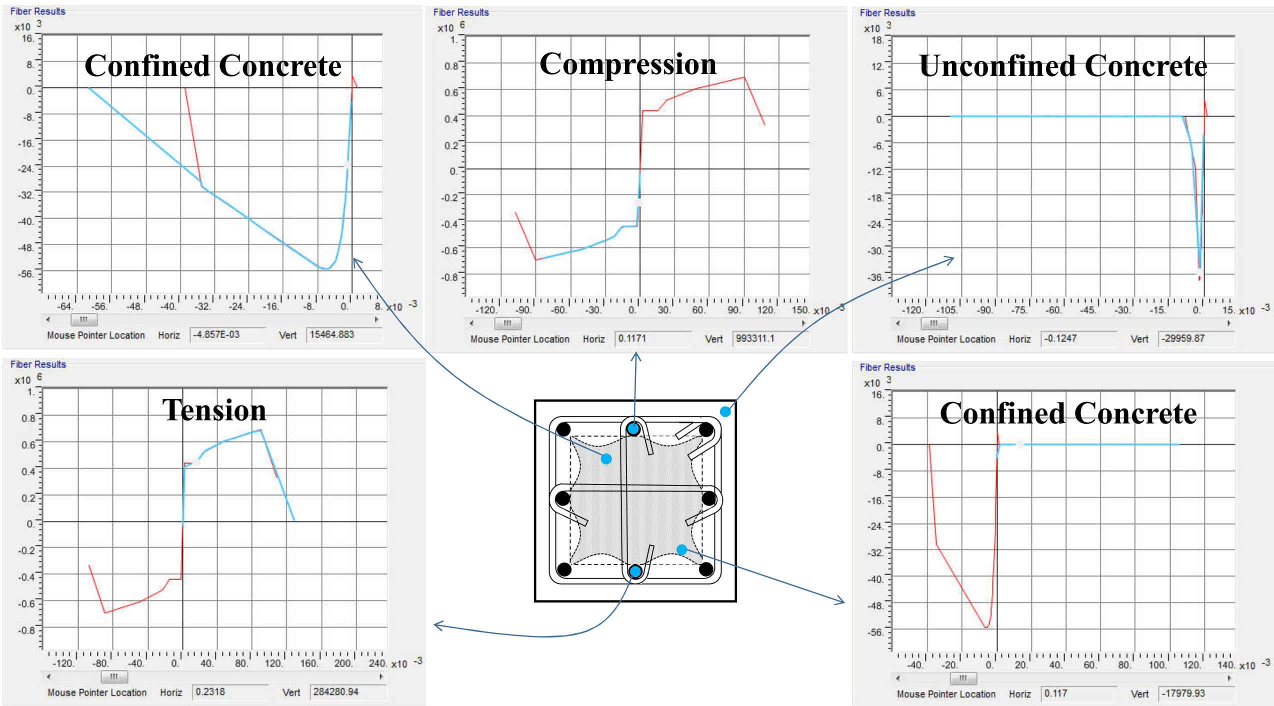


طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Material Stiffness — Section Stiffness — Element Stiffness — Structure Stiffness



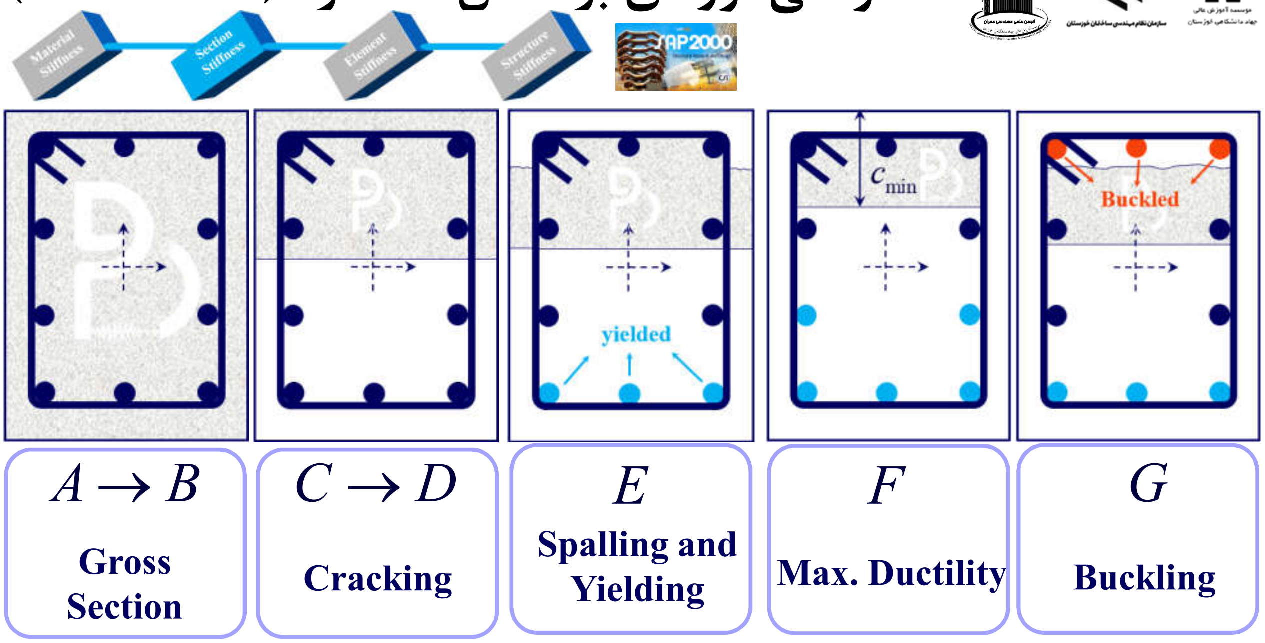


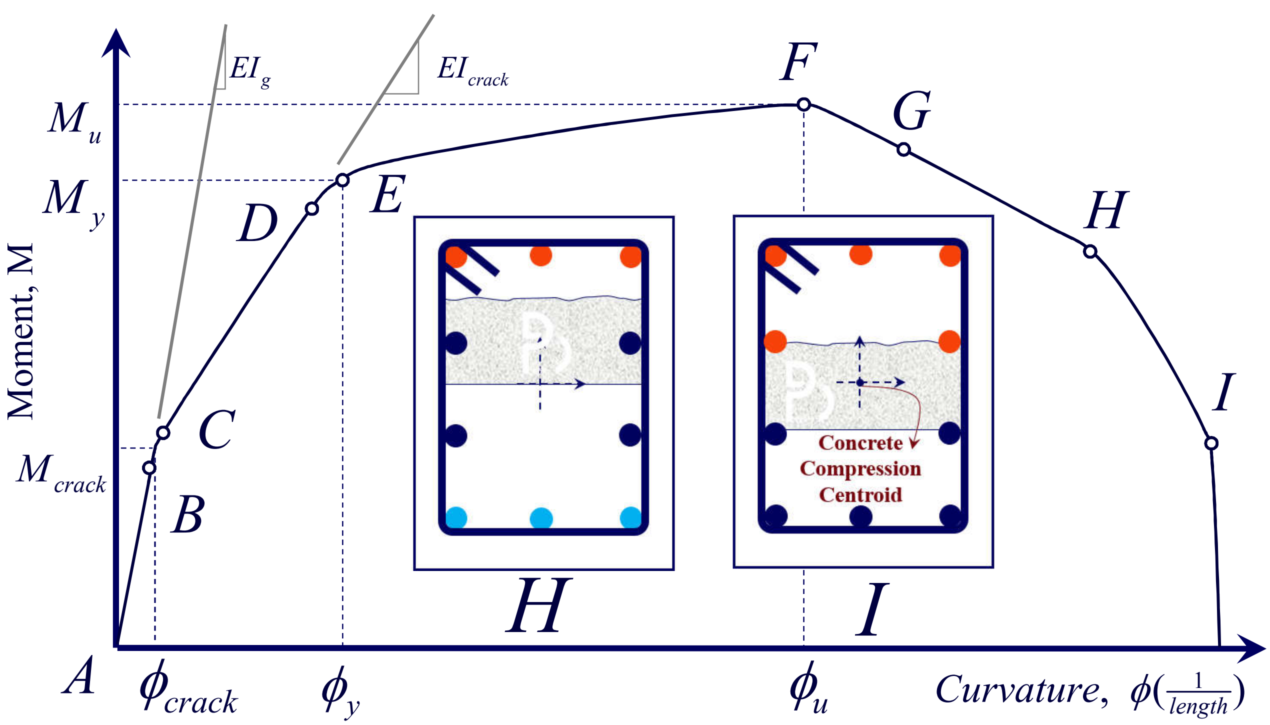


طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

$$\begin{cases} \phi_y = 1.284 \times 10^{-5} \dfrac{1}{mm} \\ M_y = 221412 kN.mm \end{cases}$$

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)



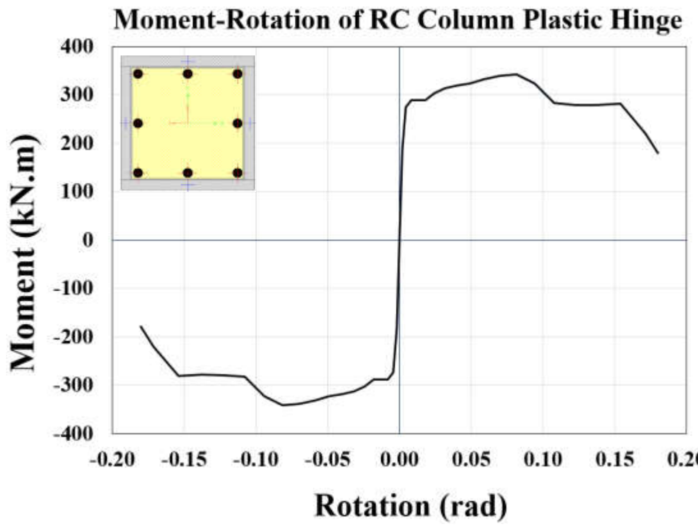


طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

NIST GCR 17-917-46v3

$$l_p = 0.05l + 0.1\frac{f_y}{\sqrt{f_c'}}d_b \le \frac{L}{4}$$

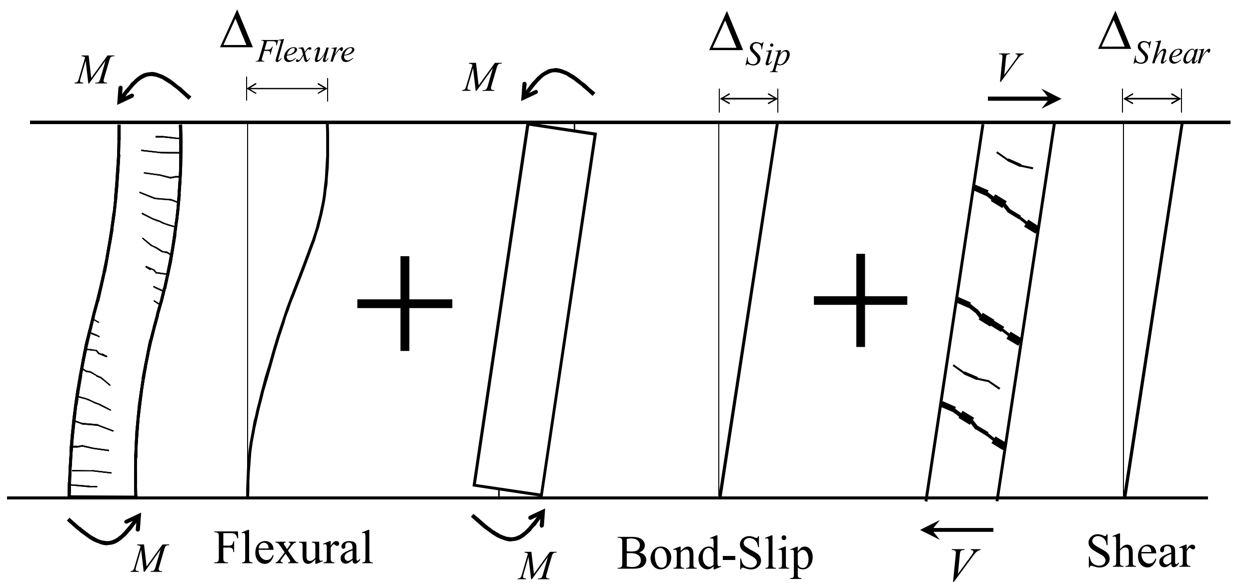
$$\Rightarrow l_p = 0.05\times1000 + 0.1\frac{437}{\sqrt{37.3}}25 = 228.88mm \le \frac{1000}{4} = 250mm$$



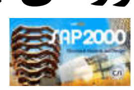
طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Total Deformations = Flexural Deformations + Bond-Slip Deformations + Shear Deformations

