English Version, 1st

Standard

for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001

Guidelines

for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001

and

Technical Manual

for Seismic Evaluation and Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001

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Technical Manual for Seismic Evaluation and Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001

Translated by :
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Translators' Foreword

In Japan, unexpectedly severe damage to buildings in a series of earthquakes, including the 1948 Fukui Earthquake, 1964 Niigata Earthquake, 1968 Tokachi-oki Earthquake, and 1975 Oita Earthquake, made it clear that the provisions of the existing seismic design method alone were inadequate to guarantee the safety of new buildings which could be designed with free structural plans. Therefore, a new seismic design method was developed under the leadership of Japan's Ministry of Construction (now Ministry of Land, Infrastructure and Transport). As a result of this effort, the Revised Building Standard Law and Enforcement Order were promulgated in 1980 and took effect in 1981.

The Revised Building Standard Law and Enforcement Order were based on evaluation of the ultimate strength of buildings, among other features, and consequently created a situation in which much of the existing stock of buildings in Japan, which had been designed in accordance with the former seismic design method, failed to conform to the new design code. Because this problem had been anticipated when study of the Revised Building Standard Law began, development of techniques for evaluating the seismic capacity of existing buildings and, when necessary, improving their seismic capacity (seismic retrofit) was considered an urgent matter. Therefore, the study which led to the Standard for Seismic Evaluation / Guidelines for Seismic Retrofit of Existing Buildings was undertaken in parallel with the establishment of the Revised Building Standard Law and Enforcement Order, resulting in publication of the 1st Edition (Japanese Ed.) of the present Standard in 1977, in advance of the enforcement of the new law itself.

Because the Standard/Guidelines took evaluation/improvement of the seismic capacity of the existing building stock as its purview, it was extremely innovative for the time and without precedent in any other country. However, as its intended range of application was existing buildings in Japan, only a Japanese edition was published. Revised editions were published in 1990 and 2001 based on subsequent technical progress, but were also limited to Japanese.

In recent years, damage to buildings with low seismic capacities has occurred in a number of earthquake-prone countries, for example, in the 1999 Kocaeli Earthquake and 1999 Chi-chi Earthquake, requiring technical development for improvement of the seismic capacity of such buildings. The translators have had the experience of participating in technical cooperation projects for seismic evaluation/seismic retrofit of existing buildings in a number of countries, including Indonesia, Mexico, Peru, Turkey, and Rumania, where we used the Japanese edition of this Standard/Guidelines for reference. However, in the absence of an English-language edition, we encountered considerable difficulties in technical cooperation, and we felt that an English edition was absolutely necessary for popularizing seismic evaluation/seismic retrofit technologies among engineers in a larger number of countries.

At this juncture, the Building Research Institute (BRI) received the translation right for an English edition from the Japan Building Disaster Prevention Center. The BRI began the translation work at once, but considering limitations on the translators' time, a decision was made to include only the minimum information necessary for performing seismic evaluation and retrofit in the English edition (1st English Ed.). To clarify the differences in the composition of the Japanese and English editions, the following compares the contents of the Japanese edition of 2001 and the 1st English edition.

(1) The Japanese edition consists of three volumes, Standard for Seismic Evaluation (300 pp.), Guidelines for Seismic Retrofit (377 pp.), and Technical Manual (107 pp.). This has been summarized in one volume in the English edition, which contains only the minimum necessary parts.

(2) In each of the volumes of the Japanese editions, the Prefaces and lists of members of the editorial or revision committees in the 1st Edition, 1990 Revision, and 2001 Revision are included before the respective Contents. Because this information is not directly necessary for users of the English edition, a section of "Prefaces and Members Lists to the Japanese Editions" has been included at the end of the English volume.

(3) The respective Contents of the three volumes of the Japanese edition have been consolidated in the Contents of the English edition. Readers should note that the parts of chapters shown in italics in the Contents of the English edition have not been translated, but are listed so that readers of the English edition can understand the composition of the Japanese edition as a whole.

(4) In the Japanese edition, the volume of the Standard for Seismic Evaluation comprises provisions and supplementary provisions, with commentaries and references provided for each. In principle, the English edition contains only the provisions and supplementary provisions, with translators' notes, as listed below, added for items which were deemed necessary and indispensable for understanding these two parts.

    1) Translators' note on concept of seismic evaluation

    2) Translators' note on column supporting the wall above

    3) Translators' note on second-class prime element

    4) Translators' note on ductility index by the 1990 version

    5) Explanatory figure for division methods into unit portions of a wall

    6) Explanatory figure for calculation of human risk index

    7) Translators' note on index for cumulative strength in ultimate limits of buildings

    Because the Technical Manual compares calculated results using the ductility index (F) based on the calculation method in the 1990 Revision and the calculated results using the ductility index in the 2001 Revision, the English edition describes the calculation method for the ductility index in the 1990 Revision, providing translators' note on this item. The reference literature listed in the commentaries of the Japanese edition of the Standard is shown collectively as References following the Technical Manual, together with various reference literature cited in the Guidelines for Seismic Retrofit and Technical Manual in the Japanese edition.

(5) The Japanese edition of the Guidelines for Seismic Retrofit comprises a main text and Appendixes, with commentary and references provided for the main text and references

provided for the Appendixes. In principle, the English edition is limited to a translation of the main text. However, as minimum items necessary for understanding the main text, the English edition also contains, as reference materials, approximately 50 figures on various strengthening methods from the original commentaries.

(6) The Technical Manual in the Japanese edition includes commentaries on examples of application of the Standard for Seismic Evaluation as Appendix 1 and commentaries on examples of application of the Guidelines for Seismic Retrofit as Appendix 2. The English edition contains translations of 2 cases of application of the Standard for Seismic Evaluation in Appendix 1 (Examples of a pure frame structure and a school building excluding details of strengthening).

In Japan, many existing buildings are being restored each year by seismic retrofit based on the Japanese edition. We hope that this English edition (1st English Ed.) will also contribute to improvement in the seismic capacity of buildings with low earthquake resistance in all earthquake-prone countries.

It should be noted that the fundamental concepts of the Standard for Seismic Evaluation can be understood by reading the overall volume of the Japanese edition, but, as mentioned previously, the English edition contains only translations of the minimum parts necessary for performing seismic evaluation and seismic retrofit. Therefore, before reading the present translation, we recommend that the user refer to the paper by Dr. Umemura *1 in order to understand the general outline.

In conclusion, we would like to express our deep appreciation to all those concerned with the publication of this work, and particularly to Mr. Shiro Kikuchi of the Japan Building Disaster Prevention Association, who was in charge of publishing the English edition (1st English Ed.), Mses. Akemi Iwasawa, Nobue Ochiai, and Kumiko Hirayama of the BRI, for their cooperation in all stages of the work, from preparation of the equations, figures, and tables to typing of the manuscript, and the members of the Review Committee, for supervising the English edition.

December, 2004

Isao Nishiyama

Director of Housing Department, National Institute of Land and Infrastructure Management, Ministry of Land, Infrastructure and Transport

*1 Umemura, H: “A Guideline To Evaluate Seismic Performance Of Existing Medium- And Low-Rise Reinforced Concrete Buildings And Its Application“. Proceedings Of The Seventh World Conference On Earthquake Engineering, September 8-13, 1980, Istanbul, Turkey, Volume 4, pp. 505-512.

Preface

In earthquake disaster prevention, one serious problem confronting the world's earthquake-prone countries is seismic retrofit of older buildings which were constructed without the benefit of recent progress in seismology and earthquake engineering. Based on progress in these two areas over the last 20 to 30 years, a number of countries are currently revising their seismic design standards. However, the buildings which enjoy stronger earthquake resistance thanks to these revised standards are new buildings. Buildings which were already constructed based on older design standards are being left behind and remain in danger. The Japanese seismic design standard was strengthened in 1981, but virtually all of the buildings which were destroyed or suffered severe damage in the 1995 Hanshin-Awaji Earthquake Disaster were those constructed prior to 1981. In fact, about half of Japan's existing building stock was constructed under the old standard. While this does not mean that all of these buildings are in danger of damage by earthquakes, it is necessary to identify those that are at risk and carry out reconstruction or seismic retrofit. This situation is not unique to Japan, but is a common problem of all earthquake-prone countries. For example, the damage in the 1994 Northridge Earthquake in the United States, the Kocaeli Earthquake in Turkey and Chi-chi Earthquake in Taiwan in 1999 is ample evidence of this problem.

This volume is an English translation of the Japan Building Disaster Prevention Association's "Standard for Seismic Evaluation and Guidelines for Seismic Retrofit of Existing R/C Buildings" and is being published to assist other earthquake-prone countries which face problems similar to Japan in their earthquake disaster prevention efforts. The 1st Edition (in Japanese) of the Standard/Guidelines was published in 1977, followed by revisions in 1990 and 2001. The English translation is based on the most recent revision, which was completed in 2001.

The English translation was entrusted to the Independent Administrative Institution, Building Research Institute (BRI) and was completed in a short period of time by Drs. Isao Nishiyama, Masaomi Teshigawara, Hiroshi Fukuyama, and Koichi Kusunoki of the BRI. I wish to express my deep appreciation to those gentlemen for their dedicated efforts. The translation was also reviewed by the principal members of the 2001 Revision Committee, Drs. Toshimi Kabeyasawa, Masaya Murakami, Yoshiaki Nakano and Hideo Katsumata. I would like to take this opportunity to thank all of these persons for their valuable contributions.

All of those concerned sincerely hope that this Standard/Guidelines will be useful in alleviating the effects of natural disasters in earthquake-prone countries around the world.

December, 2004

Tsuneo Okada, President

The Japan Building Disaster Prevention Association

Professor Emeritus, Tokyo University

Contents

Supplementary Provisions; Strength, deformation capacity (ductility index), and yield deformation of members

* Contents written in italic are not included in this English version.

Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001

* Contents written in italic are not included in this English version.

Technical Manual for Seismic Evaluation and Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001

* Contents written in italic are not included in this English version.

References

Prefaces and Members Lists to the Japanese Editions

Standard

for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001

Chapter 1 General

1.1 Basic Principle

The provisions of this standard shall be applied to seismic evaluation of existing reinforced concrete buildings. The seismic evaluation shall be based on both site inspection and structural calculation to represent the seismic performance of a building in terms of seismic index of structure I_S and seismic index of non-structural elements I_N. The seismic safety of the building shall be judged based on standard for judgment on seismic safety wherein seismic performance demands are prescribed. See the translators' note 1.

1.2 Scope

This standard shall be applied to the seismic evaluation and the verification of seismic retrofitting of existing low-rise and medium-rise reinforced concrete buildings. Three levels of screening procedures, namely the first, the second, and the third level screening procedures, have been prepared for the seismic evaluation according to this standard. Any level of the screening procedures may be used in accordance with the purpose of evaluation and the structural characteristics of the building.

The methods specified in the provisions and the commentary of this standard should be used in principle for seismic evaluation. In addition, other methods, which are based on the concept of this standard and have been verified through experimental data or detailed analyses to be equivalent to the methods of this standard, may also be used for seismic evaluation.

1.3 Definitions

(1) Indices for seismic performance of buildings

SEISMIC INDEX OF STRUCTURE I_S: An index representing the seismic performance of structure.

SEISMIC INDEX OF NON-STRUCTURAL ELEMENTS I_N: An index representing the seismic performance of non-structural elements, such as exterior walls.

SCREENING LEVEL: The degree of simplification in calculating the indices I_S and I_N. Three screening levels are provided from the first, simple level to the third, detailed level of screening.

(2) Sub indices for calculation of seismic index of structure I_S

BASIC SEISMIC INDEX OF STRUCTURE E_0: An index representing the basic seismic performance of a building, evaluated as a function of the strength index C, the ductility index F, and the story-shear modification factor.

STORY-SHEAR MODIFICATION FACTOR: A factor normalizing the strength index C of upper stories being equivalent to the base shear coefficient in consideration of the story level and the lateral earthquake force distribution.

CUMULATIVE STRENGTH INDEX C_T: Strength index accumulated for the members in a story in relation to the story drift angle (ductility index) accounting for the compatibility of the members and modified by the story-shear modification factor.

STRENGTH INDEX C: The lateral strength or the lateral-load carrying capacity of a member or a story in terms of shear coefficient, namely the shear normalized by the weight of the building sustained by the story.

DUCTILITY INDEX F: An index representing the deformation capacity of a structural member.

IRREGULARITY INDEX S_D: An index modifying the basic seismic index of structure E_0 in consideration of unbalance in stiffness distribution and/or irregularity in structural plan and elevation of a building.

TIME INDEX T: An index modifying the basic seismic index of structure E_0 in consideration of aging of a building.

MATERIAL STRENGTH: Compressive strength of concrete and yield strength of reinforcing bar that are used to calculate the flexural and shear ultimate strengths of structural members. Specified design strength may be used for the compressive strength of concrete, 294 N/mm^2 for the yield strength of round bars, and 49 N/mm^2 plus the nominal yield strength for deformed bars, in case the material tests are not performed at the site inspection.

ULTIMATE DEFORMATION: Limit deformation within which a structural member can carry its lateral strength and its axial load during an earthquake stably.

DUCTILITY FACTOR: Ratio of the deformation capacity to the yield deformation.

GROUPING: The action of collecting structural members with similar ductility indices and arranging them as a member group, for which the sum of strength indices of the group members is defined as the group strength index.

EFFECTIVE STRENGTH FACTOR α: Ratio of the lateral resistance of a member at a certain level of story deformation to the calculated lateral strength based on the compatibility.

COLUMN: A vertical member with inflection point in its deformable portion. There are columns with/without wing walls and short columns.

COLUMN WITH WING WALL: A vertical member consisting of a column and wing wall(s) attached to monolithically, which is regarded as column.

WALL WITH A (ONE) COLUMN (wing wall with a column, wall with one boundary column): A vertical member consisting of a column and wing wall(s) attached to monolithically, except for a wall with two boundary columns.

EXTREMELY SHORT COLUMN: A column with h_0/D (clear height divided by depth) less than 2.

COLUMN CLEAR HEIGHT h_0: The height of the deformable portion in a column without beams, standing walls and hanging walls.

EXTREMELY BRITTLE COLUMN: An extremely short column whose shear failure precedes flexural yielding.

FLEXURAL COLUMN: A column whose flexural yielding precedes shear failure.

SHEAR COLUMN: A column whose shear failure precedes flexural yielding.

COLUMN GOVERNED BY FLEXURAL BEAM (flexural beam-governed column): A column seismic performance of which is governed by beams whose flexural yielding precedes shear failure.

COLUMN GOVERNED BY SHEAR BEAM (shear beam-governed column): A column seismic performance of which is governed by beams whose shear failure precedes flexural yielding .

WALL: A vertical member other than columns, categorized into walls with two boundary columns, and walls without columns.

WALL WITH (TWO) BOUNDARY COLUMNS: A wall with boundary columns at both sides, including those sequential in multi spans.

WALL WITHOUT (BOUNDARY) COLUMNS: A wall without columns, including those located outside frames.

FLEXURAL WALL: A wall whose flexural yielding precedes shear failure.

SHEAR WALL: A wall whose shear failure precedes flexural yielding.

UPLIFT WALL: A wall whose rotating (uplifting) mode of failure precedes flexural yielding and shear failure.

FRAME WITH SOFT STORY: A system filled with multi-story shear walls except for one or a few stories, including so-called pilotis frame.

SOFT STORY COLUMN (COLUMN SUPPORTING THE WALL ABOVE): An column located in a frame with soft story directly under walls. See the translators' note 2.

SECOND-CLASS PRIME ELEMENT: Column or wall element, loss of whose lateral resistance is not fatal, but loss of the gravity load carrying capacity leads to collapse of the structure, even though accounting for redistribution to neighborhood elements. See the translators' note 3.

ULTIMATE STATE OF STRUCTURE (or STORY): A state in terms of inter-story deformation or ductility index at overall or partial collapse of the structure, defined by the loss of the gravity load carrying capacity leading to vertical collapse or the lateral strength decay leading to unstable lateral response.

(3) Indices for judgment on seismic safety of buildings

SEISMIC DEMAND INDEX OF STRUCTURE I_{SO}: The standard level of the seismic index required for a building to be safe against the earthquake hazard on the site of the building, defined as a product of E_S, Z, G and U.

BASIC SEISMIC DEMAND INDEX OF STRUCTURE E_S: A sub-index representing the basic seismic demand for a building.

ZONE INDEX *Z*: A sub-index accounting for the expected seismic activities and seismic intensities.

GROUND INDEX *G*: A sub-index accounting for the effects of soil profiles, geological conditions, and soil-and-structure interactions.

USAGE INDEX *U*: A sub-index accounting for the use of a building.

ULTIMATE CUMULATIVE STRENGTH INDEX C_{TU}: The cumulative strength index evaluated at the ultimate state of a building or a story.

(4) Sub indices for evaluation of seismic index of non-structural elements I_N

CONSTRUCTION INDEX *B*: An index representing the failure risk of non-structural elements depending on the building construction, calculated from the conformability index *f* and the damage record index *t*.

CONFORMABILITY INDEX *f*: An index representing the conformability of non-structural elements based on the deformability of the non-structural elements relative to that of the structural members.

DETERIORATION INDEX *t*: An index representing the deterioration of the deformability of non-structural elements due to aging and past damage.

AREA INDEX *W*: An index representing the area of non-structural elements concerned.

HUMAN RISK INDEX *H*: An index representing the risk of injury to human due to the failures of non-structural elements, evaluated by the location index *e* and the risk reduction index *c*.

LOCATION INDEX *e*: An index representing the possibility of human presence under the non-structural elements at the failure.

RISK REDUCTION INDEX *c*: An index representing the reduction of the human risk such as by the existence of fence against the failure of non-structural elements.

Chapter 2 Building Inspection

2.1 General

Building inspection shall be conducted to check the structural characteristics of the building which are necessary to calculate the seismic index of structure I_S. Appropriate methods for inspection should be selected in accordance with the screening level, such as site inspection, collection of design drawings, and material test.

2.2 Preliminary Inspection

An appropriate preliminary inspection shall be carried out to check the applicability of this standard for the seismic evaluation.

2.3 First Level Inspection

The first level inspection should be conducted on the following investigation items, which are mainly necessary for calculation of the seismic index of structure in the first level screening procedure:

(1) Material strengths and cross-sectional dimensions for calculation of strengths of structural members.

(2) Crackings in concrete and deformations of structure for evaluation of time index.

(3) Building configuration for evaluation of irregularity index.

2.4 Second Level Inspection

The second level inspection should be conducted on the following investigation items, which are necessary for calculation of the seismic index of structure in the second or the third level screening procedures:

(1) Material strengths and cross-sectional dimensions for calculation of strengths of structural members.

(2) Degrees of occurrence and ranges of structural cracking and deformation.

(3) Grades and ranges of deterioration and aging.

In the second level inspection, an inspector may conduct visual inspection or measurement without breaking finishing materials. The finishing materials should be removed if necessary accounting for the grades of cracking and aging.

2.5 Detailed Inspection

The detailed inspection should be conducted on the following investigation items in addition to the second level inspection if necessary for more precise evaluation and/or strengthening design:

(1) Strengths and Young's moduli of concrete.

(2) Arrangements, dimensions, and yield strengths of reinforcing bars.

(3) Capacity of structural members considering construction, cracking, and defect conditions.

(4) Material strengths considering carbonation and aging of concrete, and rust of reinforcing bars.

In the detailed inspection, sampling tests of concrete cylinders extracted from the building, removal of finishing and local destruction of concrete cover shall be conducted for column, beam and wall members.

2.6 Inspection in Case of Design Drawings not Available

In case design drawings of the building are not available, inspections on the structural dimensions, diameters, and arrangements of reinforcing bars shall be conducted on site, which are necessary for seismic evaluation of the building in accordance with the screening level.

Chapter 3 Seismic Index of Structure I_S

3.1 General

(1) The seismic index of structure I_s shall be calculated by Eq. (1) at each story and in each principal horizontal direction of a building. The irregularity index S_D in the first level screening and the time index T may be used commonly for all stories and directions.

$$I_S = E_0 \cdot S_D \cdot T \qquad (1)$$

where:

E_0 = Basic seismic index of structure (defined in 3.2).

S_D = Irregularity index (defined in 3.3).

T = Time index (defined in 3.4).

(2) The seismic index of structure I_S shall be calculated in either the first, the second, or the third level screening procedure.

3.2 Basic Seismic Index of Structure E_0

3.2.1 Calculation of E_0

The basic seismic index of structure E_0 , which is to evaluate the basic seismic performance of the building by assuming other sub indices as unity, shall be calculated for each story and each direction based on the ultimate strength, failure mode and ductility of the building. The basic seismic index of structure E_0 of the i-th story in a n-story building is given as a product of the strength index C defined in 3.2.2 and the ductility index F defined in 3.2.3, differently in the first, the second, or the third level screening procedure. In addition, the story-shear modification factor, which is simply expressed as $\frac{n+1}{n+i}$ in Eqs. (2), (3), (4) and (5), may be changed accounting for the lateral earthquake force distribution along the building height. In this case, the modification factor for overall collapse mechanism given in Eq. (6) shall also be changed consistently.

(1) First level screening procedure

The vertical structural members shall be classified into three categories as listed in Table 1 in the first level screening procedure, where the basic seismic index of structure E_0 shall be calculated based on approximate evaluation of the strength index C, the ductility index F, and the effective strength factor α.

Table 1 Classification of vertical members in the first level screening procedure

Vertical member	Definition
Column	Columns having h_o/D larger than 2
Extremely short column	Columns having h_o/D equal to or less than 2
Wall	Walls including those without boundary columns

Note: h_o : Column clear height (see Fig. 1)

D : Column depth

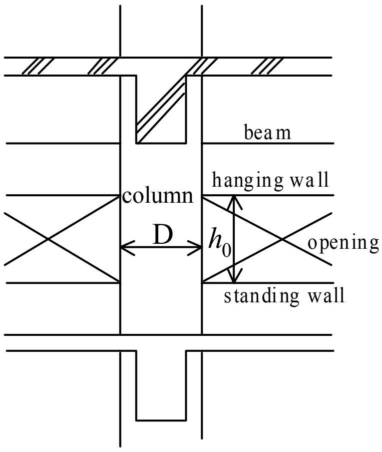


Figure 1 Clear height and depth of column

The basic seismic index of structure E_0 shall be taken as the larger value from Eqs. (2) and (3). Here, the index E_0 shall be taken as the value only from Eq. (3) in case the story consists of extremely short columns judged as the second-class prime elements defined in the item (4). See the translators' note 3.

$$E_0 = \frac{n+1}{n+i}(C_W + \alpha_1 C_C) \cdot F_W \tag{2}$$

$$E_0 = \frac{n+1}{n+i}(C_{SC} + \alpha_2 C_W + \alpha_3 C_C) \cdot F_{SC} \tag{3}$$

where:

n = Number of stories of a building.

i = Number of the story for evaluation, where the first story is numbered as 1 and the top story as n.

C_W = Strength index of the walls, calculated by Eq. (7).

C_C = Strength index of the columns, calculated by Eq. (8), except for the extremely short columns.

C_{SC} = Strength index of the extremely short columns, calculated by Eq. (9).

α_1 = Effective strength factor of the columns at the ultimate deformation of the walls, which may be taken as 0.7. The value should be 1.0 in case of $C_W \fallingdotseq 0$.

α_2 = Effective strength factor of the walls at the ultimate deformation of the extremely short columns, which may be taken as 0.7.

α_3 = Effective strength factor of the columns at the ultimate deformation of the extremely short columns, which may be taken as 0.5.

F_W = Ductility index of the walls (ductility index of columns in case C_W is nearly equal to 0) , which may be taken as 1.0.

F_{SC} = Ductility index of the extremely short columns, which may be taken as 0.8.

(2) Second level screening procedure

The vertical structural members shall be classified into five categories as listed in Table 2 in the second level screening procedure, where the basic seismic index of structure E_0 shall be calculated based on the relations between the cumulative strength index C_T and the ductility index F derived from detailed evaluation of the strength index C, the ductility index F, and the effective strength factor α accounting for the difference in the lateral stiffness of members. The strength index C and the ductility index F shall be evaluated in accordance with the provisions in 3.2.2 and in 3.2.3 respectively.

Table 2 Classification of vertical members based on failure modes in the second level screening procedure

Vertical member	Definition
Shear wall	Walls whose shear failure precede flexural yielding
Flexural wall	Walls whose flexural yielding precede shear failure
Shear column	Columns whose shear failure precede flexural yielding, except for extremely brittle columns
Flexural column	Columns whose flexural yielding precede shear failure
Extremely brittle column	Columns whose h_o/D are equal to or smaller than 2 and shear failure precede flexural yielding

The effective strength factor α may be taken as given in Table 3. The cumulative strength index C_T shall be evaluated as the sum of strength indices C corresponding to representative ductility indices for each story multiplied by the story-shear modification factor $\frac{n+1}{n+i}$. The effective strength factor shall be considered in case the yield deformation of a member is larger than the deformation for calculation of the cumulative strength index C_T, and the strength contribution shall be neglected in case the ductility index of a member is smaller than the deformation for calculation.

The basic seismic index of structure E_0 shall be taken as the larger one from Eqs. (4) and (5). Here, the index E_0 shall be evaluated within the limitation of the minimum ductility index of the second-class prime elements (see the translators' note 3) defined in the item (4) in case the story consists of these elements.

(a) Ductility-dominant basic seismic index of structure (Eq.(4))

For the calculation of E_0 by Eq. (4), vertical members shall be classified by their ductility indices F into three groups or less defined as the first, the second, and the third group in order of the smaller value of the ductility indices. The index F of the first group shall be taken as larger than 1.0 and the index F of the third group shall be less than the ductility index corresponding to the ultimate deformation of the story given in the item (4). Any grouping of members may be adopted so that the index E_0 would be evaluated as maximum. The minimum ductility index of the vertical members should be used in each group.

$$E_0 = \frac{n+1}{n+i}\sqrt{E_1^2 + E_2^2 + E_3^2} \tag{4}$$

where:

$E_1 = C_1 \cdot F_1$.

$E_2 = C_2 \cdot F_2$.

E_3 = $C_3 \cdot F_3$.
C_1 = The strength index C of the first group (with small F index).
C_2 = The strength index C of the second group (with medium F index).
C_3 = The strength index C of the third group (with large F index).
F_1 = The ductility index F of the first group.
F_2 = The ductility index F of the second group.
F_3 = The ductility index F of the third group.

(b) Strength-dominant basic seismic index of structure (Eq. (5))

For the calculation of E_0 by Eq. (5), the ductility index of the first group F_1 shall be selected as the cumulative point of strength, and the contribution of strength indices of only the vertical members with larger ductility indices than that of the first group shall be considered. The index F_1 of the first group shall be less than that corresponding to the ultimate deformation of the story given in the item (4), and may be selected so that the index E_0 by Eq. (5) would be evaluated as maximum. The effective strength factor α in the second and higher groups should be calculated considering the effects of yield deformations and clear heights of vertical members on the relationships between the story shear forces and the drift angles. The values of α given in Table 3 may be used in case no special verification. The minimum effective strength factor of the vertical members should be used in each group.

$$E_0 = \frac{n+1}{n+i}\left(C_1 + \sum_j \alpha_j C_j\right) \cdot F_1 \tag{5}$$

where:

α_j = Effective strength factor in the j-th group at the ultimate deformation R_1 corresponding to the first group (ductility index of F_1), given in Table 3.

Table 3 Effective strength factor

Cumulative point of the first group F_1 =0.8 (Drift angle R_1=R_{500}=1/500)		
	F_1	F_1=0.8
	R_1	R_1=R_{500}
Second and higher groups	Shear (R_{su}=R_{250})	α_S
	Shear (R_{250}< R_{su})	α_S
	Flexural (R_{my}=R_{250})	0.65
	Flexural (R_{250}<R_{my}<R_{150})	α_m
	Flexural (R_{my}=R_{150})	0.51
	Flexural and shear walls	0.65

Table 3 Effective strength factor (continued)

Cumulative point of the first group $F_1 \geq 1.0$ (Drift angle $R_1 \geq R_{250}$=1/250)				
	F_1	F_1=1.0	1.0<F_1<1.27	1.27≤F_1
	R_1	R_{250}	R_{250}<R_1<R_{150}	R_{150}≤R_1
Second and higher groups	Shear (R_{su}=R_{250})	1.0	0.0	0.0
	Shear (R_1<R_{su})	α_S	α_S	0.0
	Flexural (R_{my}<R_1)	1.0	1.0	1.0
	Flexural (R_1<R_{my})	α_m	α_m	1.0
	Flexural (R_{my}=R_{150})	0.72	α_m	1.0

(Note)

α_S = Effective strength factor of a shear column, calculated by $\alpha_S = Q_{(F1)}/Q_{su} = \alpha_m\, Q_{mu}/Q_{su} \leq 1.0$

α_m = Effective strength factor of a flexural column, calculated by $\alpha_m = Q_{(F1)}/Q_{mu}=0.3+0.7\times R_1/R_{my}$

R_{my} = Drift angle at flexural yielding, calculated by Eq. (A1.3-1) in the Supplementary Provisions 1.

R_{su} = Drift angle at shear strength, calculated by Eq. (A1.2-11) in the Supplementary Provisions 1.

$Q_{(F1)}$ = Shear force at the deformation capacity R_1 of a column in the second and higher groups.

Q_{su} = Shear strength of a column in the second and higher groups (3.2.2).

Q_{mu} = Shear force at flexural yielding of a column in the second and higher groups (3.2.2).

(3) Third level screening procedure

As in the similar way to the second level screening procedure, the vertical structural members shall be classified into eight categories as listed in Table 4 in the third level screening procedure. The basic seismic index of structure E_0 shall be calculated based on the relations between the cumulative strength index C_T and the ductility index F derived from detailed evaluation of the strength index C, the ductility index F, and the effective strength factor α accounting for the difference in the lateral stiffness of members. Three types of failure modes of members, namely, columns governed by flexural beams, columns governed by shear beams, and uplift walls should be considered in addition to those given in the second level screening procedure.

The strength and ductility indices of vertical members shall be evaluated based on the strength and ductility of the members governing the structural failure mode, and the strength margin of non-hinge members affecting the failure mode assumed in the evaluation. The basic seismic index of structure E_0, which shall be calculated in the same way as in the second screening procedure, may be modified as given in Eq. (6) only in case a story failure mechanism would surely be prevented so that an overall structural failure mechanism would be formed with flexural yielding of beams, flexural yielding at the wall base, or wall uplifting.

$$E_0^{'} = E_0 \cdot \frac{2}{3} \cdot \frac{2n+1}{n+1} \tag{6}$$

where:

n = Number of stories of a building.

Table 4 Classification of vertical members based on failure modes in the third level screening procedure

Vertical members	Definition
Shear wall Flexural wall Shear column Flexural column Extremely brittee column	Defined in Table 2
column governed by flexural beams	Columns governed by beams whose flexural yielding precedes shear failure
column governed by shear beams	Columns governed by beams whose shear failure precedes flexural yielding
uplift wall	Walls whose uplift (rotation) failure precedes flexural yielding or shear failure

(4) Ultimate state of a structure (for a story)

The ultimate state of a structure is defined for each story in terms of the inter-story deformations or the corresponding ductility indices of the columns when the structure or the story attains to either of the following states due to the failure of the gravity load carrying members (columns) under seismic loading.

(a) A state wherein the columns nearly lose the gravity load carrying capacity due to shear or axial compressive failure. The ultimate state of the structure can be redefined at the larger inter-story deformation in case it is verified that the structure would not collapse even after the shear or axial compressive failure of some columns. The probability that the shear or compressive failure of these columns lead to the structural failure shall be checked by whether these columns are the second-class prime element (see the translators' note 3) or not. The second-class prime element is defined as the member, the gravity axial load of which cannot be sustained not only by itself but also by any other neighborhood members instead after the shear or axial compressive failure occurs under seismic loading. In case the vertical members are the second-class prime elements, it should be judged that the failure of these members leads to structural collapse with high probability.

(b) A state wherein the cumulative strength index C_T decays down to a certain level so that the structure would be unstable in lateral resistance.

(5) Exemptions

In case the eccentricity ratio l_e exceeds 0.15 due to an unbalanced arrangement of walls, etc. in evaluating the irregularity index S_D according to the section 3.3, the basic seismic index of structure E_0 should be taken as the smaller value from the following calculations. The reduction factor for the irregularity index S_D due to the eccentricity may be taken as 0.8 for both cases.

(a) The index E_0 calculated independently for a frame or frames with tributary weight on the side where the seismic drift response would increase due to the effect of eccentricity.

(b) The index E_0 calculated by Eq. (5) on the assumption that the vertical members causing the structural eccentricity are classified into the first group.

3.2.2 Strength index *C*

The methods of calculating the strength index C of vertical members in each story of a building are provided for the first, the second, and the third screening procedure as follows.

(1) First level screening procedure

The strength index C in the first level screening procedure shall be calculated approximately using the cross-sectional areas of walls and columns as follows:

$$C_W = \frac{\tau_{W1} \cdot A_{W1} + \tau_{W2} \cdot A_{W2} + \tau_{W3} \cdot A_{W3}}{\Sigma W} \cdot \beta_c \tag{7}$$

$$C_C = \frac{\tau_C \cdot A_C}{\Sigma W} \cdot \beta_c \tag{8}$$

$$C_{SC} = \frac{\tau_{SC} \cdot A_{SC}}{\Sigma W} \cdot \beta_c \tag{9}$$

$$\begin{aligned} \beta_c &= \frac{F_c}{20} && F_c \le 20 \\ \beta_c &= \sqrt{\frac{F_c}{20}} && F_c > 20 \end{aligned} \tag{10}$$

where:

C_W = Strength index of walls.

C_C = Strength index of columns.

C_{SC} = Strength index of extremely short columns.

τ_{W1} = Average shear stress at the ultimate state of walls with two boundary columns, which may be taken as 3 N/mm^2.

τ_{W2} = Average shear stress at the ultimate state of walls with one column, which may be taken as 2 N/mm^2.

τ_{W3} = Average shear stress at the ultimate state of walls without columns, which may be taken as 1 N/mm^2.

τ_C = Average shear stress at the ultimate state of columns, which may be taken as 1 N/mm^2 or 0.7 N/mm^2 in case h_0/D is larger than 6.

τ_{SC} = Average shear stress at the ultimate state of extremely short columns, which may be taken as 1.5 N/mm^2.

A_{W1} = Total cross-sectional area of walls with two boundary columns in the story and effective to the direction concerned (see Fig. 2) (mm^2).

A_{W2} = Total cross-sectional area of walls with one boundary column in the story and effective to the direction concerned (see Fig. 2) (mm^2).

A_{W3} = Total cross-sectional area of walls without columns in the story and effective to the story concerned (see Fig. 2) (mm^2).

A_C = Total cross-sectional area of columns (mm^2) in the story concerned, where the areas of boundary columns in the walls with one or two boundary columns shall be neglected in calculation.

A_{SC} = Total cross-sectional area of extremely short columns in the story concerned (mm^2).

ΣA_f = Total floor area supported by the story concerned (m^2).

ΣW = Total weight (dead load plus live load for seismic calculation) supported by the story concerned, which may be estimated approximately by assuming the unit floor weight as 12 kN/m^2.

F_c = Compressive strength of concrete (N/mm^2), which may be taken as the specified design concrete strength in case without special inspection, but should not exceed 20 N/mm^2 .

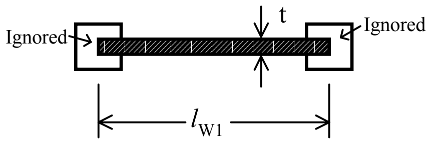
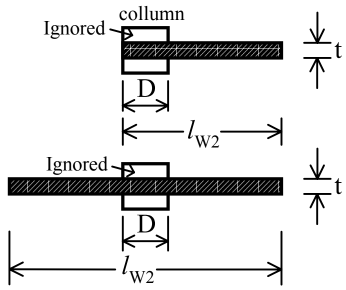
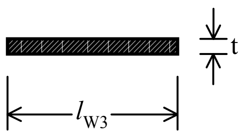
	$A_{W1} = t \times l_{W1}$
	$A_{W2} = t \times l_{W2}$ It should be considered as a column, in case $(l_{W2}-D)$ is less than 450 mm.
	$A_{W3} = t \times l_{W3}$ This wall should be ignored, in case l_{W3} is less than 450mm.

Figure 2 Definition of cross sectional area of wall

(2) Second level screening procedure

(a) Principles

The strength index C in the second level screening procedure shall be calculated from the ultimate lateral load-carrying capacity of vertical members (columns and walls) in principle based on the assumption that the beams are strong enough. The failure modes of the vertical members shall be classified in accordance with Table 2 by comparing the ultimate shear strength Q_{su} and the shear at the ultimate flexural failure Q_{mu}. Published methods, which have reliable accuracy, may be used for the calculation of the ultimate shear strength Q_{su} and the ultimate flexural strength M_u. The inflection heights for calculations of Q_{su} and Q_{mu} should be used as specified in the following item (c) in case no special considerations.

(b) Calculation of ultimate strengths of members

The formulas or methods estimating the lower bound of the actual strengths should be used in calculation of the ultimate shear strength Q_{su} while those estimating the average should be used in calculation of the ultimate flexural strength $M_{u.}$ The formulas given in the Supplementary Provisions may be used in case no special considerations. Material strength may be taken as follows in calculation of the ultimate member strengths: specified design strength of concrete F_c as compressive strength of concrete; 294 N/mm^2 as the yield strength of round reinforcing bars; and nominal yield strength plus 49 N/mm^2 as the yield strength of deformed reinforcing bars. The values estimated from material test on samples should be used in case an extreme aging is observed in the preliminary inspection or material test data are available in the detailed inspection.

(c) Identification of failure modes and calculation of ultimate lateral load-carrying capacity

The shear force Q_{mu} ($=M_u/h$) associated with the ultimate flexural strength M_u at the base of a vertical member and the ultimate shear strength Q_{su} shall be calculated using the following inflection height ($=M/Q$) in case no special considerations. The smaller value between Q_{mu} and Q_{su} shall be defined as the ultimate lateral load-carrying capacity of the vertical member Q_u.

(i) For columns: $h_{C0}=h_0/2$, where, h_0 is the clear height.

$h_{C0}=h_0 M_B/(M_T+M_B)$, in case the ultimate flexural strengths are different at the two ends, where, M_T and M_B are the ultimate flexural strengths at the top and bottom ends, respectively.

(ii) For walls with two boundary columns: $h_{W0}=h_W/2$, where, h_W is the height from the floor level concerned to the top of the wall. $h_{W0}=h_W$ in case of the wall at the top story and the wall in one-story building.

(iii) For columns with wing walls, or walls with a column:

$$
\begin{aligned}
h_{CW0} &= h_{C0} + \left(h_{W0} - h_{C0}\right)\cdot\frac{L_W}{L} && \left(0 < L_W < L - D_C\right) \\
h_{CW0} &= h_{W0} && \left(L_W \geq L - D_C\right)
\end{aligned}
\qquad (11)
$$

where:

L_W = Length of the wing wall (total length of the wing walls in case they locate at both sides of a column).

D_C = Column depth.

L = Standard or averaged length of spans in the direction concerned, which may be taken as the length of the span on the side with a longer wing wall.

h_{C0} = Inflection height calculated as columns as given in the item (i).

h_{W0} = Inflection height calculated as walls with two boundary columns as given in the item (ii).

Eq. (11) may also be used in calculation of the inflection height for multi-story walls without boundary columns, in which case the length of the wing wall shall be calculated as $L_W=L' - 2D_C$ $(L_W\geq 0)$, where L' is the wall length and D_C is the typical column depth.

(d) Calculation of strength index

The strength index C in the second level screening procedure shall be calculated by the following equation:

$$C = \frac{Q_u}{\sum W} \tag{12}$$

where:

Q_u = Ultimate lateral load-carrying capacity of the vertical members in the story concerned.

ΣW = The weight of the building including live load for seismic calculation supported by the story concerned.

(3) Third level screening procedure

(a) Principles

The strength index C in the third level screening procedure shall be calculated with the following principles:

(i) The ultimate flexural strength M_u and the ultimate shear strength Q_{su} of columns, walls, and beams should be calculated by the methods specified in the item (b).

(ii) Using the results above, the failure mode of each member and the nodal ultimate moment should be calculated by the methods specified in the item (c). The failure modes of vertical members and the ultimate lateral load-carrying capacity should be calculated by so-called nodal moment distribution method. They should be calculated by limit analysis in case of a frame with walls.

(iii) In the same way as specified for the second level screening procedure, vertical members should be classified into three groups or less according to their failure modes and ductility indices as listed in Table 4, and the strength index of each group should be calculated.

(iv) The strength in bond failure or the shear strengths of beam-column joints and their effects may be considered if necessary.

(b) Calculation of ultimate strengths of members

The ultimate flexural strength and the ultimate shear strength of columns and walls should be calculated in accordance with the methods specified for the second level screening procedure. Earthquake-induced axial forces should appropriately be evaluated and considered in the third level calculation.

The ultimate flexural strength and the ultimate shear strength of beams may be calculated by the formulas given in the Supplementary Provisions in case no special analyses. The effects of slab reinforcement and the multi-layered main bars in the beam should be considered in the calculation.

(c) Identification of failure modes and calculation of ultimate lateral load-carrying capacity

According as the structural system of the building concerned, the structure should be modeled into appropriate frames and members, the failure modes and the ultimate lateral load-carrying capacity of the vertical members should be evaluated with the so-called nodal moment distribution method. In case of a frame with walls, the ultimate lateral load-carrying capacity should be calculated by the virtual work analysis method assuming a failure mechanism of structure and a lateral force distribution along the height of the whole building or the frame.

(d) Calculation of strength index

The strength index C of the vertical members shall be calculated by the same methods as specified for the second level screening procedure.

3.2.3 Ductility index *F*

(1) Basic principles in calculation of ductility index *F*

The ductility index of a vertical member shall be evaluated in consideration of the screening level, failure mode and member deformation capacity, and response to earthquakes. A standard value of the ductility index shall be defined as the ductility index of the shear wall, in which shear failure precedes other failure modes. The ductility indices of the other members shall be determined as a relative value to this standard value.

The ductility index of the member shall be evaluated by the methods specified as in the following items (2)-(4), according to the screening level and the classification by the failure mode of the member (as shown in Table 2 or Table 4 in 3.2.1), in case no special investigations.

The ductility index by the 1990 version of the standard is shown in the translators' note 4.

(2) First level screening procedure

The ductility index of a vertical member in the first level screening procedure should be selected as listed in Table 5 according to the classification of the member.

Table 5 Ductility index in the first level screening

Vertical member	Ductility index F
Column ($h_0/D>2$)	1.0
Extremely short column ($h_0/D\leq 2$)	0.8
Wall	1.0

(3) Second level screening procedure

The ductility index of a vertical member in the second level screening procedure shall be calculated as follows according to the classification of the member listed in Table 2. The item (f) may be applied to the columns with wing wall(s).

(a) Shear wall

The ductility index of a shear wall should be defined as 1.0.

(b) Flexural wall

The ductility index of a flexural wall should be calculated by Eq. (13) based on the margin of the shear strength to the shear force at the flexural strength of the wall.

If ${}_WQ_{su}/{}_WQ_{mu} = 1.0$ then F=1.0 (13)

If ${}_WQ_{su}/{}_WQ_{mu} \geq 1.3$ then F=2.0.

(in case of wall with a column in item (f)(i), F=1.5)

If $1.0 < {}_WQ_{su}/{}_WQ_{mu} < 1.3$ then F should be calculated by interpolation.

where:

${}_WQ_{su}$ = Ultimate shear strength of the wall, calculated by Eq. (A2.1-2) in the Supplementary Provisions.

${}_WQ_{mu}$ = Shear force at the flexural strength of the wall, calculated according to the item 3.2.2(2)(c).

(c) Shear column

The ductility index of a shear column should be calculated by Eq. (14) based on the story drift angle at the ultimate deformation capacity in shear failure of the column.

$$F = 1.0 + 0.27\frac{R_{su} - R_{250}}{R_y - R_{250}} \tag{14}$$

where:

R_y = Yield deformation in terms of inter-story drift angle, which in principle shall be taken as R_y=1/150.

R_{250} = Standard inter-story drift angle (corresponding to the ductility index of the shear wall), R_{250}=1/250.

R_{su} = Inter-story drift angle at the ultimate deformation capacity in shear failure of the column member, calculated by Eq. (A1.2-11) in the Supplementary Provisions 1.2(4).

(d) Flexural column

The ductility index of a flexural column should be calculated by Eq. (15) or (16) based on the inter-story drift angle at the ultimate deformation capacity in flexural failure of the column.

(i) In case $R_{mn} < R_y$

$$F = 1.0 + 0.27\frac{R_{mu} - R_{250}}{R_y - R_{250}} \tag{15}$$

(ii) In case $R_{mn} \geq R_y$

$$F = \frac{\sqrt{2R_{mu}/R_y - 1}}{0.75 \cdot (1 + 0.05R_{mu}/R_y)} \leq 3.2 \tag{16}$$

where:

R_y = Yield deformation in terms of inter-story drift angle, which in principle shall be taken as R_y=1/150.

R_{250} = Standard inter-story drift angle (corresponding to the ductility index of the shear wall), R_{250} = 1/250.

R_{mu} = Inter-story drift angle at the ultimate deformation capacity in flexural failure of the column member, calculated by Eq. (A1.2-1) in the Supplementary Provisions 1.2(1).

(e) Extremely brittle column

The ductility index of an extremely brittle column should be selected as 0.8.

(f) Column with wing wall(s) or wall with a column

The ductility index of a column monolithically attached with one wing wall or with two wing walls should be selected based on the following three groups according to the classification specified in the Supplementary Provisions 3.

(i) Wall (Wall with a column)

The index shall be calculated according to the items (a) and (b).

(ii) Column with wing wall(s)

The index shall be calculated as follows:

$h_o / H_o > 0.75$: F=1.0. The index may be selected according to the section (b) in case flexural yielding precede shear failure.

$h_o / H_o \leq 0.75$: F=0.8. The index may be selected as 1.0 in case flexural yielding precede shear failure.

where:

h_0 = Clear height of the column.

H_0 = Standard height of the column from the bottom of the upper floor beam to the surface of the lower floor slab.

(iii) Column

The index shall be calculated according to the above items (c)-(e). However, the ductility index should be calculated by reducing the plastic rotation angle ${}_{C}R_{mp}$ to 0.5 times as specified in the Supplementary Provisions 1.2(2), and should not exceed 1/150, in case of a flexural column with wing walls.

(4) Third level screening procedure

The ductility index of a vertical member in the third level screening procedure should be selected according to the items (3)(a) and (c)-(f), and according as the classification of vertical members listed in Table 4.

(a) Ductility index of a wall

The ductility index of a wall in consideration of the uplift or rotating failure mode at the foundation should be calculated by Eq. (17) or Eq. (18).

(i) In case ${}_{w}Q_{mu} / \gamma \cdot {}_{w}Q_{ru} \geqq 1.0$ (uplift wall or shear wall),

$$ {}_{w}F = \min\{F_{sr}, F_{mr}\} \tag{17}$$

(ii) In case ${}_{w}Q_{mu} / \gamma \cdot {}_{w}Q_{ru} < 1.0$ (flexural wall or shear wall),

$$ {}_{w}F = F_{sm} \tag{18} $$

The index F_{sr} should be calculated by Eq. (19) considering the margin of the shear strength to the uplift strength:

If ${}_{w}Q_{su} / \gamma \cdot {}_{w}Q_{ru} \leq 1.0$ then F_{sr}=1.0 (19)
If ${}_{w}Q_{su} / \gamma \cdot {}_{w}Q_{ru} \geq 1.6$ then F_{sr}=3.0 for walls with two boundary columns However, F_{sr}=2.0 for others
If $1.0 < {}_{w}Q_{su} / \gamma \cdot {}_{w}Q_{ru} < 1.6$ then F_{sr} should be calculated by linear interpolation between above two.

The index F_{mr} should be calculated by Eq. (20) considering the margin of the flexural strength to the uplift strength:

If ${}_{w}Q_{mu} / \gamma \cdot {}_{w}Q_{ru} \leq 1.0$ then F_{mr}=2.0. F_{mr}=1.5 for walls with two boundary columns. (20)
If ${}_{w}Q_{mu} / \gamma \cdot {}_{w}Q_{ru} \geq 16/13$ then F_{mr}=3.0. F_{mr}=2.0 for walls with two boundary columns
If $1.0 < {}_{w}Q_{mu} / \gamma \cdot {}_{w}Q_{ru} < 16/13$ then F_{mr} should be calculated by linear interpolation between above two.

The index F_{sm} is the ductility index of the shear wall or the flexural wall with a fixed base condition by Eq. (13).

where:

${}_{w}Q_{su}$ = Ultimate shear strength of the wall calculated by Eq. (A2.1-2) in the Supplementary Provisions 2.

${}_{w}Q_{ru}$ = Uplift strength of the wall in terms of lateral shear considering the effects of the boundary and transverse beams, the transverse walls, and the tensile resistance of the foundation in the calculation.

${}_{w}Q_{mu}$ = Flexural strength of the wall in terms of lateral shear calculated in accordance with the item 3.2.2(2)(c). The shear force may be calculated from a precise analysis in case an upper bound of the shear at the flexural failure mechanism can be estimated considering three dimensional effects.

γ = Factor on the precision in calculation of the uplift strength of the wall, taken as 1.0 to 1.2.

(b) Ductility index of uplift wall or flexural wall with boundary and transverse beams

The ductility index of a wall with boundary and/or transverse beams, F shall be calculated by Eq. (21) using the ductility index of the wall ${}_{w}F$ specified in the item (a) and the ductility indices of the boundary beams ${}_{b}F$ in the item (d).

$$ F = {}_{w}q \cdot {}_{w}F + \Sigma({}_{b}q \cdot {}_{b}F) \tag{21} $$

where:

$$ {}_{w}q = \frac{{}_{w}M}{{}_{w}M + \Sigma_{b}M} . $$

$$ {}_{b}q = \frac{{}_{b}M}{{}_{w}M + \Sigma_{b}M} . $$

$_wM$ = Moment resistance of the wall at the level of the story concerned.

$_bM$ = Contribution of the boundary beam to the overturning moment resistance of the wall at the level of story concerned.

$\sum$ = Summation for all boundary beams connected to the wall and being effective to the overturning moment resistance of the wall.

The strengths of the transverse beams should also be considered as in the same way as above in case these beams affect the strength and ductility of the wall.

(c) Columns governed by flexural/shear beams

(i) Ductility index of a column governed by beams

The ductility index of a column governed by beams should be calculated by Eq. (22) using the ductility indices of the beams connected to the top and bottom ends of the column.

$$F = \sum ({}_nq_i \cdot {}_nF_i) \tag{22}$$

where:

$${}_nq_i = \frac{{}_nM_{ui}}{\sum {}_nM_{ui}}.$$

${}_nF_i$ = Ductility index of the node at the top or the bottom of the column, calculated according to the item (ii).

${}_nM_{ui}$ = Nodal moment at the top or the bottom of the column at the failure mechanism.

$\sum$ = Summation for the top or bottom ends of the column.

(ii) Ductility index of nodes

The ductility index of the node at the top or the bottom end of a column ${}_nF_i$ should be calculated by Eq. (24) according to the margin of the nodal moments of the column strengths to the beam strengths:

If $\sum_c M_{ui} / \sum_b M_{ui} \geq 1.4$ then ${}_nF_i = {}_nF_b$. (24)

If $\sum_c M_{ui} / \sum_b M_{ui} \leqq 1.0$ then ${}_nF_i = {}_nF_c$.

If $1.0 < \sum_c M_{ui} / \sum_b M_{ui} < 1.4$ then ${}_nF_i$ should be calculated by interpolation between above two,

where:

$\sum_c M_{ui}$ = Sum of the nodal moments at the ultimate strengths of the columns in the upper and the lower stories.

$\sum_b M_{ui}$ = Sum of the nodal moments at the ultimate strengths of the beams on the left and the right sides.

${}_nF_c$ = Ductility index of the column above and below the node, which shall be calculated in accordance with the provisions in the items 3.2.3(3)(c)-(f).

${}_nF_b$ = Ductility index of the node determined from the beams calculated according to the item (iii).

(iii) Ductility index of node determined from beams

The ductility index of the node determined from the beams ${}_nF_b$ should be calculated by Eq. (25), representing the weighed average of the ductility indices of the beams at the left and right sides of the node.

$$ {}_nF_b = \Sigma({}_bq_i \cdot {}_bF_i) \tag{25}$$

where:

$$ {}_bq_i = \frac{{}_bM_{ui}}{\Sigma_b M_{ui}} .$$

${}_bF_i$ = Ductility index of the beam on the left and the right sides of the node calculated according to the item (d).

${}_bM_{ui}$ = Nodal moment at the ultimate strengths of the beams on the left and the right sides of the node.

Σ = Summation for the beams on the left and the right sides of the node.

(d) Ductility index of beam

The ductility index of a beam should be calculated by Eq. (26) or (27).

(i) Beams in general (except for boundary beams of flexural or uplift walls)

If ${}_bQ_{su}/{}_bQ_{mu} \le 0.9$ then ${}_bF = 1.5$ (26)

If ${}_bQ_{su}/{}_bQ_{mu} \ge 1.2$ then ${}_bF = 3.5$

If $0.9 < {}_bQ_{su}/{}_bQ_{mu} < 1.2$ then ${}_bF$ should be calculated by interpolation between above two

where:

${}_bQ_{su}$ = Shear strength of the beam, which shall be calculated in principle by Eq. (A4-4a) in the Supplementary Provisions 4.

${}_bQ_{mu}$ = Shear force at the flexural failure of the beam, considering the effect of the shear force Q_0 due to gravity load.