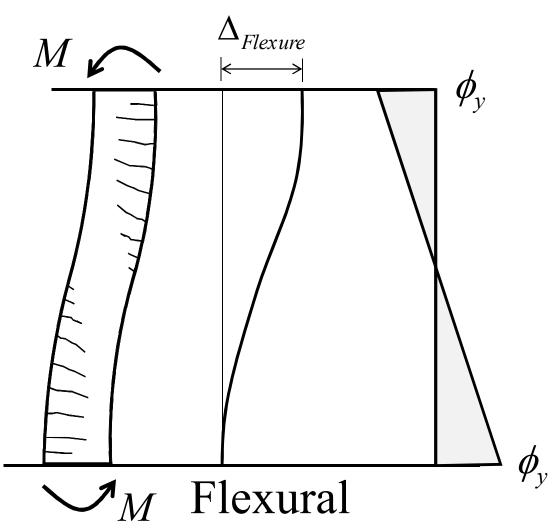
$$\Delta_T = \Delta_{Flexure} + \Delta_{Sip} + \Delta_{Shear}$$



طراحی لرزه‌ای براساس عملکرد (ACI 318-19)



Flexural Deformations:



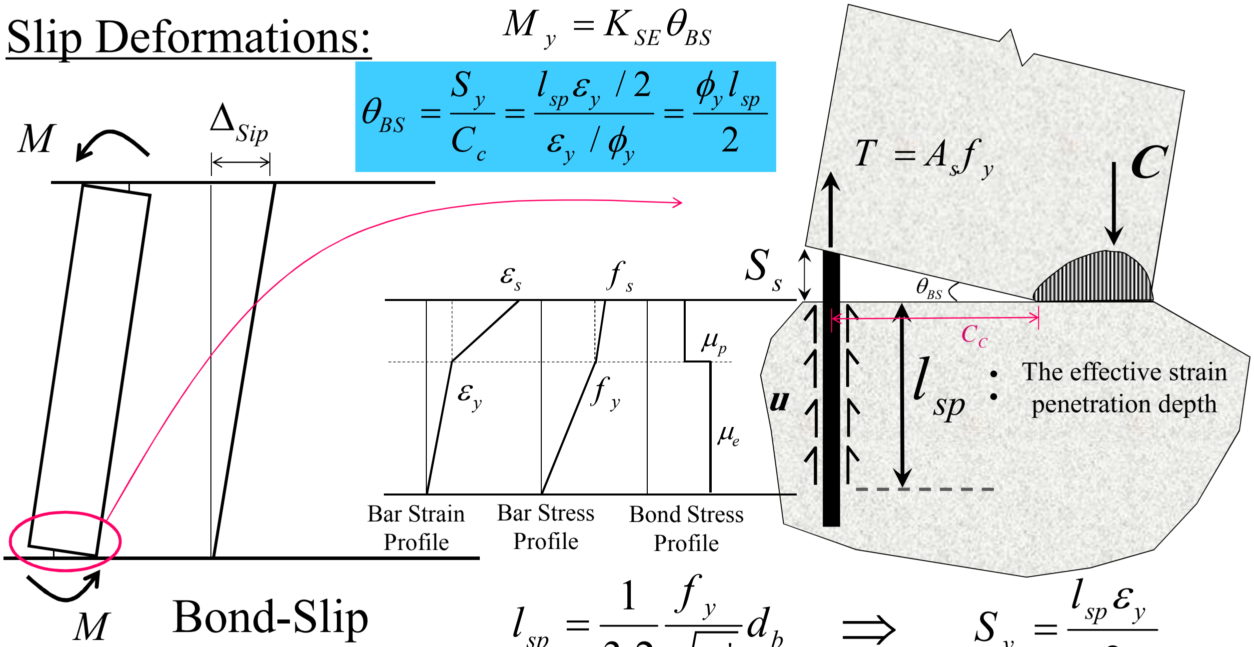
$$\Delta_{flex} = \frac{L^2}{6}\phi_y$$

Does not account for end rotations due to bar slip!

Slip Deformations:

$$M_y = K_{SE}\theta_{BS}$$

$$\theta_{BS} = \frac{S_y}{C_c} = \frac{l_{sp}\varepsilon_y/2}{\varepsilon_y/\phi_y} = \frac{\phi_y l_{sp}}{2}$$



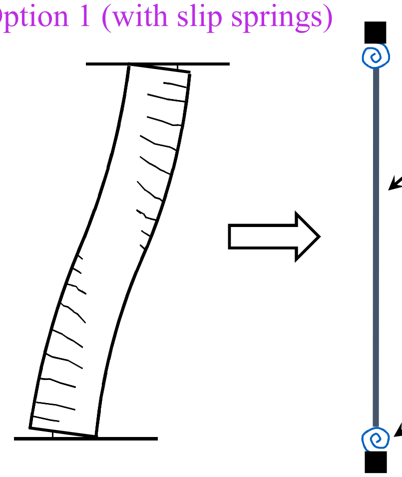
$$l_{sp} = \frac{1}{3.2}\frac{f_y}{\sqrt{f_c'}}d_b \Rightarrow S_y = \frac{l_{sp}\varepsilon_y}{2}$$

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Modeling options:

$$M_y = K_{SE}\theta_{BS} \Rightarrow K_{SE} = \frac{M_y}{\theta_{BS}} = \frac{2M_y}{\phi_y l_{sp}}$$

Option 1 (with slip springs)

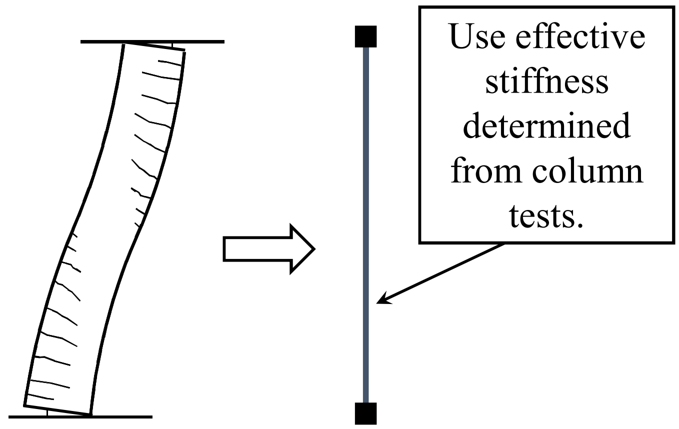


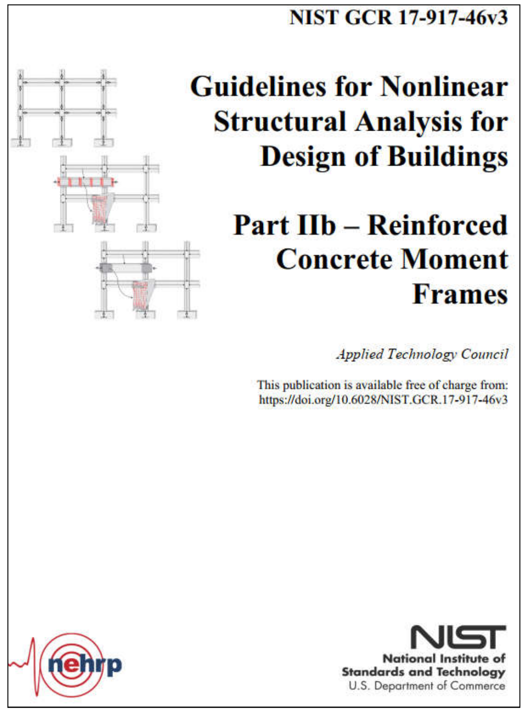
Stiffness based on moment-curvature analysis

Slip springs:

$$K_{SE} = \frac{2M_y}{l_{sp}\phi_y}$$

Option 2 (without slip springs)




NIST GCR 17-917-46v3

Guidelines for Nonlinear Structural Analysis for Design of Buildings

Part IIb – Reinforced Concrete Moment Frames

Applied Technology Council

This publication is available free of charge from: https://doi.org/10.6028/NIST.GCR.17-917-46v3

nehrp

NIST National Institute of Standards and Technology U.S. Department of Commerce

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Integrating sectional moment-curvature relations over member length and introducing bar-slip rotational flexibility provides reasonably accurate estimates of overall member lateral stiffness.

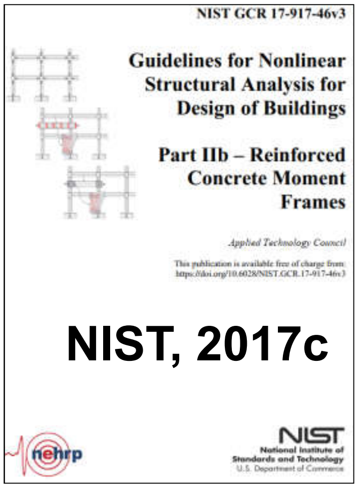
The added flexibility resulting from bar-slip rotations can be added to fiber section beam and column elements through:

1-Zero-length fiber-sections

or

2-Elastic rotational springs at element ends.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)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Guidelines for Nonlinear Structural Analysis for Design of Buildings

Part IIb – Reinforced Concrete Moment Frames

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NIST
National Institute of Standards and Technology
U.S. Department of Commerce

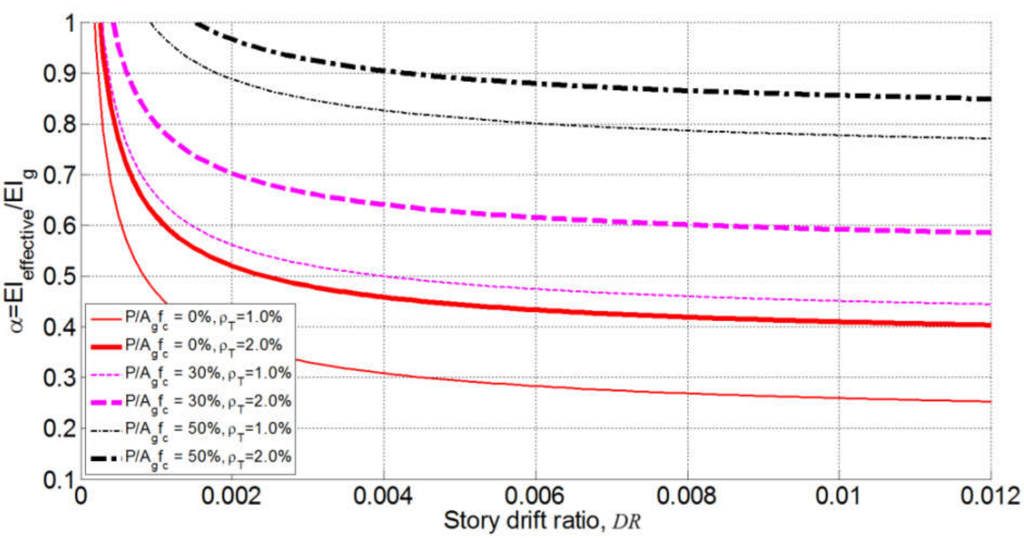
NIST, 2017c

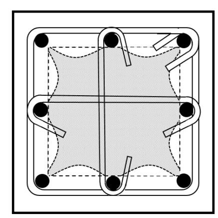
رابطه کاهش سختی ستون‌های بتن‌آرمه که در NIST GCR 17-917-46v3 معرفی شده است، براساس تحقیقات انجام شده توسط Kwon (2016) بوده و علاوه بر نسبت بارمحوری ستون‌ها، به میزان تغییرشکل نسبی و نسبت آرماتورهای کششی نیز وابسته می‌باشد. این رابطه به احتمال بسیار زیاد وارد ASCE41-23 خواهد شد.

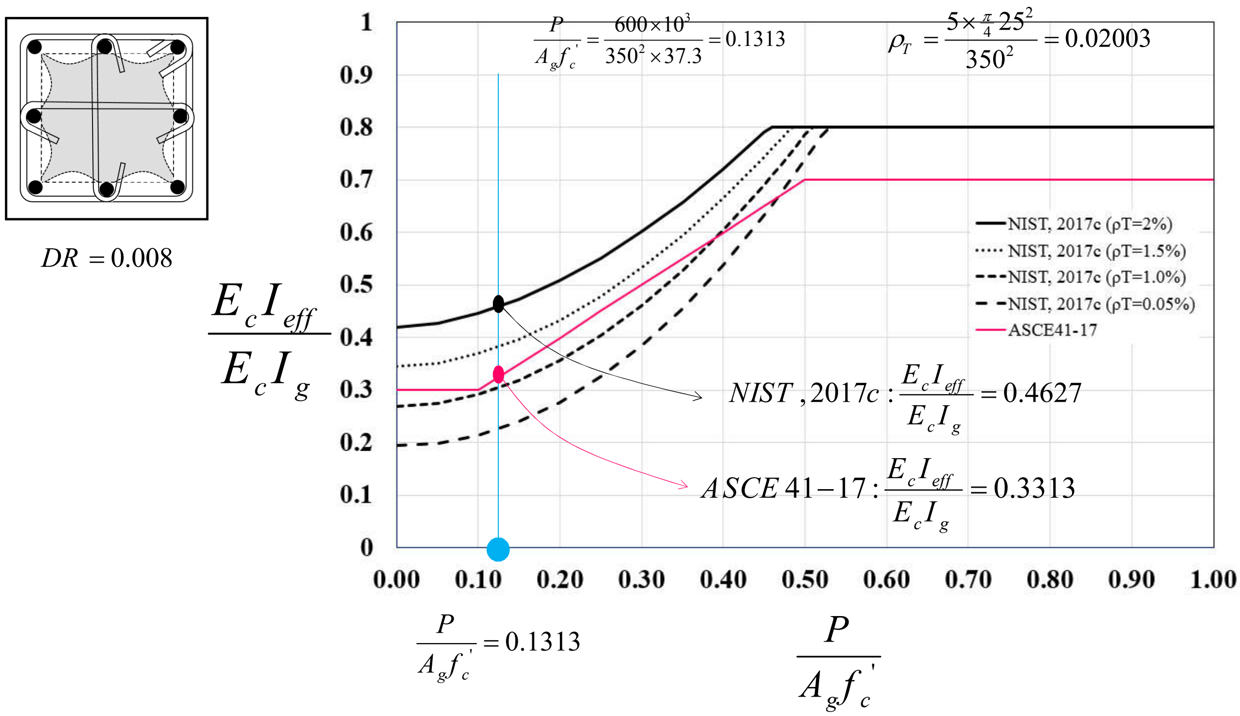
$$\frac{E_c I_{eff}}{E_c I_g} = 0.003 DR^{-0.65} + \gamma \le 0.8$$

$$\gamma = (-50\rho_T + 2.5)\left(\frac{P}{A_g f_c'}\right)^{(-20\rho_T + 2.15)} + (15\rho_T + 0.05)$$

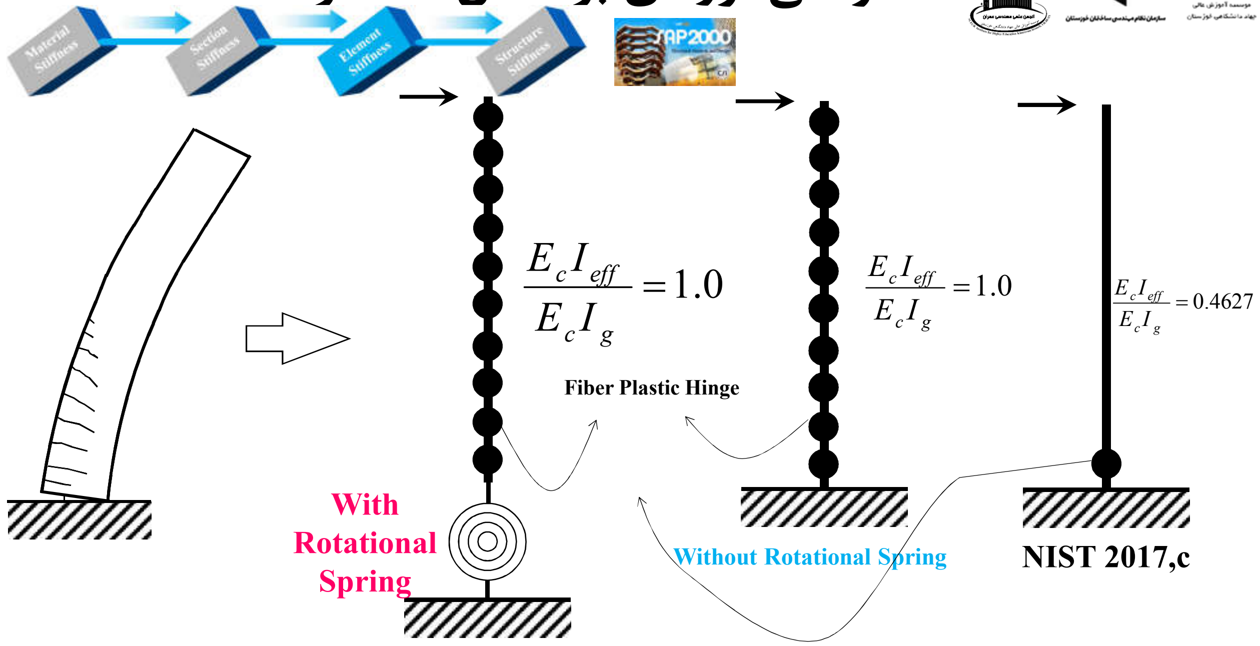
ρ_T : longitudinal tension reinforcement ratio defined as the area of longitudinal bars in tension divided by the gross section area, which can be taken as the section overall height multiplied by the section web width

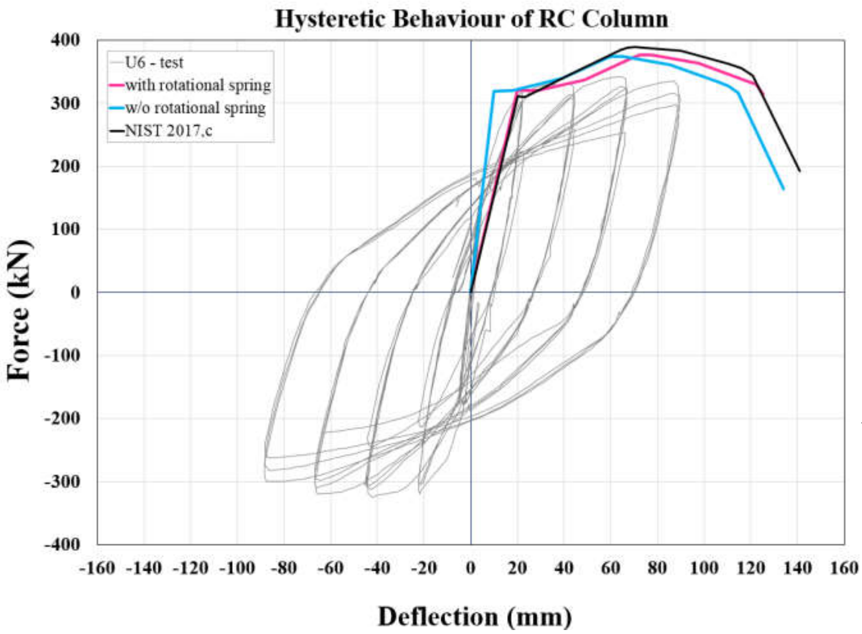






طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

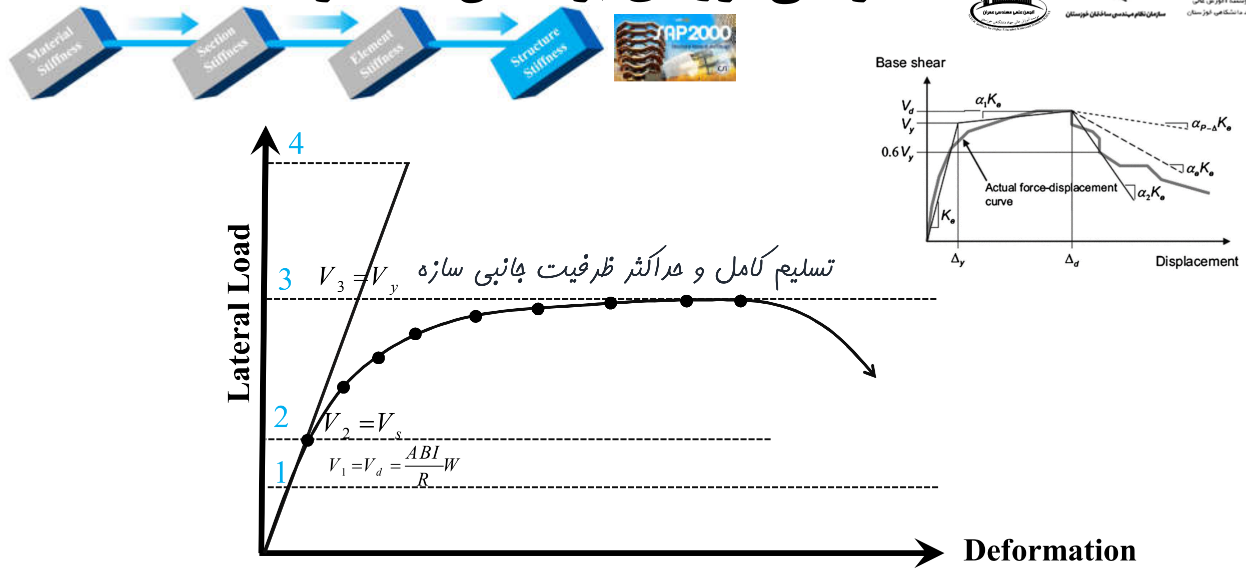




اصولاً به دلیل زوال مقاومتی درون سیکلی المان‌ها در بارگذاری سایکلیک، نمودار حاصل از بارگذاری مونوتونیک بالاتر از منحنی هسیترزیس خواهد بود. در این المان به دلیل محصور شدگی بالا اختلاف بین منحنی حاصل از تحلیل مونوتونیک با رفتار حاصل از بارگذاری سایکلیک کم می‌باشد.

مطابق با روش‌های پیشنهادی در NIST GCR 17-917-46v3 برای شبیه‌سازی اثرات لغزش آرماتورهای طولی، می‌توان سختی حاصل از دوران به دلیل لغزش آرماتورهای طولی را به عنوان یک فنر دورانی الاستیک در انتهای عضو شبیه سازی کرد. منحنی صورتی رنگ حاصل از مدلی است که در آن فنر دورانی در انتهای المان مدل شده است و مفاصل پلاستیک فایبر در طول المان اختصاص داده شده است. منحنی آبی برای مدلی است که در آن هیچ شبیه سازی برای مدل لغزش انجام نشده و مفاصل فایبر در طول عضو اختصاص داده شده است. مطابق شکل مشخص است که در صورت شبیه سازی لغزش، سختی ارتجاعی مدل بسیار کاهش یافته و شباهت مناسبی با نتایج تست دارد. در هر دو منحنی آبی و صورتی، هیچ ضریب کاهش سختی به المان اختصاص داده نشده است. در NIST GCR 17-917-46v3 پیشنهاد می‌شود که در صورت عدم استفاده از مدل فنر دورانی می‌توان از ضرائب کاهش سختی معرفی شده توسط Kwon (2016) استفاده کرد و مدل مفصل پلاستیک فایبر را در انتهای عضو اختصاص داد. همان طور که ملاحظه می‌شود ضرائب کاهش سختی این مدل دقت بسیار بالایی داشته و نتایج آن با نتایج حاصل از مدل سازی فنر دورانی لغزش، بسیار نزدیک به هم می‌باشد.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)



طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

در تحلیل‌های غیرخطی حتماً بایستی کاهش سختی المان‌های بتنی، شبیه سازی شود.

تحلیل غیرخطی

- طراحی عملکردی (ضرائب کاهش سختی پیوست A از استاندارد ACI318-19)
- بهسازی لرزه‌ای (ضرائب کاهش سختی در استاندارد ASCE41-17)

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

پیوست دوم استاندارد ۲۸۰۰

۲- مشخصات غیرخطی اعضای سازه

۲-۱ مشخصات غیرخطی اعضای سازه در مدلسازی باید به لحاظ مقاومت، سختی و شکل‌پذیری با داده‌های آزمایشگاهی و یا مدل‌های تحلیلی معتبر سازگار باشد.