(ii) Boundary beams of flexural or uplift walls

If ${}_bQ_{su}/{}_bQ_{mu} \le 0.9$ then ${}_bF = 1.5$ (27)

If ${}_bQ_{su}/{}_bQ_{mu} \ge 1.3$ then ${}_bF = 3.5$

If $0.9 < {}_bQ_{su}/{}_bQ_{mu} < 1.3$ then ${}_bF$ should be calculated by interpolation between above

(iii) Beams with spandrel wall(s)

The ductility index of a beam with spandrel wall(s) should be selected as 1.5, in principle. The index may be calculated in detail according to the provisions for the ductility index of the column with wing wall(s).

3.3 Irregularity Index S_D

3.3.1 General

The irregularity index S_D is to modify the basic seismic index of structure E_0 by quantifying the effects of the shape complexity and the stiffness unbalance distribution, and the like on the seismic performance of a structure with engineering judgment.

Methods of calculating the irregularity index for the first or the second level screening procedures should be selected respectively, considering the simplification and accuracy of calculation and the effect of index. In addition, it is recommended that the irregularity index should be calculated by the method specified in the Appendix 3 (not translated), in case the possibility of the story failure needs carefully be examined in the medium- and high-rise buildings.

3.3.2 Items to be considered

Items to be considered are listed below:

(1) First level screening

(a) items related to floor plan (to the structural integrity of floor plan)
regularity, aspect ratio, narrow part, expansion joint, well-style hall (size and location)
(b) items related to sectional plan (to the structural integrity of sectional plan)
existence of basement, uniformity of story height, existence of pilotis

(2) Second level screening

Following items shall be added to the items for the first level screening.

(a) items related to floor plan
distance between centroids of gravity and center of lateral stiffness
(b) items related to sectional plan
ratio of stiffness of lower story to upper story

3.3.3 Calculation Procedure

The irregularity index shall be calculated as the geometric product of degree of incidence q_i calculated as in Eqs. (28) and (29), which are derived from the grade index G_i and the range adjustment factor R_i for the screening level. The factors R_{1i} or R_{2i} should be used for the first or the second level screening respectively, according to the classification given in Table 6.

(1) Calculation method for index

(a) First level screening

$$S_{D1} = q_{1a} \times q_{1b} \times \cdots\cdots \times q_{1j} \tag{28}$$

where:

$$q_{1i} = \lfloor 1-(1-G_i)\times R_{1j} \rfloor \quad \cdots\cdots \quad i = a,b,c,d,e,f,i,j$$

$$q_{1i} = \lfloor 1.2-(1-G_i)\times R_{1j} \rfloor \quad \cdots\cdots \quad i = h$$

(b) Second level screening

$$S_{D2} = q_{2a} \times q_{2b} \times \cdots\cdots \times q_{2n} \tag{29}$$

where:

$$q_{2i} = \lfloor 1-(1-G_i)\times R_{2j} \rfloor \quad \cdots\cdots \quad i = a,b,c,d,e,f,i,j,l,n$$

$$q_{2i} = \lfloor 1.2 - (1 - G_i) \times R_{2j} \rfloor \quad \cdots\cdots \quad i = h$$

(c) Third level screening

The irregularity index for the third level screening shall be used as the same as for the second level screening.

$$S_{D3} = S_{D2}$$

(2) Classification of the items

The classification of the items and the corresponding values for G_i and R_i listed in Table 6 shall be used.

(3) Alternative evaluation using stiffness ratio and eccentricity ratio

The G'_ℓ and G'_n may be calculated as follows for alternative evaluation of the irregularity indices G_ℓ and G_n in Table 6, using the amplification factors (F_e and F_s) for the required lateral load-carrying capacity based on the precise calculation of the stiffness ratio (R_s) and the eccentricity ratio (R_e) as specified in the Enforcement Order of the Japanese Building Standard Law.

$$G'_l = 1/F_e \quad \text{(where it may be taken as } G_a = 1.0\text{)} \quad 1.0 \le F_e \le 1.5$$

$$G'_n = 1/F_s \quad \text{(where it may be taken as } G_i = 1.0 \text{ and } G_j = 1.0\text{)} \quad 1.0 \le F_s \le 2.0$$

Table 6 Classification of items and G, R-values

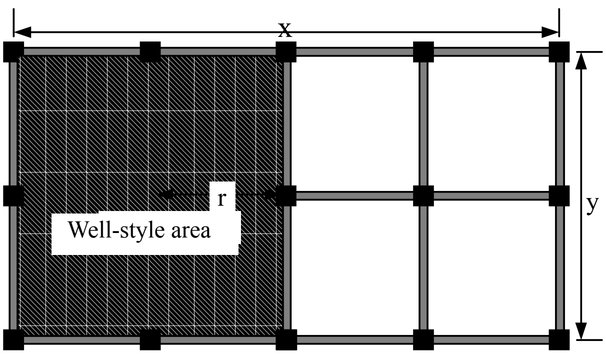
			Gi (Grade)			*R* (adjustment fator)	
			1.0	0.9	0.8	*R1i*	*R2i*
Horizontal balance	a	Regularity	Regular a1	Nearly regular a2	Irregular a3	1.0	0.5
	b	Aspect ratio of plan	b≦5	5＜b≦8	8＜b	0.5	0.25
	c	Narrow part	0.8≦c	0.5≦c＜0.8	c＜0.5	0.5	0.25
	d	Expansion joint *1	1/100≦d	1/200≦d＜ 1/100	D＜1/200	0.5	0.25
	e	Well-style area	e≦0.1	5＜e≦8	0.3＜e	0.5	0.25
	f	Eccentric well-style area*2	f_1≦0.4 & f_2≦0.1	f_1≦0.4 & 0.1＜f_2≦0.3	0.4＜f_1 or 0.3＜f_2	0.25	0
	g						
Elevation balance	h	Underground floor	1.0≦h	0.5≦h＜ 1 .0	h＜0.5	0.5	0.5
	i	Story height uniformity	0.8≦I	0.7≦I＜0.8	I＜0.7	0.5	0.25
	j	Soft story	No soft story	Soft story	Eccentric soft story	1.0	1.0
	k						
Eccentricity	l	Eccentricity*3	1≦0.1	0.1＜*l*≦0.15	0.15＜*l*		1.0
	m						1.0
Stiffness	n	(Stiffness/mass)Ratio of above and below stories	n≦1.3	1.3＜n≦1.7	1.7＜n		1.0
	o						1.0

1. Objects of the application: Items (*a*) to (*j*) should be checked at each story and the minimum value should be applied to all the stories. Items (*l*) and (*n*) should be checked at each story and in each direction.
2. In case the zoning method is applied, the S_D index should be checked for the whole building as well as for each zone.
3. The details of the applications should be referred to the "Technical Manual for Seismic Evaluation and for Seismic Retrofit of Existing RC Buildings, 2001.

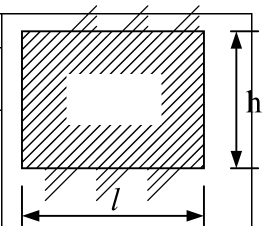
*1 For the building with zones connected by expansion joints, the zoning method shall be applied, by which each zone should be checked separately.

*2 For the symbols in calculation of the item (f), the right side figure should be referred to.

*3 The value of α in calculation of the item (l) should be adopted as the table below based on the ratio of wall height h and wall length *l*.

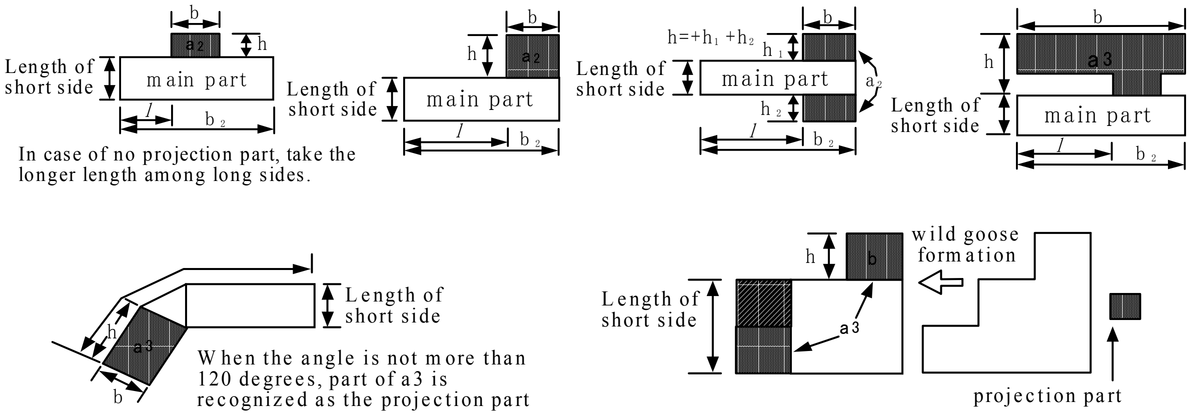


Aspect ratio of the wall h / *l*	α	
	Wall in the frame line	Wall outside of the frame line
3.0≦h / *l*	1.0	0.3
2.0≦h / *l* <3.0	1.5	0.5
1.0≦h / *l* <2.0	2.5	0.8
h / *l* <1.0	3.5	1.2



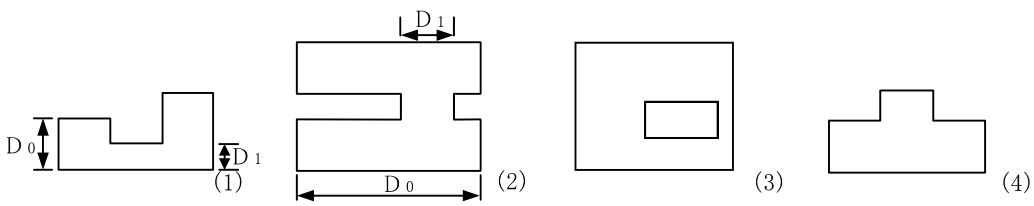
Remarks

a_1: Structural balance is good, and the area of a projection part is not more than 10% of the floor area.
a_2: Structural balance is worse than a_1, or the area of a projection part is not more than 30% of the floor area with L, T or U shaped plan.
a_3: Structural balance is worse than a_2, or the area of a projection part is larger than 30% of the floor area with L, T or U shaped plan.
If the aspect ratio (h/b) of the projection part is less than 1/2, it may not be accounted in this item. The projection part should be defined as the smaller part, while the larger rest as the main part.



b: b=(length of the long side / length of the short side). In case that the plan is not rectangular, the length of the long side may be taken ignoring the projection part when the area of the projection is less than 10% of the floor area, while otherwise, it should be taken as the longer value of $b_1=2l$ and b_2 shown in above figure. In case that the plan has "⌒" shape and no projection part, the length of the longest side should take as the length of the long side. In case of a wild goose formation plan, the length of the short side should be defined from the equivalent rectangular area with the same length of the long side.

c: $c=D_1/D_0$. It should be regarded that the buildings in the figures (1) and (2) below have narrow parts, while those in the figures (3) and (4) have no narrow parts. In case of the figure (2), the reduction factors both by the structural balance and the narrow part shall be evaluated and the only worse factor may be adopted in evaluation.



d: d=(the clear width of the expansion joint / the height from the base to the expansion joint).

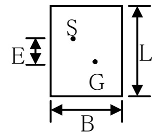
e: e=(well-style area / total floor area). The well-style area is the room or the space stretching over two stories or more. However, if it is surrounded by RC walls, it may not be regarded as the well-style area.

f: f_1= (the distance between the center of the floor area and the center of the well-style area / the length of the short side of the building) = r/y,
f_2= (the distance between the center of the floor area and the center of the well-style area / the length of the long side of the building) = r/x, where the symbols r, x, y are defined in the figure *2.

h: h= (area of the basement / area of the building).

i: i=(the height of above story / the height of the story concerned). In case of the top story, the height of the story below is take instead of above story in the equation.

j: In case that the building has the pilotis columns or the columns supporting the wall above and these columns are located eccentrically, it should be regarded as the eccentric soft story. An moment resisting frame without wall is not included. The eccentric location of the soft story may be judged in such a way that the deformation of the soft story would be larger due to the eccentricity. It may not be regarded as the eccentric soft story and taken as the grade of 0.9, in such case that the deformation of the soft story would not be larger because of the constraint of the adjacent walls.



l: $l = E/\sqrt{B^2 + L^2}$. S: the center of gravity, G = the center of rigidity, where lateral stiffness of each frame is calculated as (the summation of the column area + the wall area x α). The value of α is given as *3 above.

n: n=(the ratio of the stiffness to the weight of the story above) / (the ratio of the stiffness to the weight of the story concerned) x β. β =(N-1)/N, where, N is the number of floors sustained by the story concerned, the weight of a story is the weight of the building sustained by the story concerned, and, the story stiffness shall be calculated as {the sum of column area +the sum of (wall area x α)}/ (the story height). In case of the top story, the story above is taken as the story below in the equation, and β =2.0. In case of intermediate stories, the story above is taken as the story below and the ratio is calculated in the same way, and the larger value shall be taken.

3.4 Time Index *T*

3.4.1 General

The time index *T* evaluates the effects of the structural defects such as cracking, deflection, aging, and the like, on the seismic performance of a structure. Inspection should be carried out, according to Chapter 2 Building Inspection. The time index *T* for the seismic index of structure *Is* by the first, second, and third level screening should be calculated based on the results of three level inspections, that are the first, second, and detailed inspection, respectively.

3.4.2 First level screening procedure

The time index *T* for the first level screening should be determined based on the first level inspection results listed in Table 7. The minimum *T* value at the column [C] in the table should be taken as the time index *T* for the first level screening.

Table 7 Time index *T* by the first level inspection

<table>
<tr><th>[A]
Item to be checked</th><th>[B]
Degree</th><th>[C]
T value
(check circle at relevant degree)</th><th>[D]
Item to be checked for the second level inspection</th></tr>
<tr><td rowspan="4">Deflection</td><td>Tilting of a building or obvious uneven settlement is observed</td><td>0.7</td><td rowspan="4">Structural cracking and deflection</td></tr>
<tr><td>Landfill site or former rice field</td><td>0.9</td></tr>
<tr><td>Deflection of beam or column is observed visually</td><td>0.9</td></tr>
<tr><td>No correspondence to the foregoing</td><td>1</td></tr>
<tr><td rowspan="5">Cracking in walls and columns</td><td>Rain leak with rust of reinforcing bar is observed</td><td>0.8</td><td rowspan="5">Structural cracking and deflection</td></tr>
<tr><td>Inclined cracking in columns is obviously observed</td><td>0.9</td></tr>
<tr><td>Countless cracking is observed in external wall</td><td>0.9</td></tr>
<tr><td>Rain leak without rust of reinforcing bar is observed</td><td>0.9</td></tr>
<tr><td>No correspondence to the foregoing</td><td>1</td></tr>
<tr><td rowspan="3">Fire experience</td><td>Trace</td><td>0.7</td><td rowspan="3">Structural cracking and deflection
Deterioration and aging</td></tr>
<tr><td>Experience but traceless</td><td>0.8</td></tr>
<tr><td>No experience</td><td>1</td></tr>
<tr><td rowspan="2">Occupation</td><td>Chemical has been used</td><td>0.8</td><td rowspan="2">Deterioration and aging</td></tr>
<tr><td>No correspondence to the foregoing</td><td>1</td></tr>
<tr><td rowspan="3">Age of building</td><td>30 years or older</td><td>0.8</td><td rowspan="3">Deterioration and aging</td></tr>
<tr><td>20 years or older</td><td>0.9</td></tr>
<tr><td>19 years or less</td><td>1</td></tr>
<tr><td rowspan="3">Finishing condition</td><td>Significant spalling of external finishing due to aging is observed</td><td>0.9</td><td rowspan="3">Deterioration and aging</td></tr>
<tr><td>Significant spalling and deterioration of internal finishing is observed</td><td>0.9</td></tr>
<tr><td>No problem</td><td>1</td></tr>
</table>

3.4.3 Second level screening procedure

The time index T for the second level screening shall be calculated by Eq. (30) based on the second level inspection results listed in Table 8.

$$T = (T_1 + T_2 + T_3 \cdots\cdots + T_N) / N \tag{30}$$

where:

$T_i = (1 - p_1) \times (1 - p_2)$

T_i = Time index for the inspected story i.

N = Number of the inspected stories.

p_1 = Sum of the mark-down in Table 8 by the structural cracking and deflection for the inspected story. It may be taken as 0, in case the inspection is not necessary.

p_2 = Sum of the mark-down in Table 8 by the deterioration and aging for the inspected story. It may be taken as 0, in case the inspection is not necessary.

3.4.4 Third level screening procedure

The time index T for the third level screening should be the same as for the second level screening in principle. The calculated time index may be modified, in case the strength index and the ductility index are calculated based on the detailed inspection.

Table 8 Evaluation of time index by the second level inspection (-story) for the second level screening

Item / Degree / Portion / Range		Structural cracking and deflection			Deterioration and aging		
		a	b	c	a	b	c
		1. Cracking caused by uneven settlement. 2. Shear or inclined cracking in beams, walls, and/or columns, observed evidently.	1. Deflection of a slab and/or beam, affecting on the function of non-structural element. 2. Same as left but not visible from some distance. 3. Same as above but can be observed from some distance.	1. Minute structural cracking not corresponding to the items a or b. 2. Deflection of a slab and/or beam, not corresponding to the item a or b.	1. Cracking by concrete expansion due to the rust of reinforcing bar. 2. Rust of reinforcing bar. 3. Cracking caused by a fire disaster. 4. Deterioration of concrete caused by chemicals.	1. Seep of the rust of reinforcing bar due to rain water or water leak. 2. Neutralization to the depth of reinforcing bar or equivalent aging. 3. Spalling off of finishing materials.	1. Remarkable blemish of concrete due to rain water, water leak, and chemicals. 2. Deterioration or slight spalling off of a finishing material.
I Slab including sub-beam	1) 1/3 or more of total floor	0.017	0.005	0.001	0.017	0.005	0.001
	2) 1/3~1/9	0.006	0.002	0	0.006	0.002	0
	3) 1/9 or less	0.002	0.001	0	0.002	0.001	0
	4) 0 #	0	0	0	0	0	0
II Beam	1) 1/3 or more of total number of members for each direction	0.05	0.015	0.004	0.05	0.015	0.004
	2) 1/3~1/9	0.017	0.005	0.001	0.017	0.005	0.001
	3) 1/9 or less	0.006	0.002	0	0.006	0.002	0
	4) 0 #	0	0	0	0	0	0
III Wall & Column	1) 1/3 or more of total number of members	0.15	0.045	0.011	0.15	0.045	0.011
	2) 1/3~1/9	0.05	0.015	0.004	0.05	0.015	0.004
	3) 1/9 or less	0.017	0.005	0.001	0.017	0.005	0.001
	4) 0 #	0	0	0	0	0	0
Mark-down	Subtotal						
Total	Ground total	P1			P2		

Remark(#): The item 4) may be adopted in case where there are no areas or members with aging defect, and the maintenance condition of the building could be recognized as very good.

Chapter 4 Seismic Index of Non-Structural Elements I_N

4.1 Basic Principles

Seismic index of non-structural elements I_N is to judge the safety of human lives or the secure of evacuation routes against the fall-down or the spall-off of non-structural elements, especially external walls.

The first, the second, and the third level screening procedures are provided, either by which the seismic index of non-structural elements I_N is to be calculated for each wall in each story.

4.2 First Level Screening Procedure

4.2.1 General

The seismic index of non-structural elements I_N for the first level screening shall be calculated for each wall in each story by Eq. (31).

$$I_N = 1 - B \cdot H \tag{31}$$

where:

B = Construction index.

H = Human risk index.

In evaluation by Eq. (31), the values of B and H shall be adopted for the external wall by the most vulnerable construction method, that is, which gives the maximum value of *B,* among the walls concerned.

4.2.2 Construction index *B*

The construction index B shall be calculated from conformability index f and deterioration index t by Eq. (32).

$$B = f + (1 - f)t \tag{32}$$

(1) Conformability index *f*

The conformability index f shall be determined in combination of ductility grade of the primary structure g_S and ductility grade of non-structural elements g_N as given in Table 9. The values of g_S and g_N shall be graded according to Tables 10 and 11, respectively.

Table 9 Conformability index *f*

Non-structural elements \ Primary structure		rigid ← g_S → flexible: I	II
rigid ↑ g_N ↓ flexible	I	0.5	1.0
	II	0	0.5

Table 10 Ductility grade of primary structure g_S

g_S		Structural characteristics of primary structure
rigid ⇕ flexible	I	Structure with limited ductility, such as with many short columns
	II	Ductile structure, such as with few walls.

Table 11 Ductility grade of non-structural elements g_N

g_N		Construction method of non-structural elements
rigid ⇕ flexible	I	Non-structural elements with limited deformation capacity members, such as, concrete block, glass block, fixed window, stone finishing, tile finishing, mortar finishing, ALC panel, and the like
	II	Non-structural elements with large deformation capacity members, such as, metal / PC curtain wall, movable sash, finishing paint, tile pre-fixed form, concrete finishing, and the like.

(2) Deterioration index *t*

The deterioration index *t* shall be selected as listed in Table 12, which is based on aging and past damages.

Table 12 Deterioration index *t*

Past damage	*t*
damaged, or unknown	1.0
no	0.5

4.2.3 Human risk index *H*

The human risk index *H* shall be selected as listed in Table 13, which is based on the condition of usage below the external wall and the existence of guard such as the eaves, set back and the like.

Table 13 Human risk index *H*

Condition below the external wall \ Guard	No	Yes
passage way, square	1.0	0.3
others	0.5	0.1

4.3 Second Level Screening Procedure

4.3.1 General

The seismic index of non-structural elements I_N for the second level screening shall be calculated for each wall in each story by Eq. (33).

$$I_N = 1 - \frac{\sum_j B_j \cdot W_j \cdot H_j \cdot L_j}{\sum_j L_j} \tag{33}$$

where:

B_j = Construction index.

W_j = Area index.

H_j = Human risk index.

L_j = Wall length in unit portion.

The wall should be divided horizontally into unit portion with the vertical same sectional details in the application of Eq. (33). In the equation, symbol of Σ means to sum up the values of these unit portions. In case the sectional details of the unit portion consists of multiple construction methods, the values of B and H shall be adopted for the external wall estimated as by the most vulnerable construction method, that is, which gives the maximum value of *B,* among the elements concerned. See the translators' note 5 for the wall with different vertical construction method.

4.3.2 Construction index *B*

The construction index B shall be calculated from conformability index f and deterioration index t by Eq. (34).

$$B = f + (1 - f)t \tag{34}$$

(1) Conformability index *f*

The conformability index f shall be determined in combination of ductility grade of the primary structure g_S and ductility grade of non-structural elements g_N as given in Table 14. The values of g_S and g_N shall be graded according to Tables 15 and 16, respectively.

Table 14 Conformability index *f*

Non-structural elements \ Primary structure		rigid ← g_S → flexible			
		1	2	3	4
rigid ↑ g_N ↓ flexible	1	0.3	0.8	0.9	1.0
	2	0	0.3	0.8	0.9
	3	0	0	0.3	0.8
	4	0	0	0	0.3

Table 15 Ductility grade of primary structure g_S

g_S		Ductility grade of primary structure	Corresponding ductility Index F
rigid ↕ flexible	1	Poor ductile building governed by extremely brittle column	0.8
	2	Small ductile building governed by shear column or shear wall	1.0
	3	Ductile building governed by flexural column or flexural wall	1.3
	4	Ductile enough building with enough ductility governed by flexural column or flexural wall	3.0

Table 16 Ductility grade of non-structural elements g_N

g_N		Construction method of non-structural elements (example of non-structural wall, opening, and finishing materials)		
rigid ↕ flexible	1	Poor ductile, wet construction method		
		Concrete block, glass block	Fixed window (hardening putty sealing)	Stone finishing
	2	Small ductile, dry construction method		
		ALC panel	Fixed window (elastic sealing, glazing channel sealing)	Tile finishing, mortar finishing
	3	Ductile enough, prefabricated, members connected monolithically with a wall constructed on site		
		Metal / PC curtain wall	Movable sash	Finishing paint, tile pre-fixed form

	4	Prevention from falling or spalling, special consideration for earthquake response		
		Monolithic wall constructed on site	without opening	Concrete finishing

(2) Deterioration index *t*

The deterioration index t shall be selected as listed in Table 17, in combination of the past damage grade g_H and the aging grade g_Y.

Table 17 Deterioration index *t*

Past damage grade g_H \ Aging grade g_Y		1	2	3
		~3years	3~10years	10years~
1	not repaired damage	1.0	1.0	1.0
2	damage/trouble unknown	0.2	0.3	0.5
3	no damage/ repaired	0	0.2	0.3

4.3.3 Area index *W*

The area index W shall be calculated by Eq. (35).

$$W = a + b\frac{h_j}{h_s} \tag{35}$$

where:

a = 0.5.

b = 0.5.

h_j = Height of the portion where the construction method is used (m).

h_s = Standard height =3.5m.

4.3.4 Human risk index *H*

The human risk index H shall be calculated from the location index e and the risk reduction c by Eq. (36). See the translators' note 6 in estimating the values of c, e and H.

$$H = \sum_k e_k \cdot c_k \tag{36}$$

In the calculation by Eq. (36), the product of the indices $e_k \cdot c_k$ shall be summed up for every horizontal plane (k) inside the angle of incidence, which is defined as the line with the tangential gradient of 2/1 to the vertical line (see Figure TN.6-1). The maximum of $e_k \cdot c_k$ shall be adopted, in case there are different e_k or c_k in one horizontal plane (k). (Calculation examples are also shown in Figure TN.6-1)

(1) Location index *e*

The location index *e* shall be taken as listed in Table 18 based on the possibility of human presence below the non-structural elements.

Table 18 Location index

Environment	*e*
public passage way	1.0
private passage way, passage in the site, corridor, square, veranda	0.7
open space where human can enter, planted garden	0.2
open space where human can not enter, adjacent building	0

(2) Risk reduction index *c*

The risk reduction index *c* shall be taken as listed in Table 19 based on the existence of the effective eaves, set back and the like.

Table 19 Risk reduction index *c*

Condition of risk reduction	*c*
eaves, set-back cover the incidence angle	0
just below the eaves in case where eaves cover the incidence angle partially (horizontally projecting surface)	0
horizontal surface of the same floor as the wall surface concerned	0.5
Others	1.0

4.4 Third Level Screening Procedure

The site inspection on actual conditions of the construction method (construction details and states affecting deformability, aging, and etc.) shall be carried out to evaluate the construction index, from which the methods for the second level screening procedure shall be applied in the third level screening procedure.

Chapter 5 Judgment on Seismic Safety

5.1 Basic Principles

(1) Seismic safety of a building shall be judged by comprehensive assessment based on the seismic evaluations separately conducted on the structure and the non-structural elements.

(2) Seismic safety of structure shall be judged by Eq. (37):

$$I_S \geq I_{SO} \tag{37}$$

where:

I_S = Seismic index of structure

I_{SO} = Seismic demand index of structure

If Eq. (37) is satisfied, the building may be assessed to be “Safe - the building possess the seismic capacity required against the expected earthquake motions”. Otherwise, the building should be assessed to be “Uncertain,” in seismic safety.

(3) Seismic safety of non-structural elements of the building shall be judged based on the standard specified elsewhere.

(4) The seismic evaluation document shall be made which includes the indices for evaluation, the calculation procedures, the seismic index of structure, the seismic demand index, and comments on the seismic evaluation and the safety judgment.

5.2 Seismic Demand Index I_{SO}

(1) The seismic demand index of structure I_{SO} should be calculated by Eq. (38) regardless of the story in the building.

$$I_{SO} = E_S \cdot Z \cdot G \cdot U \tag{38}$$

where:

E_S = Basic seismic demand index of structure, standard values of which shall be selected as follows regardless of the direction of the building:

$E_S = 0.8$ for the first level screening,
$E_S = 0.6$ for the second level screening, and
$E_S = 0.6$ for the third level screening.

Z = Zone index, namely the modification factor accounting for the seismic activities and the seismic intensities expected in the region of the site.

G = Ground index, namely the modification factor accounting for the effects of the amplification of the surface soil, geological conditions and soil-and-structure interaction on the expected earthquake motions.

U = Usage index, namely the modification factor accounting for the use of the building.

(2) In case the seismic safety of a structure is judged by Eq. (37) in the second and the third level screening procedure and assessed to be "Safe," Eq. (39) shall also be satisfied.

$$C_{TU} \cdot S_D \geq 0.3 \cdot Z \cdot G \cdot U \tag{39}$$

where:

C_{TU} = Cumulative strength index at the ultimate deformation of structure.

S_D = Irregurality index.

The index C_{TU} may be modified accordingly in the same manner, in case the basic seismic index of structure E_0 is modified by Eq. (6).

See the translators' note 7 which explains the relationship between Eq. (39) and the Japanese Building Code.

Supplementary Provisions

Supplementary Provisions: Calculation of Ultimate Strength, Ultimate Deformation (Ductility Index) and Yield Deformation of Members

1 Columns

1.1 Ultimate Strength

(1) Basic principles

(a) The provisions of this section shall be applied to the calculation of the strengths and ductility indexes of columns, and columns with wing wall(s) that subject to bending moment in the out-of plane direction of the wing wall(s).

(b) Material strengths of the concrete, the round reinforcing bars, and the deformed reinforcing bars used in the calculation of the flexural strength and the shear strength of columns shall be the specified design strength (F_C), the 294 N/mm^2, and the nominal yield strength plus 49 N/mm^2, respectively. The results of the preliminary inspection, such as the compressive strength test of concrete core sample specimens or the tensile test of sample reinforcing bars, may be used as material strength instead.

(c) The varied axial force in columns due to the lateral external force at the failure mechanism of the frame shall be considered for the calculation of flexural strength and shear strength of columns, in principle. The varied axial force of columns is not necessary to be considered in the second level screening, in case that the columns are in six story or less buildings and are normal such as not the column supporting the wall above (see the translators' note 2).

(2) Ultimate flexural strength

(a) The ultimate flexural strength of columns shall be calculated with Eq. (A1.1-1).

For $N_{max} \geq N > 0.4b \cdot D \cdot F_c$

$$M_u = \left\{ 0.8a_t \cdot \sigma_y \cdot D + 0.12b \cdot D^2 \cdot F_c \right\} \cdot \left(\frac{N_{max} - N}{N_{max} - 0.4b \cdot D \cdot F_c} \right)$$

For $0.4b \cdot D \cdot F_c \geq N > 0$

$$M_u = 0.8a_t \cdot \sigma_y \cdot D + 0.5N \cdot D \cdot \left(1 - \frac{N}{b \cdot D \cdot F_c} \right) \quad \text{(N·mm)}$$

For $0 > N \geq N_{min}$

$$M_u = 0.8a_t \cdot \sigma_y \cdot D + 0.4N \cdot D$$

(A1.1-1)

where:

N_{max} = Axial compressive strength $= b \cdot D \cdot F_c + a_g \cdot \sigma_y$ (N).

N_{min} = Axial tensile strength $= -a_g \cdot \sigma_y$ (N).

N = Axial force (N).

a_t = Total cross sectional area of tensile reinforcing bars (mm^2).

a_g = Total cross sectional area of reinforcing bars (mm^2).

b = Column width (mm).

D = Column depth (mm).

σ_y = Yield strength of reinforcing bars (N/mm^2).

F_c = Compressive strength of concrete (N/mm^2).

(b) The multi layered reinforcement shall be considered in using Eq. (A1.1-1).

(c) In calculating the ultimate flexural strength of columns, another calculation method such as based on rigid-plastic theory may be used instead.

(3) Ultimate shear strength

(a) Ultimate shear strength of columns shall be calculated with Eq. (A1.1-2).

$$Q_{su} = \left\{ \frac{0.053 p_t^{0.23} (18 + F_c)}{M/(Q \cdot d) + 0.12} + 0.85\sqrt{p_w \cdot {}_s\sigma_{wy}} + 0.1\sigma_0 \right\} \cdot b \cdot j \quad \text{(N)} \qquad \text{(A1.1-2)}$$

where:

p_t = Tensile reinforcement ratio (%).

p_w = Shear reinforcement ratio, p_w=0.012 for $p_w \geq 0.012$.

${}_s\sigma_{wy}$ = Yield strength of shear reinforcing bars (N/mm^2).

σ_0 = Axial stress in column (N/mm^2).

d = Effective depth of column. D-50mm may be applied.

$\frac{M}{Q}$ = Shear span length. Default value is $\frac{h_0}{2}$.

h_0 = Clear height of the column.

j = Distance between centroids of tension and compression forces, default value is 0.8D.

(b) If the value of $M/(Q \cdot d)$ is less than unity or greater than 3, the value of $M/(Q \cdot d)$ shall be unity or 3 respectively in using Eq. (A1.1-2). And if the value of σ_0 is greater than $8N/mm^2$, the value of σ_0 shall be $8N/mm^2$ in using Eq. (A1.1-2).

1.2 Ultimate Deformation

(1) Inter-story drift angle at the ultimate flexural strength of columns R_{mu}

The inter-story drift angle at the ultimate flexural strength of columns R_{mu} shall be calculated with Eqs. (A1.2-1) and (A1.2-2).

$$R_{mu} = (h_0 / H_0) \cdot {}_c R_{mu} \geq R_{250} \qquad \text{(A1.2-1)}$$

where, $h_0 / H_0 \leq 1.0$

$${}_cR_{mu} = {}_cR_{my} + {}_cR_{mp} \leq {}_cR_{30} \qquad \text{(A1.2-2)}$$

where:

h_0 = Clear height of column.

H_0 = Standard clear height of column from bottom of the upper floor beam to top of the lower floor slab.

${}_cR_{my}$ = Yield drift angle of column (measured in clear height of column), specified in the section 1.3 of Supplementary Provisions.

${}_cR_{mu}$ = Drift angle at the ultimate flexural strength of column (measured in the clear height of column).

${}_cR_{mp}$ = Plastic drift angle of the column (measured in the clear height of column), specified in the section 1.2(2) of Supplementary Provisions.

${}_cR_{30}$ = Standard drift angle of the column (measured in the clear height of column), 1/30.

R_{250} = Standard inter-story drift angle, 1/250.

The ${}_cR_{mu}$ shall not be larger than ${}_cR_{\max}$ specified in the section 1.2(3) of Supplementary Provisions.

(2) Plastic drift angle of columns ${}_cR_{mp}$

The plastic drift angle of the column ${}_cR_{mp}$ shall be calculated with the following equations.

$${}_cR_{mp} = 10({}_cQ_{su} / {}_cQ_{mu} - q) \cdot {}_cR_{my} \geq 0 \qquad \text{(A1.2-3)}$$

$$\begin{aligned} q &= 1.0 \quad for \quad s \leq 100mm \\ q &= 1.1 \quad for \quad s > 100mm \end{aligned} \qquad \text{(A1.2-4)}$$

where:

${}_cQ_{su}$ = Ultimate shear strength of the column, calculated with Eq. (A1.1-2) in principle.

${}_cQ_{mu}$ = Shear force at the ultimate flexural strength of the column. The largest moment capacity shall be used under the working axial force, in case axial force of column is greater than the balanced axial force.

s = Spacing of hoops.

(3) Upper limit of the drift angle of flexural columns ${}_cR_{\max}$

The upper limit of the drift angle of flexural column ${}_cR_{\max}$ shall be calculated with the following equations, in principle.

$${}_cR_{\max} = \min\{{}_cR_{\max(n)}, {}_cR_{\max(s)}, {}_cR_{\max(t)}, {}_cR_{\max(b)}, {}_cR_{\max(h)}\} \quad \text{(A 1.2-5)}$$

- ${}_cR_{\max(n)}$: upper limit of the drift angle of the flexural column determined by the axial force;

$${}_cR_{\max(n)} = {}_cR_{250} \quad \textit{for} \quad \eta > \eta_H$$
$${}_cR_{max(n)} = {}_cR_{30} \cdot \left(\frac{{}_cR_{250}}{{}_cR_{30}} \right)^{n'} \le {}_cR_{30} \quad \textit{for other case} \quad \text{(A 1.2-6)}$$

where:

$n' = (\eta - \eta_L)(\eta_H - \eta_L)$.

$\eta = N_s / (b \cdot D \cdot F_c)$.

$\eta_L = 0.25$ and $\eta_H = 0.5$ for $s \le 100mm$.

$\eta_L = 0.2$ and $\eta_H = 0.4$ for $s > 100mm$.

- ${}_cR_{\max(s)}$: upper limit of the drift angle of the flexural column determined by the shear force;

$${}_cR_{\max(s)} = {}_cR_{250} \quad \textit{for} \quad {}_c\tau_u / F_c > 0.2$$
$${}_cR_{\max(s)} = {}_cR_{30} \quad \textit{for other case} \quad \text{(A1.2-7)}$$

- ${}_cR_{\max(t)}$: upper limit of the drift angle of the flexural column determined by the tensile reinforcement ratio;

$${}_cR_{\max(t)} = {}_cR_{250} \quad \textit{for} \quad p_t > 1.0\%$$
$${}_cR_{\max(t)} = {}_cR_{30} \quad \textit{for other case} \quad \text{(A 1.2-8)}$$

- ${}_cR_{\max(b)}$: upper limit of the drift angle of the flexural column determined by the spacing of hoops;

$${}_cR_{\max(b)} = {}_cR_{50} \quad \textit{for} \quad s / d_b > 8$$
$${}_cR_{\max(b)} = {}_cR_{30} \quad \textit{for other case} \quad \text{(A 1.2-9)}$$

- ${}_cR_{\max(h)}$: upper limit of the drift angle of the flexural column determined by the clear height;

$${}_cR_{\max(h)} = {}_cR_{250} \quad \textit{for} \quad h_o / D \le 2$$
$${}_cR_{\max(h)} = {}_cR_{30} \quad \textit{for other case} \quad \text{(A 1.2-10)}$$

where:

b = Column width.

D = Column depth.

h_o = Clear height of the column.

F_c = Compressive strength of concrete.

N_s = Additional axial force of column due to earthquakes.

$_C\tau_u$ = Shear stress at the column strength.

= $\min\left\{ {}_cQ_{mu} / (b \cdot j),\ {}_cQ_{su} / (b \cdot j) \right\}$.

${}_cQ_{mu}$ = Shear force at the ultimate flexural strength of the column.

${}_cQ_{su}$ = Ultimate shear strength of the column, calculated with Eq. (A1.1-2).

j = Distance between the centroids of the tension and compression forces. Default value is 0.8D.

p_t = Tensile reinforcement ratio (%).

s = Spacing of hoops.

d_b = Diameter of the flexural reinforcing bar of the column.

${}_cR_{250}$ = Standard drift angle of the column (measured in the clear height of column), 1/250.

${}_cR_{50}$ = Standard drift angle of the column (measured in the clear height of column), 1/50.

${}_cR_{30}$ = Standard drift angle of the column (measured in the clear height of column), 1/30.

The upper limit of the drift angle of the flexural column ${}_cR_{\max}$ may be increased based on the special inspection or study, in case that the column has enough hoops as a result of seismic strengthening, etc.

(4) Inter-story drift angle at the ultimate shear strength of columns R_{su}

The inter-story drift angle at the ultimate shear strength of the column R_{su} shall be calculated with Eq. (A1.2-11).

$$R_{su} = \frac{{}_cQ_{su} / {}_cQ_{mu} - 0.3}{0.7} \cdot R_{my} \geq R_{250} \quad for \quad {}_c\alpha \cdot {}_cQ_{mu} < {}_cQ_{su}$$
$$R_{su} = R_{250} \quad for \quad {}_c\alpha \cdot {}_cQ_{mu} \geq {}_cQ_{su} \qquad \text{(A1.2-11)}$$

where:

${}_cQ_{su}$ = Ultimate shear strength of the column, calculated with Eq. (A1.1-3) in principle.

${}_cQ_{mu}$ = Shear force at the ultimate flexural strength of the column.

${}_c\alpha$ = Effective strength factor of the column, calculated with the following equation.

$${}_c\alpha = 0.3 + 0.7(R_{250} / R_{my}) \qquad \text{(A1.2-12)}$$

where:

R_{my} = Yield inter-story drift angle, specified by Eq. (A1.3-1) in the section 1.3 of Supplementary Provisons.

R_{250} = Standard inter-story drift angle, 1/250.

1.3 Yield Deformation of Flexural Columns

(1) Columns

The inter-story drift angle at the flexural yielding of the column R_{my} shall be calculated with Eqs. (A1.3-1) and (A1.3-2).

$$R_{my} = (h_0 / H_0) \cdot {}_cR_{my} \geq R_{250} \tag{A1.3-1}$$

where, $h_0 / H_0 \leq 1.0$

$$\left.\begin{array}{l} {}_cR_{my} = {}_cR_{150} \quad for \quad h_0 / D \geq 3.0 \\ {}_cR_{my} = {}_cR_{250} \quad for \quad h_0 / D \leq 2.0 \end{array}\right. \tag{A1.3-2}$$

${}_cR_{my}$ is set by interporation for $2.0 < h_0 / D < 3.0$

where:

h_0 = Clear height of the column.

H_0 = Standard clear height of the column from the bottom of the upper floor beam to the top of the lower floor slab.

D = Column depth.

${}_cR_{150}$ = Standard drift angle of the column (measured in the clear height of column), 1/150.

${}_cR_{250}$ = Standard drift angle of the column (measured in the clear height of column), 1/250.

R_{250} = Standard inter-story drift angle, 1/250.

${}_cR_{my}$ = Yield drift angle of the column (measured in the clear height of column).

The value of ${}_cR_{my}$ shall not be greater than that of ${}_cR_{\max}$ specified in the section 1.2(3) of Supplementary Provisions.

(2) Columns with wing wall(s)

The inter-story drift angle at the flexural yielding of the column with wing wall R_{my} shall be calculated with using Eqs. (A1.3-1) and (A1.3-2) by replacing D in Eq. (A1.3-2) to D' specified as follows.

For $h_0 / D \leq 4.0$

$$\left.\begin{array}{l} D' = D \cdot \{1 + L_W / L\} \quad for \quad 0 < L_w < L - D \\ D' = 2D \quad for \quad L_w \geq L - D \end{array}\right. \tag{A1.3-3}$$

For $h_0 / D > 4.0$

$$D' = D \cdot \left(1 + \frac{L_w}{L} \cdot \frac{h_0 / D - 2}{2} \right) \quad for \quad 0 < L_w < L - D$$

$$D' = h_0 / 2 \quad for \quad L_w \geq L - D \qquad \text{(A1.3-4)}$$

where:

L_W = Wing wall length, calculated as the same manner in the section 3 of Supplementary Provisions.

L = Standard span length, calculated as the same manner in the section 3 of Supplementary Provisions.

2 Walls

2.1 Ultimate Strength of Wall

(1) Basic principles

(a) The provisions of this section shall be applied to the calculation of the strength and ductility index of walls with boundary columns, and walls without columns. The ductility index shall be calculated according to the section 3.2.3 of the standard.

(b) The material strengths used in the calculation of the wall ultimate strength shall follow the section 1 (Columns) of the Supplementary Provisions.

(c) The axial force used in the calculation of the wall ultimate strength shall follow the section 1 (Columns) of the Supplementary Provisions.

(2) Wall with boundary columns

(a) Ultimate flexural strength

The ultimate flexural strength of the wall with boundary columns ${}_W M_u$ shall be calculated with Eq. (A2.1-1). The sectional area of the flexural reinforcing bars in the column located at the intermediate position of the wall span shall be counted as $\sum a_{wy}$ in the equation considering them as the vertical reinforcing bars in the wall. However, the vertical reinforcing bars in the wall cut by openings shall not be counted.

$$ {}_W M_u = a_t \cdot \sigma_{sy} \cdot l_W + 0.5\sum(a_{wy} \cdot \sigma_{wy}) \cdot l_W + 0.5N \cdot l_W \quad (\mathrm{N \cdot mm}) \qquad \text{(A2.1-1)}$$

where:

N = Total axial force in the boundary columns attached to the wall.

$a_t, \sum a_{wv}$ = Cross sectional area of the flexural reinforcing bars of a boundary column and the vertical reinforcing bars in the wall, respectively (mm^2).

σ_{sy}, σ_{wy} = Yield strength of the flexural reinforcing bars of a boundary column and the vertical reinforcing bars in the wall, respectively ($\mathrm{N/mm}^2$).

l_W = Distance between the center of the boundary columns of the wall (mm).

(b) Ultimate shear strength

The ultimate shear strength of the wall with boundary columns ${}_W Q_{su}$ shall be calculated with Eq. (A2.1-2). In case that the wall with boundary columns has an opening, the ultimate shear strength of the wall shall be reduced from Eq. (A2.1-2) by multiplying the strength reduction factor γ due to the opening calculated with Eq. (A2.1-4).

$$Q_{su} = \left\{ \frac{0.053 p_{te}^{\ 0.23}(18 + F_c)}{M/(Q \cdot l) + 0.12} + 0.85\sqrt{p_{se} \cdot \sigma_{wy}} + 0.1\sigma_{0e} \right\} \cdot b_e \cdot j_e \quad \text{for } 1 \le M/(Q \cdot l) \le 3 \quad (\mathrm{N})$$

(A2.1-2)

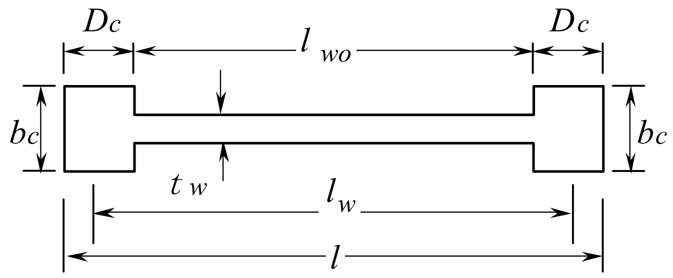


Figure A2.1-1 Wall with boundary columns

where:

$p_{te} = 100 a_t / (b_e \cdot l)$: Equivalent tensile reinforcement ratio (%).

a_t = Cross sectional area of the flexural reinforcement of the boundary column in the tension side of wall.

l = Wall length.

$b_e = \sum A / l$: Equivalent thickness of the wall.

$\sum A$ = Cross sectional area of the wall.

$p_{se} = a_h / (b_e \cdot s)$ = Equivalent lateral reinforcement ratio (%).

a_h, s = Cross sectional area of a pair of the lateral reinforcement and its spacing, respectively.

σ_{wy} = Yield strength of the lateral reinforcing bar.

$\sigma_{0e} = N / (b_e \cdot l)$: Axial stress. The σ_{0e} shall be not greater than 8N/mm^2.

j_e = Distance between the centroids of tension and compression forces, and may be taken as $j_e = l_W$ or $0.8 \cdot l$.

M / Q = In case of no special study, the inflection height of $h_W / 2$ can be applied, which is described in the section 3.2.2 of the standard.

In case that the wall height of $h_W / 2$ is higher than l_W, and the wall has beams at the location lower than l_W, the cross sectional area of the flexural reinforcement in the beams, $\sum a_{tg}$, can be counted into p_{se} as follows.

$$p_{se} = \frac{a_h}{b_e \cdot s} + \frac{\sum a_{tg}}{b_e \cdot h'} \cdot \frac{\sigma_{yg}}{\sigma_{wy}} \leq 2 \, {a_h}\big/{b_e \cdot s} \leq 1.2\% \qquad \text{(A2.1-3)}$$

where:

h' = The height from the floor level concerned to the top of the beam whose flexural reinforcement is counted into $\sum a_{tg}$.

σ_{yg} = Yield strength of the flexural reinforcing bars in the beams.

$$\gamma = 1 - \eta \tag{A2.1-4}$$

$$\eta = \max\left\{\sqrt{\frac{\sum h_i \cdot l_i}{h \cdot l_W}},\ \frac{\sum l_i}{l_W}\right\} \tag{A2.1-5}$$

where:

$\sqrt{\dfrac{\Sigma h_{i\cdot} \cdot l_i}{h \cdot l_W}}$ = Equivalent opening area ratio.

h = Story height.

h_i, l_i = Opening height and length.

In case that the equivalent opening area ratio is greater than 0.4, the wall shall be considered as the column with a wing wall or the wall with a column instead of considering as the wall with boundary columns.

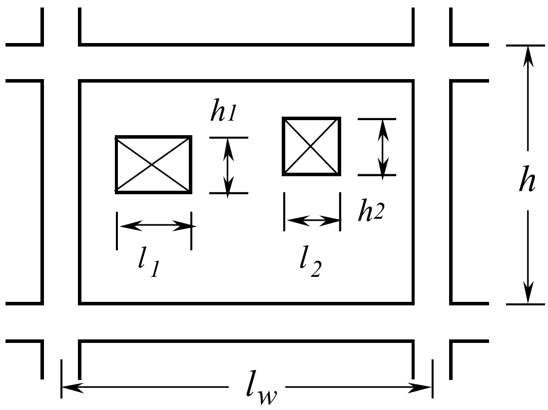


Figure A2.1-2 Wall with multiple openings

(3) Wall without column

(a) Ultimate flexural strength

The ultimate flexural strength of the wall without column shall be calculated with Eq. (A2.1-1) in consideration of the shape of the cross section and the reinforcing bar arrangement. The rational calculation method such as rigid-plastic theory may also be recommended.

(b) Ultimate shear strength

The ultimate shear strength of the wall without column shall be calculated with Eq. (A2.1-2) in consideration of the shape of the cross section and the reinforcing bar arrangement.

3 Walls with Column(s)

(1) Basic principles

(a) The provisions of this section shall be applied to the calculation of the strength and ductility index of the column with wing wall, and wall with column(s).

(b) The material strengths used in the calculation of the wall strength shall follow the section 1 (Columns) of Supplementary Provisions.

(c) The axial force used in the calculation of wall strength shall follow the section 1 (Columns) of Supplementary Provisions.

(2) Inflection height h_{CW0}

The inflection height shall be assumed based on the result of elastic or inelastic analysis. In case of not conducting elastic or inelastic analysis, the inflection height can be calculated with the following equation.

$$h_{CW0} = h_{C0} + \left(h_{W0} - h_{C0}\right) \cdot L_W / L \quad for \quad 0 < L_W < L - D_C$$
$$h_{CW0} = h_{W0} \quad for \quad L_W \geq L - D_C \tag{A3-1}$$

where:

L' = Total length including the length of the wing walls $(D_C + L_W)$.

L_W = Wing wall length or sum of wing wall lengths (see Figure A3-3).

D_C = Column depth.

L = Standard span length .

h_{W0} = Inflection height calculated as the wall with boundary columns, $h_{Wo} = h_W / 2$.

h_W = Height from the floor level concerned to the top of the multi-story wall. Here, $h_{Wo} = h_W$ at the top story.

h_{C0} = Inflection height calculated for the column, $h_{C0} = h_0 / 2$.

h_0 = Clear height of the column. Here, when the ultimate flexural strength at the top and bottom of the column are different, $h_{C0} = h_0 \cdot M_B /(M_T + M_B)$.

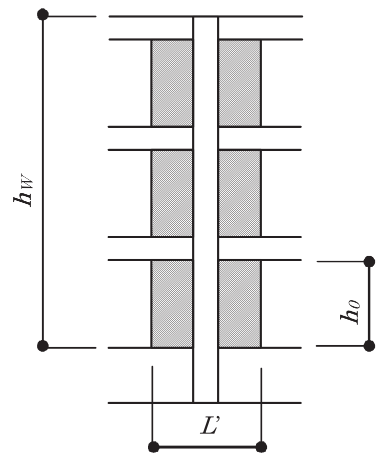


Figure A3-1 Total height of column with wing walls

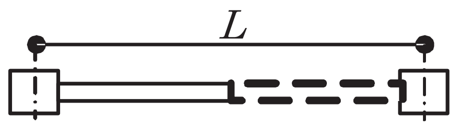


Figure A3-2 Standard span of a wall with boundary columns

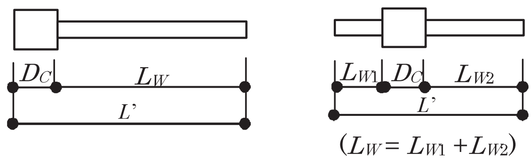


Figure A3-3 Total depth of column with wing wall

(3) Ultimate flexural strength

(a) The ultimate flexural strength M_u shall be calculated with Eq. (A3-2). The ultimate flexural strength shall be calculated with the Eq. (A1.1-1) as the rectangular column without wing walls, in case that the wing wall is attached to the one side of the column and the wing wall is located in the tension side of the column.

$$M_u = (0.9+\beta)\cdot a_t \cdot \sigma_y \cdot D + 0.5N \cdot D \cdot \left\{1 + 2\beta - \frac{N}{b_e \cdot D \cdot F_c}\left(1 + \frac{a_t \sigma_y}{N}\right)\right\} \quad \text{(A3-2)}$$

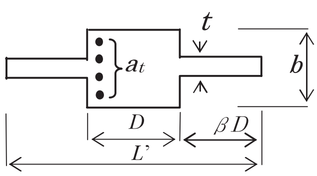


Figure A3-4 Column with wing walls

where:

a_t = See Figure A3-4.

b_e = $\sum A / L'$.

$\sum A$ = Total cross sectional area of the column with wing wall.

L' = Total depth of the column with wing wall.

β = Wing wall length in compressive side divided by D.

Other notations are according to Eq. (A1.1-1).

(b) The ultimate flexural strength can be calculated by the rational calculation methods such as the method based on rigid-plastic theory. Especially, the ultimate flexural strength of the wall with a boundary column or wing wall with a column had better be calculated by these methods.