۲-۲ رابطه نیرو-تغییرشکل اعضا را می‌توان حداقل به‌صورت دو خطی در نظر گرفت. سختی ارتجاعی در ساختمان‌های بتن‌آرمه و بنایی براساس مقاطع ترک‌خورده در نظر گرفته می‌شود. در اعضای شکل‌پذیر که انتظار می‌رود رفتار غیرخطی داشته باشند، سختی ارتجاعی در مدلسازی دو خطی، سختی سکانت تا نقطه جاری شدن محسوب می‌شود. در منحنی رفتاری اعضا می‌توان سختی بعد از جاری شدن را صفر اختیار نمود. استفاده از رابطه سه خطی نیرو-تغییرشکل که اثر سختی قبل و بعد از ترک‌خوردگی را در نظر می‌گیرد مجاز می‌باشد. استفاده از روابط داده شده در "دستورالعمل بهسازی لرزه‌ای ساختمان‌های موجود" (نشریه ۳۶۰) نیز مجاز است.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

ACI 318-19

Table 6.6.3.1.1(a)—Moments of inertia and cross-sectional areas permitted for elastic analysis at factored load level

Member and condition		Moment of inertia	Cross-sectional area for axial deformations	Cross-sectional area for shear deformations
Columns		$0.70I_g$	$1.0A_g$	$b_w h$
Walls	Uncracked	$0.70I_g$		
	Cracked	$0.35I_g$		
Beams		$0.35I_g$		
Flat plates and flat slabs		$0.25I_g$		

Table 6.6.3.1.1(b)—Alternative moments of inertia for elastic analysis at factored load

Member	Alternative value of I for elastic analysis: Minimum	I	Maximum
Columns and walls	$0.35I_g$	$\left(0.80+25\frac{A_{st}}{A_g}\right)\left(1-\frac{M_u}{P_u h}-0.5\frac{P_u}{P_o}\right)I_g$	$0.875I_g$
Beams, flat plates, and flat slabs	$0.25I_g$	$(0.10+25\rho)\left(1.2-0.2\frac{b_w}{d}\right)I_g$	$0.5I_g$

Notes: For continuous flexural members, I shall be permitted to be taken as the average of values obtained for the critical positive and negative moment sections. P_u and M_u shall be calculated from the load combination under consideration, or the combination of P_u and M_u that produces the least value of I.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

ASCE41-13

Table 10-5. Effective Stiffness Values

Component	Flexural Rigidity	Shear Rigidity	Axial Rigidity
Beams—nonprestressed[a]	$0.3E_cI_g$	$0.4E_cA_w$	—
Beams—prestressed[a]	E_cI_g	$0.4E_cA_w$	—
Columns with compression caused by design gravity loads $\geq 0.5A_g f_c'$	$0.7E_cI_g$	$0.4E_cA_w$	E_cA_g
Columns with compression caused by design gravity loads $\leq 0.1A_g f_c'$ or with tension	$0.3E_cI_g$	$0.4E_cA_w$	E_cA_g (compression) E_sA_s (tension)
Beam–column joints	Refer to Section 10.4.2.2.1		E_cA_g
Flat slabs—nonprestressed	Refer to Section 10.4.4.2	$0.4E_cA_g$	—
Flat slabs—prestressed	Refer to Section 10.4.4.2	$0.4E_cA_g$	—
Walls-cracked[b]	$0.5E_cA_g$	$0.4E_cA_w$	E_cA_g (compression) E_sA_s (tension)

[a]For T-beams, I_g can be taken as twice the value of I_g of the web alone. Otherwise, I_g should be based on the effective width as defined in Section 10.3.1.3. For columns with axial compression falling between the limits provided, flexural rigidity should be determined by linear interpolation. If interpolation is not performed, the more conservative effective stiffnesses should be used.
[b]See Section 10.7.2.2.

ASCE41-13

10.3.1.2 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, and deformation levels under gravity loads and seismic forces shall be considered.

C10.3.1.2 Stiffness For columns with low axial loads (below approximately $0.1A_g f_c'$), deformations caused by bar slip can account for as much as 50% of the total deformations at yield. The design professional is referred to Elwood and Eberhard (2009) for further guidance regarding calculation of the effective stiffness of reinforced concrete columns that include the effects of flexure, shear, and bar slip.

ACI STRUCTURAL JOURNAL TECHNICAL PAPER

Title no. 106-S45

Effective Stiffness of Reinforced Concrete Columns

by Kenneth J. Elwood and Marc O. Eberhard

ASCE41-17

Table 10-5. Effective Stiffness Values

Component	Flexural Rigidity	Shear Rigidity	Axial Rigidity
Beams—nonprestressed[a]	$0.3E_{cE}I_g$	$0.4E_{cE}A_w$	—
Beams—prestressed[a]	$E_{cE}I_g$	$0.4E_{cE}A_w$	—
Columns with compression caused by design gravity loads $\geq 0.5A_g f'_{cE}$ [b]	$0.7E_{cE}I_g$	$0.4E_{cE}A_w$	$E_{cE}A_g$
Columns with compression caused by design gravity loads $\leq 0.1A_g f'_{cE}$ or with tension[b]	$0.3E_{cE}I_g$	$0.4E_{cE}A_w$	$E_{cE}A_g$ (compression) $E_{sE}A_s$ (tension)
Beam–column joints	Refer to Section 10.4.2.2.1		$E_{cE}A_g$
Flat slabs—nonprestressed	Refer to Section 10.4.4.2	$0.4E_{cE}A_g$	—
Flat slabs—prestressed	Refer to Section 10.4.4.2	$0.4E_{cE}A_g$	—
Walls—cracked[c]	$0.35E_{cE}A_g$	$0.4E_{cE}A_w$	$E_{cE}A_g$ (compression) $E_{sE}A_s$ (tension)

[a] For T-beams, I_g can be taken as twice the value of I_g of the web alone. Otherwise, I_g should be based on the effective width as defined in Section 10.3.1.3.

[b] For columns 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] See Section 10.7.2.2.

N_{UG} = Member design axial force evaluated based on Eq. (7-3) of ASCE 41; set to zero for tension force in Eq. (10-3)

$$Q_G = Q_D + Q_L + Q_S \quad (7\text{-}3)$$

Q_D = Action caused by dead loads;
Q_L = Action caused by live load, equal to 25% of the unreduced live load obtained in accordance with ASCE 7 but not less than the actual live load; and
Q_S = Action caused by effective snow load.

ASCE41-17

10.3.1.2 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, and deformation levels under gravity loads and seismic forces shall be considered. Gravity-load effects considered for effective stiffnesses of components shall be determined using Eq. (7-3).

$$Q_G = Q_D + Q_L + Q_S \quad (7\text{-}3)$$

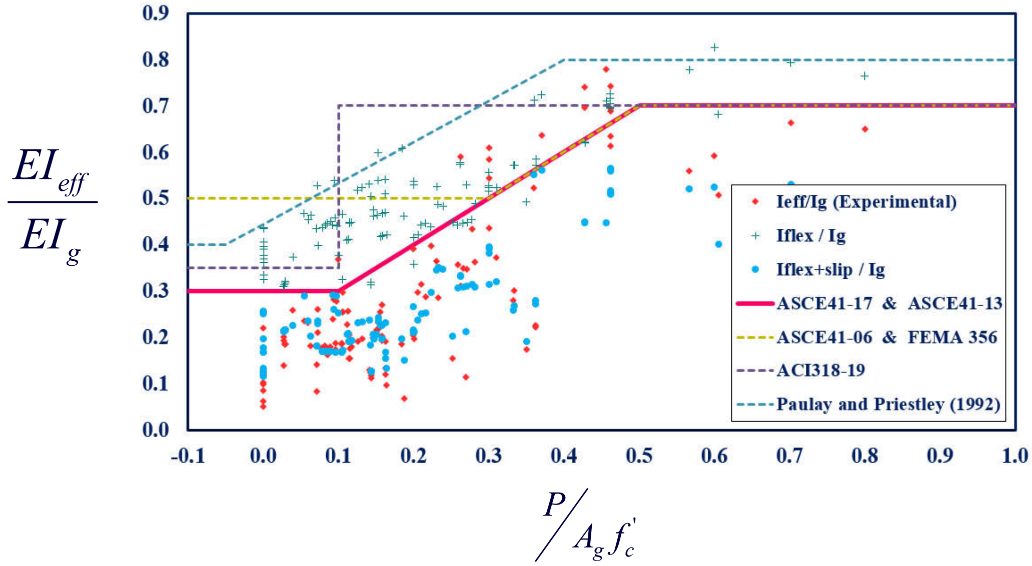
Q_D = Action caused by dead loads;
Q_L = Action caused by live load, equal to 25% of the unreduced live load obtained in accordance with ASCE 7 but not less than the actual live load; and
Q_S = Action caused by effective snow load.

C10.3.1.2 Stiffness. For columns with low axial loads (below approximately $0.1A_g f'_c$), deformations caused by bar slip can account for as much as 50% 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 $0.5E_{cE}I_g$, depending on wall reinforcement and axial load. A method for calculating wall stiffness that provides compatibility with fiber section analysis is offered in C10.7.2.2.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Effective Stiffness Ratio



طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

<u>ACI 318-19</u> & <u>ASCE 41-17</u> Effective stiffness values

ACI 318-19 (Appendix A)

A.8—Effective stiffness

A.8.1 Member stiffness shall include effects of deformations due to flexure, shear, axial elongation or shortening, and reinforcement slip along its development length.

RA.8—Effective stiffness

RA.8.1 Software for nonlinear analysis generally is capable of directly modeling deformations due to flexure, shear, and axial elongation or shortening. Additional deformation may occur due to slip of longitudinal reinforcement from adjacent anchorages. Such effects commonly occur where beams frame into beam-column joints or walls, where columns frame into beam-column joints or foundations, and where walls frame into foundations. If such effects are considered important to the performance of the structure, appropriate assumptions should be included in the analytical model, either directly or by adjustment of flexural stiffness.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

ACI 318-19 & ASCE 41-17 Effective stiffness values

ACI 318-19

RA.8.2: TBI (2017) and LATBSDC (2017) provide additional effective stiffness recommendations while NIST GCR 17-917-46v1 (NIST 2017a) and NIST GCR 17-917-46v3 (NIST 2017c) provide more detailed guidance on modeling of diaphragms and frame elements.

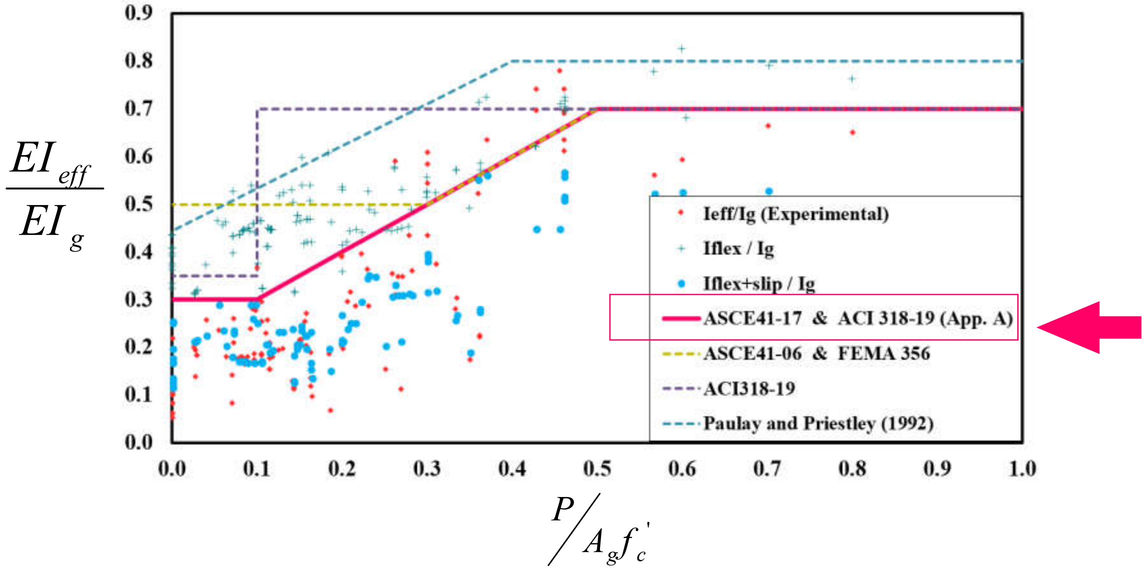
پیوست A از استاندارد ACI 318-19 ضوابط تحلیل غیرخطی تاریخچه زمانی را ارائه می‌کند. مطابق با این پیوست ضرائب کاهش سختی به صورت جدول زیر ارائه شده است. برای مقایسه، ضوابط ASCE 41-17 نیز در این جدول گنجانده شده است. استاندارد ACI 318-19 علاوه بر این جدول، به NIST GCR 17-917-46v3 هم اراجاع می‌دهد و عنوان می‌کند که ضوابط مدرک مذکور دارای جزئیات بیشتری می‌باشد.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Component		ACI 318-19 Appendix A			ASCE41-17		
		Axial	Flexural	Shear	Axial	Flexural	Shear
Beams	nonprestressed	$1.0E_cA_g$	$0.3E_cI_g$	$0.4E_cA_g$	-	$0.3E_{cE}I_g$	$0.4E_{cE}A_w$
	prestressed	$1.0E_cA_g$	$1.0E_cI_g$	$0.4E_cA_g$	-	$1.0E_{cE}I_g$	$0.4E_{cE}A_w$
Columns with compression caused by design gravity loads	$>0.5A_gf_c'$	$1.0E_cA_g$	$0.7E_cI_g$	$0.4E_cA_g$	$1.0E_{cE}A_g$	$0.7E_{cE}I_g$	$0.4E_{cE}A_w$
	$<0.1A_gf_c'$ or with tension	$1.0E_cA_g$(C) $1.0E_sA_s$(T)	$0.3E_cI_g$	$0.4E_cA_g$	$1.0E_{cE}A_g$ (C) $1.0E_{sE}A_s$ (T)	$0.3E_{cE}I_g$	$0.4E_{cE}A_w$
Structural walls	in-plane	$1.0E_cA_g$	$0.35E_cI_g$	$0.2E_cA_g$	$1.0E_{cE}A_g$	$0.35E_{cE}I_g$	$0.4E_{cE}A_w$
	out-of-plane	$1.0E_cA_g$	$0.25E_cI_g$	$0.4E_cA_g$			

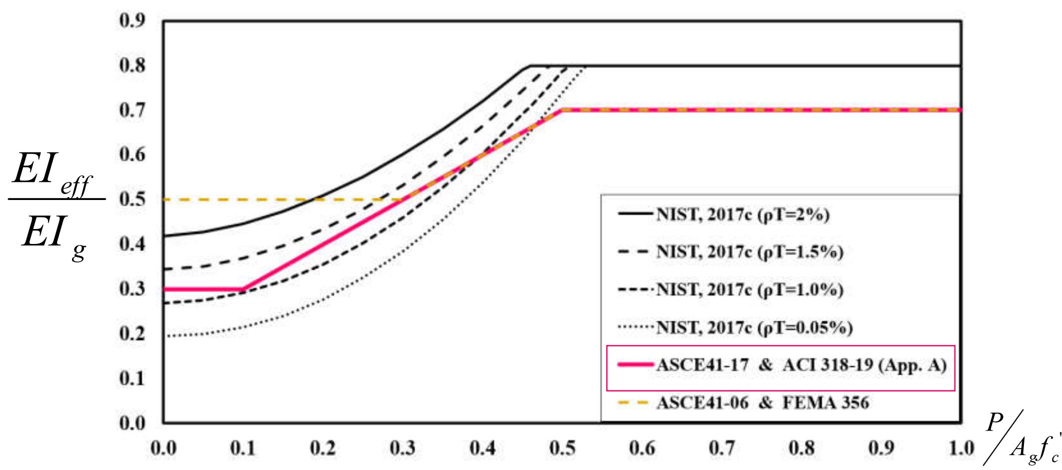
طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

RC Column Effective Stiffness Values



طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

استاندارد ACI 318-19 در پیوست A به مدرک NIST GCR 17-917-46v3 هم اراجاع می‌دهد و عنوان می‌کند که ضوابط مدرک مذکور در محاسبه سختی موثر اعضای بتنی، دارای جزئیات بیشتری می‌باشد. در این نمودار ضوابط هر دو استاندارد ASCE 41-17 و ACI 318-19 با NIST GCR 17-917-46v3 مقایسه می‌شود.



FEMA Support for ACI 369/ASCE 41

FEMA providing funding through ATC Project 140 to help ASC/SEI 41 and ACI 369 update provisions of their standards

ATC Project No. : ATC 140-1

Project Title: Update of Seismic Evaluation and Retrofit of Existing Buildings Guidance

Client: Federal Emergency Management Agency

Purpose: To investigate and address technical issues regarding the evaluation and retrofit of existing buildings and develop material for the expanded FEMA design applications document that will replace the current FEMA 275 Design Examples document.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

RC Wall Effective Flexural Stiffness Values (ACI 369.1-23)

The dissertation of Saman Ali Abdullah is approved.

Henry J. Burton

Kristijan Kolozvari

Scott Joseph Brandenberg

John Wright Wallace, Committee Chair

University of California, Los Angeles

2019

UNIVERSITY OF CALIFORNIA

Los Angeles

Reinforced Concrete Structural Walls: Test Database and Modeling Parameters

A dissertation submitted in partial satisfaction of the requirements for the degree Doctor of Philosophy in Civil Engineering

by

Saman Ali Abdullah

2019

Saman Ali Abdullah

Chair: John Wallace

7.9. Acknowledgements

Funding for this study was provided, in part, by ATC 140 Project, and the University of California, Los Angeles. The authors would also like to thank the other member of Working Group 3 (WG3) of ATC 140 Project, which include Wassim Ghannoum, Garrett Hagen, Mohamed Talaat, Laura Lowes, and Afshar Jalalian for providing thoughtful comments on the work presented. Any opinions, findings, and conclusions or recommendations expressed in this paper are those of the authors and do not necessarily reflect the views of others mentioned here.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

RC Wall Effective Flexural Stiffness Values (ACI 369.1-23)

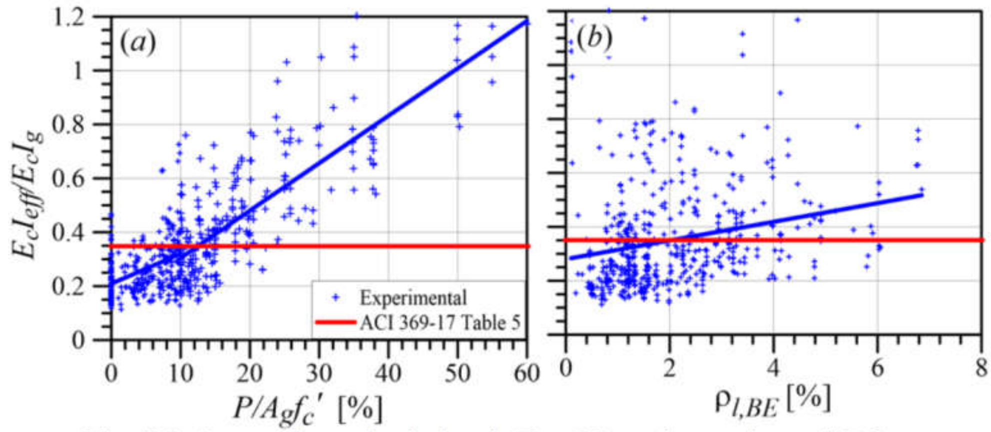


Fig. 7-9–Comparison of calculated (**Eq. 7-9**) and experimental E_cI_{eff}.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

RC Wall Effective Flexural Stiffness Values (ACI 369.1-23)

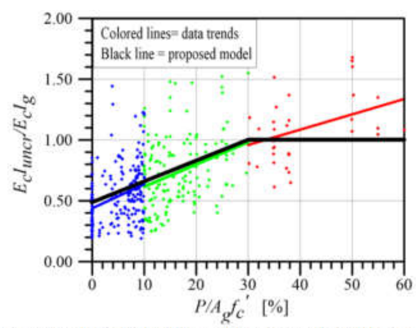


Fig. 7-13– Linear regression lines to the data and the proposed model for E_cI_{uncr}. (black line = model).

Table 7-2–Proposed values for uncracked wall flexural stiffness (E_cI_{uncr})

$\frac{P}{A_g f'_c}$	$\frac{E_c I_{uncr}}{E_c I_g}$*
≤ 0.00	0.50
≥ 0.30	1.00

* Values between those listed should be determined by linear interpolation

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

RC Wall Effective Flexural Stiffness Values (ACI 369.1-23)

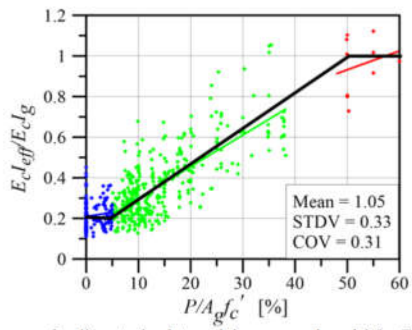


Fig. 7-14–Linear regression lines to the data and the proposed model for E_cI_{eff}. (black line = model).

Table 7-4–Proposed values for effective flexural stiffness (E_cI_{eff})

$\frac{P}{A_g f'_c}$	$\frac{E_cI_{eff}}{E_cI_g}$ *
≤ 0.05	0.20
≥ 0.50	1.00

* Values between those listed should be determined by linear interpolation

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

RC Wall Effective Flexural Stiffness Values (ACI 369.1-23)

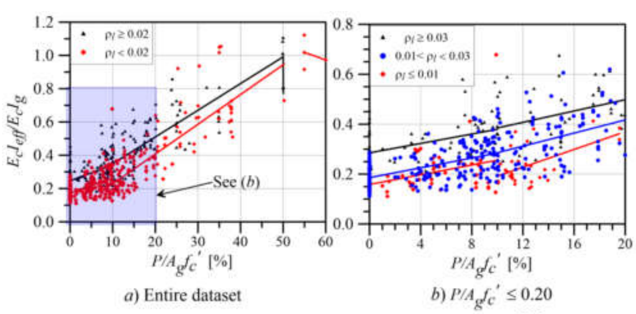


Fig. 7-15–Influence of longitudinal reinforcement ratio ($\rho_{l,BE}$) on E_cI_{eff}.

$$\frac{E_cI_{eff}}{E_cI_g} = 0.1 + 1.5\frac{P}{A_g f_c'} + 3.5\rho_{l,BE} \leq 1.0 \quad \text{(Eq. 7-17)}$$

Table 7-5–Proposed values for E_cI_{eff} as a function of $P/(A_gf'_c)$ and $\rho_{l,BE}$

$\frac{P}{A_g f'_c}$	$\rho_{l,BE}$	$\frac{E_cI_{eff}}{E_cI_g}$ *
≤ 0.05	≥ 0.01	0.20
	≤ 0.03	0.30
≥ 0.50	≥ 0.01	0.90
	≤ 0.03	1.00

* Values between those listed should be determined by linear interpolation

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

RC Wall Effective Shear Modulus (ACI 369.1-23)

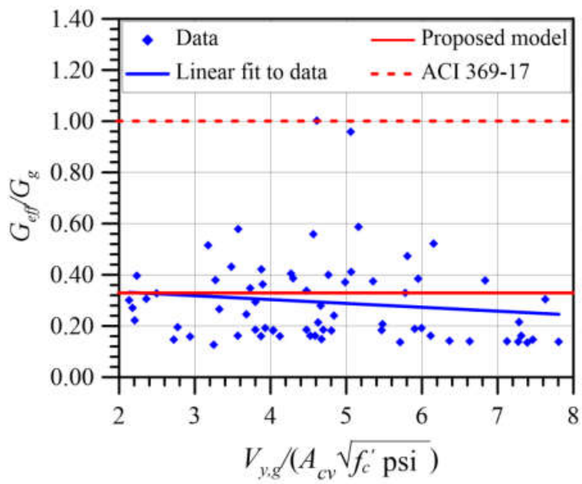


Fig. 7-17–Effective shear modulus results from 64 wall tests.

Gross shear modulus

$$G_g = 0.4E_{cE}$$

Note that shear stress at general yield for all walls exceeded the cracking shear strength of concrete

$$v_c = 0.17\sqrt{f_c'}\,(MPa)$$

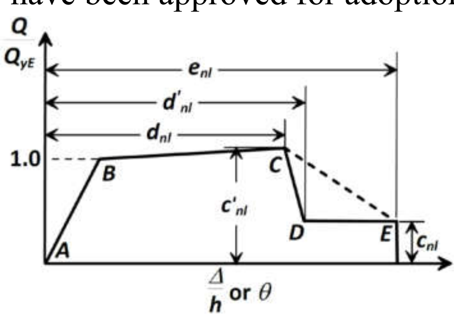
Based on the results of Fig. 7-17, a constant G_{eff} of $G_g/3$ is proposed to be used to model shear response of flexure-controlled walls.

$$G_{eff} = \frac{G_g}{3} = \frac{1}{3}\times 0.4E_{cE} = 0.13E_{cE}$$

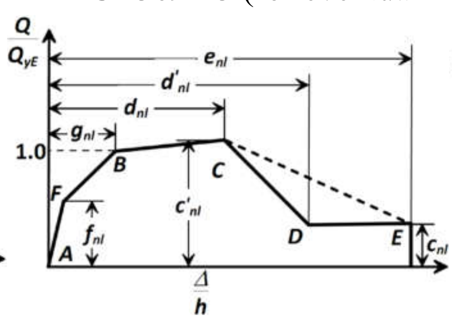
طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Expected Wall Dominant Behavior

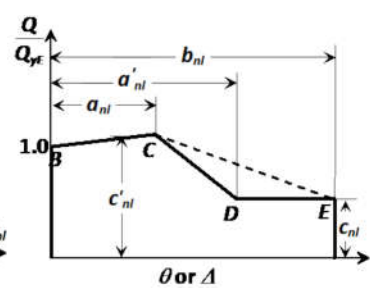
have been approved for adoption in **ACI 369-23** (for eventual inclusion into **ASCE 41-23**)