(4) Ultimate shear strength

The ultimate shear strength shall be calculated with Eq. (A3-3).

$$Q_{su} = \max\left(Q_{su1}, Q_{su2}, Q_{su3}, Q_{su4}\right) \qquad \text{(A3-3)}$$

The Q_{su1} shall be calculated with the following equation for the wing wall.

$$Q_{su1} = \left\{ \frac{0.053 p_t^{0.23}\left(18 + F_c\right)}{M/\left(Q \cdot d_e\right) + 0.12} + 0.85\sqrt{p_{we} \cdot \sigma_{wy}} + 0.1\sigma_{0e} \right\} \cdot b_e \cdot j_e \quad \text{(N)} \qquad \text{(A3-4)}$$

Here, in case that the value of $(M/Q \cdot d_e)$ is less than unity, the value of $(M/Q \cdot d_e)$ shall be unity, and the value of $(M/Q \cdot d_e)$ is greater than 2, the value of $(M/Q \cdot d_e)$ shall be 2. In case that it is expected that the shape of the member, the reinforcing bar condition, or the confinement condition have better effects on the shear strength, the value of unity can be changed to 0.5.

where:

$p_{te} = a_t / (b_e \cdot d_e)$ (See Fig. A3-4 for a_t).

d_e = Distance from the center of the tensile reinforcing bars to the extreme fiber of the wing wall in the compressive side (mm).

$p_{we} \cdot \sigma_{wy} = p_w \sigma_{wy} (b / b_e) + p_{sh} \cdot \sigma_{sy} (t / b_e)$.

$p_w \cdot \sigma_{wy}$ = Product of the shear reinforcement ratio of the column and its yield strength (N/mm^2).

$p_{sh} \cdot \sigma_{sy}$ = Product of the horizontal shear reinforcement ratio of the wing wall and its yield strength (N/mm^2).

$\sigma_{0e} = N / \left(b_e \cdot j_e\right)$.

$j_e = 7 d_e / 8$.

$b_e = \sum A / L''$. Here, $\sum A$ = Sum of the cross sectional areas of the column, and the wing wall in the compressive side, and L'' = the column depth and the wing wall depth in the compressive side.

$M / Q \cdot d_e = \dfrac{h_{CW0}}{L'}$. Here, h_{CW0} = The inflection height, and L' = The total depth including the wing walls.

The Q_{su2} shall be calculated as the wall with the actual length and the equivalent thickness which is obtained as the quotient of the total cross sectional area including the columns dividing by the actual wall length.

The Q_{su3} shall be calculated as the column ignoring the wing walls.

The Q_{su4} shall be calculated as the wall without boundary columns ignoring the columns.

(5) Lateral strength at the ultimate flexural strength of walls

The lateral strength at the ultimate flexural strength of the walls shall be calculated with the following equation.

$$Q_{mu} = \frac{M_u}{h_{CW0}} \tag{A3-5}$$

where:

M_u = Ultimate flexural strength at the bottom of the wall.

h_{CWo} = Inflection height.

(6) Ductility index

The ductility index shall be calculated as follows.

In case the ultimate shear strength is determined as Q_{su3}, the ductility index shall be calculated as for the column.

In case the ultimate shear strength is determined as Q_{su1}, the ductility index shall be calculated as for the column with wing wall.

In case the ultimate shear strength is determined as Q_{su2} or Q_{su4}, the ductility index shall be calculated as for the wall with boundary columns.

4 Beams

(1) Basic principles

(a) The provisions of this section shall be applied to the calculation of the strength and ductility index of beams.

(b) The material strength used in the calculation of the beam strength shall follow the section 1 (Columns) of Supplementary Provisions.

(2) Ultimate flexural strength

The ultimate flexural strength of the beam shall be calculated with Eq. (A4-1). In the calculation, the effect of the slab reinforcement and the intermediate reinforcement in the beam with multi layered arrangement of the flexural reinforcement shall be considered in principle.

$$M_u = 0.9 a_t \cdot \sigma_y \cdot d \qquad \text{(A4-1)}$$

where:

a_t = Cross sectional area of the tensile reinforcing bars (mm^2).

σ_y = Yield strength of the tensile reinforcing bars (N/mm^2).

d = Effective depth of the beam (the distance between the center of gravity of the tensile reinforcement and the extreme fiber of compressive zone).

(3) Ultimate flexural strength of beam with standing or hanging wall

The ultimate flexural strength of the beam with standing and/or hanging wall shall be calculated with the Eq. (A4-2).

$$M_u = a_{te} \cdot \sigma_y (d_e - 0.5 x_n) \quad \text{(N-mm)} \qquad \text{(A4-2)}$$

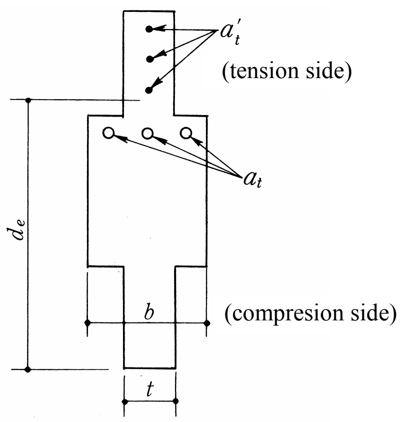


Figure A4-1 Notations used for calculation of ultimate flexural strength of beam with standing and hanging walls

where:

$$a_{te} = a_t + \sum a_t'\left(\frac{\sigma_y'}{\sigma_y}\right) \leq (0.85F_c \cdot t \cdot x_{nb} / \sigma_y) - \sum a_t'\left(\frac{\sigma_y'}{\sigma_y}\right).$$

$$x_n = a_{te} \cdot \sigma_y / (0.85F_c \cdot t).$$

$$x_{nb} = \frac{{}_c\varepsilon_B}{{}_c\varepsilon_B + {}_s\varepsilon_y} d_e.$$

a_t, a_t' = See Fig. A4-1 (mm).

σ_y, σ_y' = Yield strength of the flexural reinforcing bars in the beam or in the wall in tension side (N/mm^2).

F_c = Compressive strength of concrete (N/mm^2).

t = Wall thickness in the compression side ($t = b$ in case of no wall) (mm).

d_e = Distance between the center of gravity of the tensile reinforcement and the extreme fiber of compressive zone (see Fig. A4-1) (mm).

${}_c\varepsilon_B$ = Compressive strain at the concrete strength.

${}_s\varepsilon_y$ = Yield strain of the flexural reinforcing bar in the beam ($\sigma_y / {}_sE$ can be used).

(4) Ultimate flexural strength of beam with standing and/or hanging wall that have partial slits at their ends

In case that partial slits are placed in the compressive zone, the ultimate flexural strength of the beam with standing and/or hanging wall with partial slits at beam end shall be calculated with the Eq. (A4-3) except for the calculation based on the plastic theory assuming plane section remains plane after the deformation. In case that a partial slit is not in the compressive zone, the ultimate flexural strength shall be calculated with the Eq. (A4-1) ignoring the effect of standing and/or hanging wall.

In addition, the value of M_u calculated with the Eq. (A4-3) shall be equal to or greater than the ultimate flexural strength of the beam calculated with the Eq. (A4-1).

$$M_u = \min\ [C_{\max}, T_{\max}] \cdot j_{\max} \tag{A4-3}$$

where:

$$C_{\max} = 0.7t_s \cdot h_s \cdot F_c$$

$$T_{\max} = a_t \cdot \sigma_y$$

$$j_{\max} = 0.65h_s + d$$

t_s = Remaining concrete thickness of the partial slit (mm).

h_s = Standing or hanging wall height (mm).

a_t = Cross sectional area of the tensile reinforcing bars in the beam in case that the partial slit is in compression side (mm^2).

σ_y = Yield strength of the reinforcing bars (N/mm^2).

(5) Ultimate shear strength

The ultimate shear strength of the beam shall be calculated with the Eq. (A4-4a). In case that the beam does not have a standing or hanging wall, the effect of slab may be considered rationally.

$$Q_{su} = \left\{ \frac{0.053 p_t^{0.23}(F_c + 18)}{M/(Q \cdot d) + 0.12} + 0.85\sqrt{p_w \cdot \sigma_{wy}} \right\} b \cdot j \quad \text{(N)} \tag{A4-4a}$$

where:

p_t = Tensile reinforcement ratio (%).

F_c = Compressive strength of concrete (N/mm^2).

M/Q = Ratio of the bending moment to the shear force at the section where the strength is calculated. $1 \le M/(Q \cdot d) \le 3$

d = Effective depth of the beam (mm).

p_w = Shear reinforcement ratio (decimal number).

σ_{wy} = Yield strength of the shear reinforcing bars (N/mm^2).

b = Beam width (mm).

j = Distance between the centroids of the tension and compression portions. Default value is $(7/8 \cdot d)$ (mm).

(6) Ultimate shear strength of the beam with standing or hanging wall

The ultimate shear strength shall be calculated with Eq. (A4-5)

$$Q_{su} = \left\{ \frac{0.053 p_t^{0.23}(F_c + 18)}{M/(Q \cdot d_e) + 0.12} + 0.85\sqrt{p_{we} \cdot \sigma_{wy}} \right\} b_e \cdot j_e \quad \text{(N)} \tag{A4-5}$$

where:

$0.5 \le M/(Q \cdot d_e) \le 2$.

$p_{we} = p_w\left(\frac{b}{b_e}\right) + p_s\left(\frac{t}{b_e}\right)$.

p_t = Tensile reinforcement ratio of the beam (%).

F_c = Compressive strength of concrete (N/mm^2).

d_e = Distance between the center of gravity of the tensile reinforcement and the extreme fiber of compressive zone (see Fig. A4-2) (mm).

p_w = Shear reinforcement ratio of the beam.

p_s = Shear reinforcement ratio of the wall.

σ_{wy} = Yield strength of the shear reinforcing bars (N/mm^2).

b_e = Beam width of the equivalent rectangular shaped beam. See Fig. A4-2. (mm).

j_e = Distance between the centroids of the tension and compression portions. Default value is $(7/8 \cdot d_e)$ (mm).

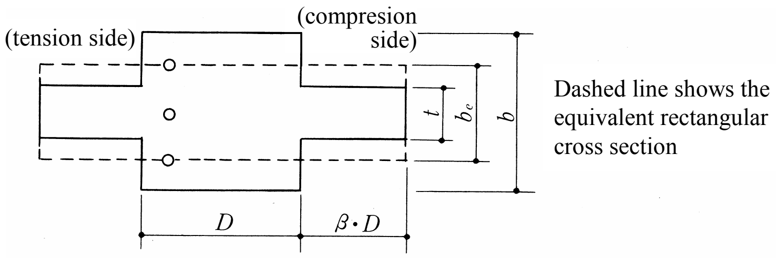


Figure A4-2 Notations used for calculation of ultimate shear strength of beam with wing wall

(7) Remarks on the strength calculation of the beam or beam with spandrel wall

In case of the strength calculation for the beam, the beam should be rationally modeled considering the effect of surrounding members, etc.

5 Others

Members other than specified in the sections of 1 to 4 in the Supplementary Provisions or the failure mode should be examined, if necessary.

Translators’ Notes

Translators' Note 1 ---

Concept of Seismic Evaluation

Seismic performance of buildings is represented by marks of 'I_S' which is the seismic index of structure. This index is evaluated by the following equation at each story and to each direction.

$$I_S = E_0 \times S_D \times T \qquad \text{(TN.1-1)}$$

where:

E_0 = Basic seismic index of structure.

S_D = Irregularity index.

T = Time index.

The overall 'I_S' evaluation method consists of three level screening procedures; first, second and third level screening procedures. The first level screening procedure is the simplest, but most conservative of the three, while the basic concept is common for all three. Since the S_D and T indices are the reduction factors less than or equal to 1.0 and the E_0 index usually predominates, the outline for evaluating the E_0 index is described here to show the concept of seismic evaluation method adopted in this Standard.

The E_0 index is a basic value that specifies the seismic performance of a building. It is known that existing RC buildings have a seismic performance of varying degrees, and that the variation is due to the diversity of strength and ductility possessed by the buildings. The E_0 index is the criteria used for evaluating the seismic performance of a building based on the strength and ductility of the building.

Fig. TN.1-1 is a diagram explaining the relationship between horizontal force and horizontal displacement when the force is applied to RC buildings. Though RC buildings have varied properties actually, two types of typical buildings, Building A and Building B, are cited here to give simplified explanation.

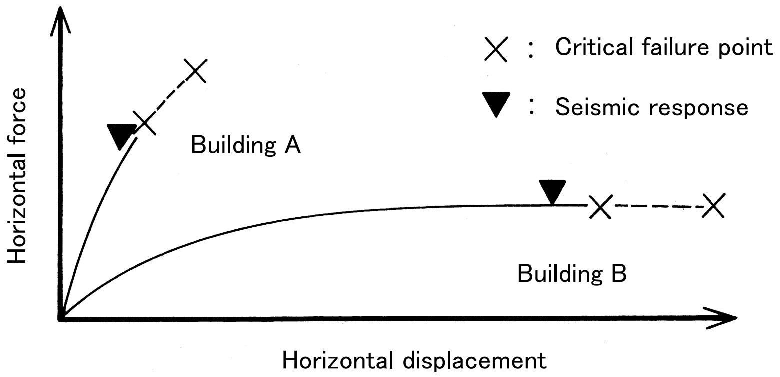


Figure TN.1-1 Relationship between horizontal force and horizontal displacement of RC buildings
(quoted from the figure on page 72 in the commentary of 3.2.1 of the Standard of 2001 Japanese version)

Building A is assumed to have many walls and considerably strong but low in ductility. In contrast, Building B is assumed to be a rigid-frame structure with less walls and not so strong but large in ductility. When these buildings are subjected to earthquake loading, if the maximum displacement indicated by the mark ▼ remains within the critical failure point shown by the 'x' mark, the building will stay safe. But if not remain within the critical failure point, the building will suffer significant damage. From various investigations to date, it is known that in order to satisfy the above requirements RC buildings with many walls but short in ductility shall have considerable strength and rigid-frame buildings not blessed with strength shall have considerable ductility.

Based on these properties of buildings, the E_0 index is introduced so as to establish evaluation criteria commonly usable for buildings with many walls and buildings of a rigid-frame structure. To put it simply, the following expression is given:

$$E_0 = (criteria\ \ of\ strength) \times (criteria\ \ of\ ductility) \tag{TN.1-2}$$

In this Standard, the criteria of strength is called Strength Index C and the criteria of ductility is called Ductility Index F. To derive the values of these indexes, three types of estimation methods - from the first level screening method with a simple and handy calculation through the third level screening method that requires a moderately detailed calculation - are provided.

The two examples cited above as Building A and Building B are very simple ones, but practically speaking, actual buildings are never so simple, or very complicated, making it hard to derive the E_0 index. Fig. TN.1-2 is a schematic description of the behavior when horizontal force is applied to a rigid-frame building with a limited number of walls. When horizontal force is gradually increased, the walls reach fracturing at the 'a' point. But the building does not completely fracture at this point. Though horizontal resistance drops for a moment, the remaining rigid-frame structure begins to resist horizontal force in accordance with an increase in deformation. The horizontal resistance continues until finally reaches the 'b' point which is the fracturing point of the rigid-frame structure.

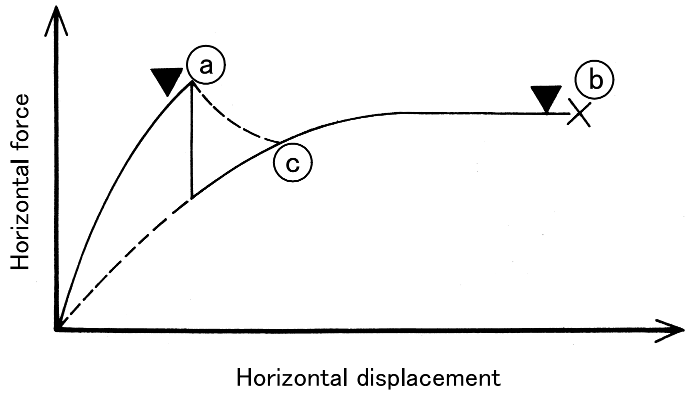


Figure TN.1-2 Behavior of rigid-frame and wall combined buildings
(quoted from the figure on page 74 in the commentary of 3.2.1 of the Standard of 2001 Japanese version)

In this scheme, the E_0 index which allows deformation up to the 'b' point is derived as follows: First, derive the value of E_0 index by presuming that the building is supported by walls only and ignoring the presence of rigid-frames. The value thus derived is taken as E_1. Next, presuming the contrary, that is, presuming that the building is supported by rigid-frames only and ignoring the presence of walls, the value of E_0 index is obtained, which is taken as E_2. Then, the square root of the sum of the square of E_1 and the square of E_2 is calculated, and the derived value is identified as the E_0 index of the building.

$$E_0 = \sqrt{E_1^2 + E_2^2} \tag{TN.1-3}$$

The value of E_0 index thus derived is naturally smaller than the value of ($E_1 + E_2$). In other words, the derived E_0 index of the building is smaller than the mere sum of the two seismic performances derived from assumptions that the building is supported by walls only and by rigid-frames only. The seismic response of buildings in which walls and rigid-frames are intermingled is highly complicated, and it has been found that it is sometimes risky to regard the seismic performance of each building as just a mere addition of the seismic performance provided by walls only and the seismic performance provided by rigid-frames only.

Fig. TN.1-3 compares the estimated *Is* index and damages of buildings during 1968 Tokachi-oki and 1978 Miyagiken-oki earthquakes. From this figure, it is seen that the *Is* index properly distinguish the damaged buildings from the non-damaged ones.

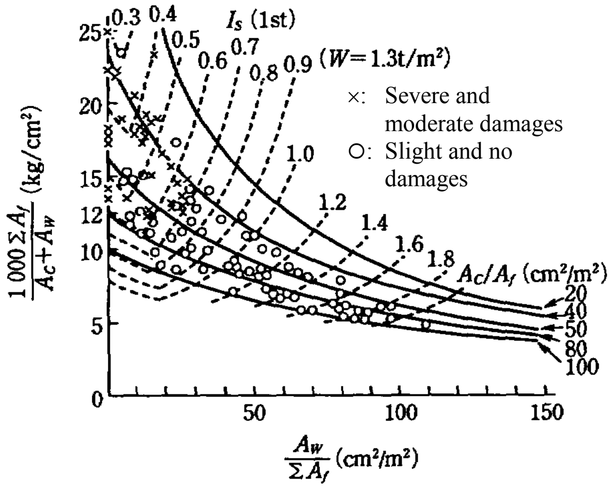


Figure TN.1-3 Index I_S and building damage
(1968 Tokachi-oki and 1978 Miyagiken-oki earthquakes)
(quoted from Figure 4 on page 511 of Ref. 1)

The translators summarized Ref.1 into this note for the purpose of briefly explaining the basic concept of the seismic evaluation method for the reader of this book.

(Ref. 1) Umemura, H: "A Guideline To Evaluate Seismic Performance Of Existing Medium-And Low-Rise Reinforced Concrete Buildings And Its Application". Proceedings Of The Seventh World Conference On Earthquake Engineering, September 8-13, 1980, Istanbul, Turkey, Volume 4, pp. 505-512.

--- End of Translators' Note 1

Translators' Note **2** ---

Column Supporting The Wall Above

In the second level screening, a column supporting the wall above at the soft story in the frame where the wall panel is taken off from the multistory shear wall (hereafter, the frame with soft story), should be examined based on the special study, or by the following procedures practically used.

(1) Estimation of shear force carried by the walls at upper stories

Assuming the seismic force distribution along the height of the total building or the frame with soft story, the shear force carried by the story just above the soft story should be estimated.

(a) In case where the seismic index of structure would be enough large, shear force carried by shear wall just above the soft story could be estimated by reducing its shear strength by a specific ratio.

(2) Estimation of axial force

The maximum axial force acting on the column supporting the wall above should be estimated in consideration of the following failure mechanism, (a) to (c). In failure mechanism of (a) and (c), the strength at their mechanisms should be calculated considering the over strength.

(a) The failure mechanism due to the flexural yielding or shear failure of shear wall at the upper story besides the soft story.

(b) The failure mechanism due to the tensional axial yielding of the column supporting the wall above (that is forming the total flexural yielding mechanism).

(c) The failure mechanism due to the uplifting.

(3) Examination of the second-class prime element

The column supporting the wall above should be examined which it is the second-class prime element or not. The column should be categorized to the second-class prime element, in case where the column would meet any case (a) to (c) describing below and could not support the redistributed sustained load.

(a) The shear failure mode is expected.

1) Flexural strength Mu of the column whose section is rectangular should be calculated by the Eq. A1.1-1 in the Supplementary Provisions of the Standard.

2) Shear strength Qsu of the column whose section is rectangular should be calculated by the Eq. A1.1-2.

(b) The shear failure mode at the balanced axial load is expected, in case where the axial force would be larger than the balanced axial load ($N/(Ac \cdot Fc) = 0.4$, approximately).

(c) In case that the axial force ratio ($N/(Ac \cdot Fc)$) would be larger than the specified limit axial force ratio (η_u). Without any further studies, the specified limit axial force ratio might be set as 0.4 for the column with more than 100 mm spacing of shear reinforcement (constructed in earlier than 1971), and 0.5 for the column with less than 100 mm spacing (constructed in not earlier than 1971).

(4) Reevaluation of the seismic index of structure

The seismic index of structure should be reevaluated in case where the column supporting the wall above would meet the following case, (a) and (b).

(a) The column supporting the wall above is expected to form the shear failure mode and to be categorized to the second-class prime element.

(b) The axial force ratio would be larger than the specified limit axial force ratio (η_u).

(5) Strengthening of the column supporting the wall above

(a) In case where the seismic index of structure would be smaller than the demand of the seismic index of structure, the strengthening for axial capacity of column should be conducted for the specified limit axial force ratio to be cover the acting axial force ratio.

(b) The strengthening of the column supporting the wall above that is categorized to the second-class prime element would not be necessary, in case where the following conditions would be satisfied.

1) Spacing of shear reinforcement is not larger than 100 mm.
2) The reduced seismic index of structure is enough larger than the demand of the seismic index of structure.
3) The lateral strength of the soft story is enough large due to the existing of the shear wall in parallel to the column concerned.

(c) In case where the in-plane stiffness of floor slab would not be enough, a relevant strengthening should be conducted based on the various studies according to the condition of the building, if necessary.

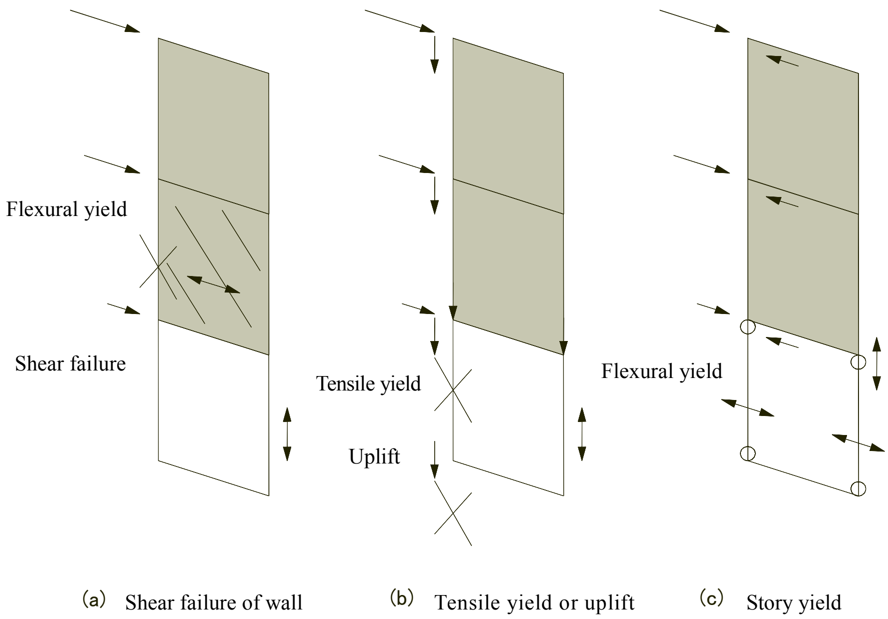


Figure TN.2-1 Collapse mechanism of the frame composed of the columns supporting the wall above
(quoted from Figure 2.2.3-1 on page 273 of the Standard of 2001 Japanese version)

This note is quoted from the appendix 2 of the Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001 in the Japanese edition.

-- End of Translators' Note 2

Translators' Note 3 --

<u>Second-Class Prime Elements</u>

If the axial load sustained by columns in brittle failure mode can be redistributed to surrounding other columns in a structure and lateral-force resisting capacity of the structure is sufficient by other structural members, the structure does not have fatal damage or fall. Otherwise the structure should have fatal damage and fall down. Then the Second-Class Prime Element is defined as following.

"The vertical structural element or frame that will fail in brittle manner and whose sustaining axial load can not be redistributed or not be sustained by the surrounding members in the structure, even if the lateral-force resisting capacity of the structure is enough."

The judgment of Second-Class Prime Element is necessary in the case that the E_0 index corresponding to the larger ductility index will be adopted allowing the brittle failure, such as shear failure, of a limited number of vertical structural elements. Specifically, it is necessary to the extremely short column in the 1st screening, and shear column, extremely short column, and column supporting the wall above which will fail in axial compression in the 2nd screening. It is not necessary to the flexural column. The column in the structure built before 1970 and the spacing of shear reinforcement is over 15 cm that is classified to the flexural column due to its little longitudinal reinforcement and long shear span should be checked the condition of the second-class prime element. In case that the flexural column carries the additional axial load from the surrounding members, the axial load carrying capacity depending on the confinement and required ductility level should be checked.

The method of calculating axial load and judgment of the second-class prime element could be done by the followings.

(1) The redistributed axial load N_1 is equal to the sustaining axial load N by the candidate of second-class prime element. In case that the residual axial capacity N_r could be expected, N_1 might be $N - N_r$.

(2) It should be checked which the surrounding structural elements, such as floor slab, beam, and wall, could carry the N_1 to the surrounding vertical structural elements. If it is not possible, the member is second-class prime element. Otherwise, N_1 might be redistribute to the surrounding vertical structural elements with ΔN_1 s.

(3) The additional axial load form the objective members and sustained axial load of itself N_0 , carried by the surrounding member, $\sum \Delta N_1 + N_0$, should be summing up. If the surrounding member could not sustain the load, the objective member is the second-class prime element. In case that the surrounding member is wall, this judgment procedure is not necessary. The objective member is not the second-class prime element except that N_1 is extremely large.

The residual axial load capacity of the objective member N_r and axial load capacity of the surrounding member N_R can be estimated according to the Table TN.3-1.

Table TN.3-1 Residual axial load capacity N_r and axial load capacity N_R
($\eta_r = N_r / A_C F_C$ [$\eta_R = N_R / A_C F_C$])
(quoted from Table 3.2.1-1 in the commentary of 3.2.1 of the Standard of 2001 Japanese version)

Column	p_w (%)	F=1.0	F=1.27	F=2	F=3
Extremely short column*3	$0.4 < p_w$ *1	0.4	0.3	0.1	0
	$0.2 \le p_w \le 0.4$ *2	0.3[0.4]	0.1	0	0
	$p_w < 0.2$	0[0.4]	0	0	0
Shear column	$0.4 < p_w$ *1	0.6	0.4	0.2	0
	$0.2 \le p_w \le 0.4$ *2	0.5	0.3[0.4]	0.1	0
	$p_w < 0.2$	0.4	0[0.4]	0	0
Flexural column	$0.4 < p_w$ *1	0.6	0.6	0.5	0.4
	$0.2 \le p_w \le 0.4$ *2	0.5	0.5	0.3[0.4]	0.2[0.3]
	$p_w < 0.2$	0.4	0.4	0[0.3]	0[0.2]

Note*1: In case that spacing is not larger than 100mm, p_w > 0.4%, and sub ties are provided at the same spacing as that of main ties. In case where p_w is different in each direction, the smaller p_w can be used.
*2: In case that spacing is not larger than 100mm.
*3: The flexural column of $h_0/D \leq 2$ and $F < 1.27$ is included.
[ ]: In case where F is greater than that listed in the table, the axial load capacity N_R in [ ] can be used. In case where F is smaller than that listed in the table the axial load capacity N_R is the same as the residual axial load capacity.

In case where load bearing walls (including wing wall) are attached, especially in the transverse direction, axial load of columns could be evaluated considering the load carrying capacity of those walls. Mean axial load stress capacity is $0.3F_c$ for flexural wall, $0.1F_c$ for shear wall in the concerned direction, and $0.5F_c$ only in the case that the wall is in the orthogonal direction and seismic performance is over the seismic demand not considering ductility.

This note is quoted from the commentary of 3.2.1 of the Standard in the Japanese version.

-- **END of Translators' Note 3**

Translators' Note 4 ---

<u>Ductility Index *F* by the 1990 version</u>

(1) Calculation of ductility index *F*

Ductility index F of each story of the building shall be calculated based on the order of screening level and the collapse mode of members.

(i) First level screening procedure

Ductility index shall be calculated from Table TN.4-1.

Table TN.4-1 Ductility index for first level screening procedure
(*quoted from 3.2.3 of the Standard of 1990 Japanese version*)

	Ductility index F
Column ($h_o/D > 2$)	1.0
Extremely short column ($h_o/D \leq 2$)	0.8
Wall	1.0

(ii) Second level screening procedure

Ductility index shall be calculated from Table TN.4-2. Column with wing wall can be F=1.0 in case of no special investigation.

(a) Flexural column

$$F = \phi\sqrt{2\mu - 1} \qquad \text{(TN.4-1)}$$

where:

μ = ductility capacity (Eq. (TN.4-3)).

$$\phi = \frac{1}{0.75(1+0.05\mu)}.$$

(b) Flexural wall

If ${}_W Q_{su} / {}_W Q_{mu} \leq 1.2$ then F=1.0 (TN.4-2)

If ${}_W Q_{su} / {}_W Q_{mu} \geq 1.3$ then F=2.0

If $1.3 > {}_W Q_{su} / {}_W Q_{mu} > 1.2$ then F should be calculated by linear interpolation between above two.

where:

${}_W Q_{su}$ = shear strength of wall.

${}_W Q_{mu}$ = shear force at flexural strength of wall.

(iii) Third level screening procedure

Ductility index shall be calculated from Table TN.4-2 similar to the case for the second level screening procedure.

Table TN.4-2 Ductility index for second and third level screening procedure
(quoted from 3.2.3 of the Standard of 1990 Japanese version)

	Ductility index, F	Applicable order of screening level
Flexural column	1.27-3.2* by Eq. (TN.4-1)	second, third
Flexural wall	1.0-2.0 by Eq. (TN.4-2)	second, third
Shear column	1.0	second, third
Shear wall	1.0	second, third
Extremely short column	0.8	second, third
Column governed by flexural strength of beam	3.0	third
Column governed by shear strength of beam	1.5	third
Wall governed by uplift strength	3.0	third

* There is a case of F=1.0 when one of the conditions of Eq. (TN.4-4) is satisfied.

(2) Calculation of ductility capacity, *m*, for flexural column

Ductility capacity μ of a flexural column can be calculated by Eq. (TN.4-3). Ductility index F should be 1.0, when one of the conditions of Eq. (TN.4-4) is satisfied.

$$1 \leq \mu = \mu_o - k_1 - k_2 \leq 5 \qquad \text{(TN.4-3)}$$

where:

$$\mu = 10\left(\frac{{}_C Q_{su}}{{}_C Q_{mu}} - 1\right).$$

$k_1 = 2.0$ (k_1 can be 1.0 when spacing of hoop reinforcement is equal or less than eight times of diameter of main reinforcing bar).

$$k_2 = 30\left(\frac{{}_C \tau_{mu}}{F_c} - 1\right) \geq 0 .$$

${}_C Q_{su}$ = Shear strength of column.

${}_C Q_{mu}$ = Shear force at flexural strength of column.

$${}_C \tau_{mu} = {}_C Q_{mu} \big/ (b \cdot j) .$$

b = Width of column.

j = Distance between centroids of tension and compression forces (j can be $0.8D$).

F_c = Compressive strength of concrete.

Conditions of taking the ductility index F to be 1.0:

$$\begin{aligned} & N_s/(bDF_c) > 0.4 \\ & {}_C \tau_{mn}/F_c > 0.2 \\ & P_t > 1\% \\ & h_o/D \leq 2.0 \end{aligned} \qquad \text{(TN.4-4)}$$

where:

N_s = Column axial force for earthquake design.

P_t = Tensile reinforcement ratio.

h_o = Clear height of column.

This note is quoted from 3.2.3 of the Standard of 1990 Japanese version.

--- End of Translators' Note 4

Translators' Note 5 ---

<u>A Wall with Different Vertical Construction Methods</u>

The seismic index of non-structural elements I_N for the second level screening is estimated by Eq. (33) as the sum of the values for divided portions of a wall. The following figure, quoted from the commentary of 4.3.1 of the Standard, shows how to divide a wall into portions with different characteristics.

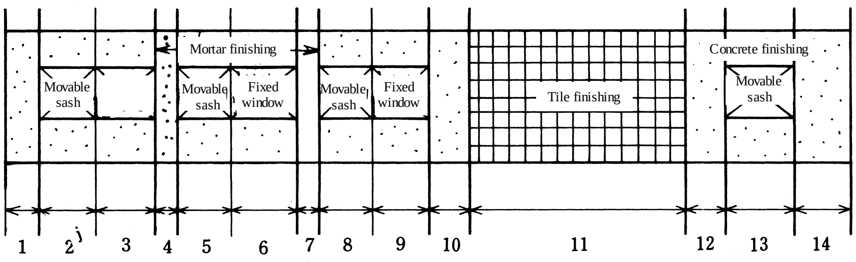


Figure TN.5-1 Division into unit portions of a wall with different vertical construction methods
(*quoted from the figure on page 167 in the commentary of 4.3.1 of the Standard of 2001 Japanese version*)

-- End of Translators' Note 5

Translators' Note 6 --

<u>Calculation Examples of *e, c, H*</u>

The following examples, quoted from the commentary of 4.3.4 of the Standard, shows how to estimate the human risk index *H*.

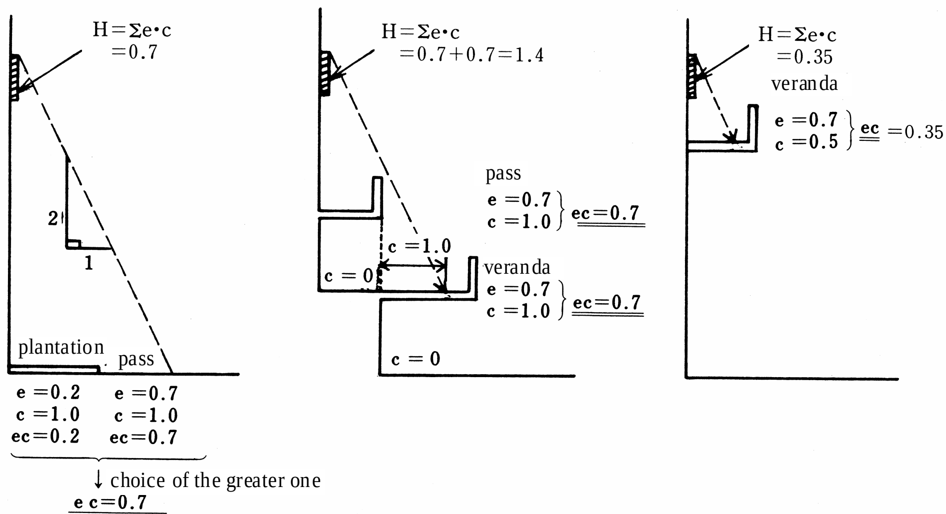


Figure TN.6-1 Calculation examples of *e, c, H*
(*quoted from the figure on page 171 in the commentary of 4.3.4 of the Standard of 2001 Japanese version)*

-- End of Translators' Note 6

Translators' Note 7 --

<u>C_T-S_D Criterion – *equation (39)*</u>

The equation (39) is derived from the relationship between the Japanese Screening method and the Japanese Building Code. The equation (39) is to ensure the structure, which satisfies the second and third level screening, has at least the required minimum story strength in the building code.

The criteria for each story of the structure can be defined as Eq. (TN.7-1) according to the Japanese Building Code.

$$Q_{un} \geq D_S \cdot F_{es} \cdot Q_{ud}$$
$$Q_{ud} = Z \cdot R_t \cdot A_i \cdot C_0 \cdot W \qquad \text{(TN.7-1)}$$
$$C_0 \geq 1.0$$

where:

Q_{un} = Calculated capacity of structure.

D_S = Deformability and damping factor of structure. The value is in the range of 0.3 to 0.55 according to the failure mode and structural system type of each story. The value becomes greater if the deformability or damping is smaller.

F_{es} = Shape factor to take the effect of vertical stiffness unbalance and eccentricity into account (greater than or equal to 1.0).

Q_{ud} = Seismic demand force for each story.

Z = Zone factor (0.7 to 1.0).

R_t = Coefficient for response in term of period and soil condition (less than 1.0).

A_i = Vertical distribution shape of lateral seismic force.

C_0 = Base shear coefficient (greater than 1.0).

W = Total weight of the story and above.

By replacing Q_{un}/W by story shear coefficient, C, Eq. (TN.7-2) can be derived from Eq. (TN.7-1).

$$\frac{1}{A_i} \cdot C \cdot \frac{1}{D_S} \cdot \frac{1}{F_{es}} \geq Z \cdot R_t \cdot C_0 \qquad \text{(TN.7-2)}$$

On the other hand, the criteria for the screening can be shown in the following fashion as described earlier.

$$I_S \geq I_{S0} \qquad \text{(TN.7-3)}$$
$$I_S = E_0 \cdot S_D \cdot T$$
$$= \frac{n+1}{n+i} \cdot C \cdot F \cdot S_D \cdot T$$
$$I_{S0} = E_S \cdot Z \cdot G \cdot U$$

From the comparison between Eqs. (TN.7-2) and (TN.7-3), followings can be pointed out;

1) $\frac{n+1}{n+i}$ and $\frac{1}{A_i}$, and C in each equation are essentially equivalent.

2) F and $\frac{1}{D_S}$ are equivalent.

3) S_D and $\frac{1}{F_{es}}$ are equivalent.

4) If G and U are 1.0, $Z \cdot R_t \cdot C_0$ and I_{S0} are equivalent.

Discussing on the regular shaped ($F_{es} = S_D = 1.0$) low-rise building ($R_t = 1.0$), if Z=1.0, U=1.0, G=1.0, C_0 =1.0, and I_{S0} =0.6, following equations can be derived from Eqs. (TN.7-2) and (TN.7-3).

$$C \cdot \frac{1}{D_S} \geq 1.0 \tag{TN.7-4}$$

$$C \cdot F \geq 0.6 \tag{TN.7-5}$$

Finally, Eq. (TN.7-6) can be derived as the relationship between D_S and F from Eqs. (TN.7-4) and (TN.7-5).

$$D_S = \frac{0.6}{F} \tag{TN.7-6}$$

Therefore, the D_S for the structure which satisfies the criteria for the second and third level screening can be calculated as 0.75 when the all members are categorized as the extremely brittle columns and the second-class prime elements (F=0.8), and calculated as 0.6 when members are categorized as the shear members (F=1.0).

It is obvious from Eq. (TN.7-6) that the D_S for the structure can be less than the required value in the building code if the F can be relatively large (greater than 2.0), even if the structure satisfies the criteria for the second and the third level screening. Therefore, the criterion of the equation (39) is defined in order to ensure that the structure has at least D_S of 0.3 that is the smallest required value in the building code.

The C_T is calculated for each member, and the S_D is calculated for each floor. Then C_T is grouped into at most three groups when E_0 is calculated. However, if the $C_T \cdot S_D$ for a group in the second and third level screening is less than the demand, the group cannot be taken into account.

This note is quoted from 5.2 of the commentary of the Standard of 2001 Japanese version.

-- **End of Translators' Note 7**

Guidelines

for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001

Chapter 1 General

1.1 Scope and Definitions

1.1.1 Scope

The Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001 referred to as “the guidelines” herein shall apply to the seismic retrofit design and construction of existing reinforced concrete buildings. The guidelines shall not apply in cases where design and construction have been performed based on special investigations. The items not mentioned in the guidelines are based on related standards and criterion such as the “Standard for Structural Calculation of Reinforced Concrete Structures” and the “Japanese Architectural Standard Specifications” published by the Architectural Institute of Japan (AIJ).

1.1.2 Definitions

The terminology used in the guidelines, unless specified otherwise, conform to the “Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001” by the Japan Building Disaster Prevention Association, hereafter referred to as “the Standard”, and the criterion and standard specifications related to other structural calculations and construction presented by AIJ.

1.2 Demand Performance for Seismic Retrofit

Demand seismic performance shall be clearly defined in the retrofit design. See the translators’ note 1.

1.3 Preliminary Inspection

When conducting retrofit design and construction planning, site investigation shall be conducted thoroughly. Meetings with building owner shall also be held to confirm various conditions related to retrofit work.

1.4 Design Procedure

Retrofit design shall follow the procedure of planning, structural design, detailed design, and evaluation of retrofit effect. The procedures shall be repeated when seismic performance cannot meet a demand performance.

1.5 Construction

The construction of retrofit work shall conform to the provisions in Chapter 4.

Chapter 2 Planning and Structural Design

2.1 Planning

2.1.1 General

When planning seismic retrofit, basic policy on how to meet the demand seismic performance by improving strength and/or ductility of building concerned shall be clearly defined. In addition, optimum retrofit methods for meeting demand performance shall be selected. An overall study shall be conducted at the planning stage considering building function after retrofit and workability of retrofit construction as well as performance upgrading by seismic retrofit. See the translators' note 2.

2.1.2 Retrofit design strategy

Reliable techniques whose upgrading effects are confirmed by structural tests or other investigations shall be adopted for seismic retrofit. Optimum techniques shall be adopted according to demand performance such as strength upgrading, ductility upgrading, and reduction of eccentricity, improvement of stiffness distribution or vulnerable spots, as well as condition of strengthening construction. The seismic performance of existing buildings shall be understood well for this purpose.

The layout of strengthening elements shall be properly planned in order to meet the demand condition for building function considering importance and use of building concerned. The strengthening elements shall be arranged in such a way that they can contribute to upgrading of seismic performance appropriately.

The influence of arranging the strengthening elements on the building function shall be minimized, e.g. by changing the use of strengthening part if necessary, in case there is a risk of disrupting the building function.

See the translators' notes 3 to 9 which provided many types of seismic upgrading methods and those effects.

2.2 Structural Design

2.2.1 General

The required seismic performance for upgrading shall be defined by difference between the demand performance and the performance of existing building concerned. Arrangements of the retrofit elements shall be planned based on the estimated amount of retrofit elements obtained from expected performance of selected retrofit method. When planning the arrangements of retrofit elements, seismic balance and influence on the building function shall be considered adequately.

2.2.2 Material strength

Material strength in the existing part used in retrofit design shall be the value which is confirmed by the site investigation. Strength of materials used in the retrofitting members or frames shall be the value which is provided in the related section of the guidelines. In case nothing particular is mentioned in the guidelines, the value provided in the Standard and related regulations and guidelines can be applied.

2.2.3 Required seismic performance and amount of retrofit members

Required amount of retrofit members shall be calculated according to the Standard and Chapter 3 of the guidelines.

2.3 Evaluation of Planning

Seismic performance of buildings to be retrofitted shall be evaluated according to the Standard, and it is confirmed that the buildings meet the criteria on demand seismic performance.

Chapter 3 Retrofit Design of Members and Frames

3.1 Installing Shear Walls

3.1.1 Outline

Installing shear walls is a retrofit method which is suitable to increase the strength of existing buildings by infilling new shear walls into open frames of existing buildings with inadequate seismic performance, filling up the opening of existing shear walls or increasing the thickness of existing shear walls. Necessary stress transfer mechanism between infilled shear wall and existing boundary frame shall be maintained by using joint devices like post-installed anchors or shear connectors (cotters), or joint methods like anchorage of wall reinforcing bars into the boundary frame or welding those with existing reinforcing bars.

In case installing shear walls, it shall be recognized that shear strength of infilled walls can not be fully expected when flexural strength including boundary frame or uplift strength of walls are smaller than shear strength of walls. Safety of foundation and ground shall be considered in the planning stage against increase in dead load by installing walls and change of axial force during earthquake due to change of failure mechanism caused by the retrofit. It is possible to improve structural balance index (S_D index) in case a building with soft-first story (pilotis) and/or large eccentricity since stiffness is extremely increased by infilling shear walls. Great care is required in the retrofit design and construction since strength and failure mechanism of infilled shear walls are highly influenced by the casting method of infilled concrete.

3.1.2 Demand performance

(1) Structural performance of wall members

Installed shear walls shall be designed so that the capacity of retrofitted building shall meet the demand capacity. However, if the expected increase in strength can not be obtained due to the strength limit determined by the flexural strength including boundary frame or uplift strength of walls, the strengthening member shall be designed to have appropriate ductility as well as strength.

Expected strength of infilled shear walls is $\tau = 0.25\ F_c$ (τ is the average shear stress of wall in the clear span of columns, F_c is compressive strength of existing concrete) in case of walls without opening, and this value shall be reduced according to the condition in case of walls with opening. Different ductility which can be expected due to the failure mode is provided. Ductility factor F is set as follows according to the section 3.2.3 of the Standard.

(i) Shear failure mode ----- 1.0

(ii) Flexural failure mode ----- 1.0 - 2.0

(iii) Foundation uplift mode ----- 1.0 - 3.0

(2) Structural performance of buildings

Buildings with infilled shear wall generally aim to be strength resistance type structures whose strength is much higher than external forces. However, as indicated in “(1) Structural performance of wall members”, there are some cases that are difficult to be strength resistance structures. In these cases, it shall be aimed to be ductility resistance type structures that dissipate input energy by its large deflection capacity after yielding. The infilled walls are designed as flexural failure mode or foundation uplift mode, and sum of the strengths with other flexural failure mode members is higher than demand strength.

3.1.3 Planning

(1) Buildings suitable for this strengthening method

Buildings to be strengthened with shear walls are those with poor lateral strength or those with brittle members failing in shear. It is effective to apply to buildings with dominant flexural members without high strength in case the strength of infilled shear wall is determined by flexural strength or uplift strength of foundation, by using its ductility effectively.

Wall shall be installed in buildings which may be less restricted in utilizing subdivided inner space and barriers against function or lighting. It is applied to buildings with enough supporting strength of foundation since infilled wall may cause increase in dead load and significant change of axial force during earthquake due to change of resistance mechanism.

(2) Installing position

Walls are recommended to be installed in a proper position considering the restriction of building utilization and a good structural balance in plan and elevation.

3.1.4 Construction method and structural details

(1) Construction method

(a) Strengthening by installing new shear wall

It is a strengthening method to fill a bare frame or a frame having window opening with shear wall. It mainly increases the strength of building. However, it is necessary to consider deeply the structural characteristics of whole buildings in the retrofit design, since the strength and restoring force characteristics of infilled shear wall will be changed due to flexural yielding of boundary frame or uplift of foundation.

(b) Strengthening by increasing thickness of existing shear wall

It is a strengthening method to increase thickness of existing shear wall. The design philosophy for infilled shear wall can be referred in the retrofit design because structural behavior of thickness-increased shear wall is similar to that of infilled shear wall. The cast-in-situ concrete shall be completely adhered with existing wall, beams and columns connected with thickness-increased wall.

(c) Strengthening by infilling of opening in existing shear wall

In case a frame with hanging wall, standing wall and wing wall having relatively small opening, existing walls can be effectively used. Those can be treated as a infilled shear wall by closing of opening with cast-in-situ concrete if the existing wall’s thickness is 15 cm or more and concrete strength of existing building is 15 N/mm^2 or more. See the translators’ note 10.

(2) Structural detail

(a) Joint methods with existing structures

(i) Joint method using post-installed anchor

It is a joint method to place post-installed anchors in the existing structures to transfer shear force between existing structures and infilled walls. See the translators' notes 11 and 12.

1. Structural detail of post-installed anchor shall follow the items shown below and in the section 3.9.
2. In general, post-installed anchors are placed along boundary columns and beams. However, they can be placed only along boundary beams considering strength reduction.
3. Strengthening against concrete splitting shall be sufficiently provided by using spiral hoops or ladder-shaped reinforcing bars.
4. Surface of the existing wall attached to the infilled wall shall be roughened in the case of retrofitting by increasing thickness of existing wall.

(ii) Other joint methods

There are following joint methods other than above-mentioned method. See the translators' notes 13 and 14.

1. joint with chipped cotter
2. joint with adhesive cotter
3. welding joint of reinforcing bars, welding joint using steel plates, welding joint using hooked bars

When using these joint methods, it is recommended to study the structural performance of the joints by structural or construction tests, if necessary.

(b) Construction methods for splitting prevention

Reinforcing bars for splitting prevention shall be sufficiently provided at or nearby the reinforcing bars to be anchored.

(i) Spiral hoop

(ii) Ladder-shaped reinforcing bar

(iii) Others

(c) Remarks on structural details

Followings are the common structural details for each joint method.

(i) Thickness of the infilled wall shall be 1/4 of column width or more, 15 cm or more, but less than beam width.

(ii) Shear reinforcement ratio of infilled wall shall be 0.25% or more but not more than 1.2% or less. Double layer reinforcement shall be arranged in the cross section in case the wall thickness is 18 cm or more.

(iii) Specified concrete strength of installed walls shall not be less than the concrete strength of existing structures.

(iv) Reinforcing bars around opening shall be designed to meet the strength of the wall when providing an opening in the installed wall.

(v) Thickness of the added wall shall be the thickness of existing wall or more, and 12 cm or more in case retrofitting by increasing wall thickness. Construction methods of infilled wall shall be as follows.

1. Casting concrete with pressure.
2. Casting concrete of infilled wall up to around 20 cm below the beam, and grouting with pressure the rest part

See the translators' note 15.

3.1.5 Design procedure

(1) Retrofit procedure

The retrofit procedure of infilled shear wall shall be as follows.

(a) Investigate the seismic capacity of the object building for retrofitting

(b) Determine the retrofitting policy, whether the building resist by strength or ductility

(c) Set the retrofitting demand due to the retrofitting policy

(d) Assume the design stress of wall panel and the specified design strength of materials

(e) Determine the wall arrangements based on the required wall length which is calculated using an assumption of wall thickness

(f) Calculate the amount of shear reinforcement of walls and design the joint reinforcements

(g) Calculate the ductility index using the calculated strength of infilled walls

(h) Judge whether the retrofitting demand is satisfied or not

When the retrofitting demand is satisfied and the retrofit is not too much, retrofit calculation is finished. In case the retrofitting demand is not satisfied or the retrofit is too much, recalculate from (e) or (f).

(2) Design of infilled wall panel

Design of infilled wall panel shall be as follows.

(a) Determine expected ductility index F of infilled wall and design shear force Q_D based on F.

(b) Determine the wall thickness in order that the average shear stress of wall panel τ_w obtained from Q_D is smaller than the value provided in Table 3.1.5-1.

$$\tau_W = Q_D / (t_W \cdot l_W) \leq \tau_D \qquad (3.1.5\text{-}1)$$

where:

τ_W = Average shear stress of wall panel (N/mm^2).

t_W = Wall thickness (mm).

l_W = Clear span of wall (mm).

τ_D = Values provided in Table 3.1.5-1.

Table 3.1.5-1

F value	Upper limit of τ_D
$3 \geqq F > 2$	$0.16F_c$
$2 \geqq F > 1$	$0.20F_c$
$F = 1$	$0.25F_c$

where, F_c =specified design strength of concrete (N/mm^2).

(c) Determine the amount of shear reinforcement to satisfy the following condition.

$$\beta \cdot Q_{Wu} \geq Q_D \quad (3.1.5\text{-}2)$$

where:

β = 0.9-1.0 (in case post-installed anchors are arranged along four sides of wall panel), and 0.8-0.9 (in other cases).

Q_{Wu} = Ultimate shear strength of wall calculated from Eq. (A2.1-2) shown in Supplementary Provisions 2.1 (2) (b) of the Standard.

(3) Calculation equations for member strength and ductility index of infilled walls

(a) Strength of the infilled shear walls shall be the minimum value of the strengths indicated as following (i), (ii) and (iii).

(i) Shear strength

It shall be the minimum value of following 1 and 2.

1. 80 - 90% of the calculated shear strength which is obtained assuming that the wall panel and the boundary frame (column and beam) are monolithically cast. The value can be 90 - 100% of the calculated shear strength if post-installed anchors are set along all interfaces between new shear band and existing boundary frame.
2. Ultimate strength which is calculated by the sum of shear strengths of the installed shear panels, a shear strength of the connections, and column strength, considering failure mechanism expected under seismic excitation.

(ii) Flexural strength including boundary frames

(iii) Uplift strength including boundary frames

(b) Calculation equations of each strength are as follows. When the installed wall has opening in both the case of (i) and (ii), the strength shall be reduced according to Eq. (A2.1-2) in Supplementary Provisions 2.1 (2) (b) of the Standard. If the opening is larger than the provision, the strength shall be calculated assuming it consists of columns with wing wall.

(i) Shear strength of monolithic walls

It shall be calculated by using Eq. (A2.1-2) in Supplementary Provisions 2.1 (2) (b) of the Standard. This equation can also be used for the existing shear walls in the case of retrofitting by increasing thickness of existing walls.

(ii) Strength of the shear walls which are connected with existing boundary frames by using connector such as post-installed anchors or shear cotters.

It shall be calculated by using the following equation in consideration of the load carrying mechanism at the connectors, wall panels and columns.

$$ {}_{W}Q_{su} = \min\left\{ {}_{W}Q'_{su} + 2\cdot\alpha\cdot Q_c\,,\ Q_j + {}_{p}Q_c + \alpha\cdot Q_c \right\} \qquad (3.1.5\text{-}3)$$

where:

${}_{W}Q_{su}$ = Shear strength of shear walls.

${}_{W}Q'_{su}$ = Shear strength of infilled shear panel (only for the panel part in the clear height and width).

Q_j = Sum of the shear strengths of connectors underneath the beam.