Flexure-controlled walls



Diagonal shear-controlled walls



Shear friction-controlled walls

$$\frac{V_n}{\omega_v V_{@M_n}} \geq 1.0 \quad \Rightarrow \quad \textbf{Flexure}\text{-controlled walls}$$

$$\frac{V_n}{\omega_v V_{@M_n}} < 1.0 \begin{cases} V_{n,d} \leq V_{n,f} & \Rightarrow \textbf{Diagonal shear}\text{-controlled walls} \\ V_{n,d} > V_{n,f} & \Rightarrow \textbf{Sliding shear}\text{-controlled walls} \end{cases}$$

$$V_n = Min\{V_{n,d}, V_{n,f}\}$$

$V_{@M_n}$: wall shear demand corresponding to the development of M_n

$V_{n,d}$: nominal diagonal shear strength according to ACI 318-19

$V_{n,f}$: nominal shear-friction strength according to ACI 318-19

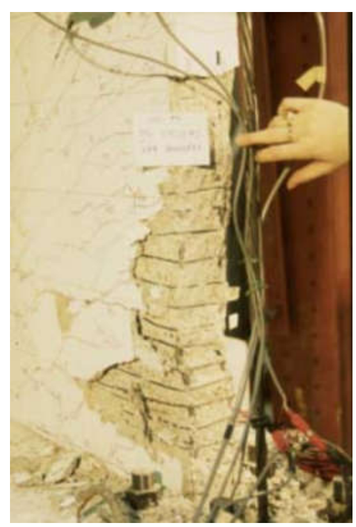
ω_v : is the dynamic shear amplification due to higher mode effects per ACI 318-19

ACI 318-19 §18.10.3
$$\begin{cases} \omega_v = 0.9 + \frac{n_s}{10} & \text{for } n_s \leq 6 \\ \omega_v = 1.3 + \frac{n_s}{30} \leq 1.8 & \text{for } n_s > 6 \end{cases}$$

Wall Flexural Failure Modes $V_n / \omega_v V_{@M_n} \geq 1.0$

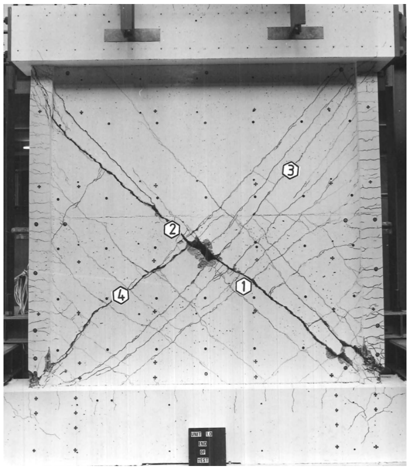
Bar buckling and concrete crushing
(Thomsen and Wallace, 1995)

Bar fracture
(Thomsen and Wallace, 1995)



Lateral instability
(Thomsen and Wallace, 1995)

Wall Shear failure modes $V_n / \omega_v V_{@M_n} < 1.0$



Diagonal tension
(Mestyanek, 1986)

$V_{n,d} \leq V_{n,f}$

Diagonal compression
(Dabbagh, 2005)

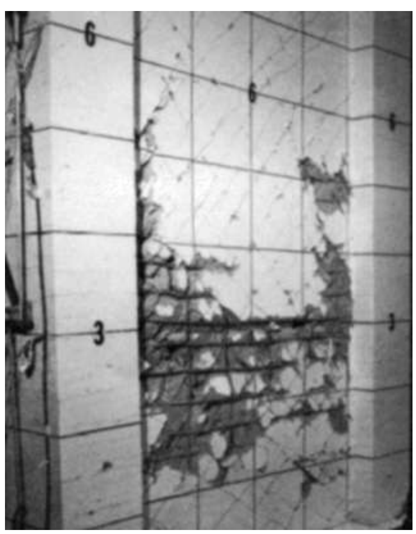
$V_{n,d} \leq V_{n,f}$

Shear-sliding
(Luna, 2015)

$V_{n,d} > V_{n,f}$

Wall Flexural-Shear failure modes

Flexure-diagonal tension
(Tran, 2012)

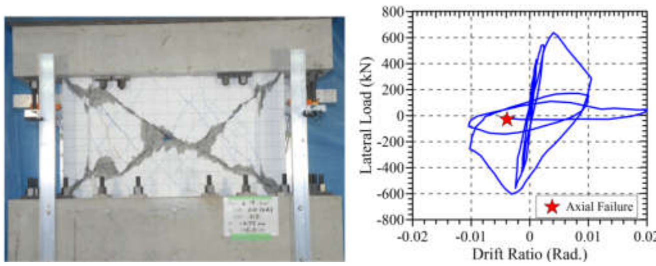


Flexure-diagonal compression
(Oesterle et al., 1976)

Flexure-shear-sliding
(Salonikios et al.,1999)

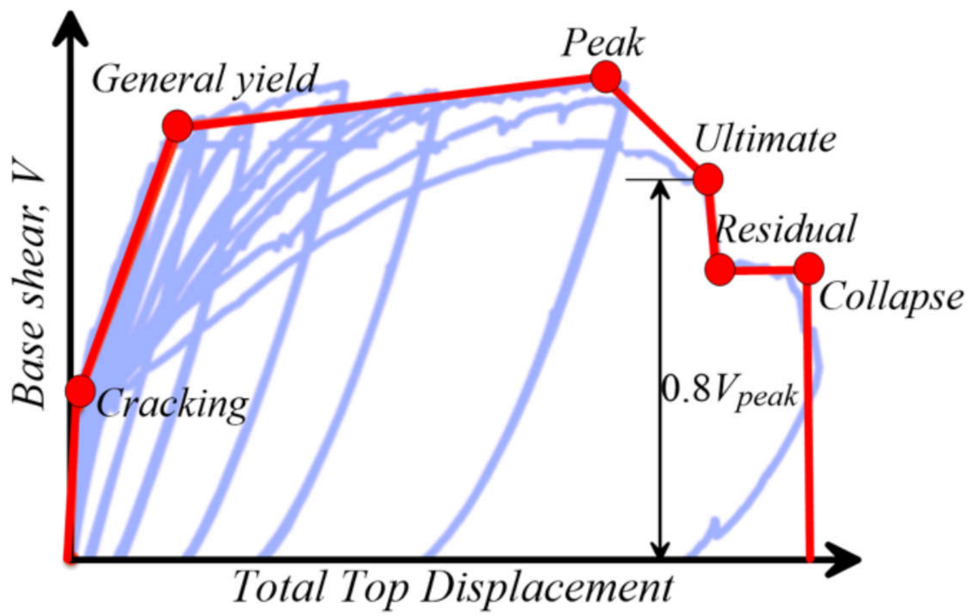
Wall Axial failure modes

Reported axial failure of a flexural-controlled wall test reported by Segura and Wallace (2018)

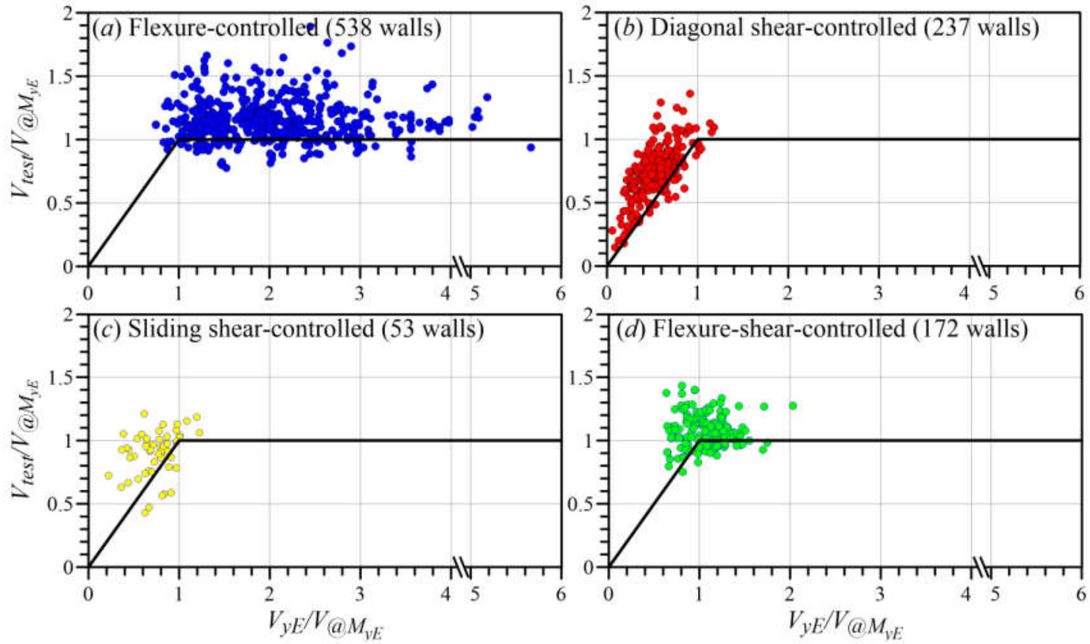


Reported axial failure of a shear-controlled wall test reported by Sanada et al. (2012)

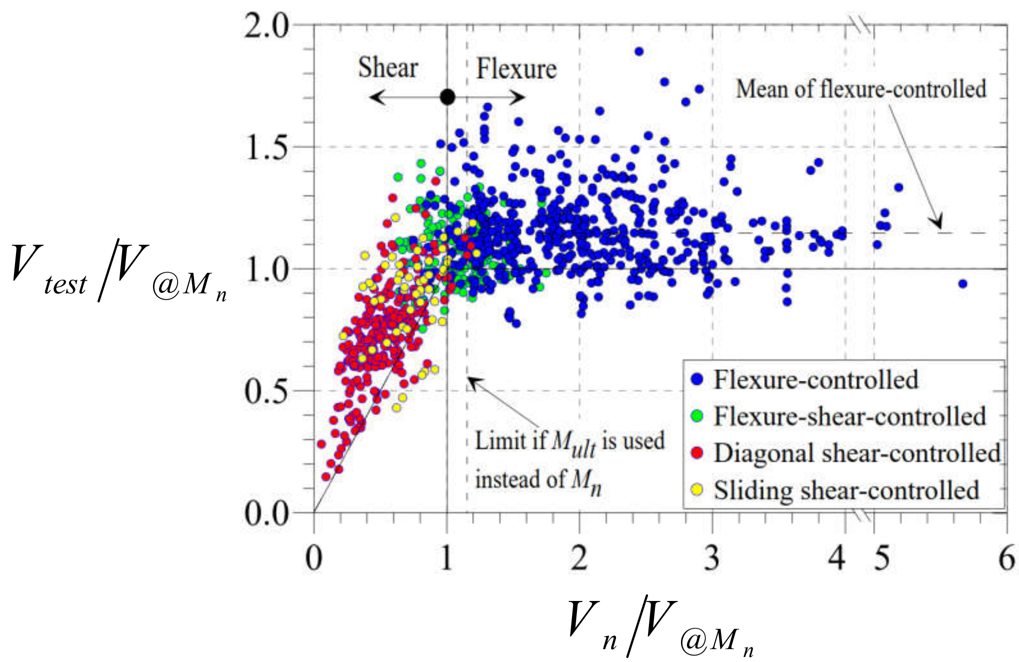
An example of backbone derivation

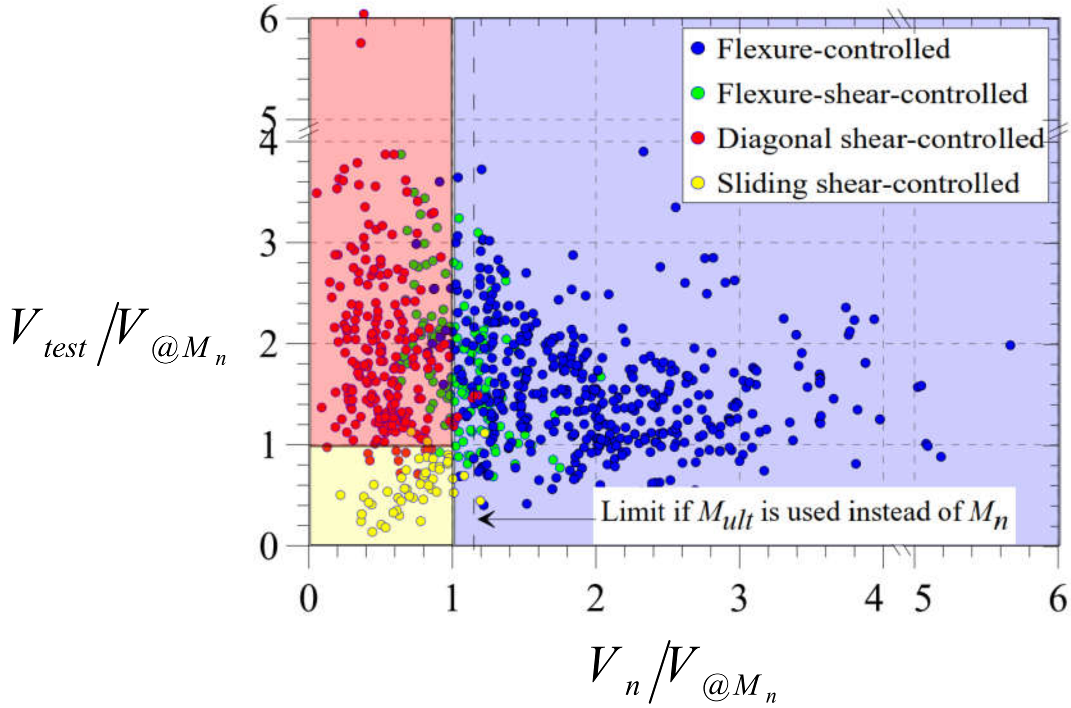


Wall failure modes results from a dataset of 1000 wall tests: failure modes separated



Wall failure modes results from a dataset of 1000 wall tests: failure modes combined





طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Expected Wall Dominant Behavior

have been approved for adoption in **ACI 369** (for eventual inclusion into **ASCE 41-23**)

ASCE 41-17

C10.7 CONCRETE STRUCTURAL WALLS

C10.7.1 Types of Concrete Structural Walls and Associated Components.

The commentary of ASCE 41-17 (C10.7.1) defines slender and squat walls as walls with aspect ratio:

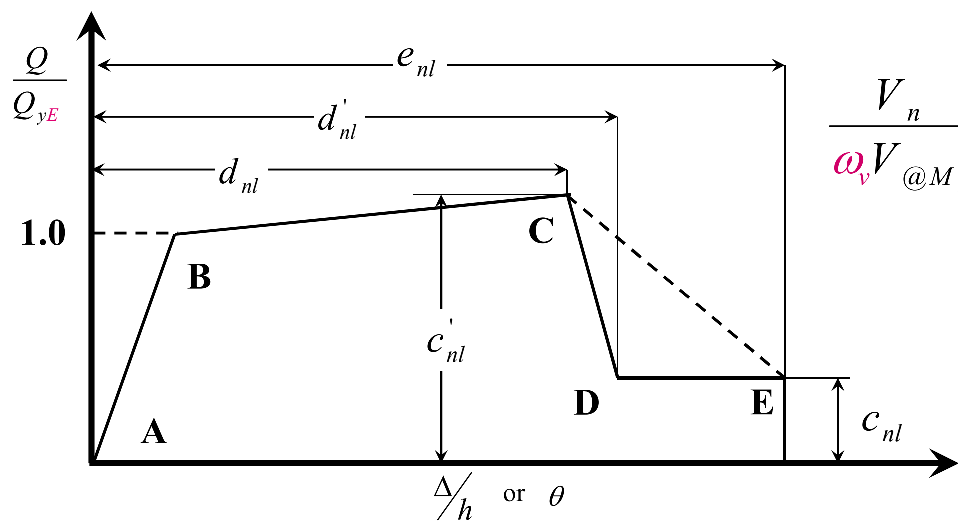
$$\begin{cases} \dfrac{h_w}{l_w} > 3.0 & \Rightarrow \text{Walls are normally controlled by flexural behavior (slender walls)} \\ 1.5 < \dfrac{h_w}{l_w} < 3.0 & \Rightarrow \text{Walls are normally controlled by flexural-shear behavior} \\ \dfrac{h_w}{l_w} < 1.5 & \Rightarrow \text{Walls are normally controlled by shear behavior (short or squat walls)} \end{cases}$$

	Shear behavior		Flexural-shear behavior		Flexural behavior	
	aspect ratio	Axial Load Limit For FC wall	aspect ratio	Axial Load Limit For FC wall	aspect ratio	Axial Load Limit For FC wall
نشریه ۳۶۰-۱۳۹۲	?	$P > 0.15A_g f'_{cl}$	?	$P > 0.35P_0$	$\frac{h_w}{l_w} > 3.0$	$P_u > 0.35P_0$
FEMA356	$\frac{h_w}{l_w} \le 1.5$	$P > 0.15A_g f'_c$	$1.5 < \frac{h_w}{l_w} < 3.0$	$P > 0.35P_0$	$\frac{h_w}{l_w} \ge 3.0$	$P > 0.35P_0$
ASCE41-06	$\frac{h_w}{l_w} < 1.5$	$P > 0.15A_g f'_c$	$1.5 < \frac{h_w}{l_w} < 3.0$	$P > 0.35P_0$	$\frac{h_w}{l_w} > 3.0$	$P > 0.35P_0$
ASCE41-13	$\frac{h_w}{l_w} < 1.5$	$P > 0.15A_g f'_c$	$1.5 < \frac{h_w}{l_w} < 3.0$	$P > 0.35P_0$	$\frac{h_w}{l_w} > 3.0$	$P > 0.35P_0$
ASCE41-17	$\frac{h_w}{l_w} < 1.5$	$P > 0.15A_g f'_{cE}$	$1.5 < \frac{h_w}{l_w} < 3.0$	$P > 0.35P_0$	$\frac{h_w}{l_w} > 3.0$	$P > 0.35P_0$

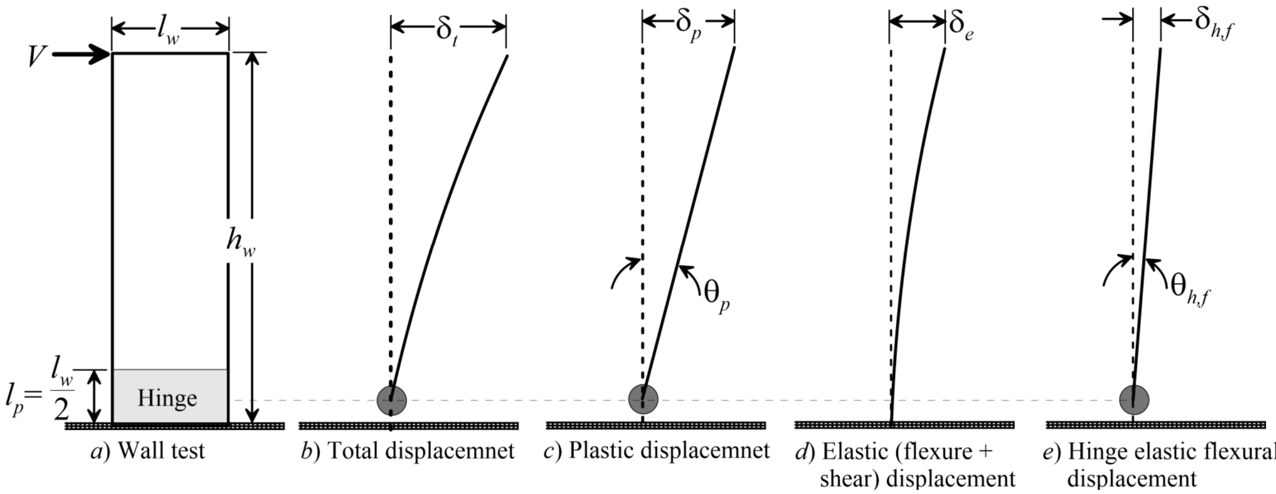
طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

Flexure-Controlled Walls (ACI 369-23 and ASCE 41-23)

تغییرات هوشمندانه در مدل مفاصل پلاستیک ACI 369-23 به قصد کاربرد در مدل‌های فایبر



$$\frac{V_n}{\omega_v V_{@M_n}} \ge 1.0$$



Displacement profiles of flexure-controlled walls

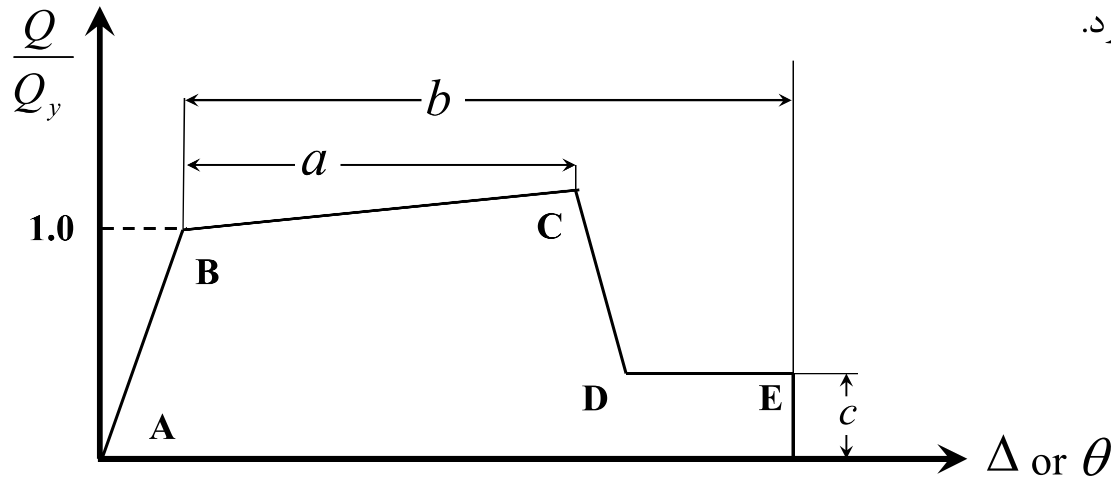
مطابق شکل بالا، مدل مفصل پلاستیک تصویب شده در ACI 369-23 به صورتی است که کل تغییرشکل دورانی (تغییرشکل ارتجاعی + تغییرشکل غیرارتجاعی) را در محدوده مفصل پلاستیک $(l_w/2)$ ارائه می‌دهد.

Table 10-19. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Structural Walls and Associated Components Controlled by Flexure

ASCE 41-17			Plastic Hinge Rotation (radians)		Residual Strength Ratio	Acceptable Plastic Hinge Rotation[a] (radians)		
						Performance Level		
Conditions			*a*	*b*	*c*	IO	LS	CP
i. Structural walls and wall segments								
$\frac{(A_s - A'_s)f_{yE} + P}{t_w l_w f'_{cE}}$	$\frac{V}{t_w l_w \sqrt{f'_{cE}}}$	Confined Boundary[b]						
≤0.1	≤4	Yes	0.015	0.020	0.75	0.005	0.015	0.020

۱- مدل مفاصل پلاستیک دیوارهایِ برشیِ خمش کنترل در استاندارد ASCE 41-17 به صورت RP (Rigid-Plastic) می‌باشد. به عبارت بهتر، پارامترهای مدل سازی a و b در این استاندارد مقدار دوران پلاستیک در مفصل پلاستیک دیوارهای برشی می‌باشد. برای مدل سازی غیرخطی دیوارهای برشی به روش مفاصل پلاستیک در نرم‌افزار SAP2000 یا Etabs عموماً از روش "ستون معادل" استفاده می‌شود. در این روش، مفاصل پلاستیک به پای دیوار (محل اتصال دیوار

به فونداسیون یا دیوارهای حائل پیرامونی) اختصاص داده می‌شود. با توجه به مدل RP مفاصل پلاستیک، تغییرشکل‌های ارتجاعی در مدل مفصل پلاستیک شبیه سازی نشده و به صورت مجزا از طریق مشخصات مکانیکی خطی مصالح و هندسه المان، توسط نرم‌افزار محاسبه می‌شود. مقادیر معیارهای پذیرش در این روش را **نمی‌توان** در روش تحلیل غیرخطی به روش فایبر استفاده کرد.