${}_{p}Q_c$ = Direct shear strength at the top of a column.

Q_c = Smaller value of the other column between the shear force at the yielding and shear strength.

α = Reduction factor in consideration of the deflection condition to allow for load bearing contribution of column(s). Following value can be used, in case without detailed study.

1.0 – in the case of shear failure of columns

0.7 – in the case of flexural failure

(iii) Flexural strength of shear walls

It shall be calculated by using Eq. (A2.1-1) in Supplementary Provisions 2.1 (2) (a) of the Standard. The strength contributed by wall reinforcing bars in this equation shall not exceed the pull-out strength of anchors if the wall panel is connected with beams by using post-installed anchors. It shall be 0 if the shear cotter connection is used.

(iv) Uplift strength of shear walls

It shall be calculated according to the provisions in the section 3.2.2 (3) of the Standard.

(v) Flexural strength and shear strength of columns, columns with wing walls, walls with columns and beams.

It shall be calculated according to the provisions in the sections 3.2.2 (2) and (3) of the Standard.

(vi) Shear strength of infilled shear panel (only for the panel part in the clear height and width)

It shall be calculated by the following equation.

$$ {}_{W}Q'_{su} = \max\left(p_w \cdot {}_{W}\sigma_y, F_{cw}/20 + 0.5 p_w \cdot {}_{W}\sigma_y\right)\cdot t_W \cdot l' \qquad (3.1.5\text{-}4)$$

where:

$p_w, {}_W\sigma_y$ = Wall reinforcement ratio and yield strength of the wall reinforcing bar (N/mm^2).

F_{cw} = Concrete strength of the installed wall panels (N/mm^2).

$t_W \cdot l'$ = Wall thickness and clear span of the installed wall panel (mm).

(vii) Direct shear strength of columns

It shall be calculated by the following equation.

$${}_pQ_c = K_{\min} \cdot \tau_o \cdot b_e \cdot D \qquad (3.1.5\text{-}5)$$

where:

$K_{\min}$ = $0.34/(0.52 + a/D)$.

τ_o = $0.98 + 0.1F_{c1} + 0.85\sigma$ in case $0 \le \sigma \le 0.33F_{c1} - 2.75$

$0.22F_{c1} + 0.49\sigma$ in case $0.33F_{c1} - 2.75 < \sigma \le 0.66F_{c1}$

$0.66F_{c1}$ in case $0.66 < \sigma$

b_e = Effective width of columns resisting against the direct shear force considering the connected members in the orthogonal direction.

D = Depth of columns resisting against the direct shear force.

a = Shear span; distance between the beam face at the column top and the point of lateral force from the infilled wall.

F_{c1} = Specified concrete strength of existing structures (N/mm^2).

σ = $p_g \cdot \sigma_y + \sigma_o$.

p_g = Ratio of a_g (gross cross section area of longitudinal reinforcing bars of a column concerned) to $b_e \cdot D$.

σ_y = Yield strength of longitudinal reinforcing bars of a column.

σ_o = $N / (b_e \bullet D)$, where N is an axial force of the column at ultimate mechanism, positive value means compression force.

(viii) Detail and strength of connector

1. Post-installed anchor

Arrangement, shear strength and tensile strength of post-installed anchor shall be determined according to the provisions in the section 3.9 of the guidelines.

2. Cotter

Details and strengths of chipped cotter and adhesive cotter shall be determined due to test, in principle.

(c) Ductility index of infilled shear wall

Ductility index of infilled shear wall shall be calculated according to the provisions in the section 3.2.3 of the Standard.

(4) Performance evaluation of strengthened buildings

Performance evaluation of strengthened buildings shall be made according to the provisions in the sections 1.2 and 2.3 of the guidelines. Stiffness of the infilled wall shall be evaluated by reducing stiffness of the monolithic shear wall appropriately.

3.2 Installing Wing Walls

3.2.1 Outline

This strengthening method is to install small wall panels which may not be considered shear walls with boundary columns. The objective of this strengthening method is to increase seismic performance of existing buildings by changing the existing independent columns to columns with wing wall for upgrading their strength. It is also possible to install wing walls to carry axial load of a column and to eliminate a problem of second-class prime elements, whose failure leads to building collapse.

However, there is a case that the seismic capacity of building is determined by the performance of existing beams, even though seismic performance of column is upgraded by installing wing walls. Thus, it shall be counted in the design.

Especially, buildings with short distance in beams shall be carefully designed to eliminate shear failure in beams due to beam shortening after installing walls adjacent to columns.

It is possible by installing wing walls to upgrade the second-class prime elements, whose failure leads to building collapse. For example, installing wing wall in the direction of lateral load concerned to increase its strength is an effective retrofitting method when they are extremely brittle columns. To enhance the axial load carrying capacity of a column, wing walls are often provided in the direction perpendicular to the lateral load concerned.

3.2.2 Demand performance

(1) Demand performance of retrofitted building

Demand performance on seismic safety of retrofitted building is determined according to the provisions in the section 1.2 of the guidelines.

In the following are two approaches to meet the demand performance in the case of installing wing walls; (1) being strength resisting type by upgrading strength index C, or (2) upgrading seismic performance by increasing ductility index F through the formation of beam yielding.

(2) Demand performance of columns with installed wing walls.

In either case that the frame will resist by strength or by ductility through the formation of beam yielding, the demand of columns with installed wing walls is to increase their strength. Thus enough width and thickness shall be provided in the installed wing walls.

3.2.3 Planning

(1) Buildings suitable for this strengthening method

This strengthening method is suitable for a building that can achieve the sufficient increase in lateral load-carrying capacity by increasing the column strength, when the shear-failure-type columns resist large lateral force and beams have enough strength. This strengthening method can be also applied to a building that can upgrade seismic performance by changing its failure mechanism from the column yielding to the beam yielding, when the flexural-yielding-type columns are predominant but their ductility is not expected, or unacceptablly large deflections are expected even if the ductility is upgraded (building with extremely small strength index *C*).

The clear span of the beam decrease by installing wing walls. Flexural yielding of beams after the reduction of their clear-spans shall be expected for securing ductility. Thus this strengthening method is generally suitable for the frame with large span.

(2) Members to be strengthened

(a) Most of the columns will be strengthened since this retrofitting method is mainly aiming to increase the strength of columns. The wing walls shall be installed with good balance in plan and elevation. The installation of wing walls causing an unbalanced distribution of stiffness or strength between frames after strengthening shall be avoided.

(b) It is desirable that l_o/D, ratio of clear span of beam l_o to beam depth D after installing the wing wall as shown in Figure 3.2.3-1, would be 4 or more, in case that the wing walls are installed to achieve the beam-yielding failure mechanism. It shall be confirmed by calculation that the beam will yield without shear failure.

(c) Installing wing wall can be applied to a short column, the clear height which is shortened due to standing wall and hanging wall attached on and under the beams. However, enough study on the strength of beams with standing wall and/or hanging wall is required when the retrofitting method is applied.

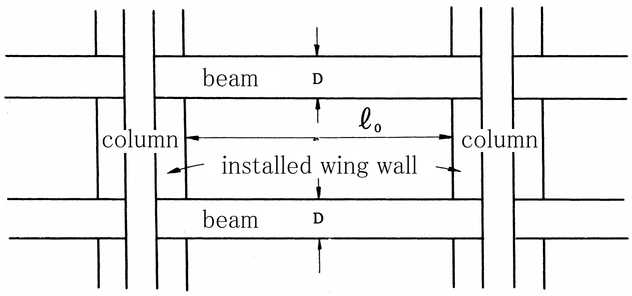


Figure 3.2.3-1

3.2.4 Construction method and structural details

(1) Construction method of installing wing walls

Construction methods of installing wing walls can be roughly classified into two methods, (a) cast-in-situ wing wall, and (b) precast wing wall which connects to existing column on site. In the construction method (a), there are two cases for connection, use of post-installed anchors and welding reinforcing bars of wing wall with those of existing structure. In the construction method (b), post-installed anchors are used for the connection.

The general construction methods for installing wing wall are based on Figures 3.2.4-1 and 3.2.4-2. Figure 3.2.4-1 indicates a case that the reinforcing bars of wing wall connect to the existing structure by using post-installed anchors. It is the most typical connection methods for installing wing wall. Although this method may allow higher flexibility in determining the position of wing wall to be fastened to the existing column, it is desirable that the centerline of installed wing wall is the same as that of column.

The amount of anchorage reinforcing bars shall be determined so as to be able to transfer the axial force in the lateral reinforcing bar of wing wall. The reinforcement to prevent splitting is required in the concrete.

The contribution of wing wall in tension side is not expected in the equation for predicting flexural strength of columns with wing wall. However, since large stress will occur at the end of wing wall, the vertical reinforcing bar at the end of wing wall shall be detailed to securely transfer the stress to the existing beam.

Figure 3.2.4-2 indicates the case that reinforcing bars of wing wall are welded to those of existing structure. This is the method that the lateral reinforcing bars of wing wall are placed on a side of existing column and welded to hoops of the column. The wing wall are therefore added to the column eccentrically. This method is advantageous in that the continuous lateral reinforcing bars at least on one side of wing wall can transfer the stress directly. However, the lateral reinforcing bars on the other side is necessary to connect to the column by post-installed anchors. The vertical reinforcing bars at the end of wing wall shall be connected to the existing beam to securely transfer the stress.

When the precast wing wall is connected to existing column on site, mortar or concrete is injected into the gap of the connection part. As for the grouting method of mortar or concrete, strengthening method by steel frame in the section 3.5 of the guidelines can be used.

In each construction method above, sufficient studies on water proof at the connection surface to the existing column is required if the wing wall is installed on exterior frames. The post-installed anchors shall be embedded in at the core concrete of existing columns and beams enclosed by transverse reinforcing bars.

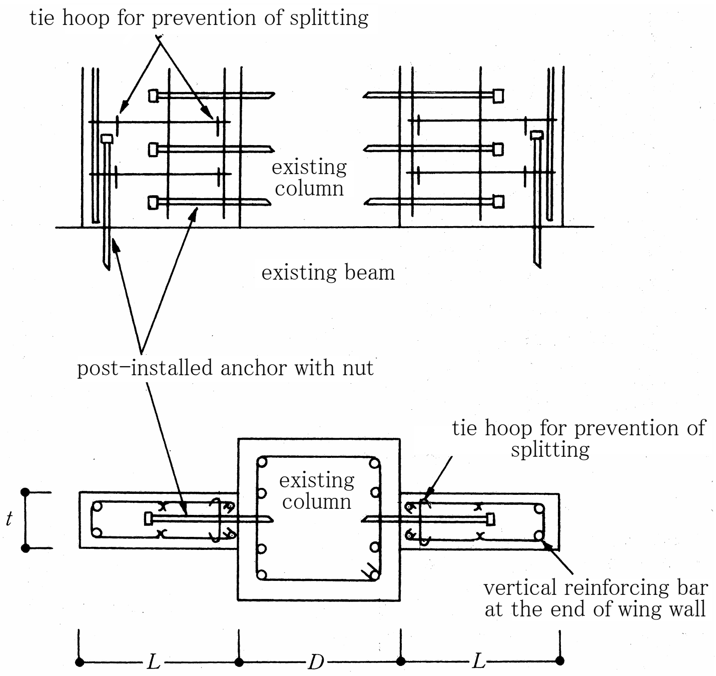


Figure 3.2.4-1

(2) Structural detail

It is recommended to follow the subsequent provisions in case of strengthening by installing wing wall.

(a) In general, wing walls shall be arranged symmetrically on both sides of column.

(b) In case of cast-in-situ method of wing wall concrete, minimum width of a wing wall L shall be smaller of 1/2 of column depth D and 500mm, and maximum width of L shall be 2 times of column depth D. Wall thickness t shall not be less than 1/3 of column width b and 200mm.

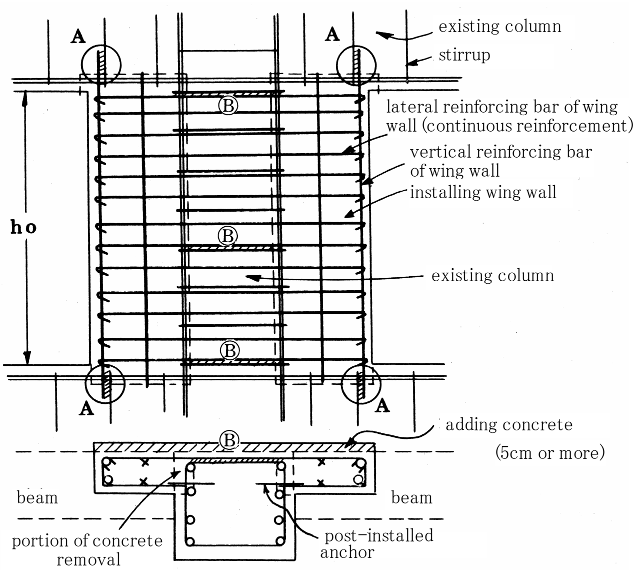
(c) In case of the precast wing wall connection to existing column on site, the ratio of width L to height of a wing wall (L/h_0) shall not be less than 1/3, L shall not be less than 800 mm, and wall thickness t shall not be less than150mm.

(d) The reinforcement ratio of vertical and lateral reinforcing bars of wing wall p_{sv}, p_{sh} shall not be less than 0.25%.

(e) Arrangement of post-installed anchor shall be made according to the provisions in the section 3.9 of the guidelines.

(f) Concrete cover from reinforcing bar of wing wall shall follow the AIJ Standard for Structural Calculation of Reinforced Concrete Structures. Additionally, it is better to cast the concrete to increase the thickness of wing wall at the portion of removal of concrete in the existing column as shown in Figure 3.2.4-2Ⓑ, in case of cast-in-situ wing wall concrete.

(g) Lateral reinforcing bars of wing wall shall be a closed shape if the wing wall is designed to be an axial force supporting member for the column identified as a second-class prime elements, whose failure leads to building collapse.



Ⓐ : Both ends of vertical reinforcing bar s near the outermost section of the wall shall be securely welded to existing stirrup s
Ⓑ : Lateral reinforcing bars shall be welded to existing hoops with the interval of 50cm or less

Figure 3.2.4-2

3.2.5 Design procedure

(1) Retrofit procedure

The retrofit procedure of installed wing wall shall be as follows.

(a) Demand value of retrofit shall be established referring to the results of seismic evaluation.

(b) Construction method and its detail of the installing wing walls shall be pre-determined.

(c) Strengths of the columns on which the wing wall is installed and the beams linking to the columns shall be calculated.

(d) The failure mechanism of the sub-assemblage consisting of the column on which the wing wall is installed and the beam linking to the columns shall be investigated. The index of basic seismic capacity E_o of the sub-assemblage after installing the wing wall shall be calculated in accordance with the third level screening method of seismic evaluation.

(e) It shall be judged that whether or not the calculated index for structural seismic performance ${}_RI_S$ meets the demand performance for retrofit. When the demand performance is not satisfied, strengthening shall be increased or the design details shall be changed. And retrofit calculation shall be performed again from (c) in this procedure.

(2) Load carrying capacity of strengthened member

(a) Load carrying capacity of the columns with wing wall which is cast-in-situ unifying with existing structures shall be the smaller value of shear force at flexural strength Q_{mu} and shear strength Q_{su} indicated as follows. Those equations are used for the case when two wing walls attached on both sides of the column. Thus in case that only one wing wall is attached to the column, the contribution of wing wall in tension side shall be ignored.

$$Q_{mu} = \phi \cdot M_u / h' \tag{3.2.5-1}$$

$$M_u = (0.9+\beta)\cdot a_t \cdot \sigma_y \cdot D + 0.5N \cdot D\left\{1 + 2\beta - \frac{N}{\alpha_e \cdot b \cdot D \cdot F_{c1}}\left(\frac{a_t \cdot \sigma_y}{N} + 1\right)^2\right\} \tag{3.2.5-2}$$

where:

α_e = $(1+2\alpha\cdot\beta)/(1+2\beta)$, α and β shall be referred to Figure 3.2.5-1.

h' = Eq. (11) in the Standard.

ϕ = Reduction factor (= 0.8).

a_t = Gross sectional area of longitudinal reinforcing bars of column in tension side (mm^2).

σ_y = Yield strength of longitudinal reinforcing bars of column (N/mm^2).

N = Axial force of column (N).

F_{c1} = Specified design strength of concrete for wing wall (N/mm^2).

b, D = Width and depth of column, respectively (mm).

$$Q_{su} = \phi\left\{\frac{0.053\cdot p_{te}^{0.23}\cdot(F_c+18)}{M/(Q\cdot d_e)+0.12} + 0.85\sqrt{p_{we}\cdot\sigma_{wy}} + 0.1\sigma_{oe}\right\}\cdot b_e \cdot j_e \tag{3.2.5-3}$$

Generally, $M/(Q\cdot d_e)$ shall be in the range of 1.0 to 2.0. A value of 0.5 can be used instead of 1 mentioned above, considering the member configuration, bar arrangement and boundary condition on confinement.

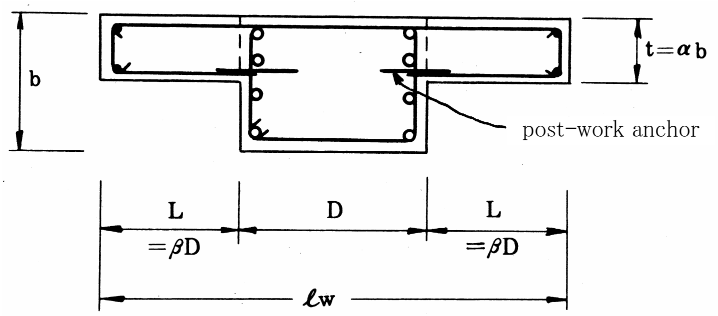


Figure 3.2.5-1

and,

$$p_{we} \cdot \sigma_{wy} = p_w \cdot \sigma_{wy}(b/b_e) + p_{sh} \cdot \sigma_{sy}(t/b_e)$$

$$\sigma_{oe} = N/(b_e \cdot j_e)$$

where,

ϕ = Reduction factor (= 0.8).

F_c = Specified design strength of concrete for existing structure (N/mm^2).

M/Q: it can be h' of column on which the wing wall is installed;

d_e = Distance between the center of the tensile reinforcing bars and the extreme fiber of wing wall in compression side (mm).

$p_w \cdot \sigma_{wy}$ = Product of hoop ratio and its yield strength in the existing columns(N/mm^2).

$p_{sh} \cdot \sigma_{sy}$ = Product of lateral reinforcement ratio of installed wing wall and its yield strength(N/mm^2).

$b_e = \alpha_e \cdot b$ (mm)

t = Wall thickness of installed wing wall (mm).

j_e = $7d_e/8$ (mm).

$p_{te} = 100a_t/(b_e \cdot d_e)$ (a_t: gross sectional area of tensile reinforcement of the column with installed wing wall).

(b) Columns with precast concrete wing wall

(i) Load carrying capacity Q_u of column with precast concrete wing wall can be calculated by Eq. (3.2.5-4). This equation considers the shear force Q_T contributed by the diagonal compression brace which models a wing wall as shown in Figure 3.2.5-2, and the shear force Q_C contributed by the existing columns.

$$Q_u = Q_T + Q_c \tag{3.2.5-4}$$

(ii) The shear force Q_T transferred by the truss model which assumes the wing wall as the diagonal compression brace shall be the smallest value of Q_{T1}, Q_{T2} and Q_{T3} indicated in Eqs. (3.2.5-5) and (3.2.5-6). Where, Q_{T1} is a shear force based on the compressive strength of diagonal brace, Q_{T2} is a shear strength of connections at top and bottom of wing wall and Q_{T3} is a shear strength of wing wall.

$$Q_{T1} = 2\alpha_B \cdot t^2 \cdot f_c \cdot (L_1 / L_2) \leq 2(N + a_g \cdot \sigma_y) \cdot (L_1 / H) \tag{3.2.5-5}$$

$$Q_{T2} = Q_A + 0.25\alpha_B \cdot t^2 \cdot f_c \cdot (H / L_2) \tag{3.2.5-6}$$

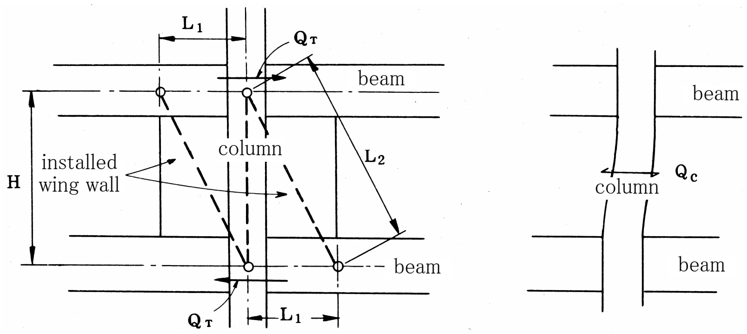


Figure 3.2.5-2

where, Q_A is a lateral shear force transferred by the post-installed anchor at the top and bottom of wing wall, which shall be calculated according to the methods in the section 3.9.

$$Q_{T3} = \sum A_W (f_s + 0.5 p_{sh} \cdot \sigma_{sy}) \tag{3.2.5-7}$$

where:

α_B = Effective width of the diagonal brace when wing wall is modeled as compression element. α_B can be 2.0 unless a special investigation is made.

t = Thickness of wing wall (mm).

N = Axial force of column due to gravity load (N).

$a_g \cdot \sigma_y$ = Product of gross sectional area of longitudinal reinforcing bars of column and their yield strength (N).

L_1, L_2, H = See Figure 3.2.5-2.

$f_c = 0.85 F_{c1}$ (N/mm^2).

F_{c1} = Specified design strength of precast concrete of installed wing wall (N/mm^2).

$\sum A_W$ = Lateral cross section area of wing walls on both sides of column (mm^2).

$p_{sh} \cdot \sigma_{sy}$ = Product of lateral reinforcement ratio of installed wing wall

($p_{sh} \leqq 1.2$ %) and its yield strength (N/mm^2).

f_s = Temporary allowable stress of concrete of precast wing wall (N/mm^2), according to the AIJ Standard for Structural Calculation of Reinforced Concrete Structures.

(iii) The shear force Q_C contributed by the existing columns used in Eq. (3.2.5-4) shall be calculated by the following equation.

$$Q_c = \min\left(\alpha_1 \cdot Q_{mu}, \alpha_2 \cdot Q_{su}\right) \tag{3.2.5-8}$$

where, Q_{mu} and Q_{su} are shear force at the flexural strength and shear strength of existing column, respectively. They shall be calculated according to the equations in Supplementary Provisions 1.1 of the Standard. The axial force N used in the calculation of Q_{mu} and Q_{su} shall be N (long term)$- Q_T / 2\left(H / L_1\right)$ (N=0 when N<0).

(c) Strength of existing beams

Strength of existing beams shall be calculated in accordance with the equations in Supplementary Provisions 4 of the Standard.

(d) Ductility index of columns with installed wing wall

Ductility index F of columns with wing wall is determined according to the section 3.2.3 (3) of the Standard. It can be 1.0 unless a special investigation is made. If the failure mechanism due to beam yielding is developed due to the presence of wing walls, F shall be 1.0 - 3.0 based on the failure pattern of sub-assemblage consisting of a column with installed wing wall and the beam linking to the column.

(3) Performance evaluation of strengthened buildings

Performance evaluation of strengthened buildings shall be done according to the provisions in 1.2 and 2.3.

3.3 Column

3.3.1 Outline

The strengthening methods indicated in this section aim to upgrade seismic performance of buildings by increasing ductility, lateral load carrying capacity or axial load carrying capacity through jacketing the existing columns or isolating the existing columns from the adjacent spandrel walls.

It is necessary to clearly define the retrofitting objective and adopt the appropriate construction methods and details since the type of construction methods and details vary with retrofitting objective of columns.

3.3.2 Demand performance

(1) Necessary strength and ductility index (F) of columns shall be set based on the demand performance of buildings concerned.

(2) In case the improvement of deflection capacity is an objective in retrofitting columns, necessary ductility index F_0 of columns to be retrofitted shall be defined as a demand performance based on the required ductility index F of retrofitted building. The required shear capacity ${}_{req}Q_{su}$ shall be determined based on the necessary ductility index F_0 according to the Standard and the amount of retrofit shall be calculated applying the equations provided in the sections 3.3.4 and thereafter of the guidelines for each strengthening method.

3.3.3 Planning

Retrofitting of columns shall be performed based on the failure mechanism of existing frames obtained by the seismic evaluation. Appropriate strengthening shall be planned for columns whose failure may reduce in seismic performance of overall building.

The buildings whose seismic performance could be effectively improved by retrofitting columns are as follows.

(1) Building with shear-failure type columns as second-class prime elements, whose failure leads to loss of its seismic performance.

(2) Building with relatively strong and stiff frames, small amount of walls and shear-failure type columns.

(3) Building with soft story.

3.3.4 RC jacketing

(1) Outline of retrofit method

(a) Basic specification

RC jacketing is a strengthening method by jacketing around the existing columns with reinforced concrete or reinforced mortar, whose thickness is around 10 to 15 cm. This method is used to upgrade ductility by increasing the shear strength of the column or to upgrade flexural strength and axial strength as well as ductility. It is necessary to follow appropriate specifications according to upgrading objectives. See the translators' note 16.

(b) Retrofit for improving ductility

When ductility of columns is planned to improve by this method, a slit with 30 to 50 mm in width shall be provided at the top and bottom of the column jacketing, in principle.

(c) Retrofit for improving ductility and strength

When ductility and strength of columns is planned to improve by this method, RC jacketing portion shall continue to columns through the floor slab in the lower and upper stories in principle. Appropriate details otherwise shall be made such that longitudinal bars of columns be anchored to the panel zone above and below the strengthened column. See the translators' note 17.

(d) Retrofit in case standing wall or hanging wall connect to the column

When a thin standing and/or hanging wall is connected to a column and the jacketing may cause damage to the wall, the column shall be isolated from the wall and strengthened over the full length including isolated region adjacent to the wall. See the translators' notes 18 and

19.

(e) Retrofit in case the walls attach to the column perpendicular to the direction concerned

When a column is strengthened by the RC jacketing method, in principle, all faces of the column shall be strengthened. Thus, if walls attach to the column, jacketing shall be done after removal of a part of the wall adjacent to the column, or strengthening shall be done with the detail to obtain the same effect as jacketing all faces of the column. See the translators' note 20.

(2) Design procedure

(a) Flexural strength of column

(i) In case of upgrading ductility

Flexural strength of RC jacketed columns with slit at their top and bottom to improve ductility shall be calculated by Eq. (3.3.4-1) as provided in the Standard. (Refer to Supplementary Provisions 1.1 (2) of the Standard).

When $0.4b \cdot D \cdot F_{c1} \geq N \geq 0$,

$$M_u = 0.8a_t \cdot \sigma_y \cdot D + 0.5 \cdot N \cdot D \cdot \left(1 - \frac{N}{b \cdot D \cdot F_{c1}}\right) \tag{3.3.4-1}$$

where:

a_t = Cross sectional area of tensile longitudinal reinforcement (mm^2).

σ_y = Yield strength of reinforcing bar (N/mm^2). The strength σ_y shall be 294 N/mm^2 for round bars, and (specified yield strength + 49 N/mm^2) for deformed bars.

b = Width of column (mm).

D = Depth of column (mm).

N = Axial force of column (N).

F_{c1} = Compressive strength of concrete for existing structures (N/mm^2).

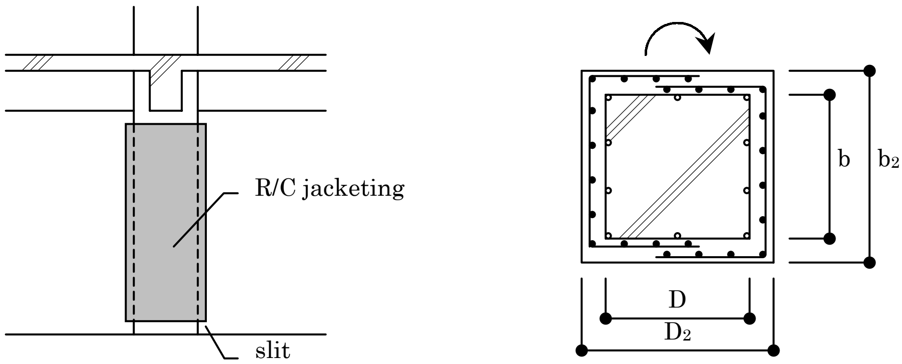


Figure 3.3.4-1 Flexural strength in case of setting a slit

(ii) In case of upgrading strength

Flexural strength of RC jacketed columns to improve their flexural strength by adding flexural reinforcement shall be calculated by Eq. (3.3.4-2). (Refer to Supplementary Provisions 1.1 (2) of the Standard)

When $0.4b \cdot D \cdot F_{c1} \geq N \geq 0$,

$$M_u = a_t \cdot \sigma_y \cdot g + a_{t2} \cdot \sigma_{y2} \cdot g_2 + 0.5 \cdot N \cdot D_2 \cdot \left(1 - \frac{N}{b_2 \cdot D_2 \cdot F_{c1}}\right) \tag{3.3.4-2}$$

where:

g = Distance between tensile and compressive longitudinal reinforcement of existing column (mm).

g_2 = g for jacketing part of the column (mm).

a_{t2} = Cross sectional area of tensile reinforcement in the jacketing part of column.

σ_{y2} = Yield strength of tensile reinforcement in the jacketing part of column (N/mm^2). The strength σ_{y2} shall be 294 N/mm^2 for round bars, and (specified yield strength + 49 N/mm^2) for deformed bars.

b_2 = Width of column after jacketing (mm).

D_2 = Depth of column after jacketing (mm).

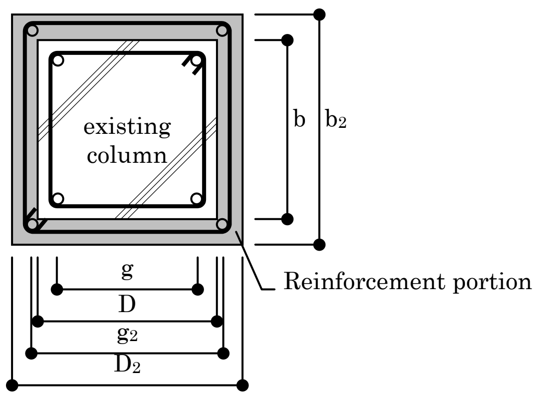


Figure 3.3.4-2 Explanation of notations on column section

(b) Shear strength of column

Shear strength of column retrofitted by RC jacketing shall be calculated by Eq. (3.3.4-3). (Refer to Supplementary Provisions 1.1 (3) of the Standard)

$$Q_{su} = \phi \left\{ \frac{0.053 \cdot p_{t2}^{\ 0.23} \cdot (F_{c1} + 18)}{M / (Q \cdot d_2) + 0.12} + 0.85 \sqrt{p_w \cdot \sigma_{wy} + p_{w2} \cdot \sigma_{wy2}} + 0.1 \frac{N}{b_2 \cdot D_2} \right\} \times 0.8 \cdot b_2 \cdot D_2$$

(3.3.4-3)

$\dfrac{M}{Q \cdot d_2}$ shall be in the range of 1.0 to 3.0.

where:

p_{t2} = Tensile reinforcement ratio calculated by using the increased cross section of jacketed column (%).

p_w = Shear reinforcement ratio of the existing column calculated by the increased cross section of jacketed column (decimal).

p_{w2} = Shear reinforcement ratio of the jacketing column calculated by the increased cross section of jacketed column (decimal), $p_w + p_{w2}$ shall be 0.012 if it is more than 0.012.

σ_{wy} = Yield strength of shear reinforcement in the existing column (N/mm^2).

σ_{wy2} = Yield strength of shear reinforcement in the jacketing column (N/mm^2). The strength σ_{wy} and σ_{wy2} shall be 294 N/mm^2 for round bars, and (specified yield strength + 49 N/mm^2) for deformed bars.

d_2 = Effective depth of the retrofitted column (mm).

M/Q = It shall be obtained by detailed calculation referring to the section 3.2.2 (2) of the Standard.

(c) Ductility factor (*F*)

Ductility factor (*F*) of columns redesigned to fail in flexure after RC jacketing shall be calculated based on the story drift angle at flexural strength provided in the Standard.

(3) Structural detail

(a) In principle, four faces of existing column shall be enclosed monolithically by RC jacket which is tightly fixed with existing column.

(b) Thickness of RC jacket shall not be less than 10 cm for post-cast concrete and not less than 6 cm for mortar.

(c) Compressive strength of post-cast concrete or mortar shall not be less than 21 N/mm^2 and the concrete strength of existing building.

(d) In case utilizing welded wire fabrics, enough lap splice length shall be provided over each fabric.

(e) In case reinforcing with hoops, the diameter of hoops shall not be less than D10 and the spacing of hoops shall not be more than 10 cm. Hoops shall be arranged to well confine the existing column. The end of hoops shall be welded or spliced to confine the concrete as effectively as welding. Longitudinal reinforcement shall be arranged within the hoops of RC jacket. See the translators' note 21.

(f) In principle, a 30 to 50 mm wide slit shall be set at both top and bottom of column in case only ductility upgrading of column is planned. See the translators' note 22.

(g) In case of strength upgrading of column, careful details such as sheer keys shall be provided for a smooth transfer of actions between new and existing concrete. New longitudinal bars shall be securely anchored in members around the column. See the translators' note 23.

3.3.5 Steel plate jacketing

(1) Outline of strengthening method

(a) Basic specification

Steel plate jacketing is a strengthening method by jacketing thin steel plate around existing column and grouting mortar into the gap between steel plate and existing concrete. This strengthening method aims to upgrade ductility of column by increasing its shear strength, and axial strength by confining existing column. The steel plate jacketing method includes square-steel-tube jacketing, circle-steel-tube jacketing, and steel strap jacketing. See the translators' note 24.

(b) Application of the method

This strengthening method may be applied to independent columns whose four faces can be retrofitted. In case of column with transverse wall, four faces of the column shall be retrofitted after removing a part of the wall, in principle. See the translators' notes 25 and 26.

(c) Retrofit for improving ductility

Square-steel-plate jacketing method, circle-steel-plate jacketing method, and steel strap jacketing method can be used for improving ductility. In this case, slits with around 30 mm shall be set in the jacketing steel plate at the top and bottom of the column, in principle. In case of no slits, ductility shall be evaluated in consideration of increase of flexural strength due to absence of slits.

(d) Retrofit for improving axial strength

Square-steel-plate jacketing and circle-steel-plate jacketing methods can be used. In case of increasing in axial strength by these methods, a slit at the bottom of the column is not necessarily required. See the translators' note 27.

(2) Design procedure

(a) Flexural strength of column

Flexural strength of column jacketed with steel plate shall be calculated by Eq. (3.3.4-1) as in case of RC jacketing. In case without a slit at the top or bottom of the column, flexural strength shall be calculated by using b_2 instead of b and D_2 instead of D in Eq. (3.3.4-1).

(b) Shear strength of column

Shear strength of column jacketed with steel plate shall be calculated by Eq. (3.3.4-3) for RC jacketing by substituting the equivalent hoop ratio of steel plate defined in Eq. (3.3.5-1). In case of circle-steel-plate jacketing methods, this equation can be applied by replacing its cross section with the equivalent square cross section.

$p_{w2} = 2 \cdot t / b_2$ for steel plate jacketing

$p_{w2} = 2 \cdot t \cdot b_s / (b_2 \cdot x_s)$ for steel strap jacketing (3.3.5-1)

where:

p_{w2} = Equivalent hoop ratio of steel plate jacketing. Upper limit of total hoop ratio shall be 0.012.

t = Thickness of steel plate.

b_2 = Column width after strengthening.

b_s, x_s = Width and spacing of steel strap, respectively.

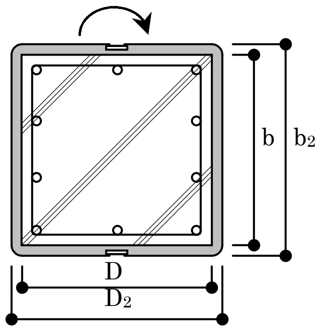


Figure 3.3.5-1 Cross section of jacketed column with steel plate

(c) Limit of axial force ratio

The axial force ratio (Supplementary Provisions 1.2 (3) of the Standard) of jacketed columns with steel plate shall follow the rule in Eq. (3.3.5-2).

$\eta_H = \eta_{Ho} + p_{w2} \cdot \sigma_{wy2} / 20 \leq 0.7$ (3.3.5-2)

where:

η_H = Limit of axial force ratio of column after jacketing.

η_{Ho} = Limit of axial force ratio of column before jacketing, 0.5 for 100 mm or less in hoop spacing, 0.4 for others.

p_{w2} = Equivalent hoop ratio of steel plate, the same as Eq. (3.3.5-1).

σ_{wy2} = Yield strength of steel plate for jacketing (N/mm^2).

(d) Ductility factor (*F*)

Ductility factor (*F*) of jacketed column with steel plate shall be calculated according to the method in the section 3.3.4 (2) of the guidelines.

(3) Structural detail

(a) In principle, four faces of existing column shall be enclosed monolithically by steel plate jacket.

(b) The gap between steel plate and existing concrete shall be securely grouted with mortar. The gap between steel plate and existing concrete shall be appropriately provided for infilling mortar securely. Strength of grouting mortar shall not be less than 21 N/mm^2 and the concrete strength of existing building.

(c) The thickness of square steel plate and circle steel plate shall not be less than 4.5 mm. Each unit part manufactured in factory shall be welded and assembled on site. The steel plate for a square section shall be rounded at four corners of a column with a radius of three times of steel plate thickness, and shall be appropriately detailed to prevent out-of-plain deformation during mortar grouting.

(d) In case of steel strap jacketing, steel straps of about 10cm wide shall be welded to the L-shaped steel angle placed at four corners of the column with an interval of around 30 cm.

(e) In case of setting slit at bottom of column, measure to prevent peeling off the grout mortar when a large earthquake hit shall be done.

3.3.6 Carbon fiber wrapping

(1) Outline of strengthening method

(a) Basic specification

This provision shall be applied for upgrading ductility due to increase in shear strength of columns by wrapping carbon fiber sheet with epoxy resin around existing column. Detail of fiber sheet wrapping with combination of continuous fiber sheets including carbon fiber sheets and impregnate adhesive resin shall be in accordance with the “Seismic Retrofit Design and Construction Guidelines for Existing Reinforced Concrete Buildings and Steel Encased in Reinforced Concrete Buildings Using Continuous Fiber Reinforced Materials (1999)”, hereafter referred to as “FRP wrapping guidelines.” See the translators’ note 28.

(b) Materials

Carbon fiber sheet used in this strengthening method shall meet the standards indicated in Table 3.3.6-1.

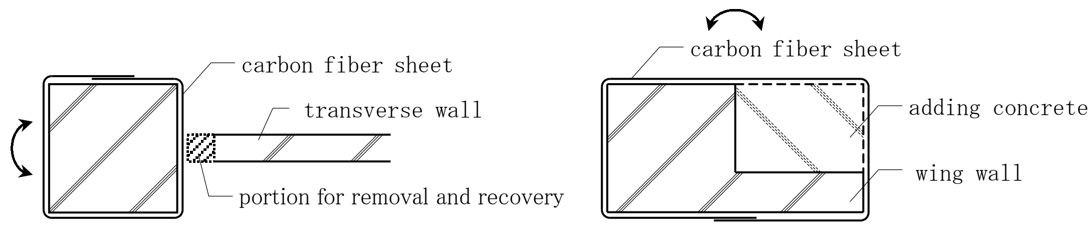
Table 3.3.6-1 Standards of carbon fiber sheet

	3400 N/mm^2 class	2900 N/mm^2 class
Type of fiber	High-strength type carbon fiber	
Shape of sheet	One directional reinforcing sheet	
Mass per unit area	300 g/m^2 or less	
Specified tensile strength*	3400 N/mm^2	2900 N/mm^2
Specified Young’s modulus*	2.30×10^5 N/mm^2	

* Value for the case of carbon fiber sheet with hardened impregnate adhesive resin

(c) Strengthening in case of column with transverse wall

In case that a column with wing wall or other walls in the longitudinal or transverse directions is strengthened with this method, carbon fiber sheets shall be wrapped around the column with square or rectangular cross section after removing the adjacent part in the wall as shown in Figure 3.3.6-1(a), or after casting concrete in the recessed portion of the section as shown in Figure 3.3.6-1 (b), in principle.



(a) In case of column with transverse wall **(b) In case of column with wing wall**

Figure 3.3.6-1 Strengthening of column with wall

(d) Strengthening in case of column with spandrel wall

When a spandrel wall (= standing wall and/or hanging wall) is connected to a column and the wrapping may cause damage to the wall, the column shall be isolated from the wall and strengthened over the full length including isolated region adjacent to the wall.

(e) Other remarks

Construction procedures shall be well discussed and confirmed, and the construction shall be done by skilled workers, since strengthening effects by this method maybe highly dependent on construction conditions.

(2) Calculation methods for strengths

(a) Flexural strength of column

Flexural strength of column wrapped with carbon fiber sheet shall be calculated by Eq. (3.3.4-1) for RC jacketing. In this calculation, the influence of multi-layered longitudinal bars shall be taken into account.

(b) Shear strength of column

Shear strength of column wrapped with carbon fiber sheet shall be calculated by Eq. (3.3.6-1).

$$Q_{su} = \left\{ \frac{0.053 \cdot p_t^{0.23} \cdot \left(F_{c1} + 18\right)}{M/\left(Q \cdot d\right) + 0.12} + 0.85\sqrt{p_w \cdot \sigma_{wy} + p_{wf} \cdot \sigma_{fd}} + 0.1\sigma_o \right\} \cdot b \cdot j \qquad (3.3.6\text{-}1)$$

$M/\left(Q \cdot d\right)$ shall be in the range of 1.0 to 3.0.

$p_w \cdot \sigma_{wy} + p_{wf} \cdot \sigma_{fd}$ shall be not more than 9.8 N/mm^2.

where:

p_t = Tensile reinforcement ratio of existing column (%).

p_w = Shear reinforcement ratio of existing column (decimal).

σ_{wy} = Yield strength of shear reinforcement of existing column (N/mm^2).

p_{wf} = Shear reinforcement ratio of carbon fiber sheet (decimal).

σ_{fd} = $\min\{E_{fd} \cdot \varepsilon_{fd}, (2/3) \cdot \sigma_f\}$, tensile strength of carbon fiber sheet for shear design.

E_{fd} = Specified Young's modulus of carbon fiber sheet. A value indicated in the Table 3.3.6-1 can be used.

ε_{fd} = Effective strain of carbon fiber sheet at shear failure. A value of 0.7% can be used.

σ_f = Specified tensile strength of carbon fiber sheet. A value indicated in Table 3.3.6-1 can be used.

M/Q = Shear span. It shall be defined by calculating the height of inflection point according to (c) of the section 3.2.2 (2) of the Standard.

b, D = Width and depth of column, respectively (mm).

j = Distance between the centroids of tension and compression forces; A value of 0.8D can be used.

σ_o = Axial compressive stress. The value shall not be more than 7.8 N/mm^2.

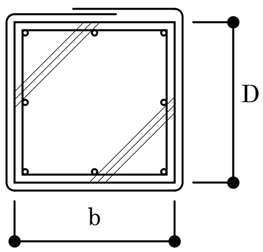


Figure 3.3.6-2 Cross section of column

(c) Ductility factor (*F*)

Ductility factor (*F*) of column wrapped with carbon fiber sheet shall be determined according to the section 3.3.4 (2) of the guidelines.

(3) Structural detail

(a) Pre-treatment shall be appropriately made on the surface of a column to be wrapped with carbon fiber sheets.

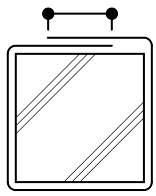
(b) Corners of cross section of column shall be rounded with a corner radius of 20 mm or larger. This rounded portion must be straight and uncurved along the column height.

(c) The column shall be securely and tightly wrapped with carbon fiber sheets. The fiber direction shall be perpendicular to the column axis.

(d) Overlap of carbon fiber sheets shall be long enough to ensure the rupture in materials. It shall be not less than the value indicated in Table 3.3.6-2.

Table 3.3.6-2 Lap length of carbon fiber sheets

Type of sheet	Lap length (mm)
200 g/cm² type	200 or larger
300 g/cm² type	200 or larger



(e) Carbon fiber sheet shall wrap closely around the column. Position of lap splice shall be provided alternately.

(f) Impregnate adhesive resin shall be the one which has appropriate properties in construction and strength to bring the strength characteristics of carbon fiber sheet.

(g) After impregnation of adhesive resin has completed the initial hardening process, mortar, boards or painting must be provided, for fire resistance, surface protection or design point of view.

3.3.7 Slit between column and spandrel /wing wall

(1) Outline of strengthening method

(a) Retrofit objectives

This section describes a strengthening method which provides new structural slits in existing buildings for the following objectives. See the translators' note 29.

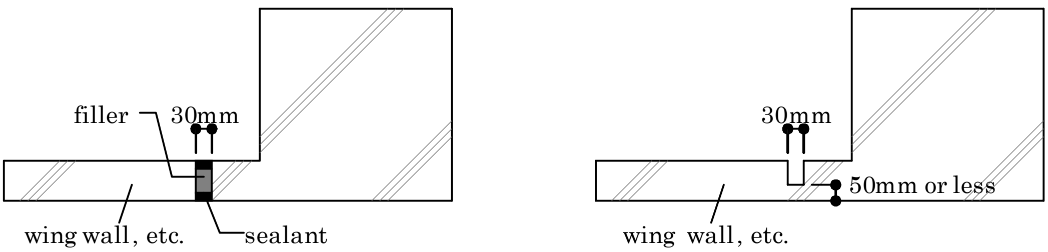
(i) Increase height-to-depth ratio so that the column should not be categorized in the second-class prime elements due to its extremely brittle response.

(ii) Alter the shear failure dominant columns to the flexural failure dominant columns.

(iii) Improve ductility of column with wing wall by changing its configuration as independent column

(b) Basic specification

A structural slit is, by using a concrete cutter, provided in the existing standing wall, hanging wall or wing wall. The slit is classified into full slit and partial slit. Full slit shall be used for strengthening of columns in principle.



(a) Full slit **(b) Partial slit**

Figure 3.3.7-1 Structural slit

(c) Other remarks

The following points shall be taken into account when this method is used for seismic retrofit.

(i) Confirm the retrofitting effect carefully

(ii) Secure safety against out-of -plain behaviors of wall to be cut

(iii) Secure waterproof performance

(iv) Secure fireproof performance

(2) Design procedure

Flexural strength, shear strength and ultimate ductility factor of columns with structural slit shall be calculated by the equations for independent columns in the Standard in consideration of increase of clear height due to the slit. See the translators' note 30.

(3) Structural detail

Detail of the structural slit shall be provided so as to maintain the waterproof performance after small or medium-scale earthquakes and not to be damaged during a large-scale earthquake. See the translators' notes 31 and 32.

3.4 Steel framed brace/panel

3.4.1 Outline

Retrofit with steel sections is a seismic upgrading technique of existing RC frames by steel braces or steel panels.

Retrofit with steel sections is classified into two cases; steel framed brace/panel and non-framed brace/panel. Connection details may have the following two schemes; direct connection by bolting, welding or other methods and indirect connection through mortar and anchors provided between existing RC frame and steel frame for strengthening. The Guidelines, in principle, applies to steel framed members which are securely connected by indirect scheme with existing RC members along their four interfaces of steel frame. See the translators' notes 33 and 34.

3.4.2 Demand Performance

(1) Resistance mechanism of structure strengthened with steel frame

The structure strengthened with steel frame consists of three structural components, existing RC frame, steel frame and connection. Resistance mechanism after strengthening is strength dominant type, ductility dominant type, or strength and ductility dominant type, depending on the strength-ductility relationship of each component and failure mechanism of the whole structure strengthened with steel frame. It can be classified into four types as shown in Table 3.4.2-1. The Guidelines recommend the strength and ductility dominant resistance mechanism, Type I, when strengthening with steel frame.

Table 3.4.2-1 Failure mechanisms of structure strengthened with steel frame

Failure mechanism	Existing R/C frame	Steel frame	Connection
Type I Strength and ductility dominant (failure at steel brace or steel panel)	- Flexural failure of columns or beams - Shear failure of columns or beams	- Strengthening with steel brace: Yielding or buckling of brace - Strengthening with steel panel: Shear yielding of panel or flexural yielding of flange	No failure
Type II Strength dominant (failure at connection)	- Direct shear failure of tension columns and shear failure of compression columns - Direct shear failure of beams	Neither yielding nor buckling	Shear slip failure
Type III Ductility dominant	- failure of tensile yielding of tension columns - Compressive failure of compression columns	Neither yielding nor buckling	No failure
Type IV Strength dominant	Extremely brittle failure of columns	- Strengthening with steel brace: Yielding or buckling of brace - Strengthening with steel panel: Shear yielding of panel or flexural yielding of flange	No failure

Note: Type III is a flexural failure of whole structure strengthened with steel frame

(2) Other resistance mechanisms

Rotation of braced frame which absorbs seismic energy by its uplift deflection can be a objective performance when the strength and ductility dominant type seismic performance can not be expected.

(3) Resistance mechanisms

Ultimate strengths of each resistance mechanisms of Table 3.4.2-1 and the type due to rotation of braced frame shall be calculated, and the resistance mechanism with the smallest ultimate capacity shall be identified to represent the mechanism. In the calculation, provisions in the Standard shall be used for RC frame. Provisions in the “Standard for calculation of steel structures” and the “Design guidelines for buckling of steel structures” published by Architectural Institute of Japan shall be used for steel frame, and provisions in the section 3.9 of the guidelines shall be used for the connections of strengthening.

(4) Ductility Index

Ductility index of the structure strengthened with steel frame shall be provided as Table 3.4.2-2 for each resistance mechanism of Table 3.4.2-1 and a type as rotation of braced frame. Valued in the table applies only to steel framed members.

Table 3.4.2-2 Ductility index of structures strengthened with steel framed brace

Failure type	Failure type of RC frame	Ductility index, F value
Type I	Flexural column or flexural beam dominant Shear column or shear beam dominant	$F = 2.0$ In case F value of RC frame > 2.0, F value of brace frame = F value of RC frame. In case of $Q_{SU2} / Q_{SU1} < 1.1$, $F = 1.5$
Type II	Direct shear and connection failure dominant	$F = 1.0$
Type III	Total flexural yielding of RC frame dominant (Capacity governed by the amount of longitudinal bars in column)	- Simple frame without beams framing into the strengthened member: $F = 2.0$ In case of $Q_{SU2} / Q_{SU1} < 1.1$, $F = 1.5$ - When link beams and/or orthogonal beams are framing into the strengthened member, F shall be calculated considering their influences in accordance with the Standard (3.2.3 (3) (iii) of the Standard).
Type IV	Extremely brittle column dominant	$F = 1.0$
Other	Rotation of braced frame	- Simple frame without beams framing into the strengthened member: $F = 2.0$ In case of $Q_{SU2} / (\gamma \cdot Q_{ru}) < 1.1$, $F = 1.5$ *In case,* $Q_{SU1} / (\gamma \cdot Q_{ru}) > 1.1$, and $Q_{SU2} / (\gamma \cdot Q_{ru}) > 1.1$, and $Q_{mu} / (\gamma \cdot Q_{ru}) > 1.21$, then $F = 3.0$ - When link beams and/or orthogonal beams are framing into the strengthened member, F shall be calculated considering their influences in accordance with the Standard (3.2.3 (3) (iii) of the Standard).

Where, Q_{SU1} : Strength governed by buckling or tensile yielding of brace
Q_{SU2} : Strength governed by direct shear and connection capacity
Q_{mu} : Strength of total flexural yielding (Capacity governed by the amount of longitudinal bars in column)
Q_{ru} : Strength of rotation
γ : See the provisions in the section of uplift wall of the Standard

(5) When seismic capacity evaluation of building after retrofit is conducted, the strength contribution factor α_j of structure retrofitted with steel framed brace shall be $\alpha_j = 0.65$ in case that ductility factor F of the total building is 0.8, and $\alpha_j = 1.0$ in case that 1.0 or higher F value is allowed.

3.4.3 Planning

Exterior frames may be most suitable to minimize construction difficulties when the steel framed braes are applied to retrofit RC buildings. In case that total flexural yielding and failure due to rotation of braced frame are expected, retrofit effects may not be fully achieved in general, and the new elements should be carefully arranged to maximize their performance. Since the connection is a most important part for structures strengthened with steel frame, it shall be designed so that stress can be transferred smoothly. Existing RC frame adjacent to the steel frame therefore should also be strong enough against actions. Also, eccentricity in plan and stiffness / weight distribution in height shall be carefully investigated because ductility, strength and failure pattern of structures strengthened with steel frame are different from those of conventional RC post-cast shear wall.

3.4.4 Construction method and structural details

(1) Construction method

Following items shall be investigated in strengthening with steel frame. See the translators' note 35.

(a) Construction method shall be selected from brace, panel or their combination.

(b) In case of brace strengthening, K-shape or X-shape member with aspect ratio of not larger than 58 shall be arranged. The brace shall be designed to have a symmetric capacity in both positive and negative loading direction.

(c) In case of panel strengthening, shear yielding strength of panel shall be studied in consideration of opening location. Stiffener arrangement shall be studied not to cause shear buckling of the panel. If the opening is relatively large, the panel shall be carefully designed to have flexural strength larger than shear yielding strength since it is likely to behave in frame-like manner.

(d) Indirect connection shall be used between steel frame and existing RC beam and column members, and the connection details shall be designed to meet the strength demand.

(2) Structural detail

In case of strengthening with steel frame, the structural details should follow the recommendations described below. See the translators' notes 36 to 39.

(a) In case of steel framed brace, the bracing members shall be centered in the steel frame.

(b) The cross section of steel brace and steel frame shall be strong enough not to cause local buckling. The strength of the connection between brace and frame shall be strong enough not to fail in the connection.

(c) The connection shall be designed not to cause stress connection.

(d) Post-installed anchors used in the connection shall be bonded anchors or expansion anchors provided in section 3.9, unless in particular specified. However, these anchors shall not be used together. Post-installed anchors, in general, shall be arranged in all beam and column around the steel frame for strengthening. Concrete surface in the connection shall be roughened appropriately.

(e) Pitch, gauge, and lap length of post-installed anchors and studs installed on steel frame shall be determined so as to transfer smoothly the stress acting in the connection. Mortar shall be injected with pressure in the connection part which shall be confined with spiral reinforcement, hoop reinforcement, or ladder reinforcement, etc.

(f) Strengthening with steel frame shall also meet the following specifications.

- Strengthening with steel brace

  Brace element with cross section performance equivalent to or better than H-150x150x7x10 shall be used.

- Strengthening with steel panel

Steel plate with thickness of not less than 4.5 mm shall be used for steel panel. Stiffeners shall be provided at a space of not wider than1000 mm.

- Detail of indirect connection along steel frame

(i) Bonded anchors of not smaller than D16, or expansion anchors of not smaller than 16φ, shall be installed at a space not wider than 250mm.

(ii) Headed studs not smaller than 16 φ shall be installed at a space of not wider than 250 mm.

(iii) Lap length between post-installed anchor and headed stud shall be not less than 1/2 length of post-installed anchor and headed stud in the injected mortar.

(iv) Compressive strength of mortar injected with pressure shall be not less than 30 N/mm^2 .

(v) Reinforcement ratio p_s of spiral reinforcement, hoop reinforcement, or ladder reinforcement in the injected mortar, shall be not less than 0.4% . The value of p_s shall be calculated by the following equation.

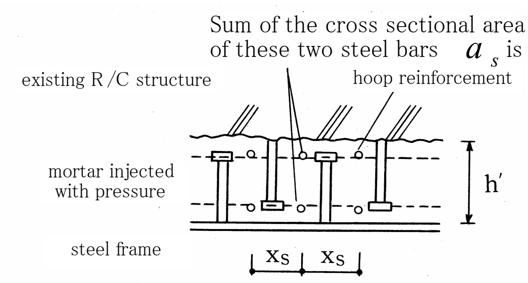
$$p_s = a_s / \left(h' \cdot X_s\right)$$

where:

X_s = Interval of reinforcement (mm).

a_s = Cross sectional area of one set of reinforcement (mm^2).

h' = Height of injected mortar (mm).



3.4.5 Design procedure

(1) Principles

(a) Lateral load carrying capacity of structures retrofitted with steel frame shall be the smallest one calculated considering ultimate strength of existing RC frame, ultimate strength of steel frame, and strength of connection for strengthening.

(b) Ultimate strength of each column strengthened with steel framed member shall be calculated primarily based on existing RC cross section and the steel frame and mortar at the connection shall be neglected unless specified.

(c) Ultimate strength of steel framed brace shall be calculated, in general, assuming that all cross sections of compression and tension braces simultaneously reach their limit stresses. Ultimate compressive stress of compression brace shall be obtained by the following equation.

$$f_{cr} = \left\{1 - 0.4(\lambda/\Lambda)^2\right\} \cdot F \quad for \quad \lambda \le \Lambda$$
$$f_{cr} = 0.6F/(\lambda/\Lambda)^2 \quad for \quad \lambda > \Lambda \tag{3.4.5-1}$$

where:

f_{cr} = Limit compressive stress (N/mm^2).

Λ = Limit aspect ratio ($\Lambda = \sqrt{(\pi^2 \cdot E)/(0.6F)}$).

λ = Effective aspect ratio.

F = Specified strength of steel(N/mm^2).

E = Young's modulus of steel (N/mm^2).

(d) Steel framed panel shall be, in general, designed to fail in shear yielding of the panel. It is therefore essential not to cause flexural yielding of the flange and the shear buckling of the panel by providing stiffeners spaced at an appropriate distance.

(e) Flexural strength of the sub-structure strengthened with steel framed member shall be the smaller value obtained when the RC column yields in tension or compression. In calculating the strength, no contribution of steel brace and panel to the total flexural strength shall be considered.

(f) The contribution of link beam, orthogonal beam and weight of foundation to the frame resistance shall be considered in calculating the strength due to rotation of steel framed members.

(g) Ultimate strength of connection shall be calculated according to the related sections in the guidelines. Shear strength contributed by each stud shall be obtained by the following equation.

$$q_{ds} = 0.64 \cdot \sigma_{\max} \cdot a_s \tag{3.4.5-2}$$

where:

q_{ds} = Shear strength contributed by each stud ((N) for one stud).

$\sigma_{\max}$ = Tensile strength of stud, equal to or less than 400 (N/mm^2).

a_s = Cross sectional area of stud (mm^2).

(2) Procedure of retrofit design

(a) Design of braced frame

Following are standard design procedures of braced frame.

(i) Lateral force carried by braced frame shall be determined.

(ii) Cross section of steel frame and brace shall be determined.

(iii) Studs and post-installed anchors shall be proportioned. The guidelines recommend that the shear strength of the connection should be not lower than the lateral load carrying capacity of the braced frame.

(iv) Connections between the steel frame and the brace ends shall be designed.

(b) Design of steel panel

Following are standard design procedures of steel panel.

(i) Anchorage arrangement around steel frame shall be preliminarily determined.

(ii) Location and size of opening shall be determined.

(iii) Lateral force carried by steel panel shall be determined.

(iv) Thickness of the web plates shall be determined assuming that the steel panel around the opening yields in shear.

(v) The flange section shall be determined not to cause flexural yielding in steel panels around opening, where steel panels around the opening are assumed as beams and columns of a portal frame. The section shall be sized not to cause local buckling and lateral buckling.

(vi) Mid-stiffeners shall be placed if shear buckling is expected in the panel.

3.5 Beam Strengthening

3.5.1 Outline

A main objective of this retrofit method is to improve ductility index of column or shear wall by preventing shear failure and improving ductility of beams framing into the column or wall.

It is desirable that this strengthening method is applied to all beams which have similar structural characteristics in the building concerned. It should be noticed, however, that the ductility of short-span beams with large amount of flexural reinforcement ratio may not be effectively improved.

3.5.2 Performance objectives

(1) Performance of members

In general shear strength shall be higher than flexural strength by shear strengthening since the objective of this method is to provide ductile beams by retrofitting existing beams. In case of increasing longitudinal bars, the beams shall be strengthened for shear and redesigned to fail in flexure.

(2) Performance of buildings

The strengthened building shall fail in flexure. Seismic performance of the whole building shall be upgraded by flexural yielding in beams and improved the ductility index F.

3.5.3 Planning

(1) Buildings suitable for this retrofit

Following three types are buildings suitable for this retrofit method. In each case, it is assumed that ductility and strength of columns or shear walls into which the beams are framing are relatively well-provided or can be improved by retrofit.

(a) A coupled shear wall building which typically has brittle short-span beams between walls.

(b) A building with ductile columns and brittle beams.

(c)A building with brittle beams which essentially contribute to the seismic performance of entire building.

(2) Strengthening region

In general, this strengthening method shall apply to all beams which have similar structural detail. Beams shall be designed such that the lateral load carrying capacity of a frame including the beams concerned shall not be determined by shear failure of the beam. See the translators' note 40.

3.5.4 Construction method and structural details

(1) Retrofit method

Following retrofit methods can be used for upgrading the ductility of beams. See the translators' note 41.

(a) Jacketing with reinforced concrete

(b) Jacketing with steel plate

(c) Wrapping with continuous fiber

(2) Structural detail

The retrofit methods in general shall following the structural detail described below.

(a) Jacketing with reinforced concrete

(i) Added shear reinforcement shall be covered by concrete or mortar.

(ii) Ends of added stirrups shall pass through the slab and be a closed shape by welding or anchoring with plate.

(iii) The added stirrups shall be not smaller than 13 mm, and its interval shall be not more than 100mm at the ends of members and not more than 150 mm in the middle of members.

(iv) Concrete cover of added stirrups shall be not less than 20 mm.

(v) Slits of 10 mm to 20 mm in width shall be provided at beam ends unless the beam strength is increased with new longitudinal reinforcing bars. See the translators' note 42.

(b) Jacketing with steel plate

(i) U-shaped steel plates shall jacket the beam under the slab and they are tightly fastened with through-bolts to anchor plates on RC slab.

(ii) The gap between steel plate and existing beam shall be ground with injected resin or pressurized mortar.

(iii) Thickness of steel plates shall be determined considering both reguired strength and construction practice.

(c) Wrapping with continuous fiber

(i) Corners of the beam shall be rounded so that the continuous fiber will not rupture at the corners under its specified tensile strength.

(ii) The continuous fiber shall enclose the beam and the fiber shall be provided in the direction perpendicular to the beam's longitudinal axis.

(iii) The continuous fiber shall be securely bonded, by using adhesion resin, on the smooth surfaced concrete of the existing beam.

(iv) In case the fire-resistance is expected to the retrofitted members, fireproof cover shall be appropriately provided.

3.5.5 Design procedure

(1) Strengthening Procedure

The strengthening procedure of calculation shall be as follows;

(a) Demand performance shall be determined based on the results of seismic evaluation.

(b) Beams to be strengthened shall be identified through feasibility studies for strengthening.

(c) Assume the retrofit method and the structural detail.

(d) The lateral load carrying capacity of the frame including strengthened beams shall not be governed by beams failing in shear.

(e) Judge whether or not the calculated seismic performance of strengthened frame meets the demand performance.

(2) Strength of retrofitted beams

Flexural and shear strength of retrofitted beam shall be calculated according to the equations in Supplementary Provisions 4 of the Standard.

Following equation shall be satisfied to provide ductile behaviors in the strengthened beam.

$$Q_{su} > \alpha \cdot Q_{mu}$$

where:

Q_{su} = Shear strength of beam.

Q_{mu} = Shear force at the flexural capacity of beam.

α = Safety factor for flexural failure.

3.6 Other Techniques

3.6.1 Outline

Basic technical issues for adding buttress, adding columns (spatial frames), improving stiffness distribution, and other techniques are shown in this section.

3.6.2 Adding buttress

(1) Outline

This strengthening method mainly aims to increase lateral strength of building by constructing new buttress connecting with exterior frames of a building.

(2) Performance objectives

The main objective of the new buttress shall be improving seismic performance due to increase in lateral strength of the building.

However, when ductile performance can be expected in the new buttress, both strength and ductility of a building can be improved.

(3) Planning

This strengthening method is suitable for buildings holding large space with important function difficult to be strengthened inside, or susceptible to overturning or severe damage, but those having spaces wide enough to add buttress around them. In general, the buttress shall be arranged to connect with existing structural frames at both ends of the building and at all floor levels. See the translators' note 43.

(4) Construction method and structural details

(a) Construction method

(i) The buttress shall be arranged symmetrically at both ends of the building in the direction where strengthening is required.

(ii) The buttress shall have columns on its both ends and beams in each floor level.

(iii) In case that two or more buttresses are arranged on the same end of a building, lateral link elements such as beams and slabs shall be arranged between them.

(iv) The corner columns of the existing building shall be also buttress columns. Connection between the corner columns and buttress walls and between existing beams and buttress beams shall be carefully detailed to be strong enough against actions.

(b) General structural detail

(i) In case adding new buttress, pre-loading or supporting pile shall be applied to avoid uneven settlement.

(ii) Foundation beam shall be constructed under the buttress, and connected firmly with the existing foundation or the foundation beam.

(iii) The connection at buttress beams and existing beams shall be detailed to resist tensile actions expected in the design.

(iv) The vertical connections between buttress wall and existing column shall be detailed to resist shear actions expected in the design.

(v) Wall thickness of the buttress shall be not less than 150mm, and its wall reinforcement ratio shall be not less than 0.2%.

3.6.3 Adding spatial frame

(1) Outline

This strengthening method mainly aims to increase lateral strength of building by constructing new spatial frame connecting with exterior frames of a building.

(2) Performance objectives

The main objective of the retrofit by adding spatial frame is to increase lateral strength. This method may contribute to the ductility improvement of a building when the ductility of existing frame is also upgraded.

(3) Planning

This strengthening method is suitable for buildings which can not be strengthened inside since it has functionally-important space and so on, and those with a few structural members such as single-span frames.

However, it is necessary that enough spaces should be provided around the building if this strengthening method is applied.

The spatial frame shall be arranged with good balance in plane and in elevation, and connected with existing structural frames in general, at both ends of the building. The structural detail of the connection between added frames and existing building, and the influence of increased weight due to added frames, in addition to the structural detail of each part of the spatial frame shall be carefully studies. See the translators' note 44.

(4) Construction method and structural details

(a) Issues to be investigated

(i) Construction method of foundation to decrease in settlement of added spatial frame and influence of the settlement on the structural strength.

(ii) Effects of retrofit on existing frame.

(iii) Stiffness, strength and ductility of each part of added spatial frame.

(iv) Strength and detail of the connection including slab between added spatial frame and existing frame.

(v) In-plane shear force transfer between new and existing slab.

(b) Structural detail

(i) The center of each column and floor level of added special frame shall lie on the same line of existing frame.

(ii) In connecting the beam of the added spatial frame with existing frame, re-bars located at four corners in a new beam shall be securely connected to those in existing beam by welding or equivalently effective methods. They shall also be designed to effectively transfer the acting shear forces including out-of-plane shear force.

(iii) Frictional resistance piles, in general, shall not be applied to the spatial frame foundation.

3.6.4 Attaching planar frame on existing buildings

(1) Outline

This retrofit method mainly aims to increase lateral strength of building by constructing new planar frame attaching to exterior frames of the existing building.

(2) Objectives performance

Main objective of this retrofit method is to increase lateral strength. This method may also contribute to the ductility improvement of a building when the ductility of existing frame is upgraded since it is relatively easy to secure the ductility of the added planar frame.

(3) Planning

This strengthening method is suitable for the building which can not be strengthened inside since it has functionally-important space, and the building which should be operational during the strengthening construction. It has a major advantage over other schemes such as buttress and spatial frames since it does not necessarily need a large space around the building.

The planar frame shall be arranged with good balance of stiffness and strength in plane and in elevation. Careful studies are also required on connection details between the added planar frame and existing building. See the translators' note 45.

(4) Essential issues in retrofit design and construction

Following are issues essential for retrofit design of steel framed braces.

(a) Essential points

(i) Location of the added planar frame

(ii) Stiffness, strength and ductility of each part of the added planar frame

(iii) Strength and detail of connection between the added planar frame and the existing building

(b) Retrofit design

(i) Beams of the added planar frame shall be placed at the same height as existing beams and both beams shall be connected to transfer the acting forces. The connections shall be detailed so that they can transfer tensile force and vertical shear force acting on the connection as well as lateral sheer forces. See the translators' note 46.

(ii) The added planar frame shall be designed not to fail in connections with existing frame.

(iii) The interface between the added planar frame and the existing building shall be detailed not to cause loose connections.

(iv) The braces of the added planar frame shall be continuous to the lowest floor, in general.

(v) The vertical members of the added planar frame shall be located adjacent to the existing columns and shall be continuous to the lowest floor, in general. The members shall be proportioned not to yield or buckle.

(vi) The influence of vertical force transfer between vertical members of the added planar frame and existing columns shall be considered in evaluating ductility index.

(vii) The new foundation constructed under the added planar frame shall be firmly connected with the foundation or foundation beam of existing building.

(5) Design procedure

(a) The structural performance of existing building shall be confirmed and then the demand performance for retrofit shall be determined. The demand *F* values shall be as uniform as possible along building height. The demand capacity (strength) of added frame shall not reduce in lower stories.

(b) The number and location of added planar frames which meet the demand capacity shall be identified.

(c) The shear force to be transferred at connections from the existing building to the added planar frame shall be calculated. The connection shall be designed strong enough to resist satisfactorily lateral shear forces, vertical shear forces, and tensile forces acting on the connection.

(d) The brace members and stiffness shall be designed to meet demand strength..

(e) The vertical members of the added planar frame shall be designed not to yield or buckle under tension and compression forces resulting from the shear carried by the brace members.

(f) The foundation shall be designed against the force transferred from the added planar frame.
(g) Confirmed that the target performance for strengthening is satisfied.

3.6.5 Other techniques

Other retrofit techniques not described in the guidelines such as adding columns and improving stiffness distribution may be applied in general based on experimental investigations unless technical information is fully available.

3.7 Foundation

3.7.1 Outline

(1) It may be desired that the strengthening of foundation is not required in seismic retrofit of buildings. In general, strengthening of foundation shall be performed only when the retrofit scheme is simple, practical, cost effective, and reliable for drastic improvement of seismic performance.

3.7.2 Determination of demand performance

(1) The strengthening of foundation shall aim to help retrofitted superstructures effectively contribute to the overall seismic performance demand of buildings.

(2) Foundation shall safely support the permanent load of superstructure after strengthening.

(3) When adverse effects to the structural performance of building concerned is expected in future due to settlement of ground, negative friction of pile, and liquefaction of sandy soil at the time of earthquake, improve the soil performance appropriately.

3.7.3 Evaluation of bearing capacity and settlement of existing foundation

(1) Bearing capacity of soil and pile, soil settlement, negative friction and lateral resistance of pile shall be calculated according to the "Design Guideline of Structural Foundation of Buildings" published by Architectural Institute of Japan.

(2) The bearing capacity of soil and pile after strengthening of building shall be the same as the case before strengthening; that is 1/3 and 2/3 of ultimate bearing capacity for long-term and short-term design, respectively. However, the ultimate bearing capacity can be allowed against seismic loads.

3.7.4 Evaluation of bearing capacity of retrofitted foundation

Bearing capacity of new foundations shall be added to that of existing foundation in general.

3.7.5 Structural details and others

(1) Added foundation shall not be arranged eccentrically, in general.

(2) Different types of foundation shall not be applied together, in general.

(3) Strength and stiffness at the connection between new and existing foundation shall be as close to those of monolithic foundation as possible.

(4) In constructing new foundation, attention shall be carefully paid not to leave damage to the existing foundation.

(5) Construction safety and practice shall be investigated in determining retrofit method for foundation.

See the translators' notes 47 to 50.

3.8 Non-Structural Elements

3.8.1 Outline

The objective of the retrofit method is to prevent non-structural elements such as exterior finishing from peeling off or falling off at the time of earthquake. This method applies only to exterior walls that may cause life-threatening hazard due to their failing-off or blocking evacuation routes.

3.8.2 Performance objectives

(1) Performance of members

The main objective of repair or retrofit is to secure safety of human life from existing non-structural elements peeling off or falling off the structure at the time of earthquake. Target performance for retrofitting non-structural elements may differ due to building location, main structural type and materials used in the non-structural members

(2) Performance of building

It is not desired that the structural performance should be affected by strengthening non-structural elements. Comprehensive investigations shall be needed if it may affect the structural performance.

3.8.3 Planning

(1) Retrofit elements

The non-structural elements necessary to retrofit are as follows.

(a) Exterior wall like concrete block, glass block, or curtain wall, etc.

(b) Window glass or sash on exterior walls

(c) Exterior finishing like bonded stone or tile

(d) Signboard or lighting instrument fixed to exterior wall

Note that the guidelines do not cover relatively large-scaled elements on roof top.

3.8.4 Repair and strengthening method

The retrofit method which can increase the seismic performance index of non-structural element I_N defined by the Standard shall be applied. Repair and strengthening method are as follows.

(1) Exterior wall, opening of exterior wall, and exterior finishing

(a) Material used in the exterior wall etc. shall be changed to improve the deflection capacity so as to remove falling hazard of elements.

(b) In case that the same (original) material is used for strengthening, construction method allowing for its deflection capacity and rigidity with base material shall be adopted.

(2) Signboard or lighting instrument fixed to exterior wall

(a) Removal of the signboard or lighting instrument

(b) Re-construction of the connection part of the signboard or lighting instrument

3.9 Design Procedure for Post-Installed Anchor

3.9.1 General

(1) Scope

This provision is applied to the design of post-installed anchors used in the connection in case of adding reinforced concrete (RC) shear wall (including wing wall) or steel framed member for retrofit into the RC frame.

(2) Type and construction method of post-installed anchor

The types of post-installed anchor covered in this provision are expansion anchor and bonded anchor. See the translators' note 51.

(3) Material, shape, and size of post-installed anchor

Material, shape, and size of post-installed anchor shall be carefully examined befor installation.

(4) Others

Issues not described in this section are based on the "Standard for Structural Calculation of Reinforced Concrete Constructions" and the "Design Recommendations for Composite Constructions " of AIJ, and the "Common Specification of Retrofit Construction of Building" and the "Guideline for Management of Retrofit Construction" of the Building Maintenance and Management Center and other related standards, criterion or specifications.

3.9.2 Definitions

Notations used in this section is as follows.

T_a = Tensile capacity of an anchor (N).

T_{a1} = Tensile capacity of an anchor determined by yielding of steel material (N).

T_{a2} = Tensile capacity of an anchor determined by concrete cone failure (N).

T_{a3} = Tensile capacity of an anchor determined by bond failure (N).

l = Depth of drilled hole or embedment length of bonded anchor (mm).

l_e = Effective embedment length of an anchor (mm).

l_a = Length of expansion anchor (mm).

l_1 = Embedment length of expansion anchor to the existing concrete structure (mm).

l_2 = Exposed length of expansion anchor from the connection surface (mm).

l_d = Full length of connection bar or anchorage bar (mm).

l_n = Effective anchorage length of added wall (mm).

d_a = Diameter of anchor; nominal diameter of anchorage bar for bonded anchor or diameter of sleeve of expansion anchor (mm).

d_b = Nominal diameter of steel bar threaded into expansion anchor (mm).

D_a = Diameter of drilled hole of existing concrete structure (mm).

A_c = Projected area of concrete cone failure surface of a single anchor (mm^2).

a_e = Minimal cross section area of expansion anchor (mm^2).

a_o = Effective cross section area of threaded steel bar, or nominal cross section area of anchorage bar (mm^2).

${}_s a_e$ = Cross section area of expansion anchor at concrete interface, or cross section area of bonded anchorage bar (mm^2).

σ_B = Compressive strength of existing concrete (N/mm^2).

E_c = Young's modulus of existing concrete (N/mm^2).

F_c = Specified compressive strength of existing concrete (N/mm^2).

σ_y = Specified yield strength of steel bar (N/mm^2).

${}_m\sigma_y$ = Yield strength of expansion anchor (N/mm^2).

Q_a = Shear capacity of an anchor (N).

Q_{a1} = Shear capacity of an anchor determined by steel strength (N).

Q_{a2} = Shear capacity of an anchor determined by bearing strength of concrete (N).

τ = Shear strength of anchor (N/mm^2).

τ_a = Bond strength of bonded anchor against pull-out force (N/mm^2).

τ_o = Basic bond strength of bonded anchor (N/mm^2).

3.9.3 Material strength of anchors

Material strength of anchors, steel strength of anchor itself and connection bar for expansion anchors, steel strength and bond strength for bonded anchors shall be as specified in JIS (Japan Industrial Standard).

3.9.4 Design strength

Design strength of post-installed anchor shall be minimum values of strengths of resistance mechanisms calculated based on the material strength of archons.

(1) Shear capacity *Qa*

The shear capacity Q_a is defined as the capacity resisted by a single anchor at the concrete interface. Shear capacity shall be the smaller value of Q_{a1} and Q_{a2}, which are determined by steel strength and bearing strength of concrete, respectively.

(a) Expansion anchor in case of $4d_a \le l_e < 7d_a$

$$Q_a = \min[Q_{a1}, Q_{a2}] \tag{3.9.4-1}$$

$$Q_{a1} = 0.7\,{}_m\sigma_y \cdot {}_s a_e \tag{3.9.4-2}$$

$$Q_{a2} = 0.3\sqrt{E_c \cdot \sigma_B} \cdot {}_s a_e \tag{3.9.4-3}$$

But $\tau(= Q_a / {}_s a_e)$ shall not be greater than 245 N/mm^2.

(b) Expansion anchor in case of $l_e \ge 7d_a$

$$Q_a = \min[Q_{a1}, Q_{a2}] \tag{3.9.4-4}$$

$$Q_{a1} = 0.7\,{}_m\sigma_y \cdot {}_s a_e \tag{3.9.4-5}$$

$$Q_{a2} = 0.4\sqrt{E_c \cdot \sigma_B} \cdot {}_s a_e \tag{3.9.4-6}$$

But $\tau(= Q_a / {}_s a_e)$ shall not be greater than 294 N/mm^2.

(c) Bonded anchor in case of $l_e \geq 7d_a$

$$Q_a = \min[Q_{a1}, Q_{a2}] \tag{3.9.4-7}$$

$$Q_{a1} = 0.7\sigma_y \cdot {}_s a_e \tag{3.9.4-8}$$

$$Q_{a2} = 0.4\sqrt{E_c \cdot \sigma_B} \cdot {}_s a_e \tag{3.9.4-9}$$

But $\tau(= Q_a / {}_s a_e)$ shall not be greater than 294 N/mm^2.

where:

σ_B = Compressive strength of existing concrete. In general, the strength shall be obtained by compression test of concrete cores. When the test value is larger than specified concrete strength F_c, σ_B shall be determined according to the Standard.

E_c = Young's modulus calculated based on σ_B. The test value can be used when measured during compression test.

(2) Tensile capacity T_a

The tensile capacity T_a is defined as the capacity resisted by a single anchor at the concrete interface. Tensile capacity shall be the smallest value of T_{a1} which is determined by steel strength, T_{a2} which is determined by concrete cone failure, and in case of bonded anchor additionally T_{a3} which is determined by bond strength.

(a) Expansion anchor

$$T_a = \min[T_{a1}, T_{a2}] \tag{3.9.4-10}$$

$$T_{a1} = \min\lfloor {}_m\sigma_y \cdot a_e, \sigma_y \cdot a_o \rfloor \tag{3.9.4-11}$$

$$T_{a2} = 0.23\sqrt{\sigma_B} \cdot A_c \tag{3.9.4-12}$$

(b) Bonded anchor

$$T_a = \min[T_{a1}, T_{a2}, T_{a3}] \tag{3.9.4-13}$$

$$T_{a1} = \sigma_y \cdot a_o \tag{3.9.4-14}$$

$$T_{a2} = 0.23\sqrt{\sigma_B} \cdot A_c \tag{3.9.4-15}$$

$$T_{a3} = \tau_a \cdot \pi \cdot d_a \cdot l_e \tag{3.9.4-16}$$

$$\tau_a = 10\sqrt{(\sigma_B / 21)} \tag{3.9.4-17}$$

3.9.5 Structural details

(1) General requirements

(a) Bonded anchors shall be used to anchor wall reinforcement to develop yielding. Their effective embedment length l_e shall be not less than $10d_a$.

(b) The diameter, pitch and arrangement method of post-installed anchor shall follow those described below. (see Figure 3.9.5-1)

(i) Diameter of anchor d_a shall be in the range of 13mm to 22mm.

(ii) Pitch shall not be less than $7.5d_a$, but shall not exceed 300mm.

(iii) Gauge shall not be less than 5.5 d_a for double layer bar arrangement, and shall not be less than 4 d_a for stagger bar arrangement.

(iv) Distance to wall end shall be not less than $5d_a$ but not greater than pitch.

(v) Distance to wall free edge shall not be less than $2.5d_a$. The anchor shall be installed inside concrete cone.

(c) Reinforcement for splitting prevention shall be sufficiently provided around anchors of new wall or steel framed members to prevent splitting failure.

(d) Post-installed anchors shall be installed into all beams and columns connected with new wall.

(2) Expansion anchor

Steel bars embedded in the new wall shall be deformed steel bars. Their effective anchorage length shall be not less than 30 d_a, in general. However, the length shall be not less than $20d_a$ in case of bars with hook or nut at one end.

The effective embedment length of anchor shall be not less than $4d_a$.

(3) Bonded anchor

Anchorage bars embedded in the new wall shall be deformed steel bars with nut, in general. Their effective anchorage length shall be not less than $20d_a$. The effective embedment length of anchorage bar shall be not less than $7d_a$.

Pitch not less than $7.5d_a$
but not greater than 300mm

Pitch not less than $7.5d_a$
but not greater than 300mm)

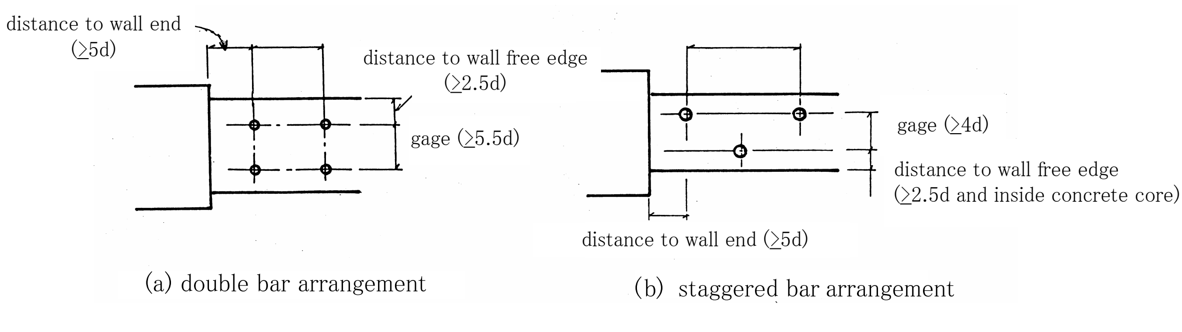


Figure 3.9.5-1 Interval and name of post-installed anchor

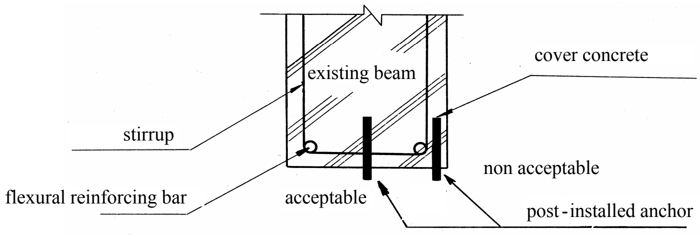


Figure 3.9.5-2 Location of post-installed anchors

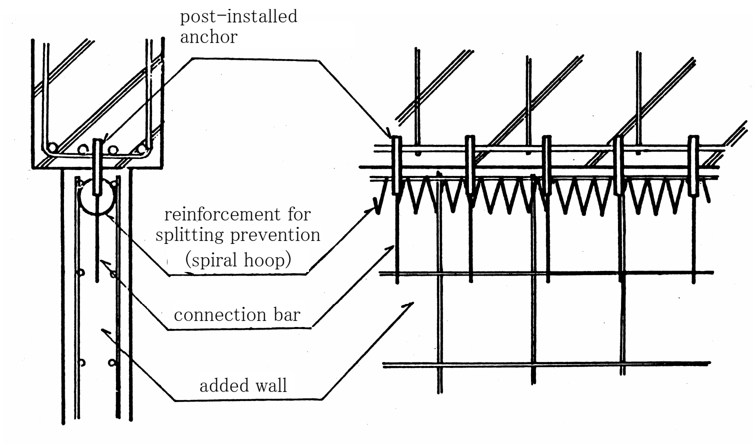


Figure 3.9.5-3 Spiral hoop reinforcement against splitting failure

See the translators' note 52.

3.10 Press-Joint Method with PC Tendon

3.10.1 Outline

The objective of this method is to improve shear transferring capacity between added member and existing member by pressure induced friction of PC tendons such as PC bars and PC strands.

3.10.2 Demand performance

The amount of PC tendons and prestressing force shall be large enough so that the friction capacity due to press-joint is larger than shear force acting on the connection between added member and existing member.

3.10.3 Construction method and structural details

(1) Design essentials

(a) Stress and strength of the connection between added member and existing member.

(b) Selection of suitable PC tendon, anchorage device and grouting material etc.

(c) Influence of prestressing of PC tendons.

(2) Recommended structural detail

(a) Penetrated hole large enough to injecting grout around the PC tendon shall be able to set in the existing members.

(b) Protection of PC tendons etc.

See the translators’ note 53.

Chapter 4 Construction

4.1 General

(1) Scope

This section shall be applied for construction of seismic strengthening methods mentioned in the sections from 3.1 to 3.10. The issues not mentioned in this section shall be determined based on the "Japanese Architectural Standard Specifications" published by the Architectural Institute of Japan (AIJ) and other related specifications.

(2) Construction plan

Construction plan shall be determined to achieve the strengthening effects expected in design. In the construction plan, measures as well as safey of occupants, users, and workers during construction shall be taken into account to minimize noise, dust and pollution,

4.2 Materials

(1) Mortar and concrete

(a) Cement

Cement shall be Portland cement provided in JIS R 5210 "Portland cement". The type-A cement provided in JIS R 5211 "Blast furnace slag cement", JIS R 5212 "Silica cement" and JIS R 5213 "Flyash cement" can also be used.

(b) Aggregate

Gravel, sand, crashed sand and crashed gravel shall be used for aggregate. The maximum size of coarse aggregate shall be determined due to casting part. Fine aggregate used in the mortar for column strengthening shall be the fine sand provided in JASS 5 4.3.

(c) Air entraining admixture

The air entraining and water reducing agent provided in JIS A 6204 "Chemical admixture for concrete" or high performance air entraining and water reducing agent which accommodate to the classification type-I due to chloride ion shall be used. The air entraining agent can also be used.

(d) Other admixture

Blast furnace slag powder for concrete or fly ash can be used if necessary only in case of utilization of normal Portland cement. The blast furnace slag powder for concrete shall conform to the provision of JIS A 6206 "Blast furnace slag powder for concrete" and its amount shall be limited to that of type-A regulated in JIS A 5211 "Blast furnace slag cement". The fly ash shall conform to type-I or type-II regulated in JIS A6201 "Fly ash" and its amount shall be limited to that of type-A regulated in JIS A 5213 "Fly ash cement".

(e) Expansive additive

Expansive additive provided in JIS A 6202 "Expansive additive for concrete" can be used if necessary.

(f) Viscosity agent

Viscosity agent can be used if necessary.

(2) Materials for grouting mortar

(a) Cement

Cement shall follow provisions in the section 4.2 (1) (a).

(b) Aggregate

Aggregate shall be the fine aggregate provided in JASS5 4.3.

(c) Admixture

The chemical admixture for concrete provided in the section 4.2 (1) (c) can be used.

(d) Expansive additive

Expansive additive shall be used whenever grouting with mortar. When the cellular like aluminum powder etc. is used, management by experienced engineers shall be required.

(e) Viscosity agent

Viscosity agent can be used if necessary.

(3) Steel bar etc.

Steel bar shall conform to the standard of JIS G 3112 “Steel bar for reinforced concrete”. It shall be deformed bar, unless specified.

Welded wire fabric shall conform to the standard of JIS G 3551 “Welded wire fabric”. The diameter of steel bar used shall be not less than 4 mm. Post-installed anchors shall follow the provisions of the section 3.9.

(4) Steel material etc.

Steel material shall conform to the provisions of JIS G 3101 “Rolled steels for general structure”, JIS G 3106 “Rolled steels for welded structure”, or JIS G 3136 “Rolled steels for building structure”. Thickness of the steel material or steel strap shall be not less than 4.5mm. The headed stud shall follow the provisions in JIS.

(5) Epoxy resin etc.

Epoxy resin and resin mortar can be used for post-installed anchors, crack repair, and bonding new and old concrete when they have sufficient durability and fire resistance. Epoxy resin or methacrylate resin, provided in the “Seismic Retrofit Design and Construction Guidelines for Existing Reinforced Concrete Buildings and Concrete Encased Steel Buildings Using Continuous Fiber Reinforced Materials (1999 version)” published by the Japan Building Disaster Prevention Association, referred to as “FRP” hereafter, shall be used for the strengthening with continuous fiber reinforcement

(6) Carbon fiber and Aramid fiber

Carbon fiber and aramid fiber shall follow the “FRP Strengthening Guidelines”.

(7) Other material

Investigations shall be required to use materials that appear in the sections 3.1 through 3.10 but those not described the provisions of (1) to (6) in this section.

4.3 Removal of Finishing and Concrete Chipping

(1) Removal of existing finishing

Finishing such as interior decorations, plaster and mortar at the surface of concrete member shall be removed before retrofitting work.

Interior decorations, fixture and piping equipment etc. in and around the construction area shall be removed to facilitate the retrofitting work.

(2) Treatment or chipping of the surface of existing concrete

The surface of existing concrete on which new concrete is placed shall be appropriately roughened or chipped. Chipping of concrete shall be minimized. The existing steel bar shall not be damaged in case of chipping and drilling. The suitable supporting members shall be set when the deflection of existing structure is expected to increase against permanent load due to the chipping or drilling.

4.4 Post-Installed Anchor

(1) General

(a) The material and shape of post-installed anchor shall follow the section 3.9.1.

(b) Post-installed anchors shall be installed by skilled engineers with enough knowledge and construction technique.

(c) Construction management shall be done to secure the necessary construction quality based on the pre-determined check items.

(d) Consultation with designer and site manager shall be done for the various problems raised on the construction site.

(2) Construction procedure

Post-installed anchors shall be installed in accordance with the standard construction procedure.

(3) Management and inspection

Compressive strength of existing concrete and material strength of post-installed anchors found in the test reports shall be confirmed to be higher than the design strength. The pull-out strength of post-installed anchors shall be inspected on site to confirm that they are properly installed.

4.5 Reinforcing Bar Arrangement and Steel Construction

(1) In case of adding new reinforced concrete members, the new steel bars shall be effectively anchored to the existing structure or its longitudinal bar.

(2) Interval and embedment length of post-installed anchors shall follow the section 3.9.

(3) Anchorage bars shall be hooked with 135 degree or welded to existing steel bars.

(4) The lap splice length of welded wire fabric, which is used to improve column ductility,

shall be greater than hoop spacing plus 100mm or 200mm, whichever is larger.

(5) Welding to longitudinal reinforcing bars in existing member shall be carefully made not to deteriorate their mechanical properties. Also welding shall be done by licensed welders qualified according to JIS Z 3801 “Test Methods and Judgment Criterion of the Technical Examination of Welding”, or certificated by Japan Welding Association.

(6) Prefabricated elements shall be applied to steel framework, in principle. Construction shall be based on a well-considered work plan.

4.6 Concrete Casting

(1) Plan of concrete casting

(a) General

Since the retrofit is made in various parts of a building and the small amount of concrete is cast in the retrofit work, concrete shall be carefully placed so as to secure the required quality of concrete at each casting.

(b) Selection of ready-mixed concrete plant

In case using ready-mixed concrete, a plant shall be selected so that the whole process from concrete mixing to casting, shall be completed within a certain acceptable time. The plant shall be certificated as JIS plant.

(c) Decision of area for concrete placing

Area for concrete placing shall be determined so that the placing can be completed in consideration of carrying method to the site, casting in each building part, required time for consolidation, practicable volume for casting in a day, time limit from mixing to the completing of casting etc.

(2) Mix proportion

(a) Concrete strength for mix proportion

Specified design strength of concrete F_c shall not be less than the specified design strength of existing concrete nor 21N/mm^2.

(b) Unit volume of water

Maximum unit volume of water shall be 185 kg/m^3.

(c) Unit volume of cement

Minimum unit volume of cement shall be 270 kg/m^3.

(d) Water to cement ratio

Maximum water to cement ratio shall be 65 %.

(e) Slump

Slump value shall not be greater than 18cm. It shall be as small as possible when concrete casting can be properly conducted.

(f) Air content

The target air content shall be 4.5 % for concrete with air entraining agent, air entraining and water reducing agent, or high performance air entraining and water reducing agent.

(g) Content of chloride

Content of chloride in concrete shall be not more than 0.3 kg/m^3 in content of chloride ion, Cl^-.

(h) Concrete shall be the quality unlikely to cause alkali-aggregate reaction.

(3) Preparation before casting

(a) Before casting new concrete, chipped surface of existing concrete shall be sufficiently cleaned up and blown off with compressed air or vacuum cleaner.

(b) The surface of forms and existing concrete shall be soaked before casting new concrete not to absorb its water.

(4) Casting and consolidation

(a) There are three casting methods; placing from bucket-shaped continuous forms provided below beams, placing through opening perforated in the slab of upper floor, and injecting with pressure from the bottom.

(b) Concrete shall be first cast up to around 20cm below the beam, and then the remaining portion shall be grouted with pressure not to leave unfilled gaps, and/or openings.

(c) Vibrators as well as hitting shall be applied in concrete consolidation.

(5) Curing

(a) After casting concrete, watering on the surface of forms and covering shall be done so as not to leave the concrete dry.

(b) Sprinkling water and covering concrete surface with sheet shall be done if wet condition is required for curing after forms are removed.

(c) When the expansive admixture is used, concrete shall be cured for more than 7 days under wet condition.

(6) Form

(a) Forms shall be designed in consideration of concrete lateral pressure, casting method and setting method of separators and so on.

(b) Forms shall be carefully assembled so that members should be accurately sized and positioned. Appropriate leakage resistive measures shall be taken at the form adjacent to existing member.

(c) Appropriate measures shall be taken to prevent deformation of steel plates due to concrete lateral pressure in the case of steel plate jacketing.

(d) Forms shall be assembled so that vibrators can be easily applied.

4.7 Mortar

(1) Scope

This section applies to mortar used for strengthening of columns and beams.

(2) Mix proportion of mortar

(a) Compressive strength of mortar shall not be less than the specified design strength of existing concrete.

(b) Consistency shall be as hard as possible when the concrete cause properly and densely cast.

(c) Mixture shall follow Table 4.7-1 depending on the consistency of mortar obtained by the flow test specified in JIS R 5210 “Physical Test Method of Cement”.

Table 4.7-1

Flow value f (mm)	Cement : sand (ratio in weight)
$f<180$	1 : 3
$180 \leqq f<240$	1 : 2.5
$240 \leqq f$	1 : 2

(3) Casting or spraying of mortar

(a) When mortar is cast in forms or steel plates for strengthening, it shall be placed from the top or injected with pressure from the bottom to obtain uniform and solid condition.

(b) Spraying shall be made according to JASS 15 “Plasterer works”.

(c) Surface of the existing concrete and forms shall be soaked before casting or spraying.

(4) Curing

Mortar shall be cured in the same manner as concrete.

4.8 Grouting

(1) Scope

This section shall be applied to grouting material injected, either with or without pressure, between existing concrete member and new retrofit member. Grouting shall be applied in spaces such as between the top of added wall and the beam above, between the steel framed brace and existing concrete member, between the steel jacket and existing concrete column.

(2) Mix proportion

(a) Grout material shall be pre-mixed non-shrinkage mortar. Compressive strength of the mortar shall not be less than the specified design strength of concrete of retrofitting member nor the specified design strength of mortar for structure.

(b) Consistency shall be determined by the part and method of injection.

(c) Trial mixing shall be done before injection.

(3) Preparation of injection with or without pressure

(a) Laitance at the joint surface shall be removed completely before injection of grout.

(b) Surface of form and joint shall be soaked by spraying water.

(4) Injection with pressure

(a) Grouting shall be made by injecting with pressure at the around 20cm gap left between the wall top and beam above the wall, the gap between the steel framed brace and existing concrete structure and the gap between steel plate and existing concrete of steel jacketing of column.

(b) Water temperature shall be properly controlled at the time of mixture and injection with pressure.

(c) Injection with pressure shall be continuously done with appropriate pressure.

(d) Mortar shall be injected until the overflow from the air outlet is confirmed.

(5) Form

(a) Forms shall be set leakage resistive.

(b) Forms shall be stiff enough to resist the pressure during mortar injection and to confine the expansion pressure during hardening.

(c) Forms shall be removed after the mortar is sufficiently hardened and therefore the confinement of the expansion pressure is not necessary.

(d) It shall be confirmed after removing forms that gaps between new member and existing concrete are properly grouted.

(6) Curing

Three days shall be required for standard, curing and the temperature of grouting mortar shall not be below 5 degrees of centigrade. Other curing practices shall be the same as concrete works.

4.9 Continuous Fiber

(1) Work specification

The seismic retrofitting work using continuous fiber reinforcement covers four different combinations of continuous fibers and impregnate adhesive resin in the guidelines as shown below.

(a) Carbon fiber / epoxy resin work method

(b) Carbon fiber / methacrylate resin work method

(c) Aramid fiber / epoxy resin work method

(d) Glass fiber / epoxy resin work method

The work specification of each method shall be determined according to the “Seismic Retrofit Design and Construction Guidelines for Existing Reinforced Concrete Buildings and Concrete Encased Steel Building Using Continuous Fiber Reinforced Materials (1999 version)” and the specifications of each method required for technical approval.

(2) Planning

(a) Contractor shall make a document of construction planning with appropriate work plan and allocation plan of the personnel based on the design drawings so that the effects of applied method is fully achieved.

(b) Contractor shall make a document of construction procedure in accordance with the document of construction planning.

(c) Construction shall follow the document of construction planning and the document of construction procedure.

(3) Construction procedure

The construction procedure of seismic retrofit method with continuous fiber reinforcement shall follow the standard construction flow as shown in Figure 4.9-1.

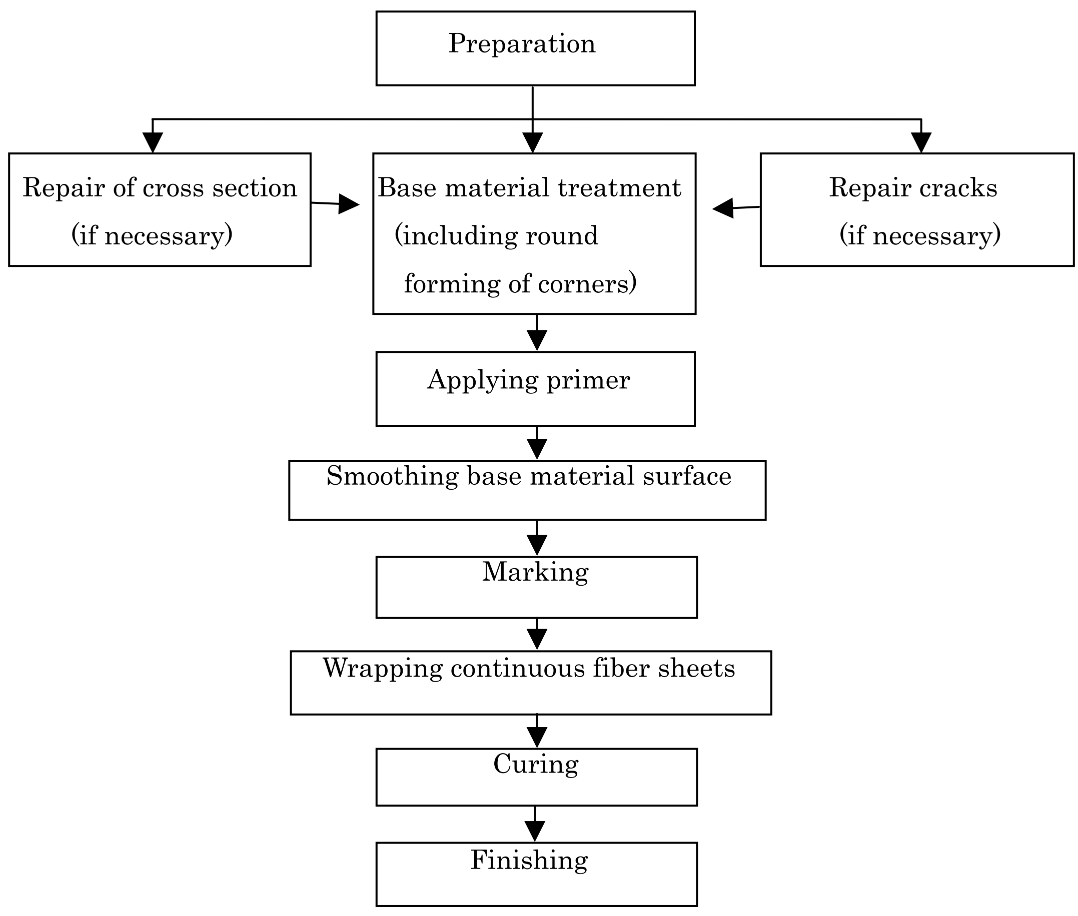


Figure 4.9-1 Flow of standard construction procedure

4.10 Press-Joint Method with PC Tendon

(1) General

Material and configuration, qualification of contractor and management of construction site shall be provided to achieve the performance of press-joint with PC tendon sufficiently.

(a) Material and configuration

Property of PC tendon shall meet JIS on PC tendons.

(b) Construction with PC tendon

Construction with PC tendon shall be done by skilled engineers with enough knowledge. Any problems raised on the construction site shall be solved through discussions by designers and the on-site manager.

(2) Construction procedure

Construction with PC tendons shall follow the provisions regulated in the document on construction planning.

4.11 Plastering, Finishing, and Carpentry Work

(1) Finishing and carpentry work shall be done by following other specifications after completing the retrofitting work.

(2) Enough water-proof finishing shall be done at the joint between new and existing concrete exposed to natural environment.

(3) When mortar finish is provided on jacketing steel or continuous fiber, the base material shall be appropriately pretreated to prevent mortar from peeling off.

4.12 Quality Control

Quality inspection of materials and productions used in the retrofit work and construction management shall be done based on the document of standard specifications. The lot for inspection the sampling number of tests shall be rationally determined to represent the quality of materials and productions.

A quality manager shall document the quality management plan and determine the necessary items, methods, number, time etc. of test or inspection, to ensure the required quality.