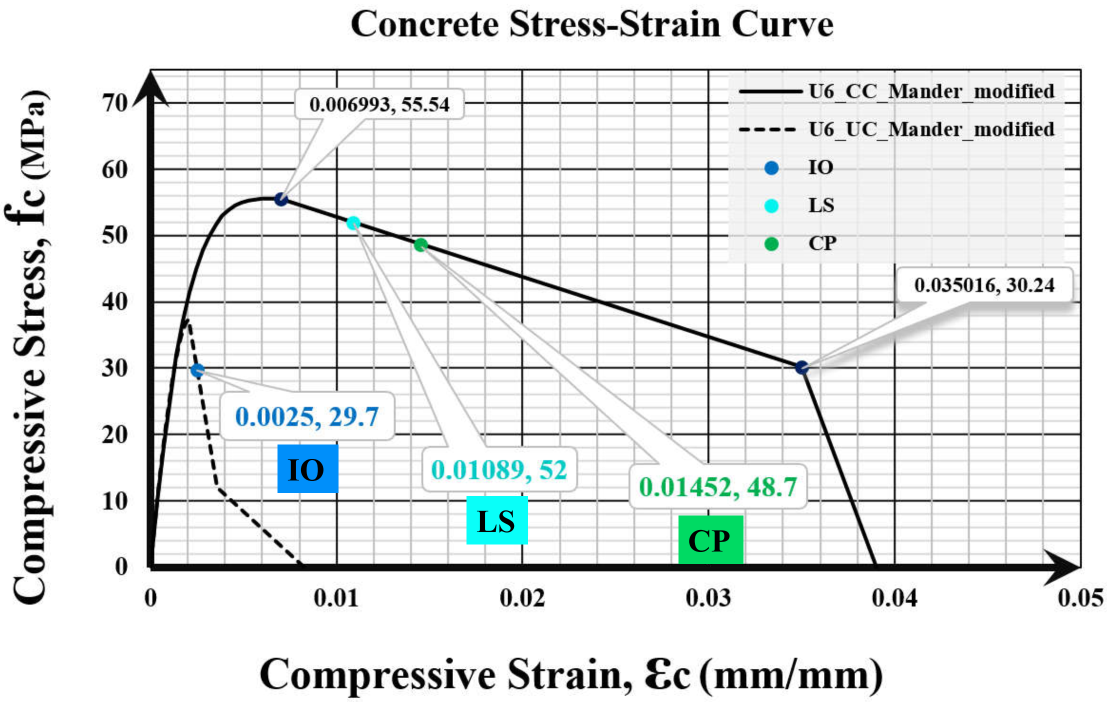


۲- مدل مفصل پلاستیک تصویب شده در ACI 369-23 به صورتی است که کل تغییرشکل دورانی (تغییرشکل ارتجاعی + تغییرشکل غیرارتجاعی) را در محدوده مفصل پلاستیک $(l_w/2)$ ارائه می‌دهد. **این روش کاملاً هوشمندانه انجام شده و بسیار کاربردی خواهد بود.** دلایل استفاده از این روش به صورت زیر ارائه شده است:

۲-۱- پارامترهای مدل سازی به روش‌های محاسباتی دوران تسلیم (θ_y) حساس نخواهد بود (ضوابط دیوارهای کنترل شونده با خمش براساس نتایج تست ۴۴۴ دیوار برشی (۱۸۸ دیوار با ضوابط لرزه‌ای تایید شده ACI و ۲۵۶ دیوار تایید نشده) تدوین شده‌اند. با توجه به اینکه دوران تسلیم در دل پارامترهای مدل سازی نهفته است بنابراین مدل غیرخطی حساس به روش‌های محاسباتی دروان تسلیم نخواهد بود).

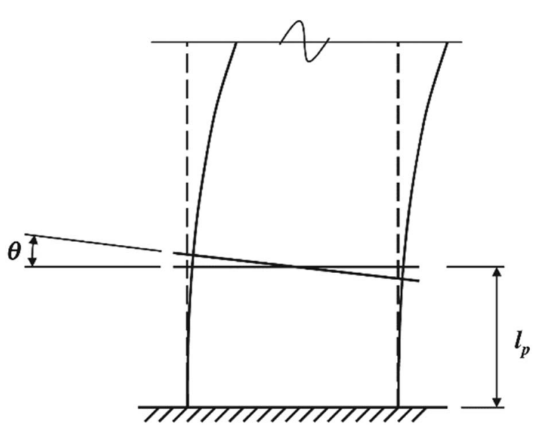
۲-۲- پارامترهای مدل‌سازی دیوارهای برشی و تیرهای همبند کنترل شونده با برش در ASCE 41-17 به صورت نسبت دریفت کلی (total drift ratio) و دوران کلی (دوران وتری یا chord rotation) ارائه می‌شود. مدل‌های جدید ارائه شده در ACI 369-23 باعث می‌شود سایر المان‌ها نیز به لحاظ روش شبیه‌سازی با این دو المان همسان می‌شوند.

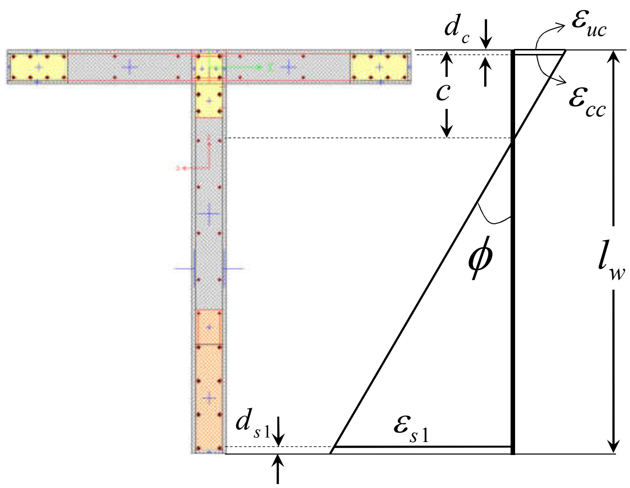
۲-۳-یکی از معضل‌های مدلسازی دیوارهای برشی و ستون‌های بتن‌آرمه به روش فایبر، عدم وجود ضوابط معتبر در "حدود کرنش مصالح" و "معیارهای پذیرش در سطح مصالح" می‌باشد. با تقسیم پارامترهای مدلسازی و معیارهای پذیرش ارائه شده در ACI 369-23 به مقدار طول مفصل پلاستیک می‌توان مقدار انحنای نظیر را به دست آورده و در مرحله بعدی با استفاده از هندسه دیوار، حدود کرنش‌های نظیر با پارامترهای مدلسازی و معیارهای پذیرش محاسبه خواهد شد. **این مورد بسیار پرکاربرد خواهد بود و به نحو احسن معضل مذکور را مرتفع می‌سازد.**



طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

ASCE 41-17: Figure 10-4. Plastic Hinge Rotation in Shear Wall Where Flexure Dominates Inelastic Response





C Point: $\dfrac{d_{nl}}{l_p} = \phi_{nl}$

ASCE 41-17 Bar Buckling Strain Limit

$$\Rightarrow \begin{cases} \phi_{nl} \times (c - d_c) = \begin{cases} \varepsilon_{cc@C} \\ \varepsilon_{sc@C} \le 0.02 \end{cases} \\ \phi_{nl} \times (l_w - c - d_{s1}) = \varepsilon_{s1@C} \le 0.05 \end{cases}$$

ASCE 41-17 Tensile Strain Limit

IO: $$\frac{\theta_{yE} + 0.1(d_{nl} - \theta_{yE})}{l_p} = \phi_{IO} \Rightarrow \begin{cases} \phi_{IO} \times (c) = \varepsilon_{uc@IO} \\ \phi_{nl} \times (l_w - c - d_{s1}) = \varepsilon_{s1@IO} \end{cases}$$

Proposed modeling parameters and numerical acceptance criteria for Conforming reinforced concrete structural walls (special detailing) and associated components controlled by flexure

Conditions[e]				Acceptance Criteria Performance Level
$\frac{l_w c_{DE}}{b_s^2}$	$\frac{w_v V_{MCultDE}}{A_{cv}\sqrt{f'_{cE}}}$ [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[e]						Acceptance Criteria Performance Level	
$\frac{l_w c_{GE}}{b_s^2}$	$\frac{N_{UD}}{A_g f'_{cE}}$	c_{nl}	c'_{nl}	$d'^{\,b}_{nl}$	e^{b}_{nl}	LS	CP
≤ 10	≤ 0.10	0.5	1.15	0.036	0.040	$0.75e_{nl}$	$0.85e_{nl}$
≤ 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 $V_{MCultDE}$ is obtained from nonlinear analyses procedures

[de] Linear interpolation between the values given in the table shall be permitted; however, interpolation between the values Specified for conforming walls and Non-conforming walls shall not be permitted

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

پارامترهای موثر بر ظرفیت دریفت جانبی دیوارهای برشی براساس UCLA-RCWalls database

Abdullah and Wallace (2019) پارامترهای موجود در ادبیات فنی را که می‌توانند در ظرفیت دریفت جانبی دیوارها تاثیرگذار باشند مطابق با جدول زیر گردآوری کرده‌اند:

Correlation coefficients, *R*, for design parameters and wall drift capacity

Design parameter	c/b	l_w/b	$v_{max}/\sqrt{f_c'}$	$P/A_g f_c'$	$A_{sh,provided}/A_{sh,required}$	s/d_b	h_x/b	$\rho_{long\,BE}$	$\rho_{t\,web}$*	f_u/f_y	l_{BE}/l_w*	c/l_w	$l_w c/b^2$
Correlation coefficient, R	–0.66	–0.56	–0.30	–0.08	0.13	–0.02	–0.25	–0.32	–0.14	–0.07	0.06	–0.32	–0.68

*$\rho_{t\,web}$ is web transverse reinforcement ratio; l_{BE}/l_w is length of confined boundary normalized by wall length.

مطابق با این تحقیق پارامترهای زیر بیشترین تاثیر (بزرگترین ضرائب همبستگی) را در ظرفیت دریفت جانبی دیوارهای خمش کنترل دارند:

۱- نسبت عمق تارخنثی دیوار به عرض ناحیه فشاری دیوار (c_{DE}/b_s)، عمق تارخنثی براساس مقدار کرنش 0.003 برای دورترین تارفشاری بتن محاسبه می‌شود.

۲- نسبت طول دیوار به عرض ناحیه فشاری دیوار (l_w/b_s)

۳- نسبت تنش برشی حداکثر $v_{\max}/\sqrt{f_c^{'}}$

۴- هندسه آرماتورهای عرضی المان مرزی (استفاده از دورگیرهای دارای همپوشانی با یکدیگر در مقایسه با استفاده از یک دورگیر پیرامونی به همراه سنجاقی‌های میانی)

سایر پارامترهای مورد بررسی که در جدول بالا ذکر شده‌اند، تاثیر قابل ملاحظه‌ای بر ظرفیت دریفت جانبی دیوارهای برشی ندارند. بنابراین در محاسبات پیشنهادی برای ACI 369-23 حذف شده‌اند.

طراحی لرزه‌ای براساس عملکرد (ACI 318-19)

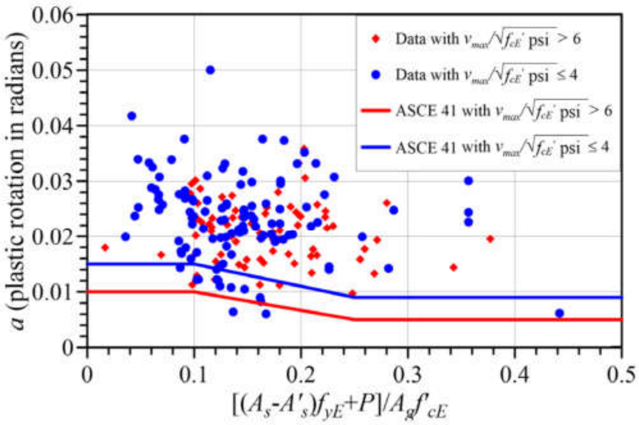
مقایسه ظرفیت دوران پلاستیک دیوارهای برشی براساس نتایج تست و پارامتر مدلسازی a در استاندارد ASCE 41-17

۱-مطابق با نمودارهای زیر، پر واضح است که پارامتر مدلسازی a در استاندارد ASCE 41-17 یک تخمین محافظه کارانه از کرانه پایین نتایج آزمایشگاهی برای دیوارهای با دیتیل لرزه‌ای "تایید شده" و "تایید نشده" می‌باشد.

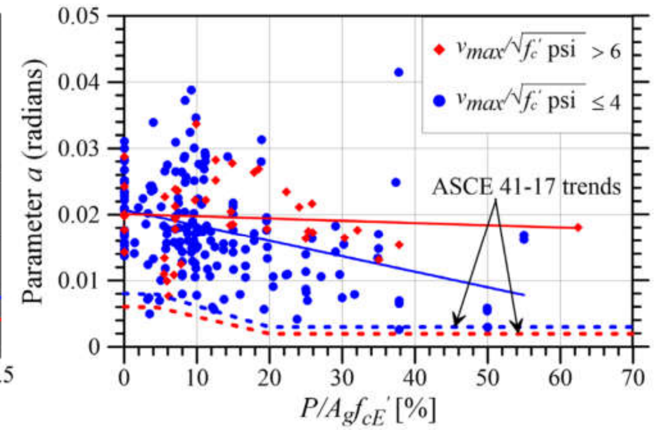
۲-مطابق با نمودارهای زیر، پر واضح است که پارامتر "نسبت بار محوری" معرفی شده در استاندارد ASCE 41-17 برای محاسبه مقدار a با نتایج آزمایشگاهی همبستگی بسیار پایینی دارد (0.08-) بنابراین نمودار a برحسب این پارامتر، پراکندگی بسیار زیادی نشان می‌دهد.

$$\frac{(A_s - A_s^{'})f_{yE} + P}{A_g f_{cE}^{'}}$$

۳- در مدل جدیدِ ظرفیت دریفت جانبی دیوارهای برشی، نسبت بارمحوری حذف شده است. به عبارت بهتر براساس ACI 369-23 ظرفیت دریفت جانبی دیوارهای برشی کنترل شونده توسط خمش، **مستقل از بارمحوری است.**



(a) Conforming walls



(b)Non-conforming walls