Translators’ Notes

Translators' Note 1 ---

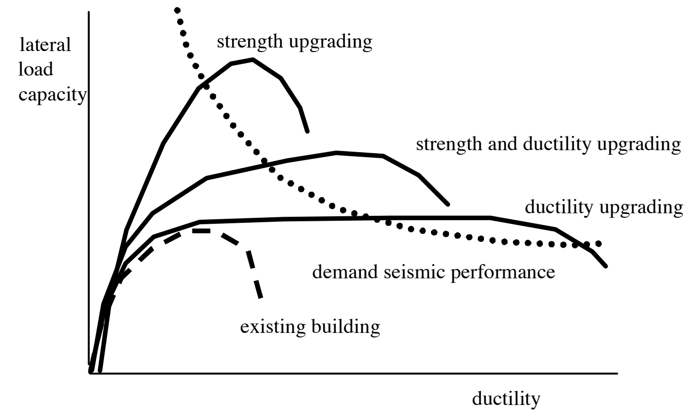


Figure TN.1 Seismic performance upgrading of existing building by retrofit

(quoted from the figure on page 58 in the commentary of 1.2 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 1

Translators' Note 2 ---

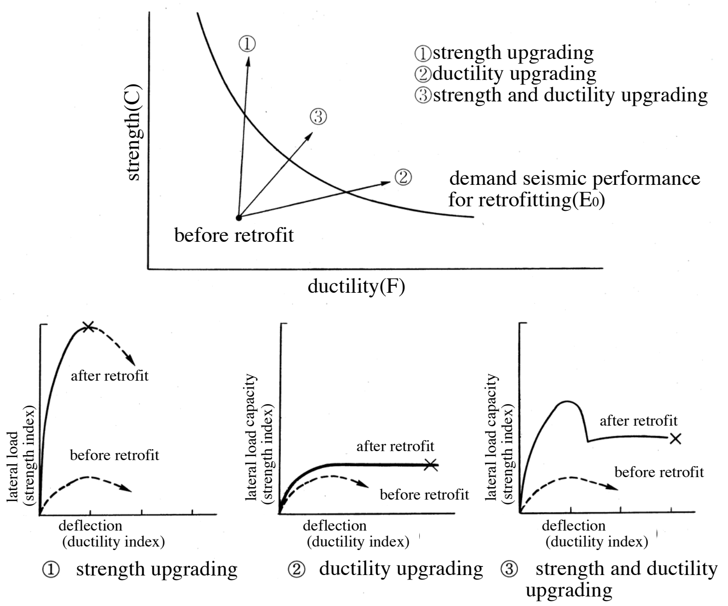


Figure TN.2 Concept of seismic retrofit

(quoted from the figure on page 63 in the commentary of 2.1.2 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 2

Translators' Note 3 --

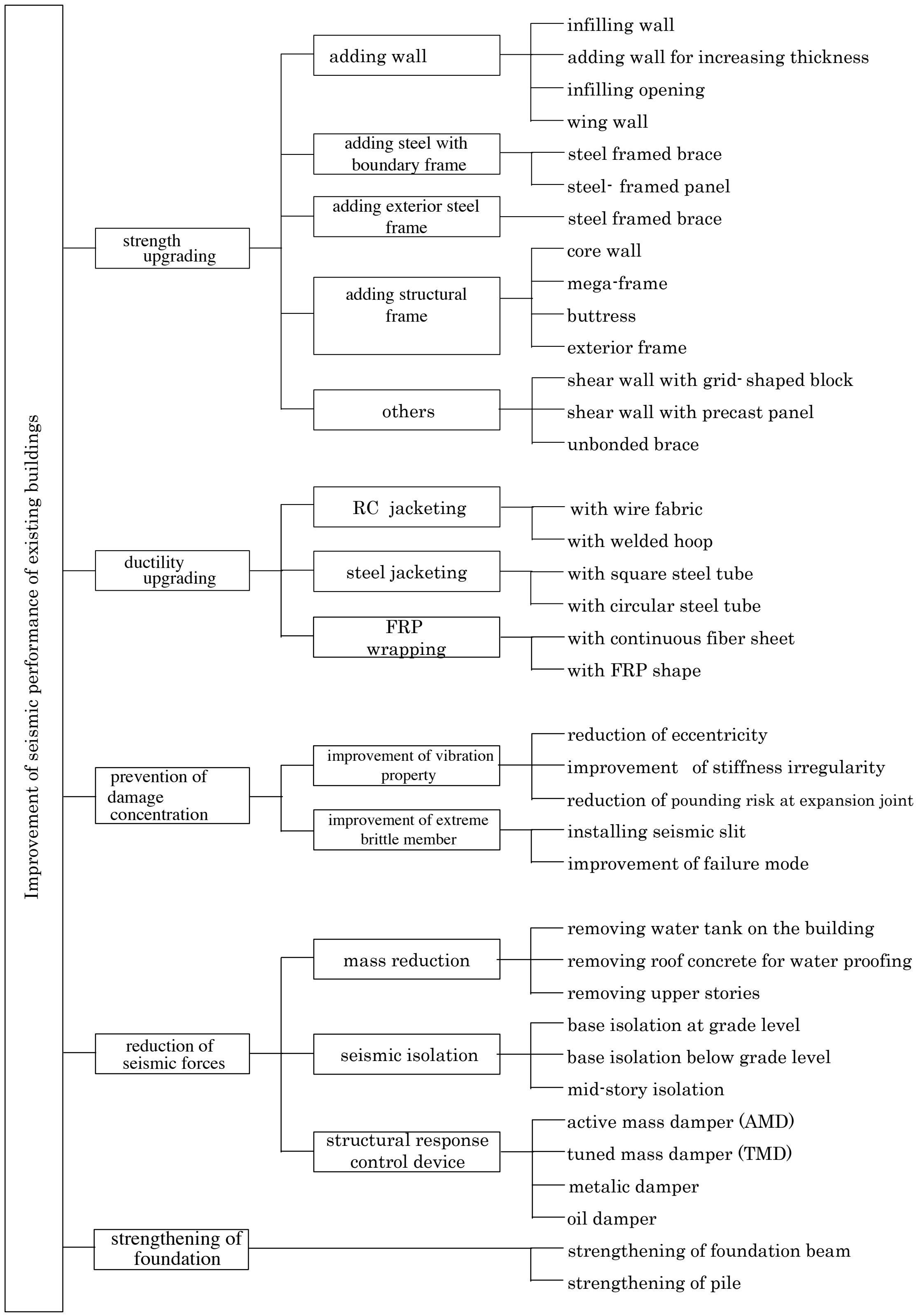


Figure TN.3 Classification of seismic upgrading methods

(quoted from the figure on page 67 in the commentary of 2.1.2 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 3

Translators' Note 4 --

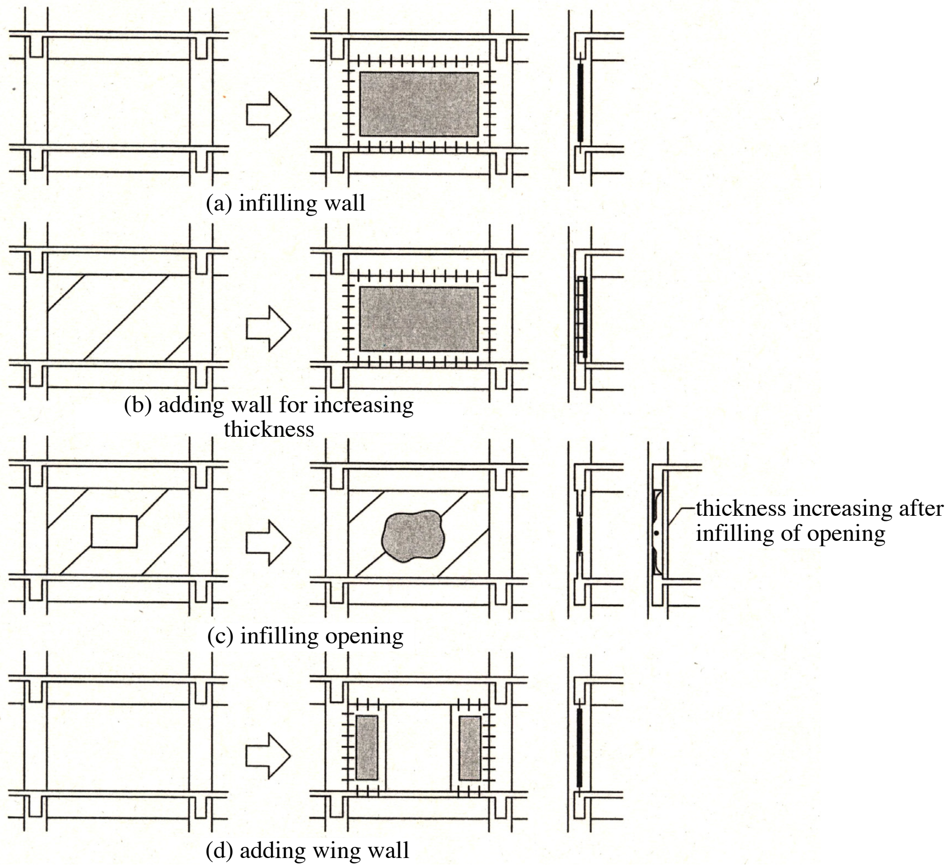
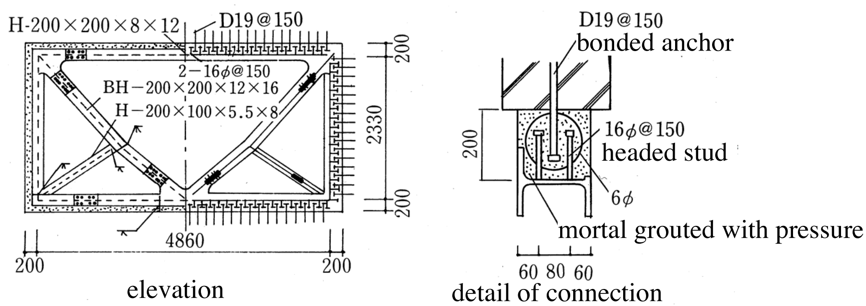


Figure TN.4 Construction methods for adding wall

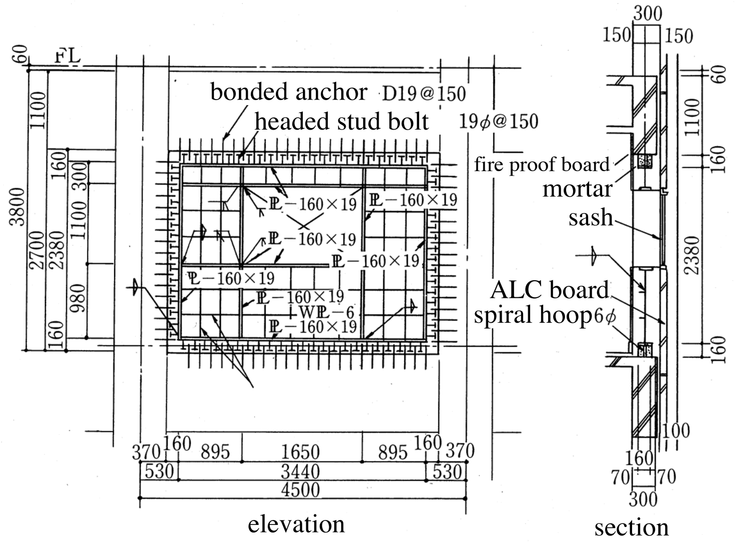
(quoted from the figure on page 68 in the commentary of 2.1.2 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 4

Translators' Note 5 --



(a) steel brace

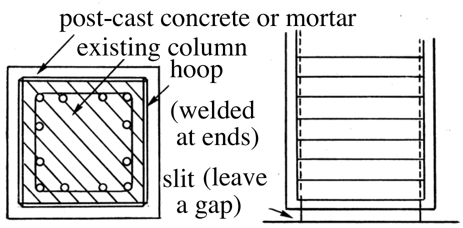


(b) steel panel

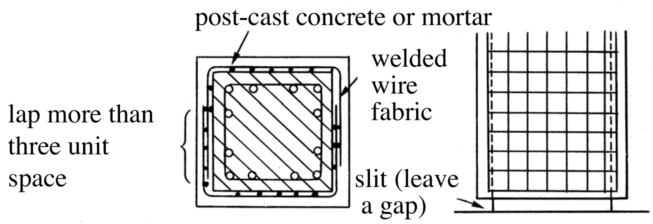
Figure TN.5 Examples of adding steel sections with boundary frame
(quoted from the figure on page 69 in the commentary of 2.1.2 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 5

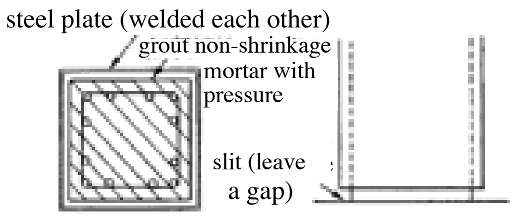
Translators' Note 6 --



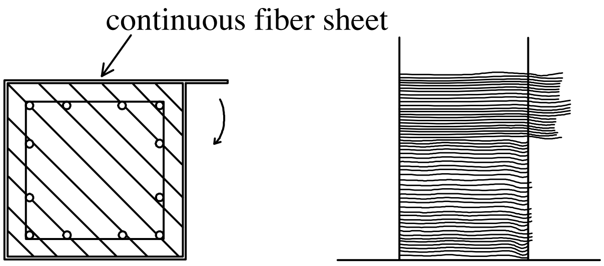
(a) Strengthening with welded closed hoop and concrete or mortar



(b) Strengthening with wire fabric and concrete or mortar



(c) Strengthening with steel jacketing



(d) Strengthening with FRP wrapping

Figure TN.6 Ductility upgrading methods of column

(quoted from the figure on page 70 in the commentary of 2.1.2 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 6

Translators' Note 7 ---

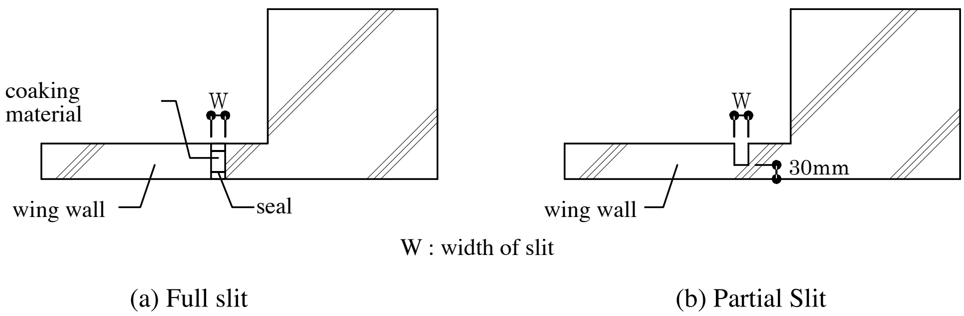
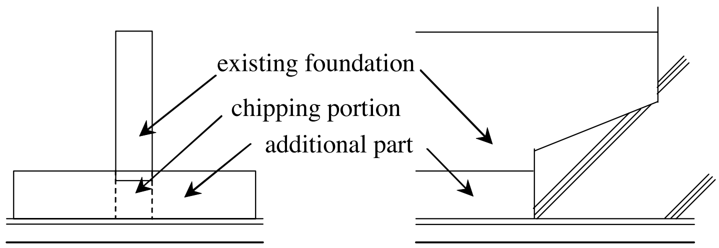


Figure TN.7 Examples of seismic slit

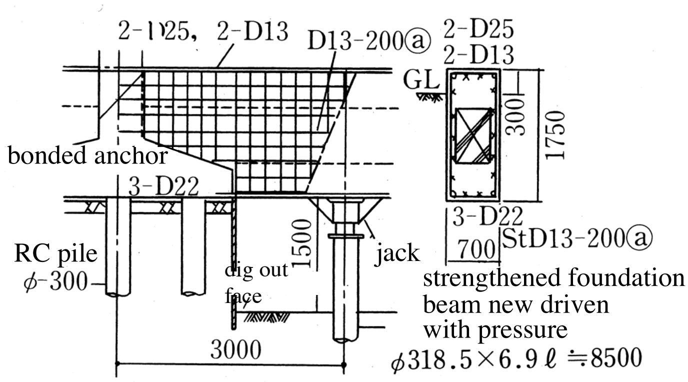
(quoted from the figure on page 71 in the commentary of 2.1.2 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 7

Translators' Note 8 ---



(a) additional foundation



(b) additional pile

Figure TN.8 Examples of strengthening of foundation

(quoted from the figure on page 72 in the commentary of 2.1.2 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 8

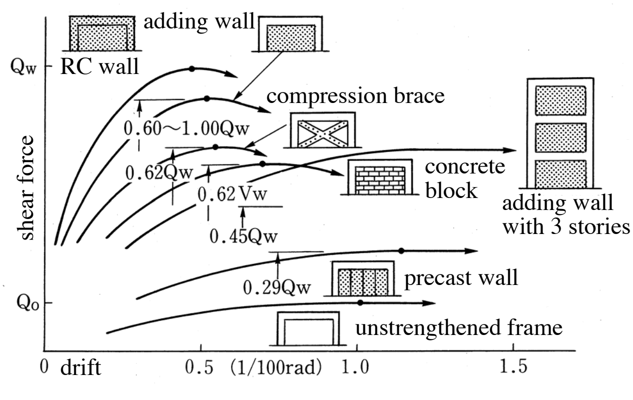
Translators' Note 9 ---

Figure (a) indicates 3.5 to 5.5 times in strength are obtained by infilling wall. 0.6 to 1.0 times in strength and a little bit increased ductility can be seen in case of infilling wall compared with monolithic RC wall.

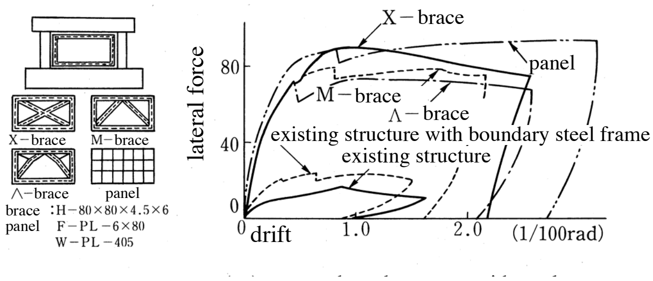
Figure (b) indicates remarkable increase in strength and ductility are obtained in case strengthening with steel brace or steel panel.

Figure (c) indicates both strength and ductility are increased by adding wing wall, ductility is increased remarkably by RC jacketing, steel jacketing, FRP wrapping and installing seismic slit.

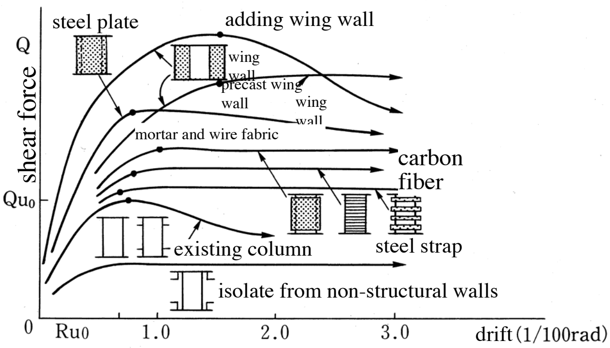
Those can be referred for predicting upgraded structural performance.



(a) strengthening of frame



(b) strengthened structure with steel brace with boundary steel frame



(c) strengthening of column

Figure TN.9 Strengthening effect observed in previous structural tests

(quoted from the figure on page 73 in the commentary of 2.1.2 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 9

Translators' Note 10 ---

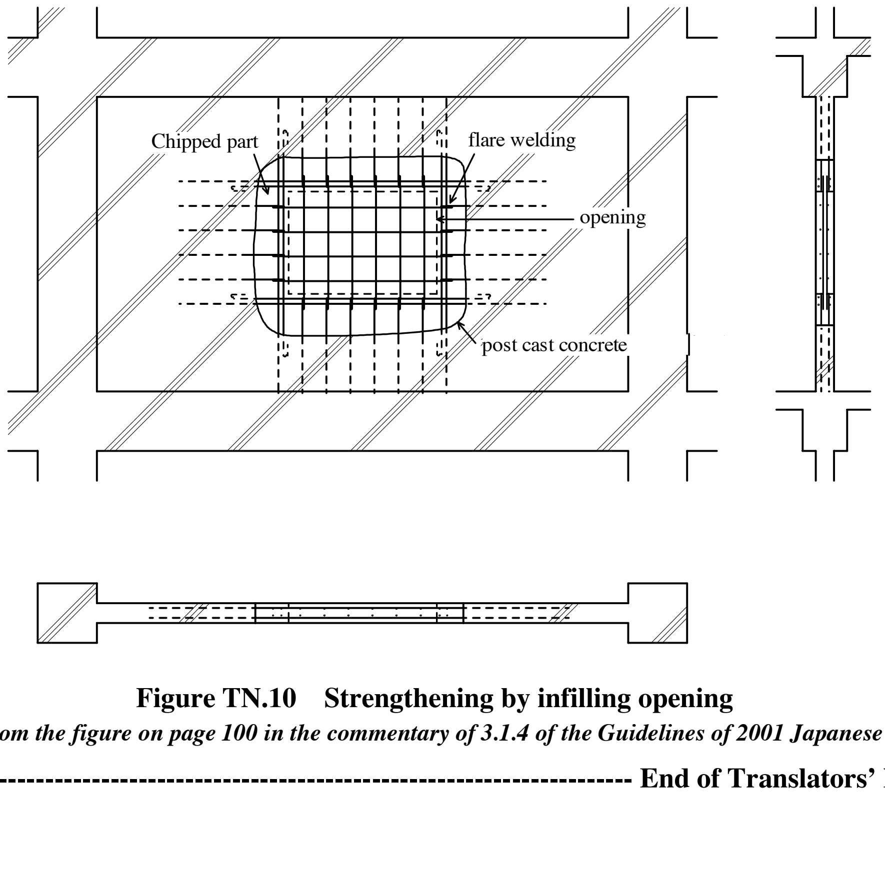


Figure TN.10 Strengthening by infilling opening

(quoted from the figure on page 100 in the commentary of 3.1.4 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 10

Translators' Note 11 ---

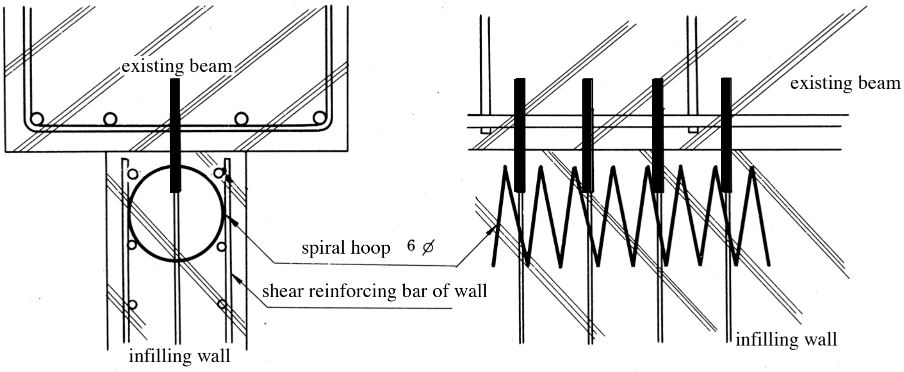


Figure TN.11 Strengthening against splitting with spiral reinforcing bars
(quoted from the figure on page 98 in the commentary of 3.1.4 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 11

Translators' Note 12 ---

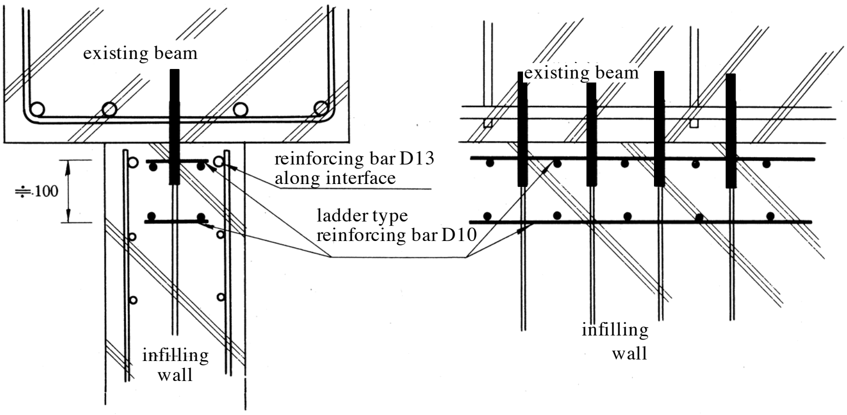


Figure TN.12 Strengthening against splitting with ladder type reinforcing bars
(quoted from the figure on page 98 in the commentary of 3.1.4 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 12

Translators' Note 13 ---

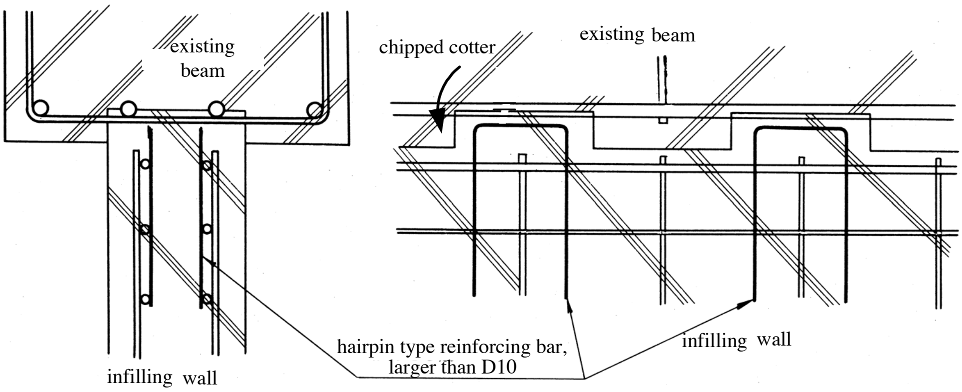


Figure TN.13 Strengthening with chipped cotter
(quoted from the figure on page 99 in the commentary of 3.1.4 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 13

Translators' Note 14 --

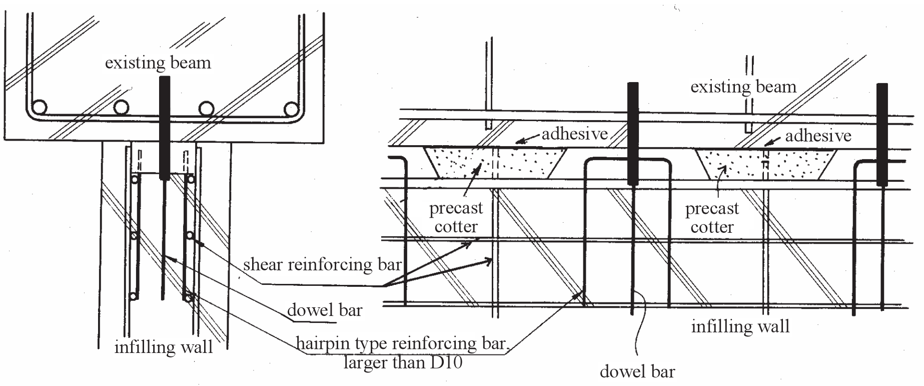


Figure TN.14 Strengthening with adhesive cotter

(quoted from the figure on page 99 in the commentary of 3.1.4 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 14

Translators' Note 15 --

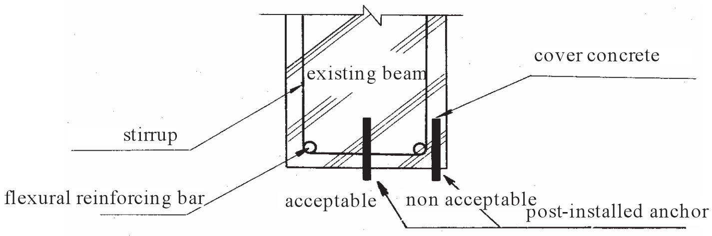
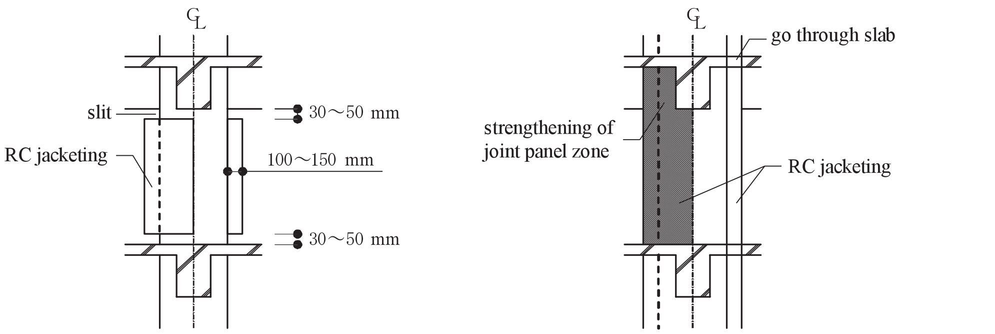


Figure TN.15 Installation position of post-installed anchor

(quoted from the figure on page 98 in the commentary of 3.1.4 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 15

Translators' Note 16 --



(a) in case of increase in shear strength (b) in case of increase in flexural, shear and axial strength

Figure TN.16 Column strengthening with RC jacketing

(quoted from the figure on page 145 in the commentary of 3.3.4 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 16

Translators' Note 17 ---

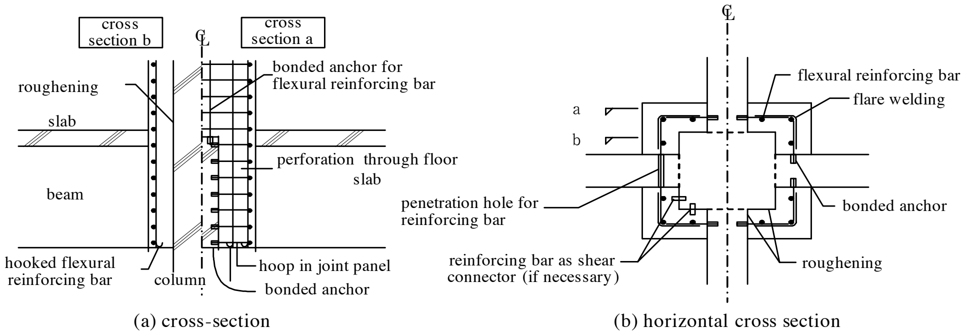


Figure TN.17 Strengthening example of joint panel zone
(quoted from the figure on page 146 in the commentary of 3.3.4 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 17

Translators' Note 18 ---

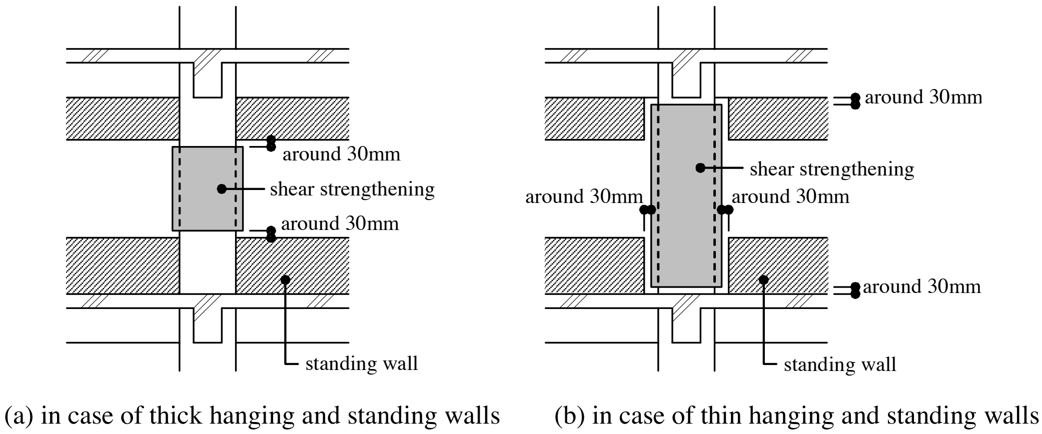


Figure TN.18 Strengthening of columns with hanging and standing walls
(quoted from the figure on page 147 in the commentary of 3.3.4 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 18

Translators' Note 19 ---

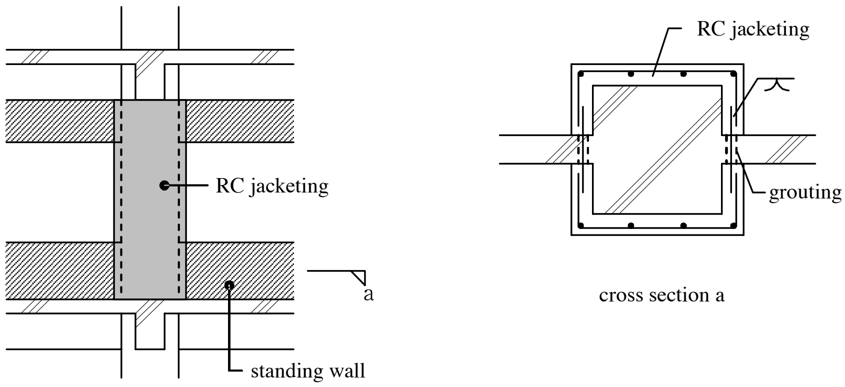


Figure TN.19 Strengthening of column together with hanging and standing walls
(quoted from the figure on page 147 in the commentary of 3.3.4 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 19

Translators' Note 20 --

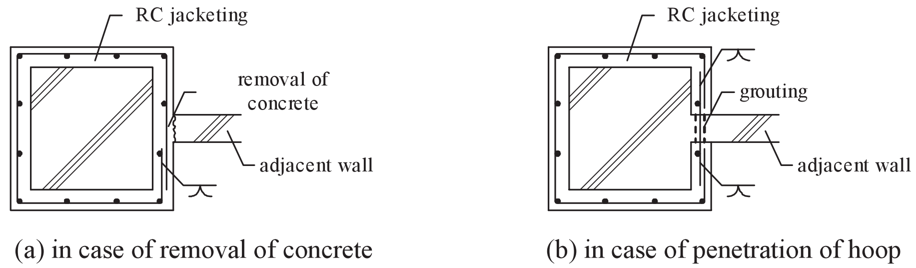


Figure TN.20 Strengthening of column with adjacent wall
(quoted from the figure on page 147 in the commentary of 3.3.4 of the Guidelines of 2001 Japanese version)

-- **End of Translators' Note 20**

Translators' Note 21 --

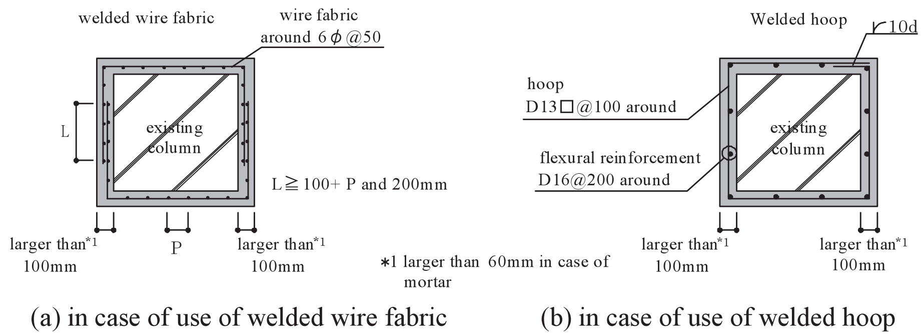


Figure TN.21 Examples of RC jacketing
(quoted from the figure on page 153 in the commentary of 3.3.4 of the Guidelines of 2001 Japanese version)

-- **End of Translators' Note 21**

Translators' Note 22 --

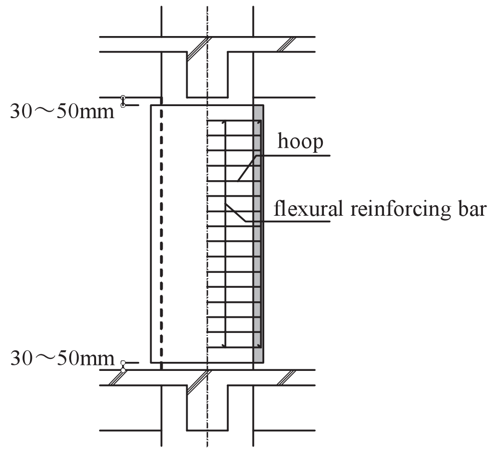


Figure TN.22 Location of slit
(quoted from the figure on page 153 in the commentary of 3.3.4 of the Guidelines of 2001 Japanese version)

-- **End of Translators' Note 22**

Translators' Note 23 --

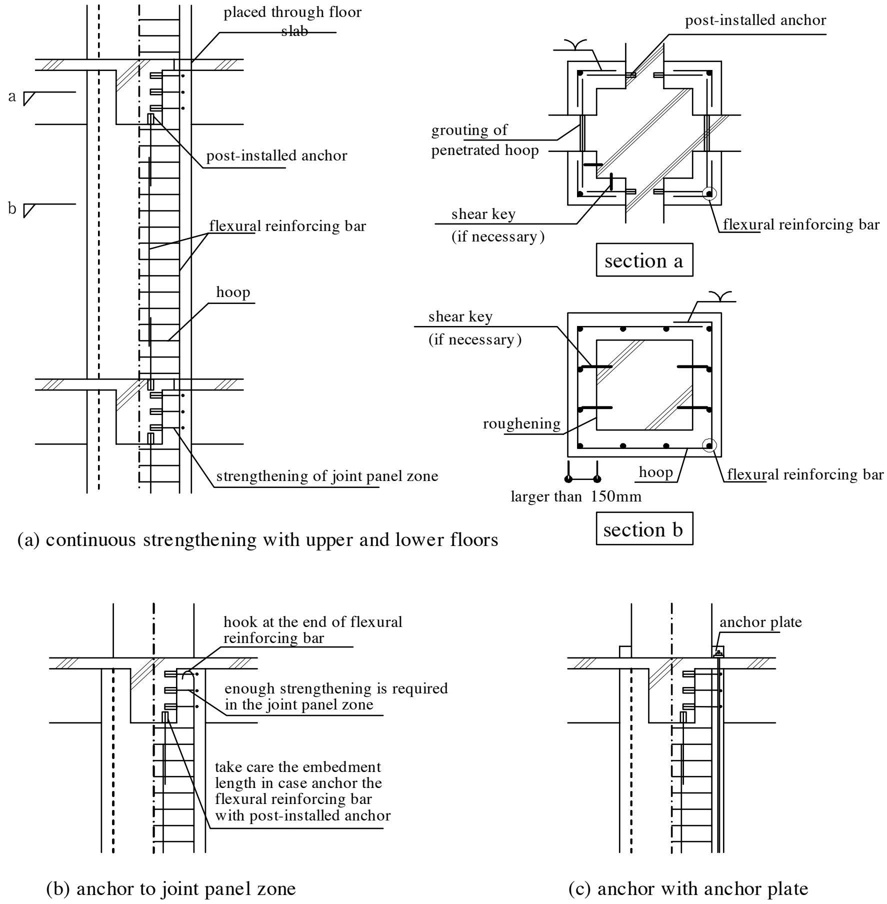


Figure TN.23 RC jacketing to increase flexural strength

(quoted from the figure on page 154 in the commentary of 3.1.4 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 23

Translators' Note 24 --

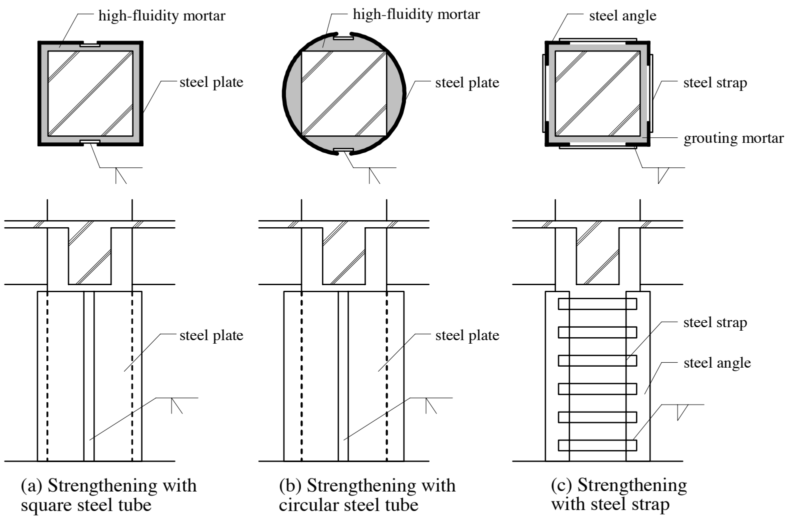


Figure TN.24 Steel jacketing

(quoted from the figure on page 155 in the commentary of 3.3.5 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 24

Translators' Note 25 ---

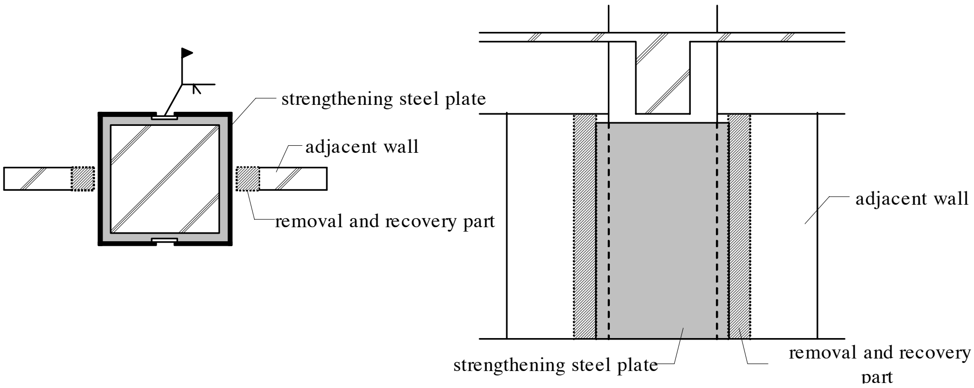


Figure TN.25 Strengthening of column with attached walls

(quoted from the figure on page 156 in the commentary of 3.3.5 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 25

Translators' Note 26 ---

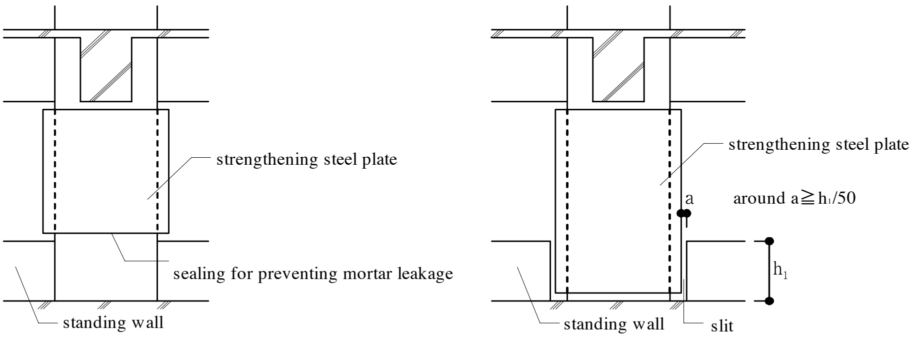
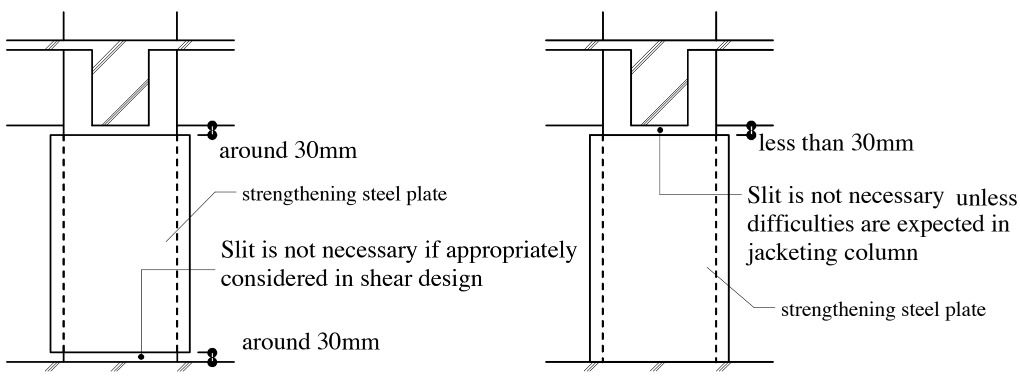


Figure TN.26 Strengthening of column with standing wall

(quoted from the figure on page 156 in the commentary of 3.3.5 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 26

Translators' Note 27 ---



(a) in case of ductility upgrading (b) in case of axial strength upgrading

Figure TN.27 Slit position for steel jacketing

(quoted from the figure on page 157 in the commentary of 3.3.5 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 27

Translators' Note 28 --

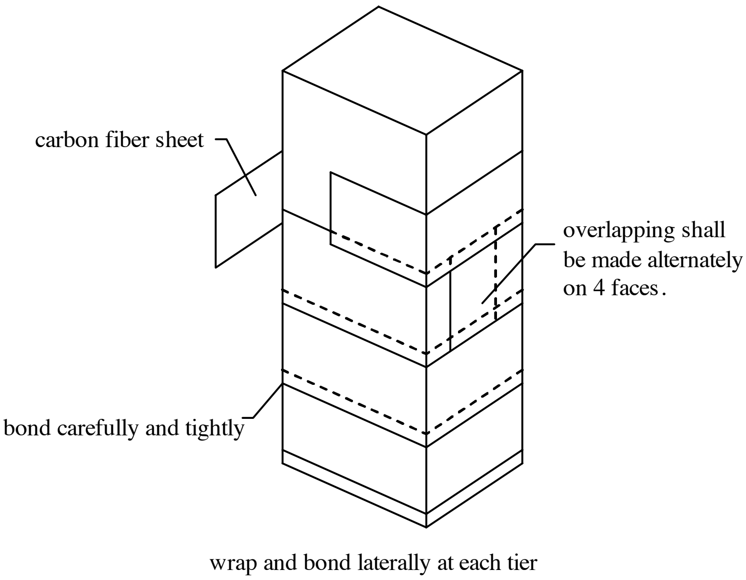


Figure TN.28 Strengthening with carbon fiber sheet wrapping

(quoted from the figure on page 163 in the commentary of 3.3.6 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 28

Translators' Note 29 --

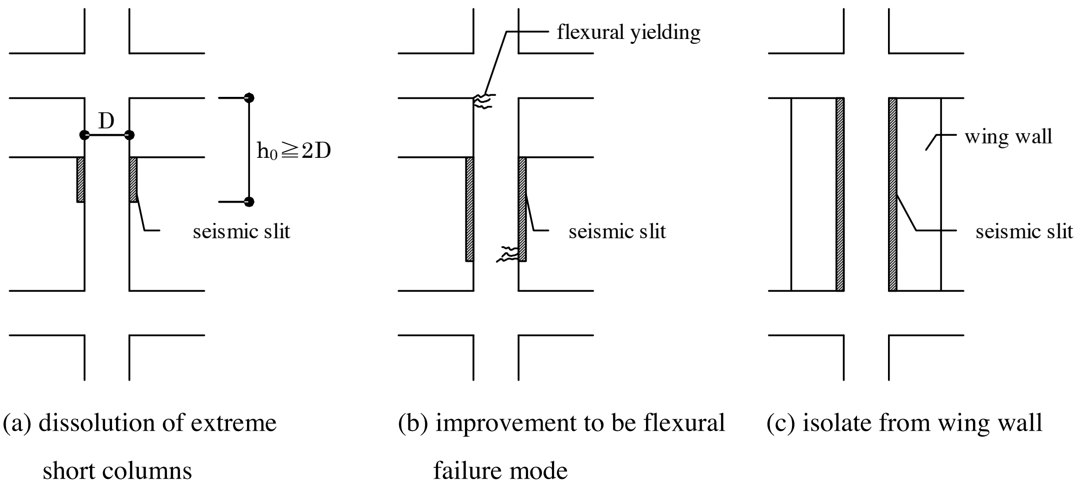
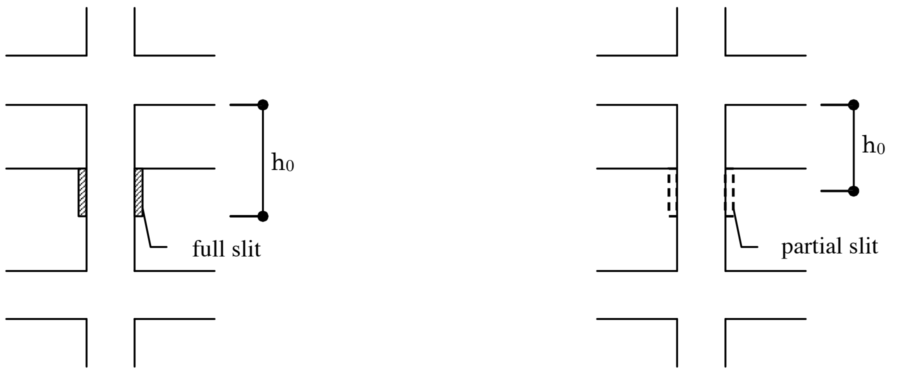


Figure TN.29 Objectives and location of seismic slit

(quoted from the figure on page 171 in the commentary of 3.3.7 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 29

Translators' Note 30 --



(a) in case of full slit (b) in case of partial slit

Figure TN.30 Clear span of columns with seismic slit (h_0)

(quoted from the figure on page 172 in the commentary of 3.3.7 of the Guidelines of 2001 Japanese version)

-- **End of Translators' Note 30**

Translators' Note 31 --

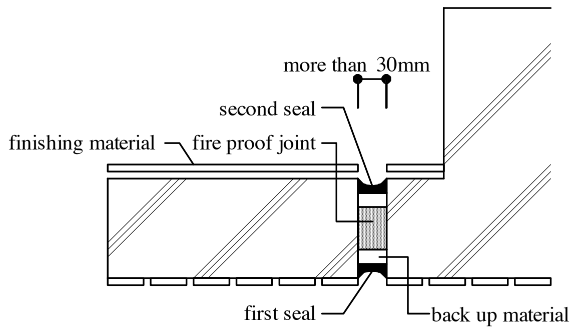


Figure TN.31 Detail of seismic slit

(quoted from the figure on page 173 in the commentary of 3.3.7 of the Guidelines of 2001 Japanese version)

-- **End of Translators' Note 31**

Translators' Note 32 --

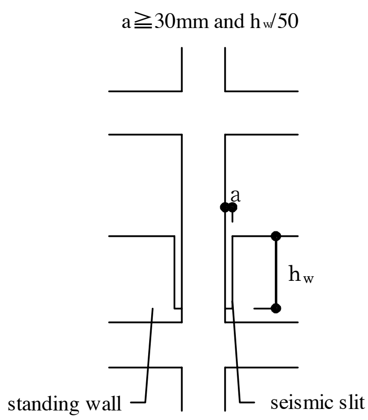


Figure TN.32 Width of seismic slit (a)

(quoted from the figure on page 173 in the commentary of 3.3.7 of the Guidelines of 2001 Japanese version)

-- **End of Translators' Note 32**

Translators' Note 33 ---

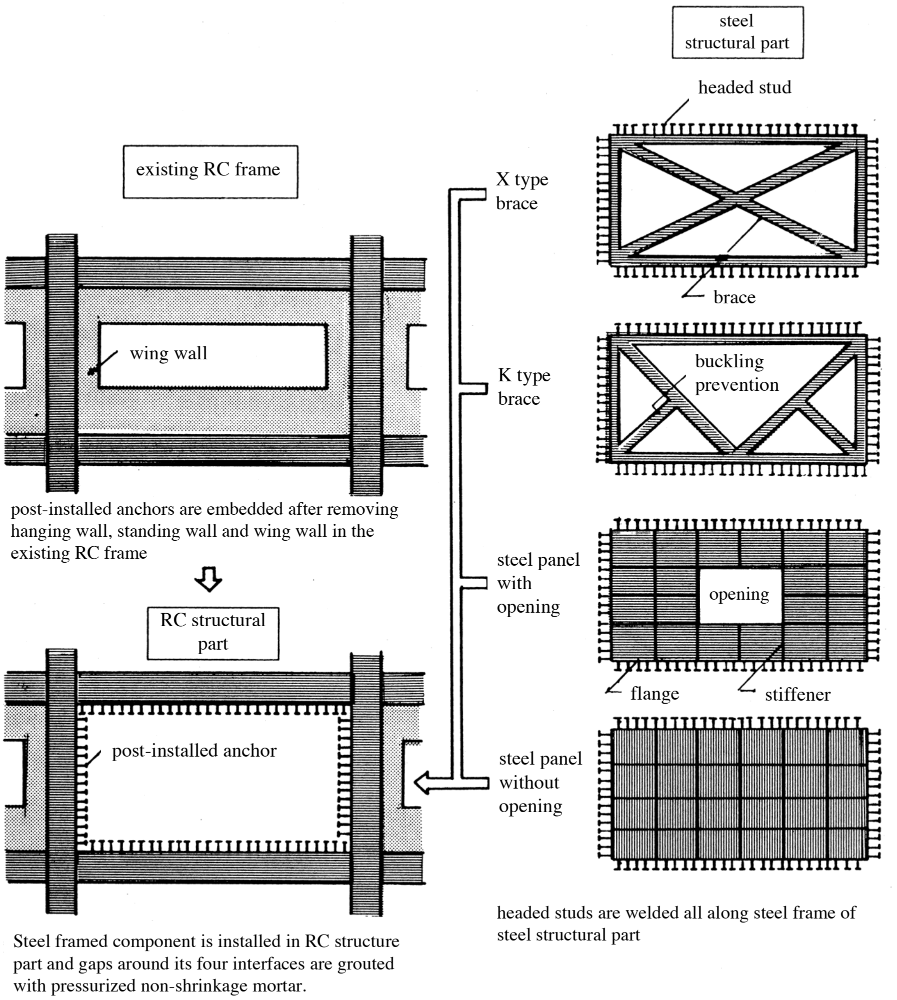


Figure TN.33 Examples of steel framed strengthening component

(quoted from the figure on page 180 in the commentary of 3.4.1 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 33

Translators' Note 34 ---

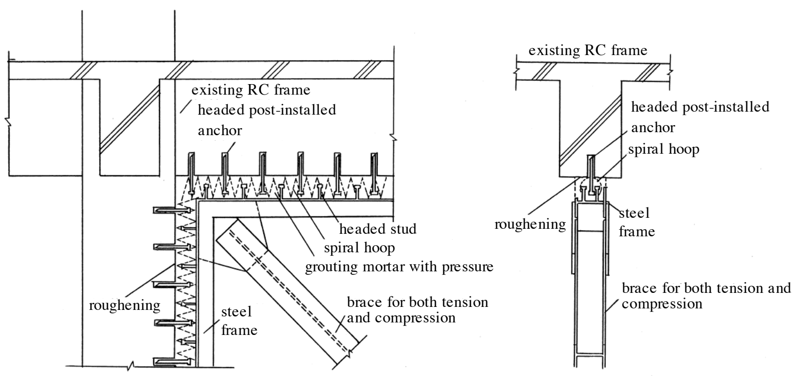


Figure TN.34 Example detail of indirect connection

(quoted from the figure on page 181 in the commentary of 3.4.1 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 34

Translators' Note 35 ---

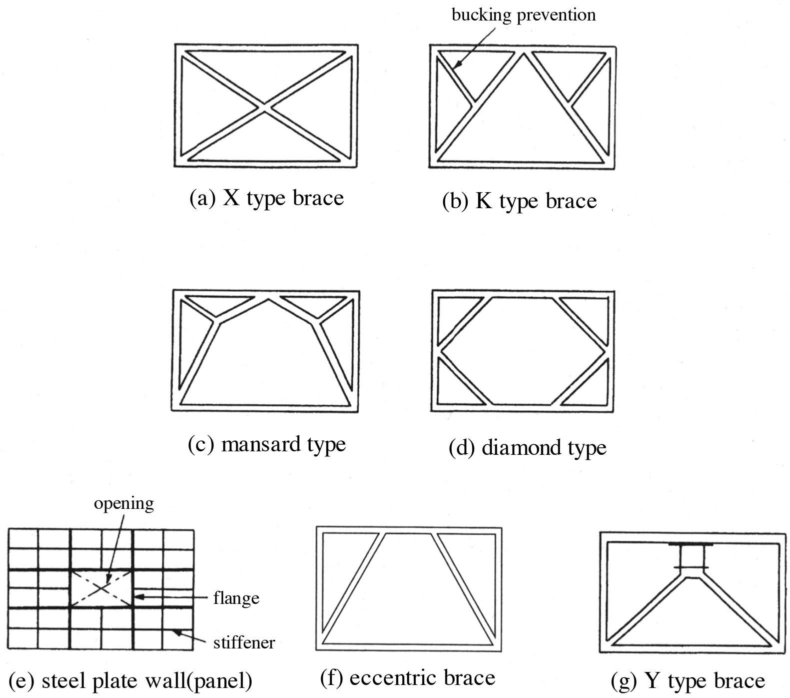
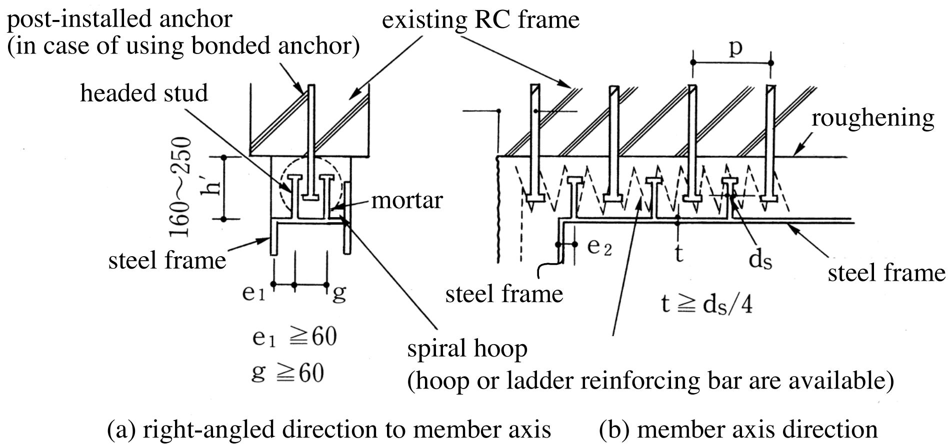


Figure TN.35 Shape of brace structures

(quoted from the figure on page 195 in the commentary of 3.4.4 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 35

Translators' Note 36 ---



the pitch p and end space e_2 would follow to provisions provided in section 3-9

e_1: space between headed stud and edge of steel frame

e_2: space between headed stud and end of steel frame

g : gauge of headed stud

Figure TN.36 Indirect connection of steel frame with existing RC member (unit: mm)

(quoted from the figure on page 198 in the commentary of 3.4.4 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 36

Translators' Note 37 --

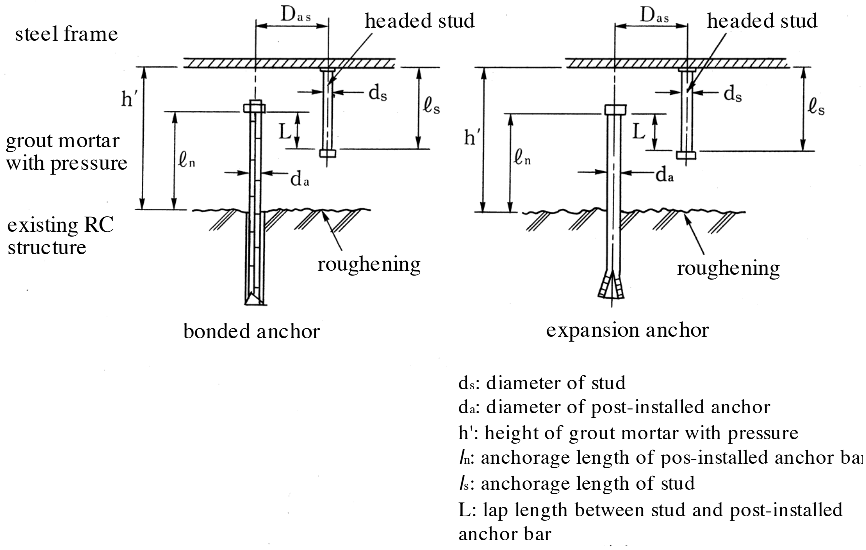


Figure TN.37 Lap length between post-installed anchor and headed stud

(quoted from the figure on page 200 in the commentary of 3.4.4 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 37

Translators' Note 38 --

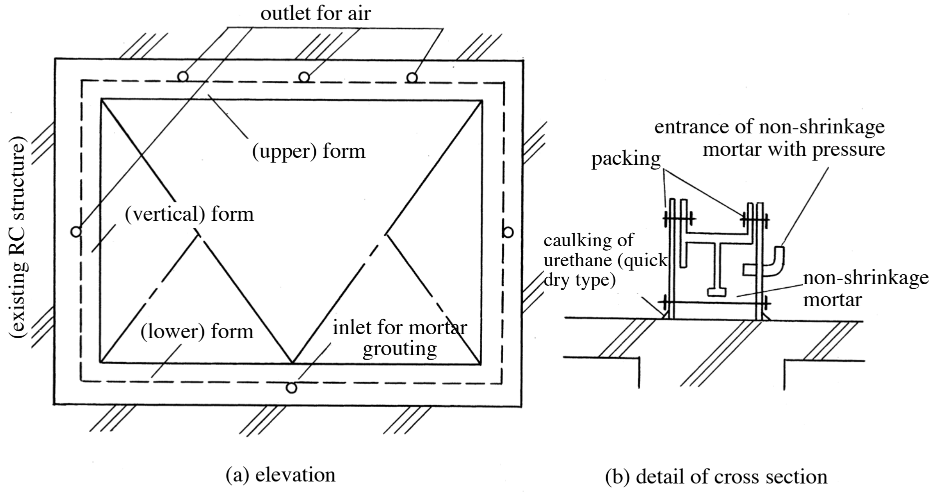


Figure TN.38 Grouting method of non-shrinkage mortar with pressure

(quoted from the figure on page 202 in the commentary of 3.4.4 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 38

Translators' Note 39 --

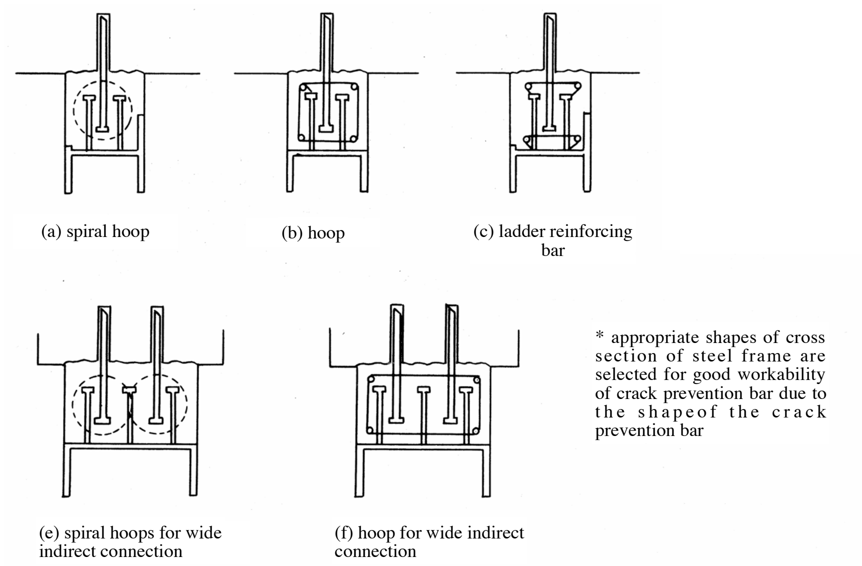


Figure TN.39 Examples of crack prevention bar

(quoted from the figure on page 202 in the commentary of 3.4.4 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 39

Translators' Note 40 --

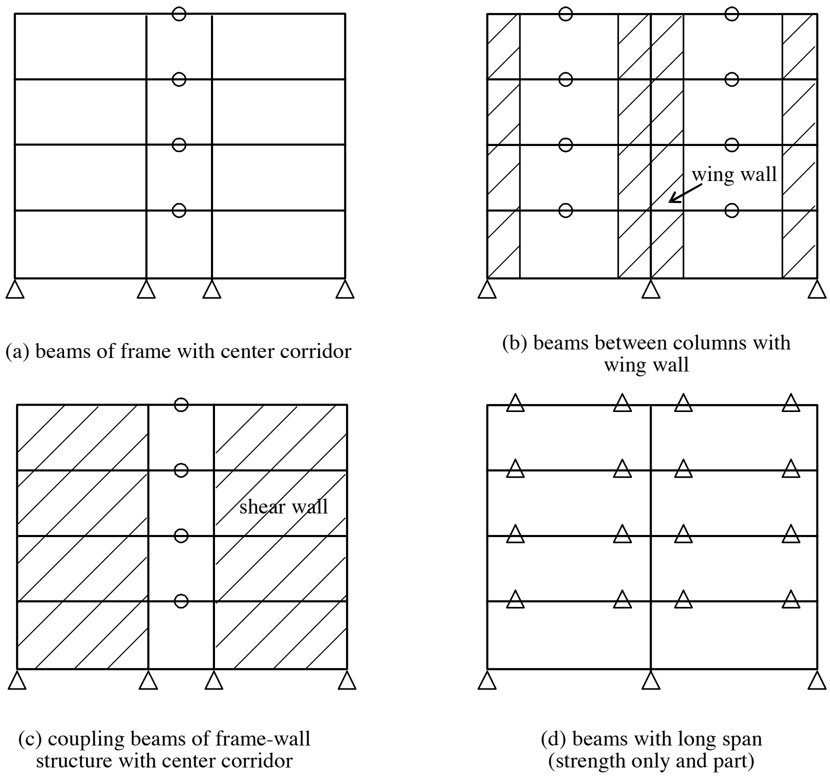


Figure TN.40 Examples of beam strengthening effectively contributing to whole building's performance (○:strengthening over whole length, △:strengthening only member's end)

(quoted from the figure on page 220 in the commentary of 3.5.1 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 40

Translators' Note 41 --

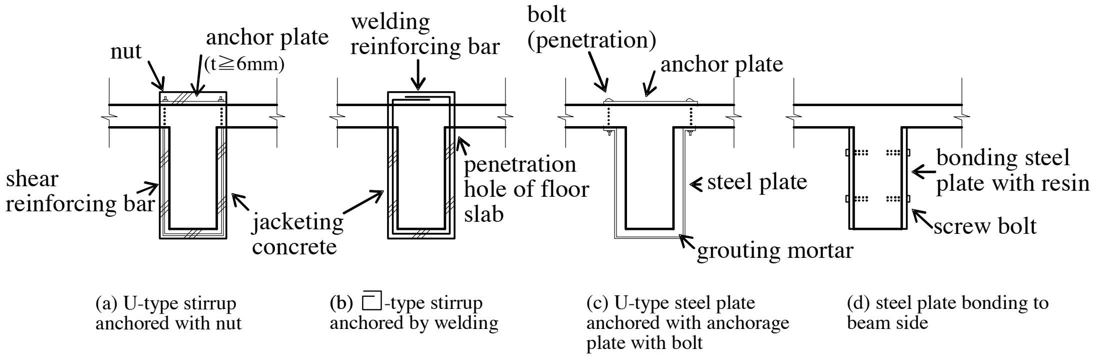


Figure TN.41 Examples of beam strengthening

(quoted from the figure on page 224 in the commentary of 3.5.4 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 41

Translators' Note 42 --

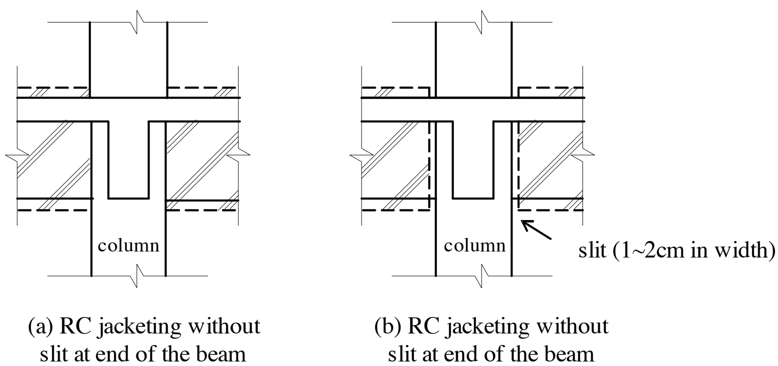


Figure TN.42 Details of RC jacketing of beams

(quoted from the figure on page 225 in the commentary of 3.5.4 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 42

Translators' Note 43 --

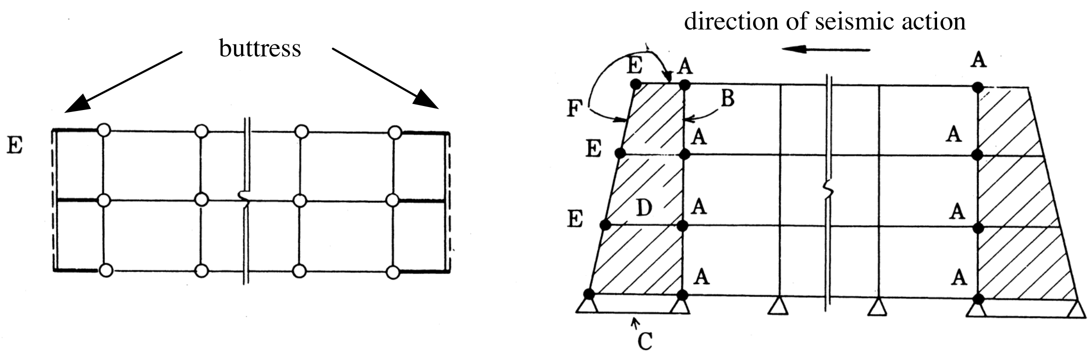


Figure TN.43 Examples of appropriately located buttress

(quoted from the figure on page 228 in the commentary of 3.6.2 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 43

Translators' Note 44 ---

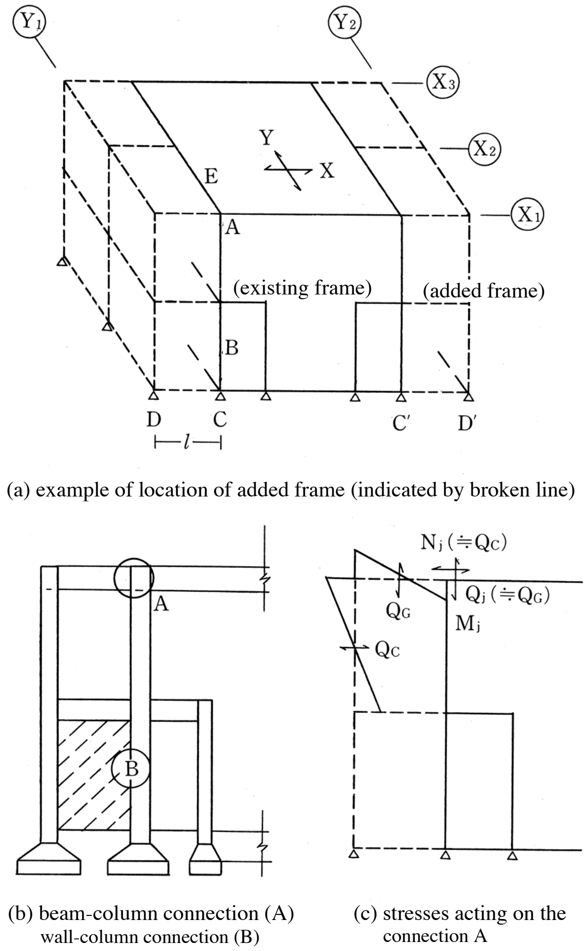


Figure TN.44 Example of adding spatial frame

(quoted from the figure on page 232 in the commentary of 3.6.3 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 44

Translators' Note 45 ---

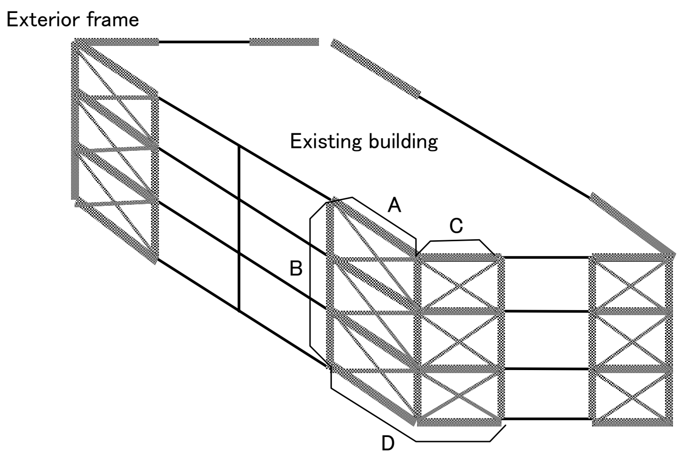
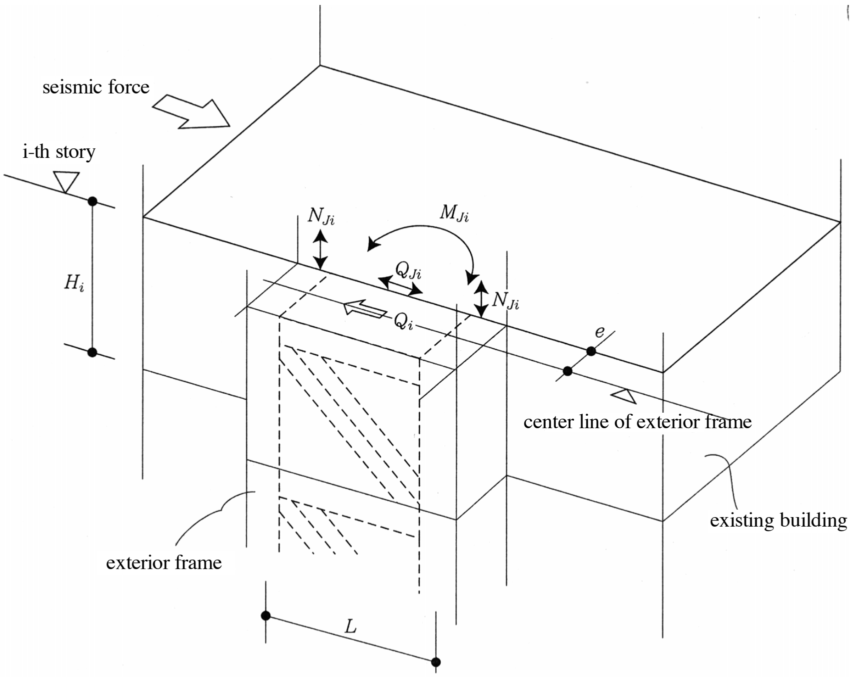


Figure TN.45 Exterior frame (steel framed brace)

(quoted from the figure on page 236 in the commentary of 3.6.4 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 45

Translators' Note 46 --



$$Q_{Ji} = Q_i - Q_{i+1}$$

$$M_{Ji} = Q_{Ji} \times e$$

$$N_{Ji} = Q_i \times \left(\frac{H_i}{L} \right) \times \left(\frac{K_O}{K_O + K_F} \right)$$

where,

Qi	: lateral shear force of exterior frame (i-th story)
QJi	: lateral shear force of joint(i-th story)
MJi	: moment of joint (i-th story)
NJi	: axial force of joint (t-th story)
e	: lateral distance between center of exterior frame and joint
Ko	: axial stiffness of existing column adjacent to the column of exterior frame
KF	: axial stiffness of column of exterior frame
Hi	: story height (i-th story)
L	: span of exterior frame

Figure TN.46 Stresses acting on the joint between exterior frame and existing building

(quoted from the figure on page 237 in the commentary of 3.6.4 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 46

Translators' Note 47 ---

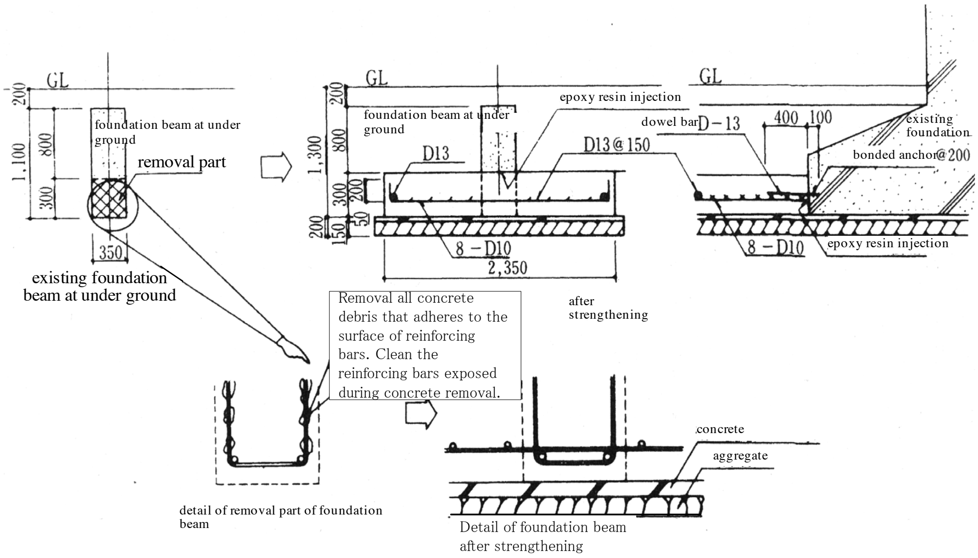


Figure TN.47 Strengthening of foundation

(quoted from the figure on page 243 in the commentary of 3.7.5 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 47

Translators' Note 48 ---

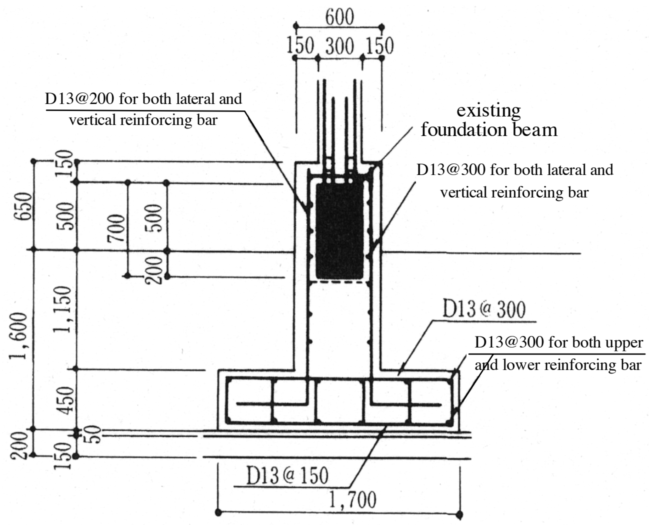


Figure TN.48 Strengthening of foundation beam

(quoted from the figure on page 244 in the commentary of 3.7.5 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 48

Translators' Note 49 ---

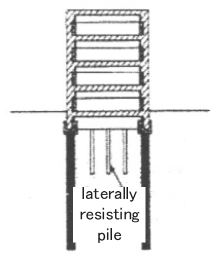
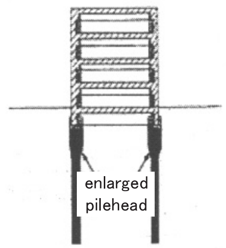
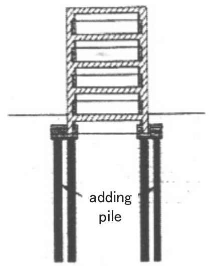
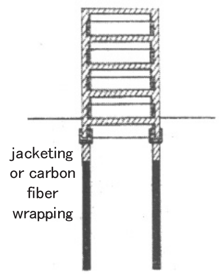
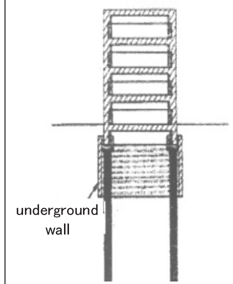
Construction method	Lateral resistance pile	Enlargement of existing pile head	Adding pile	Steel jacketing or carbon fiber wrapping	Strengthening with underground wall
Image					
Outline	add piles resisting only laterally by steel pipe or others	enlarge pile head with concrete	cast new pile and connect its head with existing footing	strengthen a part of pile under large stress steel tube or carbon fiber	decrease lateral force acting on the piles due to contribution of underground wall
Objective	increase in lateral resistance	increase in stiffness and strength of pile head	increase in lateral and vertical supporting capacity by adding piles	increase in lateral strength of pile head	decrease in lateral force of piles
Recommendations in design	- connection at pile head - group effect of piles	special care is required for the part whose stiffness is suddenly changed	- connection design of pile head - group effect of piles	evaluation of contribution effects by pile and steel tube or carbon fiber	appropriate embedment length of underground wall and joint detail
Recommendations in construction	special care is required in welding steel pipe joint	careful details in the connection between existing pile and added concrete	- careful construction of connection at pile head - avoid damage to existing pile	care is required on adhesion of steel tube on carbon fiber with existing pile	enough consolidation of filling soil is required with attention to the joints of underground wall
Cost	1.5	1.2	1.5～1.8	steel jacketing 0.8 carbon fiber wrapping 1.0～1.5	1.0
Others	careful design derails are required in connecting foundation beam and piles	this shape is similar to those strengthened after cut the pile head and jack up	often used for foundation of bridges	also used for strengthening of superstructures	this is suitable for seismic strengthening of existing sound pile without enough lateral capacity

Figure TN.49 Seismic strengthening method of pile foundation

(quoted from the figure on page 249 in the commentary of 3.7.5 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 49

Translators' Note 50 --

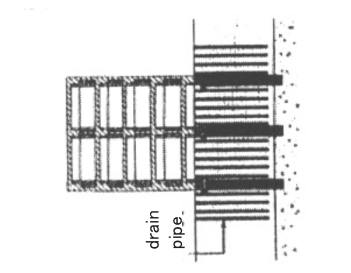
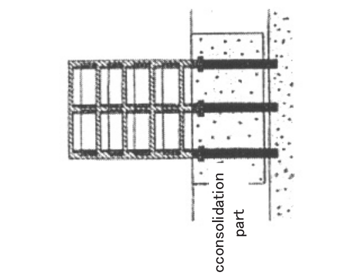
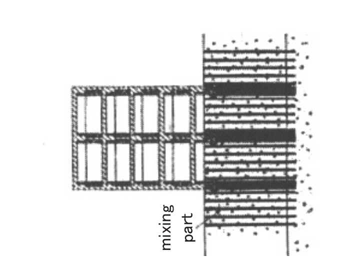
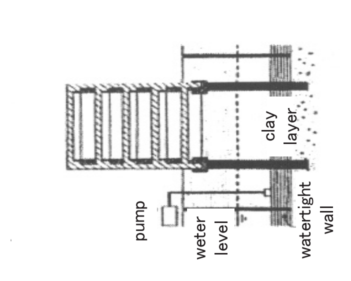
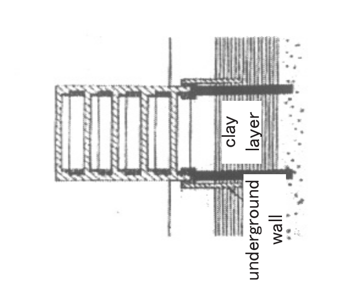
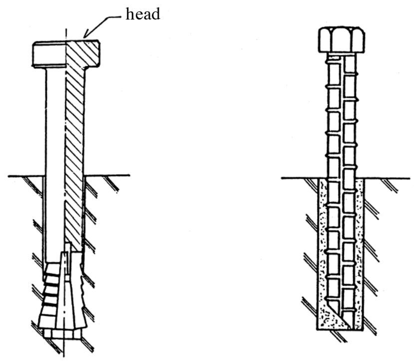
Construction method	Drain method	Consolidation method	Solidarity method	Water level decrease method	Deflection decrease method
Image					
Outline	decrease excessive water pressure at the time of liquefaction by setting aggregate or drainpipe in sand soil	prevent liquefaction by increasing density of soil due to consolidation	increase strength of soil by mixing hardening material and soil	lower the water level below currant liquefaction layer by providing watertight wall and pumping the water inside the wall	prevent liquefaction during earthquake by confinement of soil with underground wall
Objective	- prevent liquefaction	- prevent liquefaction - increase strength of soil	- prevent liquefaction - increase strength of soil	- prevent liquefaction	- prevent soil deflection - prevent liquefaction
Recommendations in design	- area to be drained, pitch of drain pipe	- area to be consolidated - level of consolidation	- area to be solidified - evaluation property of mixed soil	pumping plan meeting with volume of pumping water	effective set of stiffness of underground wall
Recommendations in construction	keep drainpipe shape and draining performance	management of consolidation	mix uniformly	prevent water leakage from the watertight wall	- secure concrete quality in case of RC continuous underground wall - secure joint performance in case of sheet pile
Cost	1.2	1.0	1.5	more than 2.0 (include maintenance of pump)	1.5
Others	it is effective only for measure against liquefaction	vibration problem may occur	plant is necessary	it is possible to prevent liquefaction perfectly	secure joint performance of continuous underground wall

Figure TN.50 Construction as measures against liquefaction

(quoted from the figure on page 253 in the commentary of 3.7.5 of the Guidelines of 2001 Japanese version)

-- End of Translators' Note 50

Translators' Note 51 ---



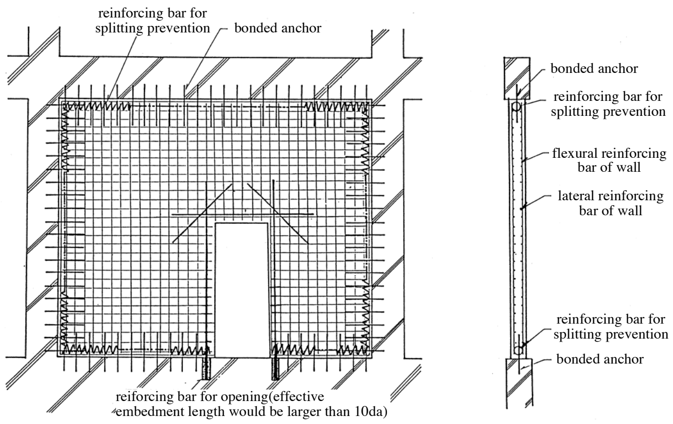
(a) expansion anchor (example) (b) donded anchor (example)

Figure TN.51 Headed anchor

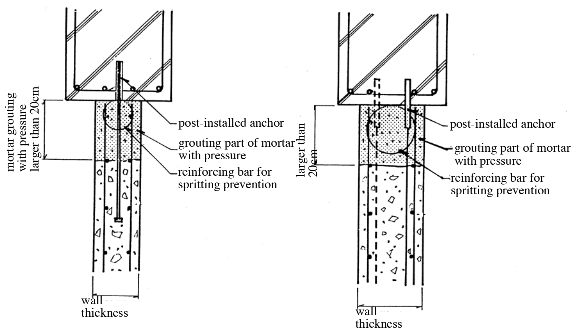
(quoted from the figure on page 279 in the commentary of 3.9.5 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 51

Translators' Note 52 ---



(a) bar arrangement of infillinf RC wall



(b) example of single arrangement (c) example of stugger arrangement

Figure TN.52 Typical arrangement of post-installed anchor used for infilling wall

(quoted from the figure on page 279 in the commentary of 3.9.5 of the Guidelines of 2001 Japanese version)

--- End of Translators' Note 52

Translators' Note 53 --

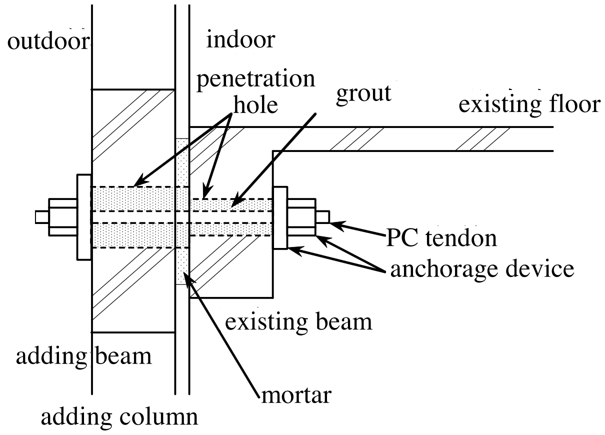


Figure TN.53 Example detail of compressive contact connection

(quoted from the figure on page 291 in the commentary of 3.10.1 of the Guidelines of 2001 Japanese version)

-- **End of Translators' Note 53**

Technical Manual

for Seismic Evaluation and Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001

Appendix-1
Technical Manual for Adoption of the Standard

Appendix 1-1 Commentary with Evaluation Examples

A Moment Resisting Frame Structure

The procedure of the seismic capacity evaluation is shown in this example with a moment resisting structure which is outlined in the section 1. Since the main purpose of the example is to show how to calculate the Basic Seismic Index of Structure (E_0) based on the "Standard for seismic evaluation of existing reinforced concrete buildings, 2001" (referred to as the current Standard, hereafter), the procedure for the Second-Class Prime Elements is ignored, and the Irregularity Index (S_D) and the Time Index (T) are assumed as 1.0.

Compared with the Standard 1990 (referred to as the previous Standard), the methods for calculating the Ductility Index (F) in the second and third level screening, and the Effective Strength Factor (α) used to evaluate the E_0 index have been considerably revised. Therefore, the procedure of these calculations is described in detail, and the result based on the current and previous Standards is compared.

1. Outline Of The Structure

Outline of the example structure is described in this section. The structure has 4-story and 2-span in the transverse direction. The frame in the area surrounded with the dashed line in Fig. 1.1.A-1 in the longitudinal (*x*) direction is studied in this example. The columns are categorized into three models, namely short column with standing wall, long column, and extremely short column with standing and hanging wall. The weight for unit area is assumed as 11.8 kN/m^2. Three different hoop spacing of columns, 300, 200 and 100mm, are studied to compare the *F* and E_0 indices with these values.

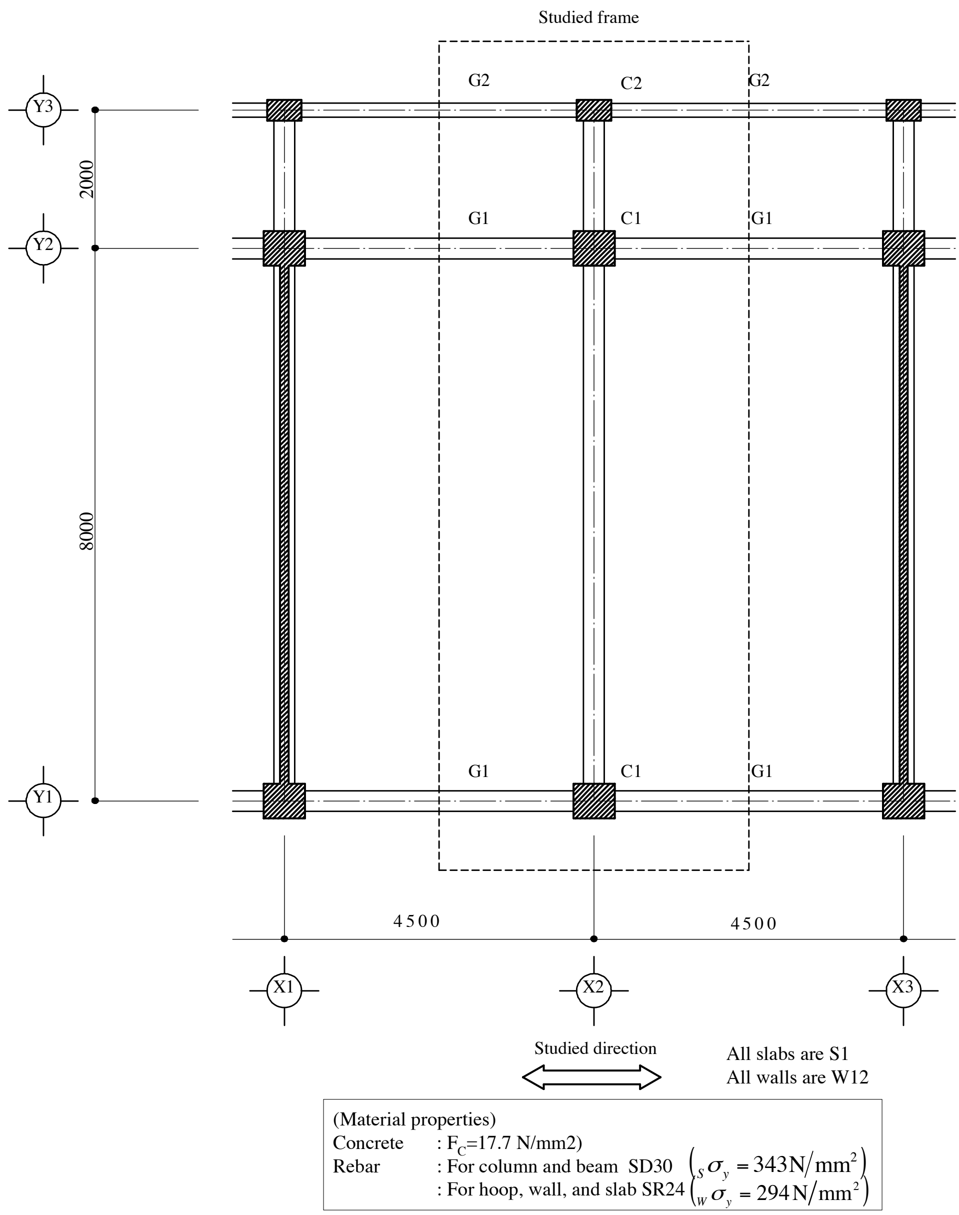
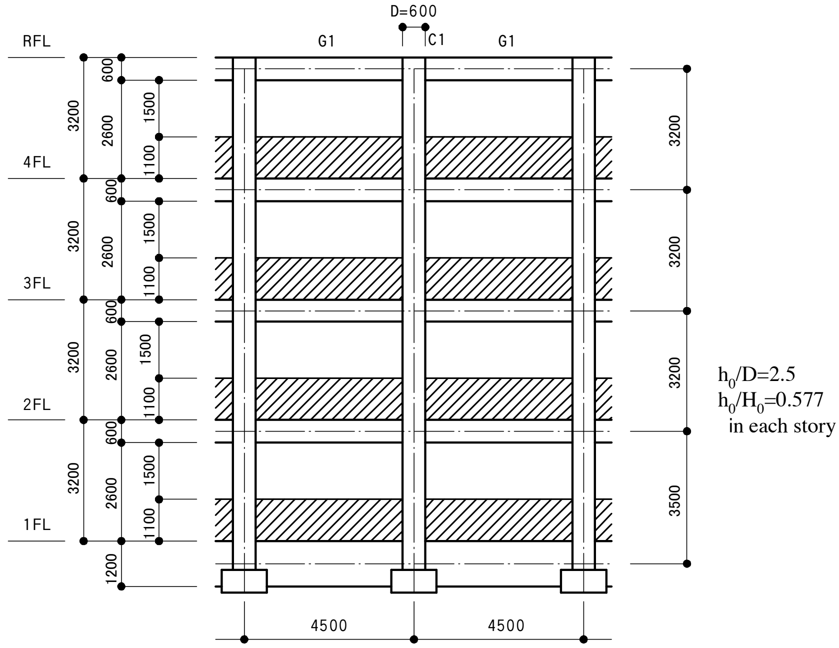


Fig. 1.1.A-1 The standard floor plan



*All walls are W12

Fig. 1.1.A-2 *Y1* framing elevation

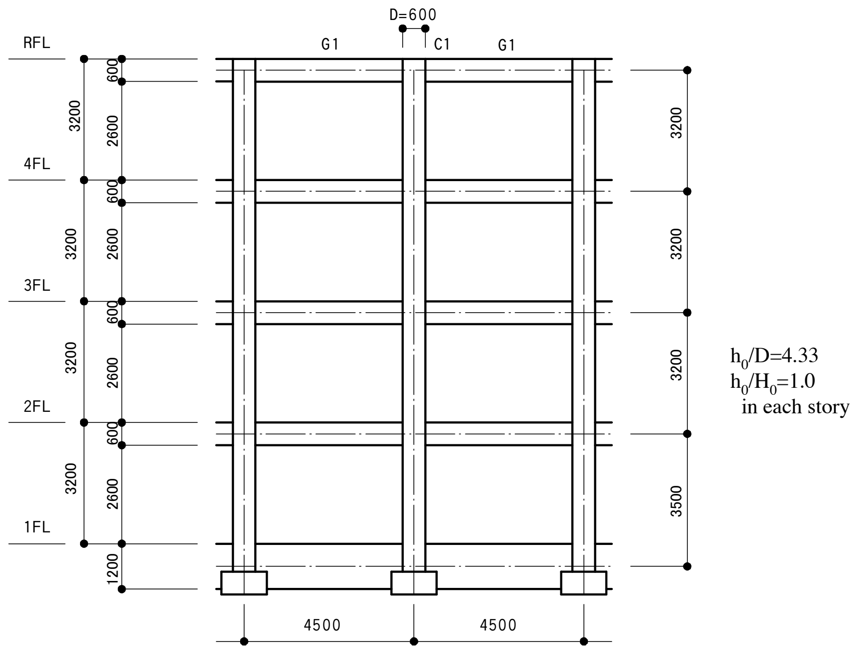
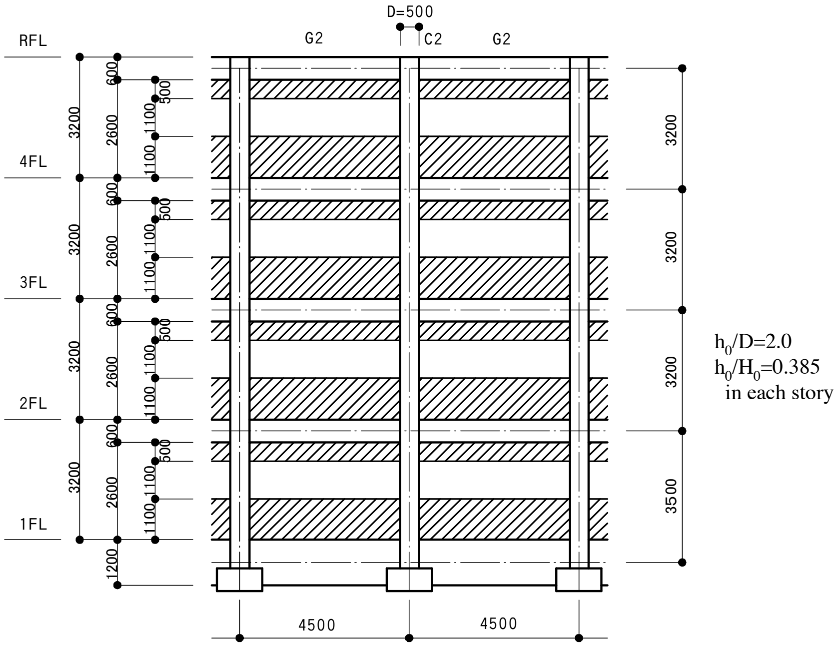
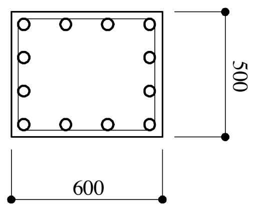
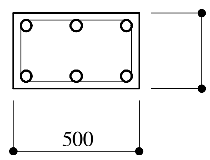


Fig. 1.1.A-3 *Y2* framing elevation



*All walls are W12

Fig. 1.1.A-4 *Y3* framing elevation

Story		C1	C2
1~4	Y Section X		
	$b \times D$	500×600	300×500
	Main bar	12-D22	6-D22
	Hoop	2-ϕ9	2-ϕ9

*Hoop spacing of 100, 200, and 300mm will be applied for the second level screening, and that of 100mm will be applied for the third level screening

Fig. 1.1.A-5 Member list (columns)

Wall list

Remark	Thickness (t)(mm)	Wall reinforcement	End reinforcement
W12	120	ϕ 9@300 Single layered (Vertical and horizontal)	1- ϕ 13

Slab list

Remark	Thickness (t)(mm)	Slab reinforcement
S1	120	ϕ 9 @300 Double layered (Cross arrangement)

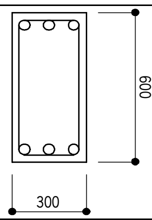
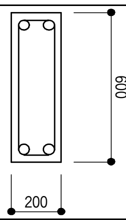
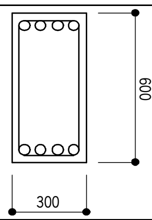
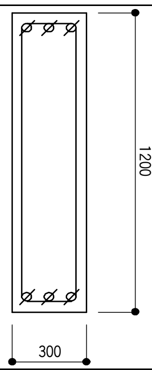
Story		G1	G2
4~R	Section		
	$b \times D$	300×600	200×600
	Main bar	3-D22 (Top & Bottom)	2-D22 (Top & Bottom)
	Hoop	2- ϕ 9@300	2- ϕ 9@300
2~3	Section		Ditto
	$b \times D$	300×600	
	Main bar	4-D22 (Top & Bottom)	
	Hoop	2- ϕ 9@300	
Underground	Section		Same as in the left
	$b \times D$	300×1200	
	Main bar	3-D19 (Top & Bottom)	
	Hoop	2- ϕ 9@300	

Fig. 1.1.A-6 Member list (girders)

2. Preliminary Calculation

2.1 Structural weight and sustained force by columns

The weight of the structure is calculated based on the assumed weight for unit area of W=11.8 kN/m^2. The weight of each floor sustained by a column is calculated according to the area supported by the column and the W. The calculated values are listed as follows;

Table 1.1.A-1 Structural weights

Floor	Floor Area A_f (m^2)	Floor weight Wi (kN)	ΣWi (kN)
4	45.0	529.6	529.6
3	45.0	529.6	1059.1
2	45.0	529.6	1588.7
1	45.0	529.6	2118.2

Table 1.1.A-2 Column sustaining force

Frame	Story	Supporting area A (m^2)	11.8*A (kN)	Sustaining force N (kN)
Y3	4	$4.5 \times 1.0 = 4.5$	53.1	53.1
	3			106.2
	2			159.3
	1			212.4
Y2	4	$4.5 \times 5.0 = 22.5$	265.5	265.5
	3			531.0
	2			796.5
	1			1062.0
Y1	4	$4.5 \times 4.0 = 18.0$	212.4	212.4
	3			424.8
	2			637.2
	1			849.6

2.2 Story-shear modification factors for E_0

The values shown in the section 3.2.1 of the current Standard are applied to the story-shear modification factors. The values are listed as follow;

Table 1.1.A-3 Story-shear modification factors

Story	Modification factor $\frac{n+1}{n+i}$
4	$\frac{5}{8} = 0.625$
3	$\frac{5}{7} = 0.714$
2	$\frac{5}{6} = 0.833$
1	$\frac{5}{5} = 1.000$

3. The First Level Screening Method

The seismic capacity of structures is evaluated based on the sectional area of vertical elements, the column shape, and the concrete strength in the first level screening method. The average shear stress of column at ultimate state is defined according to its shape. Thus, the stress multiplied by the modified factor β_c based on concrete strength and the area of the column becomes ultimate strength of the column. The ductility of column is defined based on its shape.

3.1 Vertical elements categorization and shear stress at ultimate state

The vertical elements are categorized according to Table 1 in the current Standard. The average shear stresses at ultimate state are defined according to the section 3.2.2(1) of the current Standard. The results are listed in Table 1.1.A-4.

Table 1.1.A-4 Vertical elements categorization and shear stress at ultimate state

Story		*Y1*	*Y2*	*Y3*
4	Column	$4C_1$	$4C_1$	$4C_2$
	h_0/D	2.5	4.3	2.0
	Category	Column	Column	Extremely Short Column
	τ (N/mm^2)	1.0	1.0	1.5
	Sectional area A (mm^2)	300000	300000	150000
3	Column	$3C_1$	$3C_1$	$3C_2$
	h_0/D	2.5	4.3	2.0
	Category	Column	Column	Extremely Short Column
	τ (N/mm^2)	1.0	1.0	1.5
	Sectional area A (mm^2)	300000	300000	150000
2	Column	$2C_1$	$2C_1$	$2C_2$
	h_0/D	2.5	4.3	2.0
	Category	Column	Column	Extremely Short Column
	τ (N/mm^2)	1.0	1.0	1.5
	Sectional area A (mm^2)	300000	300000	150000
1	Column	$1C_1$	$1C_1$	$1C_2$
	h_0/D	2.5	4.3	2.0
	Category	Column	Column	Extremely Short Column
	τ (N/mm^2)	1.0	1.0	1.5
	Sectional area A (mm^2)	300000	300000	150000

3.2 Strength index *C*

The strength index is calculated with Eqs. (7) to (10) in the section 3.2.2 (1) of the current Standard. Since the F_c (=17.7 N/mm^2) <20, with Eq. (10) of the current Standard;

$$\beta_c = \frac{F_c}{20} = \frac{17.7}{20} = 0.885$$

As for the 4th story, from Eqs. (8) and (9) of the current Standard;

$$C_C = \frac{\tau_C \cdot A_C}{\Sigma W} \cdot \beta_c = \frac{1.0\ (N/mm^2) \times (300000 + 300000)\ (mm^2)}{529.6\ (kN) \times 1000} \times 0.885 = 1.000$$

$$C_{SC} = \frac{\tau_{SC} \cdot A_{SC}}{\Sigma W} \cdot \beta_c = \frac{1.5\ (N/mm^2) \times 150000\ (mm^2)}{529.6\ (kN) \times 1000} \times 0.885 = 0.375$$

The values for each story are calculated as follows.

Table 1.1.A-5 Strength index *C* (the first level screening)

	Member category	*C*
4	Column	1.000
	Extremely short column	0.375
3	Column	0.500
	Extremely short column	0.188
2	Column	0.333
	Extremely short column	0.125
1	Column	0.250
	Extremely short column	0.094

3.3 Basic seismic index of structure E_0

The E_0 index for the first level screening method is calculated with Eqs. (2) and (3) in the section 3.2.1 (1) of the current Standard. The calculation procedure for the 4th story is shown as follows.

Calculation with Eq. (2);

$$E_{0(eq.2)} = \frac{n+1}{n+i}\left(C_W + \alpha_1 \cdot C_C\right) \cdot F_W$$

where, $\alpha_1 = 1.0$ since $C_W = 0.0$. Therefore,

$$E_{0(eq.2)} = \frac{5}{8}\left(0.0 + 1.0 \times 1.000\right) \times 1.0 = \frac{5}{8} = 0.625$$

Calculation with Eq. (3);

$$E_{0(eq.3)} = \frac{n+1}{n+i}\left(C_{SC} + \alpha_2 \cdot C_W + \alpha_3 \cdot C_C\right) \cdot F_{SC}$$

$$= \frac{5}{8}\left(0.375 + 0.7 \times 0.0 + 0.5 \times 1.000\right) \times 0.8 = 0.438$$

The values for each story are calculated as follows.

Table 1.1.A-6 Basic seismic index of structure E_0 (the first level screening)

Story	$\frac{n+1}{n+i}$	Member category*	C	F	E_0
4	$\frac{5}{8}$	C SC	1.000 0.375	1.0 0.8	Eq. (2): 0.625 Eq. (3): 0.438
3	$\frac{5}{7}$	C SC	0.500 0.188	1.0 0.8	Eq. (2): 0.357 Eq. (3): 0.250
2	$\frac{5}{6}$	C SC	0.333 0.125	1.0 0.8	Eq. (2): 0.278 Eq. (3): 0.194
1	1	C SC	0.250 0.094	1.0 0.8	Eq. (2): 0.250 Eq. (3): 0.175

* C: Column, SC: Extremely short column

3.4 Seismic index of structure I_S

The E_0 indices for all stories are calculated with Eq. (2) of the current Standard, if the extremely short columns on each story are assumed not to be the second-class prime elements. Since the irregularity index and time index are both assumed as 1.0, the I_S index can be calculated as follows.

$$I_S = E_0 \cdot S_D \cdot T$$

(4th story) $I_S = 0.625 \times 1.0 \times 1.0 = 0.625$

(3rd story) $I_S = 0.357 \times 1.0 \times 1.0 = 0.357$

(2nd story) $I_S = 0.278 \times 1.0 \times 1.0 = 0.278$

(1st story) $I_S = 0.250 \times 1.0 \times 1.0 = 0.250$

4. The Second Level Screening Method

According to the second level seismic capacity evaluation, the seismic capacity of a structure is evaluated based on the performance of the vertical element on the assumption that girders are strong enough not to fail. The strength of members is calculated with available equations. The deflection angle at flexural yielding is derived from the column shape. Then the deflection angle at ultimate flexural strength and the deflection angle at ultimate shear strength are calculated considering the strength margin for shear failure. Finally, the ductility index is calculated based on the deflection angle. The detailed procedure how to calculate the ductility index is mentioned in the section 4.2.

The examples with three different hoop spacing of 100, 200 and 300 mm are shown in this section.

4.1 Member strengths

The equations listed in the main text of the previous Standard, that is, Eq. (A1.1-1) and Eq. (A1.1-2) in the supplementary provisions of the current Standard, are applied in this example. Here, the stationary axial loads for columns are considered.

(1) The ultimate flexural strength

The ultimate flexural strength is calculated with the Eq. (A1.1-1).

If $N < 0.4 \cdot b \cdot D \cdot F_c$, then

$$M_u = 0.8 \cdot a_t \cdot \sigma_y \cdot D + 0.5 \cdot N \cdot D \cdot \left(1 - \frac{N}{b \cdot D \cdot F_c}\right)$$

As for the *Y1* column on the 4th floor,

$$a_t = 387 \times 4 = 1548 \ (mm^2)$$

$$\sigma_y = 343 \ (N/mm^2)$$

$$D = 600 \ (mm)$$

$$N = 212.4 \ (kN)$$

$$F_c = 17.7 \ (N/mm^2)$$

$$M_u = 0.8 \times 1548 \times 343 \times 600 \times 10^{-6} + 0.5 \times 212.4 \times 600 \times 10^{-3} \times \left(1 - \frac{212.4 \times 1000}{500 \times 600 \times 17.7}\right)$$

$$= 254.9 + 61.2 = 316.1 \ (kN \cdot m)$$

The shear force at the ultimate flexural strength Q_{mu} can be calculated as follows on the assumption that the M_u at the top and the bottom of the column are the same.

$$Q_{mu} = 2 \cdot M_u / h_0 = 2 \times 316.1/1.5 = 421.5 \ (kN)$$

The ultimate flexural strength of each column can be calculated in the same procedure.

(2) The ultimate shear strength

The ultimate shear strength is calculated with the Eq. (A1.1-2).

$$Q_{su} = \left\{ \frac{0.053 \cdot p_t^{0.23} \cdot (18 + F_c)}{M/(Q \cdot d) + 0.12} + 0.85\sqrt{p_w \cdot {}_s\sigma_{wy}} + 0.1\sigma_0 \right\} \cdot b \cdot j$$

As for the *Y1* column on the 4th floor, hoop spacing of 100mm is calculated as follows;

$$b = 500\ (mm)$$

$$D = 600\ (mm)$$

$$d = D - 50 = 550\ (mm)$$

$$j = 0.8 \cdot D = 480\ (mm)$$

$$p_t = \frac{a_t}{b \times D} = \frac{4 \times 387}{500 \times 600} \times 100 = 0.516\ (\%)$$

$$F_c = 17.7\ (N/mm^2)$$

$$M/_{Q \cdot d} = {}^{1500/2}/_{550} = 1.364 \quad \text{with assumption of } M/_{Q \cdot d} = {}^{h_0/2}/_{d}$$

$$p_w = \frac{2 \times 64}{500 \times 100} = 0.00256\ (\%)$$

$${}_s\sigma_{wy} = 294\ (N/mm^2)$$

$$\sigma_0 = \frac{N}{b \times D} = \frac{212.4 \times 1000}{500 \times 600} = 0.71 (N/mm^2)$$

$$Q_{su} = \left\{ \frac{0.053 \times 0.516^{0.23} \times (18 + 17.7)}{1.364 + 0.12} + 0.85\sqrt{0.00256 \times 294} + 0.1 \times 0.71 \right\} \times 500 \times 480 \times 10^{-3}$$

$$= (1.095 + 0.737 + 0.071) \times 240.0 = 256.7\ (kN)$$

The ultimate shear strength of each column can be calculated in the same procedure.
The calculated strength of each member is listed as follows.

Table 1.1.A-7 The strengths of members (hoop spacing of 100mm)

Frame	Story	h_0/D	M_u (kN*m)	Q_{mu} (kN)	Q_{su} (kN)	Failure mode
Y1	4	2.5	316.1	421.5	456.7	Flexural
	3	2.5	372.0	496.0	470.5	Shear
	2	2.5	422.8	563.7	487.5	Shear
	1	2.5	468.6	624.8	504.5	Shear
Y2	4	4.3	330.5	254.2	352.8	Flexural
	3	4.3	398.1	306.2	374.0	Flexural
	2	4.3	457.6	352.0	395.3	Flexural
	1	4.3	509.3	391.8	416.5	Flexural
Y3	4	2.0	119.2	238.4	269.9	Flexural
	3	2.0	131.7	263.4	274.0	Flexural
	2	2.0	143.6	287.2	278.2	Extremely brittle
	1	2.0	154.9	309.8	282.3	Extremely brittle

Table 1.1.A-8 The strengths of members (hoop spacing of 200mm)

Frame	Story	h_0/D	M_u (kN*m)	Q_{mu} (kN)	Q_{su} (kN)	Failure mode
Y1	4	2.5	316.1	421.4	402.0	Shear
	3	2.5	377.0	495.9	418.9	Shear
	2	2.5	422.8	563.7	436.0	Shear
	1	2.5	468.6	624.8	453.0	Shear
Y2	4	4.3	330.5	254.2	301.3	Flexural
	3	4.3	390.1	306.2	322.5	Flexural
	2	4.3	457.6	352.0	343.7	Shear
	1	4.3	509.3	391.7	365.0	Shear
Y3	4	2.0	119.2	238.5	237.2	Extremely brittle
	3	2.0	131.7	263.4	241.3	Extremely brittle
	2	2.0	143.6	287.2	245.6	Extremely brittle
	1	2.0	154.9	310.0	249.7	Extremely brittle

Table 1.1.A-8 The strengths of members (hoop spacing of 300mm)

Frame	Story	h_0/D	M_u (kN*m)	Q_{mu} (kN)	Q_{su} (kN)	Failure mode
Y1	4	2.5	316.1	421.4	379.1	Shear
	3	2.5	372.0	495.9	396.2	Shear
	2	2.5	422.8	563.7	413.2	Shear
	1	2.5	468.6	624.8	430.1	Shear
Y2	4	4.3	330.5	254.2	278.5	Flexural
	3	4.3	398.1	306.2	299.7	Shear
	2	4.3	457.6	352.0	321.0	Shear
	1	4.3	509.3	391.7	342.2	Shear
Y3	4	2.0	119.2	238.5	222.7	Extremely brittle
	3	2.0	131.7	263.4	226.9	Extremely brittle
	2	2.0	143.6	287.2	231.0	Extremely brittle
	1	2.0	154.9	310.0	235.3	Extremely brittle

4.2 Ductility index *F*

The F index for the independent column is calculated according to its failure mode, considering the strength margin for shear failure (ultimate shear strength/shear force at ultimate flexural strength) and deflection angle. The deflection angles to be considered are as follows; the maximum deflection angle of column to the deformable length ${}_cR_{max}$, the deflection angle of column at the yielding ${}_cR_{my}$, the plastic deflection angle of column ${}_cR_{mp}$, the deflection angle of column at the ultimate flexural strength ${}_cR_{mu}$, the deflection angle of story at flexural yielding modified by the clear height (h_0) and standard height (H_0) R_{my}, the deflection angle of story at ultimate flexural strength R_{mu}, the deflection angle of story at the ultimate shear strength R_{su}, and the deflection angle at story yielding R_y. The practical procedure is shown as Fig. 1.1.A-7. Thus, the F index can vary from 1.0 to 3.2 continuously for shear and flexural column.

The deflection angle at story yielding R_y of 1/150 is applied to the example.

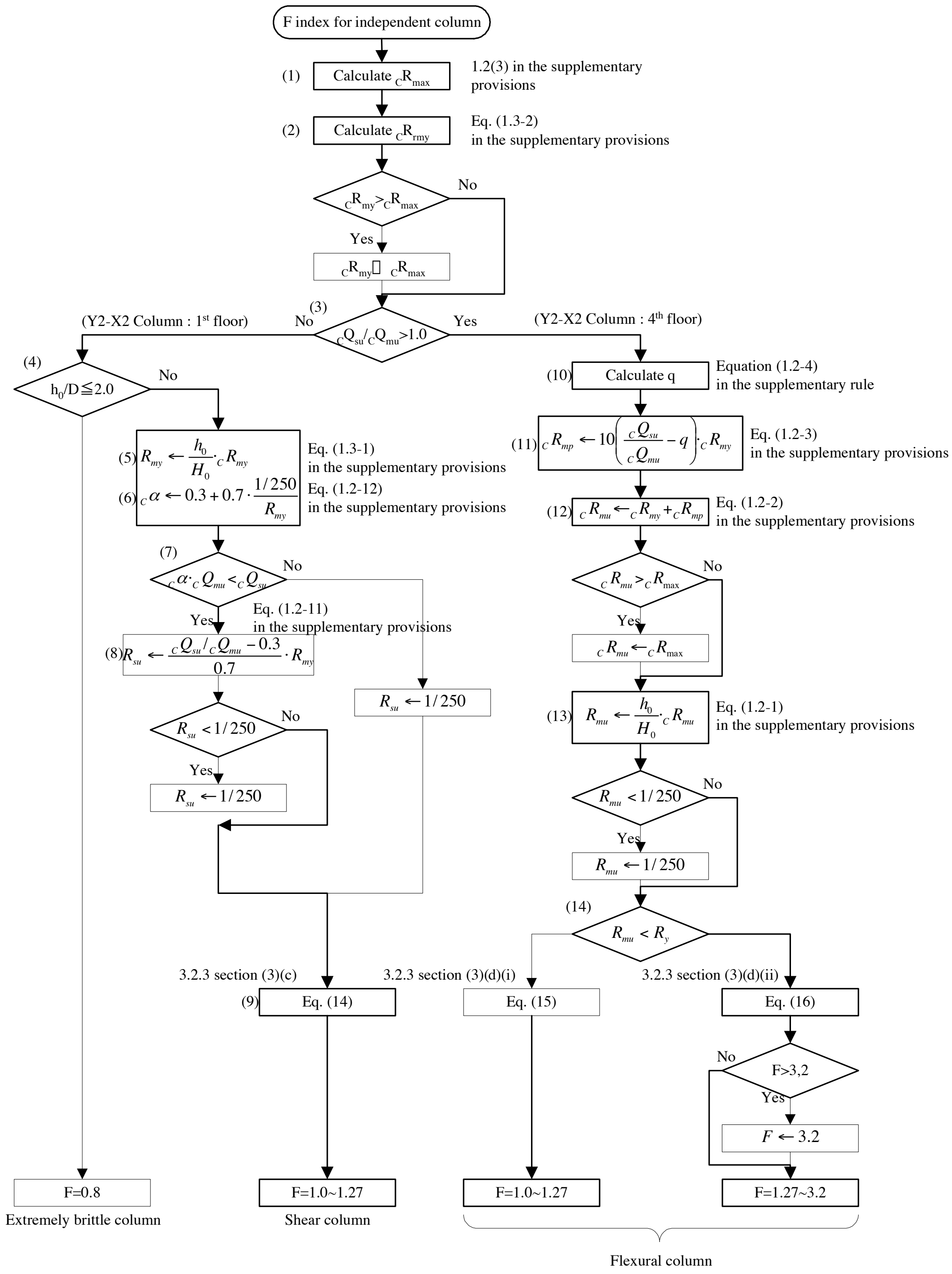
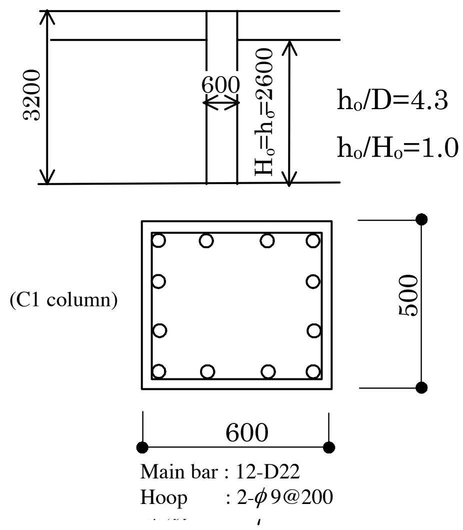


Fig. 1.1.A-7 Flowchart to calculate the ductility index for individual column

Here, the calculation procedure for the *Y2* column on the 1st and 4th floor with hoop spacing of 200mm is described as an example.

(1) *Y2-X2* column : 1st floor (Failure mode : shear)



Dimension $b \times D = 500 \times 600$, $j = 480(mm)$
(F_c=17.7N/mm^2)
Hoop $2-\phi 9@200$
N_s= 1062.0(kN)
Moment capacity : M_u = 509.3(kN·m)
Shear force at the moment capacity
: ${}_cQ_{mu}$ = 391.7 (kN)
Ultimate shear strength: ${}_cQ_{su}$ = 365.0 (kN)
(failure mode : shear)

(a) Upper limit of the deflection angle of flexural column ${}_cR_{max}$ ((1) in Fig. 1.1.A-7)

It is calculated according to the supplementary provisions 1.2 (3) of the current Standard.

$${}_cR_{\max x} = \min\left\lfloor {}_cR_{\max(n)}, {}_cR_{\max(s)}, {}_cR_{\max(t)}, {}_cR_{\max(b)}, {}_cR_{\max(h)} \right\rfloor$$

-- ${}_cR_{max(n)}$

Since $s > 100mm$, $\eta_L = 0.2$, $\eta_H = 0.4$

$$\eta = N_s / b \cdot DF_c = 1062.0 \times 1000 / (500 \times 600 \times 17.7) = 0.2$$

$$n' = (\eta - \eta_L)/(\eta_H - \eta_L) = 0.0$$

$${}_cR_{\max(n)} = R_{30} \times \left({R_{250}}\big/{R_{30}} \right)^{n'} = R_{30} \quad \rightarrow \quad 1/30$$

-- ${}_cR_{max(s)}$

$${}_c\tau_u = \frac{365.0 \times 1000}{500 \times 480} = 1.52$$

Since $s = {}_c\tau_u / F_c = 0.086$, $s < 0.2$, ${}_cR_{\max(s)} = 1/30$

-- ${}_cR_{max(t)}$

$$t = P_t = 0.516(\%)$$

Since $t < 1.0(\%)$, ${}_cR_{\max(t)} = 1/30$

-- ${}_cR_{max(b)}$

$$b = s / d_b = 200 / 22 = 9.09$$

Since $8 < b$, ${}_cR_{\max(b)} = 1/50$

-- ${}_cR_{max(h)}$

$h = h_0 / D = 4.3$

Since $h > 2$, ${}_cR_{\max(h)} = 1/30$

$$\therefore \quad {}_cR_{\max x} = \min\left\lfloor {}_cR_{\max(n)}, {}_cR_{\max(s)}, {}_cR_{\max(t)}, {}_cR_{\max(b)}, {}_cR_{\max(h)} \right\rfloor = 1/50$$

(b) The yielding deflection angle of the column ${}_c\boldsymbol{R_{my}}$ ((2) in Fig. 1.1.A-7)

It is calculated with Eq. (A1.3-2) in the supplementary provisions 1.3(1) of the current Standard.

Since $h_0 / D(= 4.3) > 3.0$, ${}_cR_{my} = R_{150} = 1/150$

Since $({}_cR_{\max} > {}_cR_{my})$, ${}_cR_{my} = 1/150$

(c) Failure mode categorization according to the strength margin for shear failure ((3) in Fig. 1.1.A-7)

The ultimate shear strength: ${}_cQ_{su} = 365.0(kN)$

The shear force at flexural yielding: ${}_cQ_{mu} = 391.7(kN)$

${}_cQ_{su} / {}_cQ_{mu} = 365.0/391.7 = 0.93 < 1.0$

Therefore, the failure mode of the column is categorized as “shear”.

(d) Extremely brittle column check ((4) in Fig. 1.1.A-7)

$h_0 / D = 2600/600 = 4.3 > 2.0$

Therefore, the ductility index for “shear” column is applied.

(e) The yielding inter-story deflection angle of the column $\boldsymbol{R_{my}}$ ((5) in Fig. 1.1.A-7)

It is calculated with Eq. (A1.3-1) in the supplementary provisions 1.3(1) of the current Standard.

Since $R_{my} = (h_0 / H_0) \times {}_cR_{my} \geqq 1/250$, $R_{my} = 1.0 \times 1/150 = 1/150$

(f) The effective strength factor of the column ${}_c\alpha$ to calculate $\boldsymbol{R_{su}}$ ((6) in Fig. 1.1.A-7)

It is calculated with Eq. (A1.2-12) in the supplementary provisions 1.2(4) of the current Standard.

$${}_c\alpha = 0.3 + 0.7 \times \left({}^{1/250}\!\big/_{R_{my}} \right) = 0.3 + 0.7 \times \left(\frac{1/250}{1/150} \right) = 0.72$$

(g) The inter-story deflection angle at the ultimate limit state of the shear column $\boldsymbol{R_{su}}$

It is calculated with Eq. (A1.2-11) in the supplementary provisions 1.2(4) of the current Standard.

$${}_c\alpha \cdot {}_cQ_{mu} = 0.72 \times 391.7 = 282.0 < {}_cQ_{su} (= 365.0) \qquad ((7) \text{ in Fig. 1.1.A-7})$$

Therefore, the R_{su} is calculated with Eq. (A1.2-11) in the supplementary provisions rule 1.2(4) of the current Standard.

$$R_{su} = \frac{{}_cQ_{su} / {}_cQ_{mu} - 0.3}{0.7} \cdot R_{my} = \frac{365.0/391.7 - 0.3}{0.7} \cdot \frac{1}{150} = \frac{1}{166}$$

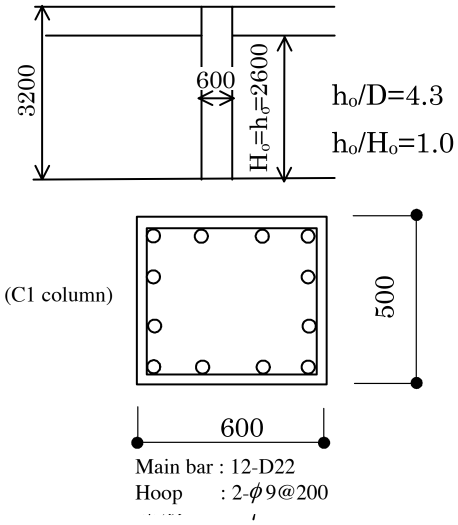
((8) in Fig. 1.1.A-7)

(h) The ductility index *F* ((9) in Fig. 1.1.A-7)

It is calculated with Eq. (14) in section 3.2.3 (3)(c) of the current Standard for shear column.

$$F = 1.0 + 0.27 \times \frac{R_{su} - R_{250}}{R_y - R_{250}} = 1.0 + 0.27 \times \frac{\frac{1}{166} - \frac{1}{250}}{\frac{1}{150} - \frac{1}{250}} = 1.20$$

(2) *Y2-X2* column : 4th floor (Failure mode : shear)



Dimension $b \times D = 500 \times 600$, $j = 480(mm)$
(F_c=17.7N/mm^2)
Hoop $2-\phi 9@200$
$N_s = 265.5$(kN)
Moment capacity : $M_u = 330.5$ (kN·m)
Shear force strength at the moment capacity
: ${}_cQ_{mu} = 254.2$ (kN)
Ultimate shear strength: ${}_cQ_{su} = 301.3$ (kN)
(failure mode : flexure)

(a) Upper limit of the deflection angle of flexural column ${}_cR_{max}$ ((1) in Fig. 1.1.A-7)

It is calculated according to the supplementary provisions 1.2 (3) of the current Standard

$${}_cR_{\max} = \min\left\lfloor {}_cR_{\max(n)}, {}_cR_{\max(s)}, {}_cR_{\max(t)}, {}_cR_{\max(b)}, {}_cR_{\max(h)} \right\rfloor$$

-- ${}_cR_{\max(n)}$

Since $s > 100mm$, $\eta_L = 0.2$, $\eta_H = 0.4$

Since $\eta = N_s/(b \cdot D \cdot F_c) = 265.5 \times 1000/(500 \times 600 \times 17.7) = 0.05 < \eta_L$, ${}_cR_{\max(n)} = 1/30$

-- ${}_cR_{\max(s)}$

$${}_c\tau_u = \min\left(\frac{{}_cQ_{mu}}{bj}, \frac{{}_cQ_{su}}{bj}\right) = \frac{{}_cQ_{mu}}{bj} = \frac{254.2 \times 1000}{500 \times 480} = 1.06$$

$s = {}_c\tau_u / F_c = 1.06/17.7 = 0.060$

Since $s < 0.2$, ${}_cR_{\max(s)} = 1/30$

-- ${}_cR_{\max(t)}$

$t = p_t = 0.516(\%)$

Since $t < 1.0(\%)$, ${}_cR_{\max(t)} = 1/30$

-- ${}_cR_{\max(b)}$

$b = s/d_b = 200/22 = 9.09$

Since $8 < b$, ${}_cR_{\max(b)} = 1/50$

-- ${}_cR_{\max(h)}$

$h = h_0 / D = 4.3$

Since $h > 2$, ${}_cR_{\max(h)} = 1/30$

$$\therefore \quad {}_cR_{\max} = \min\left\lfloor {}_cR_{\max(n)}, {}_cR_{\max(s)}, {}_cR_{\max(t)}, {}_cR_{\max(b)}, {}_cR_{\max(h)} \right\rfloor = 1/50$$

(b) The yielding deflection angle of the column $_cR_{my}$ ((2) in Fig. 1.1.A-7)

It is calculated with Eq. (A1.3-2) in the supplementary provisions 1.3(1) of the current Standard.

Since $h_0 / D(=4.3) > 3.0$, $_cR_{my} = R_{150} = 1/150$

Since $(_cR_{\max} > _cR_{my})$, $_cR_{my} = 1/150$

(c) Failure mode categorization according to the strength margin for shear failure

((3) in Fig. 1.1.A-7)

Ultimate shear strength : $_cQ_{su} = 301.3(kN)$

Shear force at ultimate flexural strength : $_cQ_{mu} = 254.2(kN)$

$$_cQ_{su} / _cQ_{mu} = 301.3/254.2 = 1.19 > 1.0$$

Therefore, the failure mode of the column is categorized as "flexural".

(d) The modification factor *q* for the $_cR_{my}$ according to the hoop spacing

((10) in Fig. 1.1.A-7)

It is calculated with Eq. (A1.2-4) in the supplementary provisions 1.2(2) of the current Standard.

Since $s(=200_{mm}) > 100_{mm}$, $q = 1.1$

(e) The plastic deflection angle of the column $_cR_{mp}$ ((11) in Fig. 1.1.A-7)

It is calculated with Eq. (A1.2-3) in the supplementary provisions 1.2(2) of the current Standard.

$$\begin{aligned} _cR_{mp} &= 10 \times (_cQ_{su} / _cQ_{mu} - q) \cdot _cR_{myp} \\ &= 10 \times (301.3/254.2 - 1.1) \times 1/150 = 1/176 \end{aligned}$$

(f) The deflection angle at the moment capacity of the column $_cR_{mu}$

((12) in Fig. 1.1.A-7)

It is calculated with Eq. (A1.2-2) in the supplementary provisions 1.2(1) of the current Standard.

$$_cR_{mu} = _cR_{my} + _cR_{mp} = 1/150 + 1/176 = 1/81 < _cR_{\max}$$

(g) The inter-story deflection angle at the deformation capacity R_{mu}

((13) in Fig. 1.1.A-7)

It is calculated with Eq. (A1.2-1) in the supplementary provisions 1.2(1) of the current Standard.

$$R_{mu}=\frac{h_0}{H_0}\cdot{}_cR_{mu}=1.0\times\frac{1}{81}=\frac{1}{81}>\frac{1}{250}$$

(h) The ductility index *F*

It is calculated according to the section 3.2.3 (3)(d) of the current Standard.

$R_{mu}(=1/81)\geqq R_y(=1/150)$ ((14) in Fig. 1.1.A-7)

Therefore, it is calculated with Eq. (16) in the section 3.2.3 (3)(d) of the current Standard.

((15) in Fig. 1.1.A-7)

$$F=\frac{\sqrt{2\cdot R_{mu}/R_y-1}}{0.75\times\left(1+0.05R_{mu}/R_y\right)}=\frac{\sqrt{2\cdot(1/81)/(1/150)-1}}{0.75\times\left(1+0.05\times(1/81)/(1/150)\right)}=2.01\ (\leq 3.2)$$

The ductility indices for each column with different hoop spacing are calculated in the same procedure. The result is listed in the table below.

Table 1.1.A-10 The ductility indices (hoop spacing:100mm)

Location	Story	${}_cQ_{mu}$ *(kN)*	${}_cQ_{su}$ *(kN)*	Failure mode	${}_cR_{max}$	${}_cR_{my}$	R_{my}	R_{su}	${}_cR_{mp}$	${}_cR_{mu}$	R_{mu}	F index Current	F index Previous
Y_1	4	421.5	456.7	CB	1/30	1/188	1/250	—	1/244	1/106	1/185	1.14	1.27
	3	496.0	470.5	CS	1/30	1/188	1/250	1/250	—	—	—	1.0	1.0
	2	563.7	487.5	CS	1/30	1/188	1/250	1/250	—	—	—	1.0	1.0
	1	624.8	504.5	CS	1/30	1/188	1/250	1/250	—	—	—	1.0	1.0
Y_2	4	254.2	352.8	CB	1/30	1/150	1/150	—	1/39	1/31	1/31	3.17	2.90
	3	306.2	374.0	CB	1/30	1/150	1/150	—	1/68	1/47	1/47	2.68	2.22
	2	352.0	395.3	CB	1/30	1/150	1/150	—	1/122	1/67	1/67	2.23	1.52
	1	391.8	416.5	CB	1/30	1/150	1/150	—	1/238	1/92	1/92	1.86	1.27
Y_3	4	238.4	269.9	CB	1/250	1/250	1/250	—	1/189	1/250	1/250	1.0	1.0
	3	263.4	274.0	CB	1/250	1/250	1/250	—	1/625	1/250	1/250	1.0	1.0
	2	287.2	278.2	CSS	1/250	1/250	1/250	1/250	—	—	—	0.8	0.8
	1	309.8	282.3	CSS	1/250	1/250	1/250	1/250	—	—	—	0.8	0.8

*Failure mode CSS : Extremely brittle, CS : Shear, CB : Flexure

Table 1.1.A-11 The ductility indices (hoop spacing:200mm)

Location	Story	$_cQ_{mu}$ (kN)	$_cQ_{su}$ (kN)	Failure mode	$_cR_{max}$	$_cR_{my}$	R_{my}	R_{su}	$_cR_{mp}$	$_cR_{mu}$	R_{mu}	F index Current	F index Previous
Y_1	4	401.4	402.0	CS	1/50	1/188	1/250	1/250	—	—	—	1.0	1.0
	3	495.9	418.9	CS	1/50	1/188	1/250	1/250	—	—	—	1.0	1.0
	2	563.7	436.0	CS	1/50	1/188	1/250	1/250	—	—	—	1.0	1.0
	1	624.8	453.0	CS	1/50	1/188	1/250	1/250	—	—	—	1.0	1.0
Y_2	4	254.2	301.3	CB	1/50	1/150	1/150	—	1/169	1/79	1/79	2.04	1.27
	3	306.2	322.5	CB	1/50	1/150	1/150	—	0	1/150	1/150	1.27	1.27
	2	352.0	343.7	CS	1/50	1/150	1/150	1/156	—	—	—	1.25	1.0
	1	391.7	365.0	CS	1/50	1/150	1/150	1/166	—	—	—	1.20	1.0
Y_3	4	238.5	237.2	CSS	1/250	1/250	1/250	1/250	—	—	—	0.8	0.8
	3	263.4	241.3	CSS	1/250	1/250	1/250	1/250	—	—	—	0.8	0.8
	2	287.2	245.6	CSS	1/250	1/250	1/250	1/250	—	—	—	0.8	0.8
	1	310.0	249.7	CSS	1/250	1/250	1/250	1/250	—	—	—	0.8	0.8

*Failure mode CSS : Extremely brittle, CS : Shear, CB : Flexure

Table 1.1.A-12 The ductility indices (hoop spacing:300mm)

Location	Story	$_cQ_{mu}$ (kN)	$_cQ_{su}$ (kN)	Failure mode	$_cR_{max}$	$_cR_{my}$	R_{my}	R_{su}	$_cR_{mp}$	$_cR_{mu}$	R_{mu}	F index Current	F index Previous
Y_1	4	421.4	379.1	CS	1/50	1/188	1/250	1/250	—	—	—	1.0	1.0
	3	495.9	396.2	CS	1/50	1/188	1/250	1/250	—	—	—	1.0	1.0
	2	563.7	413.2	CS	1/50	1/188	1/250	1/250	—	—	—	1.0	1.0
	1	624.8	430.1	CS	1/50	1/188	1/250	1/250	—	—	—	1.0	1.0
Y_2	4	254.2	278.5	CB	1/50	1/150	1/150	—	0	1/150	1/150	1.27	1.27
	3	306.2	299.7	CS	1/50	1/150	1/150	1/154	—	—	—	1.25	1.0
	2	352.0	321.0	CS	1/50	1/150	1/150	1/172	—	—	—	1.18	1.0
	1	391.7	342.2	CS	1/50	1/150	1/150	1/182	—	—	—	1.15	1.0
Y_3	4	238.5	222.7	CSS	1/250	1/250	1/250	1/250	—	—	—	0.8	0.8
	3	263.4	226.9	CSS	1/250	1/250	1/250	1/250	—	—	—	0.8	0.8
	2	287.2	231.0	CSS	1/250	1/250	1/250	1/250	—	—	—	0.8	0.8
	1	310.0	235.3	CSS	1/250	1/250	1/250	1/250	—	—	—	0.8	0.8

*Failure mode CSS : Extremely brittle, CS : Shear, CB : Flexure

In comparison with the F index by the previous Standard, the F index for the *Y1* column on the 4th floor with hoop spacing of 100mm is less than that calculated by the previous Standard. It is because the *F* index for a flexural column can vary continuously from 1.0 to 3.0 in the current Standard, whereas, it is greater than 1.27 in the previous Standard if the flexural column satisfies specific conditions. As shown in Fig. 1.1.A-23 in 6. Background Data, the *F* index can be less than 1.27 if the strength margin for shear failure ($_CQ_{su}/_CQ_{mu}$) and the R_{my} are small.

As for the *F* indices for all columns in the *Y2* frame ($h_0/D = 4.3, h_0/H_0 = 1.0$) that do not have standing and hanging wall, the values by the current Standard are greater than those by the previous Standard regardless of hoop spacing nor failure mode. The *F* index can be calculated as greater than 1.0 even if the strength margin for shear failure ($_CQ_{su}/_CQ_{mu}$) is relatively small, since the R_{my} for the column in the $Y2$ frame, of which failure mode is shear, is calculated as 1/150 (cf. 6. Background Data).

4.3 Basic seismic index of structure E_0

(1) The effective strength factor

The effective strength factor indicates the ratio of the restoring force at the ultimate deflection angle of the first group (R_1) to the ultimate strength. The practical calculation method is as follows; The effective strength factor is calculated using the ratio of R_1 to R_{my} where R_{my} is the deflection angle at yielding. As for the effective strength factor of the shear column, it is modified by the inverse number of the margin for shear failure (ultimate flexural strength / ultimate shear strength).

The effective strength factors for the column on the 1st floor with hoop spacing of 200mm are calculated as follows;

Y1 column:

$$ {}_cQ_{su} = 453.0 \ (kN), \ {}_cQ_{mu} = 624.8 \ (kN), \ F = 1.0, \quad R_{my} = 1/250, \quad R_{su} = 1/250 $$

: Shear column

Y2 column:

$$ {}_cQ_{su} = 365.0 \ (kN), \ {}_cQ_{mu} = 391.7 \ (kN), \ F = 1.20, \quad R_{my} = 1/150, \quad R_{su} = 1/166 $$

: Shear column

Y3 column:

$$ {}_cQ_{su} = 249.7 \ (kN), \ {}_cQ_{mu} = 310.0 \ (kN), \ F = 0.8, \quad R_{my} = 1/150, \quad R_{su} = 1/250 $$

: Extremely brittle column

a) *Y1* column

The effective strength factor α_s for the column of which failure mode is shear ($R_{su} = 1/250$) when the F index is 0.8 for the first group is calculated as follows according to Table 3 in the 3.2.1 (2)(b) of the current Standard.

$$ \alpha_s = \alpha_m \cdot \frac{Q_{mu}}{Q_{su}} $$

where, $\alpha_m = 0.3 + 0.7 \cdot \dfrac{R_1}{R_{my}}$

$R_1 = 1/500$ when F=0.8 for the first group.

Therefore,

$$ \alpha_m = 0.3 + 0.7 \cdot \frac{1/500}{1/250} = 0.65 \quad \text{since} \quad R_{my} = 1/250 $$

Finally,

$$ \alpha_s = 0.65 \times \frac{624.8}{453.0} = 0.897 $$

The effective strength factor for the *Y1* column becomes 0.897 for the first group with the F index of 0.8.

b) *Y2* column

(In case that the *F* index for the first group is 0.8)

The effective strength factor α_s for the column of which failure mode is shear ($R_{su} = 1/166$) when the *F* index is 0.8 for the first group is calculated as follows according to Table 3 in the 3.2.1 (2)(b) of the current Standard.

The same procedure as the *Y1* column is followed.

$$\alpha_m = 0.3 + 0.7 \cdot \frac{R_1}{R_{my}} = 0.3 + 0.7 \cdot \frac{1/500}{1/150} = 0.51$$

$$\alpha_s = \alpha_m \cdot \frac{Q_{mu}}{Q_{su}} = 0.51 \times \frac{391.7}{365.0} = 0.547$$

The effective strength factor of the *Y2* column becomes 0.547 for the first group with the *F* index of 0.8.

(In case that the *F* index for the first group is 1.0)

$R_1 = 1/250$ when *F*=1.0 for the first group.

Since the R_1 is less than R_{su}, the effective strength factor α_s for the column with the *F* index of 1.0 is calculated according to Table 3 in the 3.2.1 (2)(b) of the current Standard.

The same procedure as the *Y1* column is followed.

$$\alpha_m = 0.3 + 0.7 \cdot \frac{R_1}{R_{my}} = 0.3 + 0.7 \cdot \frac{1/250}{1/150} = 0.72$$

$$\alpha_s = \alpha_m \cdot \frac{Q_{mu}}{Q_{su}} = 0.72 \times \frac{391.7}{365.0} = 0.773$$

The effective strength factor of the *Y2* column becomes 0.773 for the first group with the *F* index of 1.0.

The relationship between restoring force and deflection angle of the *Y2* column on the first floor is shown in Fig. 1.1.A-8.

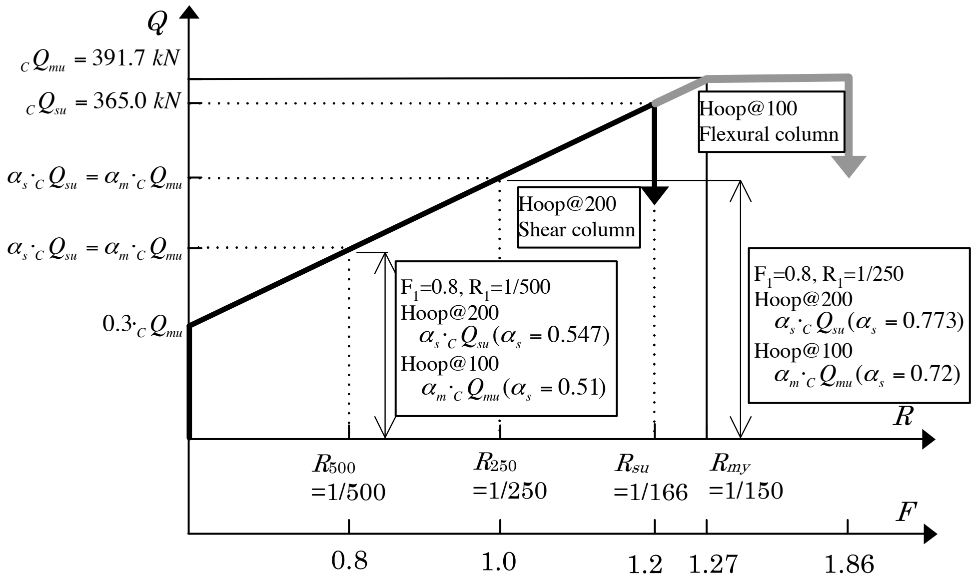


Fig. 1.1.A-8 The relationship between restoring force and deflection angle of the *Y2* column on the 1st floor

The effective strength factor for each column can be calculated in the same way. The factors calculated by the current and previous Standards are compared in the following tables.

Table 1.1.A-13 Effective strength factors (hoop spacing of 100mm)

Story	Frame	R_{my}	cQ_{mu} (kN)	cQ_{su} (kN)	F		1st group							
							F_I=0.8		F_I=1.0		1.0<F_I<1.27		1.27≦F_I	
					Current	Previous	**Current**	Previous	**Current**	Previous	**Current**	Previous	**Current**	Previous
4	Y3	1/250	238.4	269.9	**1.0**	1.0	—	–	**1.0**	1.0	—	–	—	–
	Y2	1/150	254.2	352.8	**3.14**	2.90	—	–	**0.72**	0.7	**0.87**	–	**1.0**	1.0
	Y1	1/250	421.5	456.7	**1.14**	1.27	—	–	**1.0**	0.7	**1.0**	–	—	1.0
3	Y3	1/250	263.4	274.0	**1.0**	1.0	—	–	**1.0**	1.0	—	–	—	–
	Y2	1/150	306.2	374.0	**2.68**	2.22	—	–	**0.72**	0.7	—	–	**1.0**	1.0
	Y1	1/250	496.0	470.5	**1.0**	1.0	—	–	**1.0**	1.0	—	–	—	–
2	Y3	1/250	287.2	278.2	**0.8**	0.8	**1.0**	1.0	—	–	—	–	—	–
	Y2	1/150	352.0	395.3	**2.23**	1.52	**0.51**	0.5	**0.72**	0.7	—	–	**1.0**	1.0
	Y1	1/250	563.7	487.5	**1.0**	1.0	**0.752**	0.7	**1.0**	1.0	—	–	—	–
1	Y3	1/250	309.8	282.3	**0.8**	0.8	**1.0**	1.0	—	–	—	–	—	–
	Y2	1/150	391.8	416.5	**1.86**	1.27	**0.51**	0.5	**0.72**	0.7	—	–	**1.0**	1.0
	Y1	1/250	624.8	504.5	**1.0**	1.0	**0.805**	0.7	**1.0**	1.0	—	–	—	–

Table 1.1.A-14 Effective strength factors (hoop spacing of 200mm)

Story	Frame	R_{my}	cQ_{mu} (kN)	cQ_{su} (kN)	F		1st group							
							F_1=0.8		F_1=1.0		1.0<F_1<1.27		1.27≦F_1	
					Current	Previous	**Current**	Previous	**Current**	Previous	**Current**	Previous	**Current**	Previous
4	Y3	1/250	238.5	237.2	**0.8**	0.8	**1.0**	1.0	**—**	–	**—**	–	**—**	–
	Y2	1/150	254.2	301.3	**2.01**	1.27	**0.51**	0.5	**0.72**	0.7	**—**	–	**1.0**	1.0
	Y1	1/250	421.4	402.0	**1.0**	1.0	**0.681**	0.7	**1.0**	1.0	**—**	–	**—**	–
3	Y3	1/250	263.4	241.3	**0.8**	0.8	**1.0**	1.0	**—**	–	**—**	–	**—**	–
	Y2	1/150	306.2	322.5	**1.27**	1.27	**0.51**	0.5	**0.72**	0.7	**—**	–	**1.0**	1.0
	Y1	1/250	495.9	418.9	**1.0**	1.0	**0.769**	0.7	**1.0**	1.0	**—**	–	**—**	–
2	Y3	1/250	287.2	245.6	**0.8**	0.8	**1.0**	1.0	**—**	–	**—**	–	**—**	–
	Y2	1/150	352.0	343.7	**1.25**	1.0	**0.522**	0.7	**0.737**	1.0	**1.0**	–	**—**	–
	Y1	1/250	563.7	436.0	**1.0**	1.0	**0.840**	0.7	**1.0**	1.0	**—**	–	**—**	–
1	Y3	1/250	310.0	249.7	**0.8**	0.8	**1.0**	1.0	**—**	–	**—**	–	**—**	–
	Y2	1/150	391.7	365.0	**1.20**	1.0	**0.547**	0.7	**0.773**	1.0	**1.0**	–	**—**	–
	Y1	1/250	624.8	453.0	**1.0**	1.0	**0.897**	0.7	**1.0**	1.0	**—**	–	**—**	–

Table 1.1.A-15 Effective strength factors (hoop spacing of 300mm)

Story	Frame	R_{my}	cQ_{mu} (kN)	cQ_{su} (kN)	F		1st group							
							F_1=0.8		F_1=1.0		1.0<F_1<1.27		1.27≦F_1	
					Current	Previous	**Current**	Previous	**Current**	Previous	**Current**	Previous	**Current**	Previous
4	Y3	1/250	238.5	222.7	**0.8**	0.8	**1.0**	1.0	**—**	–	**—**	–	**—**	–
	Y2	1/150	254.2	278.5	**1.27**	1.27	**0.51**	0.5	**0.72**	0.7	**—**	–	**1.0**	1.0
	Y1	1/250	421.4	379.1	**1.0**	1.0	**0.722**	0.7	**1.0**	1.0	**—**	–	**—**	–
3	Y3	1/250	263.4	226.9	**0.8**	0.8	**1.0**	1.0	**—**	–	**—**	–	**—**	–
	Y2	1/150	306.2	299.7	**1.25**	1.0	**0.521**	0.7	**0.736**	1.0	**1.0**	–	**—**	–
	Y1	1/250	495.2	413.2	**1.0**	1.0	**0.814**	0.7	**1.0**	1.0	**—**	–	**—**	–
2	Y3	1/250	287.2	231.0	**0.8**	0.8	**1.0**	1.0	**—**	–	**—**	–	**—**	–
	Y2	1/150	352.0	321.0	**1.18**	1.0	**0.559**	0.7	**0.790**	1.0	**1.0**	–	**—**	–
	Y1	1/250	563.7	413.2	**1.0**	1.0	**0.887**	0.7	**1.0**	1.0	**—**	–	**—**	–
1	Y3	1/250	310.0	235.3	**0.8**	0.8	**1.0**	1.0	**—**	–	**—**	–	**—**	–
	Y2	1/150	391.7	342.2	**1.15**	1.0	**0.591**	0.7	**0.834**	1.0	**1.0**	–	**—**	–
	Y1	1/250	624.8	430.1	**1.0**	1.0	**0.944**	0.7	**1.0**	1.0	**—**	–	**—**	–

(2) Basic seismic capacity index E_0

The E_0 index is calculated with Eq. (4) in the 3.2.1(2)(a) of the current Standard and with Eq. (5) in the 3.2.1 (2)(b) using the effective strength factor calculated in the previous section and the C index ($= Q_u / \Sigma W$). The calculated results for the hoop spacing of 100mm are listed below.

Table 1.1.A-16 C, F indices and effective strength factors (hoop spacing of 100mm)

Story	Frame (X2)	ΣW (kN)	Q_u (kN)	C	F	Effective strength factor, α_i for the first group			
						F_1=0.8	F_1=1.0	1.0<F_1<1.27	1.27≦F_1
4	Y3	529.6	238.4	0.442	1.0	—	1.0	—	—
	Y2		254.2	0.473	3.14	—	0.72	0.87	1.0
	Y1		421.5	0.783	1.14	—	1.0	1.0	—
3	Y3	1059.1	263.4	0.245	1.0	—	1.0	—	—
	Y2		306.2	0.285	2.68	—	0.72	—	1.0
	Y1		470.5	0.439	1.0	—	1.0	—	—
2	Y3	1588.7	278.2	0.176	0.8	1.0	—	—	—
	Y2		352.0	0.219	2.23	0.51	0.72	—	1.0
	Y1		487.5	0.302	1.0	0.752	1.0	—	—
1	Y3	2118.2	282.3	0.134	0.8	1.0	—	—	—
	Y2		391.8	0.183	1.86	0.51	0.72	—	1.0
	Y1		504.5	0.235	1.0	0.805	1.0	—	—

Table 1.1.A-17 The E_0 index (hoop spacing of 100mm)

Story	$\frac{n+1}{n+i}$	Eq.(5) : $(C_1+\Sigma\alpha_i \cdot C_i)\times F_i$						E_{01}
		1st group		2nd group		3rd group		
		F_1	C_1	α_2	C_2	α_3	C_3	
4	0.625	1.0	0.450	1.0	0.796	0.72	0.480	0.99
		1.14	0.796	0.87	0.480	—	—	0.86
		3.14	0.480	—	—	—	—	0.94
3	0.714	1.0	0.245 0.444	0.72	0.289	—	—	0.64
		2.68	0.289	—	—	—	—	0.55
		—	—	—	—	—	—	—
2	0.833	0.8	0.175	0.752	0.307	0.51	0.222	0.35
		1.0	0.307	0.72	0.222	—	—	0.39
		2.23	0.222	—	—	—		0.41
1	1.0	0.8	0.133	0.805	0.238	0.51	0.185	0.34
		1.0	0.238	0.72	0.188	—	—	0.37

Story	Group	Eq.(4) : $\sqrt{(C_1 \cdot F_1)^2+(C_2 \cdot F_2)^2+(C_3 \cdot F_3)^2}$						E_{02}
		1st group		2nd group		3rd group		
		C_1	F_1	C_2	F_2	C_3	F_3	
4	2	0.450 0.796	1.0	0.480	3.14	—	—	1.22
	3	0.450	1.0	0.796	1.14	0.480	3.14	1.13
3	2	0.249 0.444	1.0	0.289	2.68	—	—	0.74
	3	—	—	—	—	—	—	—
2	2	0.307	1.0	0.222	2.23	—	—	0.49
	3	—	—	—	—	—	—	—
1	2	0.238	1.0	0.185	1.86	—	—	0.42
	3	—	—	—	—	—	—	—

The E_0 indices for the hoop spacing of 200mm and 300mm can be calculated as well. The indices calculated by the current and previous Standards are compared in the following tables. The limitation for the $C_T \times S_D$ is not considered, since the main purpose of the tables is to compare the result of the current and previous Standards. The C indices in the table are the value not multiplied by the effective strength factors.

Table 1.1.A-18 E_0 index (hoop spacing of 100mm)

Story	Previous Standard						**Current Standard**					
	Group	Failure Mode	*C*	*F*	E_0		Group	Failure Mode	*C*	*F*	E_0	
4	1	CB	0.450	1.0	Eq.(4)	1.11	1	CB	0.450	1.0	Eq.(4)	1.22
	2	CB	0.796	1.27	Eq.(5)	0.84	2	CB	0.796	1.14	Eq.(5)	0.99
	3	CB	0.480	2.90	Considering Extremely brittle column		3	CB	0.480	3.14	Considering Extremely brittle column	
3	1	CS,CB	0.693	1.0	Eq.(4)	0.67	1	CS,CB	0.693	1.0	Eq.(4)	0.74
	2	CB	0.289	2.22	Eq.(5)	0.64	2	CB	0.289	2.68	Eq.(5)	0.64
	3				Considering Extremelybrittle column		3				Considering Extremely brittle column	
2	1	CSS	0.175	0.8	Eq. (4)	0.38	1	CSS	0.175	0.8	Eq.(4)	0.58
	2	CS	0.307	1.0	Eq. (5)	0.41	2	CS	0.307	1.0	Eq.(5)	0.41
	3	CB	0.222	1.52	Considering Extremely brittle column	0.33	3	CB	0.222	2.23	Considering Extremely brittle column	0.35
1	1	CSS	0.133	0.8	Eq.(4)	0.33	1	CSS	0.133	0.8	Eq.(4)	0.42
	2	CS	0.238	1.0	Eq.(5)	0.37	2	CS	0.238	1.0	Eq.(5)	0.37
	3	CB	0.185	1.27	Considering Extremely brittle column	0.31	3	CB	0.185	1.86	Considering Extremely brittle column	0.34

Table 1.1.A-19 E_0 index (hoop spacing of 200mm)

Story	Previous Standard						**Current Standard**					
	Group	Failure Mode	*C*	*F*	E_0		Group	Failure Mode	*C*	*F*	E_0	
4	1	CSS	0.448	0.8	Eq.(4)	0.61	1	CSS	0.448	0.8	Eq.(4)	0.77
	2	CS	0.759	1.0	Eq.(5)	0.68	2	CS	0.759	1.0	Eq.(5)	0.68
	3	CB	0.480	1.27	Considering Extremely brittle column	0.61	3	CB	0.480	2.01	Considering Extremely brittle column	0.60
3	1	CSS	0.228	0.8	Eq.(4)	0.39	1	CSS	0.228	0.8	Eq.(4)	0.39
	2	CS	0.396	1.0	Eq.(5)	0.43	2	CS	0.396	1.0	Eq.(5)	0.43
	3	CB	0.289	1.27	Considering Extremely brittle column	0.37	3	CB	0.289	1.27	Considering Extremely brittle column	0.39
2	1	CSS	0.155	0.8	Eq. (4)	0.41	1	CSS	0.155	0.8	Eq.(4)	0.32
	2	CS	0.490	1.0	Eq. (5)	0.41	2	CS	0.274	1.0	Eq.(5)	0.36
	3				Considering Extremely brittle column	0.33	3	CS	0.216	1.25	Considering Extremely brittle column	0.33
1	1	CSS	0.118	0.8	Eq.(4)	0.39	1	CSS	0.118	0.8	Eq.(4)	0.30
	2	CS	0.386	1.0	Eq.(5)	0.39	2	CS	0.214	1.0	Eq.(5)	0.34
	3				Considering Extremely brittle column	0.31	3	CS	0.172	1.20	Considering Extremely brittle column	0.32

Table 1.1.A-20 E_0 index (hoop spacing of 300mm)

Story	Previous Standard						Current Standard					
	Group	Failure Mode	C	F	E_0		Group	Failure Mode	C	F	E_0	
4	1	CSS	0.421	0.8	Eq.(4)	0.61	1	CSS	0.421	0.8	Eq.(4)	0.77
	2	CS	0.716	1.0	Eq.(5)	0.68	2	CS	0.716	1.0	Eq.(5)	0.68
	3	CB	0.480	1.27	Considering Extremely brittle column	0.61	3	CB	0.480	1.27	Considering Extremely brittle column	0.60
3	1	CSS	0.214	0.8	Eq.(4)	0.39	1	CSS	0.214	0.8	Eq.(4)	0.39
	2	CS	0.673	1.0	Eq.(5)	0.43	2	CS	0.390	1.0	Eq.(5)	0.43
	3				Considering Extremely brittle column	0.37	3	CS	0.283	1.25	Considering Extremely brittle column	0.39
2	1	CSS	0.145	0.8	Eq.(4)	0.41	1	CSS	0.145	0.8	Eq.(4)	0.32
	2	CS	0.462	1.0	Eq.(5)	0.41	2	CS	0.260	1.0	Eq.(5)	0.36
	3				Considering Extremely brittle column	0.33	3	CS	0.202	1.18	Considering Extremely brittle column	0.33
1	1	CSS	0.111	0.8	Eq.(4)	0.39	1	CSS	0.111	0.8	Eq.(4)	0.30
	2	CS	0.365	1.0	Eq.(5)	0.39	2	CS	0.203	1.0	Eq.(5)	0.34
	3				Considering Extremely brittle column	0.31	3	CS	0.162	1.15	Considering Extremely brittle column	0.32

(3) Summary of the E_0 indices by the current and previous Standards

The values calculated by the current and previous Standards are compared and the reason of the difference is discussed below.

(a) The result calculated with Eq. (4)

If the grouping result is the same independently of the Standard edition, the E_0 index calculated by the current Standard tends to be greater than that by the previous Standard. This is because the F index calculated by the current Standard for the flexural column in the third group becomes greater than that by the previous Standard.

On the other hand, the E_0 indices by the current Standard for the columns with hoop spacing of 200mm on the 1st and 2nd floor and the columns with hoop spacing of 300mm on the 1st to 3rd floor tend to be smaller than that by the previous Standard, since the F index for some columns of which failure mode is shear is greater than 1.0. One of the reasons is that the shear failure columns were categorized into two groups by the current Standard, however, they are done into one group by the previous Standard (cf. Fig. 1.1.A-25 in 6. Background Data).

(b) The result calculated with Eq. (5)

Since the C indices by the current and previous Standards are all the same, the difference of the E_0 indices calcutaled with Eq. (5) comes from the difference of the effective strength factor, no matter if the extremely brittle failure condition is considered or not. In case of the columns with the hoop spacing of 200mm on the 1st and 2nd floor and the columns with hoop spacing of 300mm on the 1st to 3rd floor, which are categorized into two groups, the E_0 index by the current Standard becomes smaller than that by the previous Standard, since the C index for the shear column with greater F index is multiplied by the effective strength factor of less than 1.0.

4.4 Seismic index of structure I_S

The calculation procedure for the columns with hoop spacing of 100mm is shown as an example in this section. The structure is assumed not to have the second-class prime element. The irregularity index and time index are also assumed as 1.0.

(1) $C_{TU} \times S_D$ index

The C_{TU} index is the index for cumulative strength of a structure at the ultimate limit state. Since the structure is assumed not to have the second-class prime element, if Eq. (4) is applied, the ultimate limit state is the deformation corresponding to the maximum F index of the groups. Thus the C_{TU} index is the accumulated strength at the maximum F index of the groups. On the other hand, if Eq. (5) is applied, the ultimate state is the deformation at the F_1 index, and the C_{TU} is the strength at the F_1 index.

Eq. (39) in the 5.2 (2) of the current Standard, that is, $C_{TU} \cdot S_D \geq 0.3 \cdot Z \cdot G \cdot U$, should be confirmed. Where, the zone index Z, the ground index G, and the usage index U are assumed as 1.0 in the example. Furthermore, since the irregularity index S_D is assumed as 1.0 for each story, the C_{TU} is $C_{TU} \times S_D$ for each story.

The maximum value of the F indices and the C_{TU} indices for each group are listed below. They are the result for the column with hoop spacing of 100mm.

Table 1.1.A-21 C_{TU} indices

Story	$\frac{n+1}{n+i}$	Maximum of F	C_{TU}	$C_{TU} \cdot S_D$	Evaluation	E_0 index	
						Eq.(4)	Eq.(5)
4	0.625	3.14	0.300	0.300	OK	1.22	0.94
		1.14	0.759	0.759	OK	0.86	0.86
		1.0	0.994	0.994	OK	–	0.99
3	0.714	2.68	0.206	0.206	NG	0.74	0.55
		1.0	0.643	0.643	OK	–	0.64
2	0.833	2.23	0.222	0.222	NG	0.58	0.41
		1.0	0.389	0.389	OK	–	0.39
		0.8	0.519	0.519	OK	–	0.35
1	1.0	1.86	0.185	0.185	NG	0.42	0.34
		1.0	0.371	0.371	OK	–	0.37
		0.8	0.419	0.419	OK	–	0.34

It can be found in the table that the E_0 index calcualed with Eq. (4) cannot be applied due to the $C_{TU} \times S_D$ limitation, although the value is greater than the E_0 index calcualed with Eq. (5).

(2) I_S index

From the results in the previous section, the I_S index is calculated using the E_0 index in case that $C_{TU} \cdot S_D$ is greater than or equal to $0.3 \cdot Z \cdot G \cdot U$. Here, the S_D and the T indices are both assumed as 1.0. The I_S index is calculated as $I_S = E_0 \cdot S_D \cdot T$.

(a) 4th story

The E_0 indices calcualed with Eqs. (4) and (5) are both able to be applied to the 4th story. The E_0 index with Eq. (4) is applied since it is greater than that with Eq. (5). Therefore, E_0=1.22.

$$I_S = 1.22 \times 1.0 \times 1.0 = 1.22$$

(b) 3rd story

The E_0 index calculated with Eq. (5) at the F_1 of 1.0 is applied to the 3rd story. Therefore, E_0=0.64.

$$I_S = 0.64 \times 1.0 \times 1.0 = 0.64$$

(c) 2nd story

The E_0 index calculated with Eq. (5) at the F_1 of 1.0 is applied to the 2nd story. Therefore, E_0=0.39.

$$I_S = 0.39 \times 1.0 \times 1.0 = 0.39$$

(d) 1st story

The E_0 index calculated with Eq. (5) at the F_1 of 1.0 is applied to the 1st story. Therefore, E_0=0.37.

$$I_S = 0.37 \times 1.0 \times 1.0 = 0.37$$

5. The Third Level Screening Method

In the third level screening, the seismic capacity should be evaluated supposing the yield mechanism of the structure by considering yielding in beams. The flowchart for the third level screening is shown in Fig. 1.1.A-9.

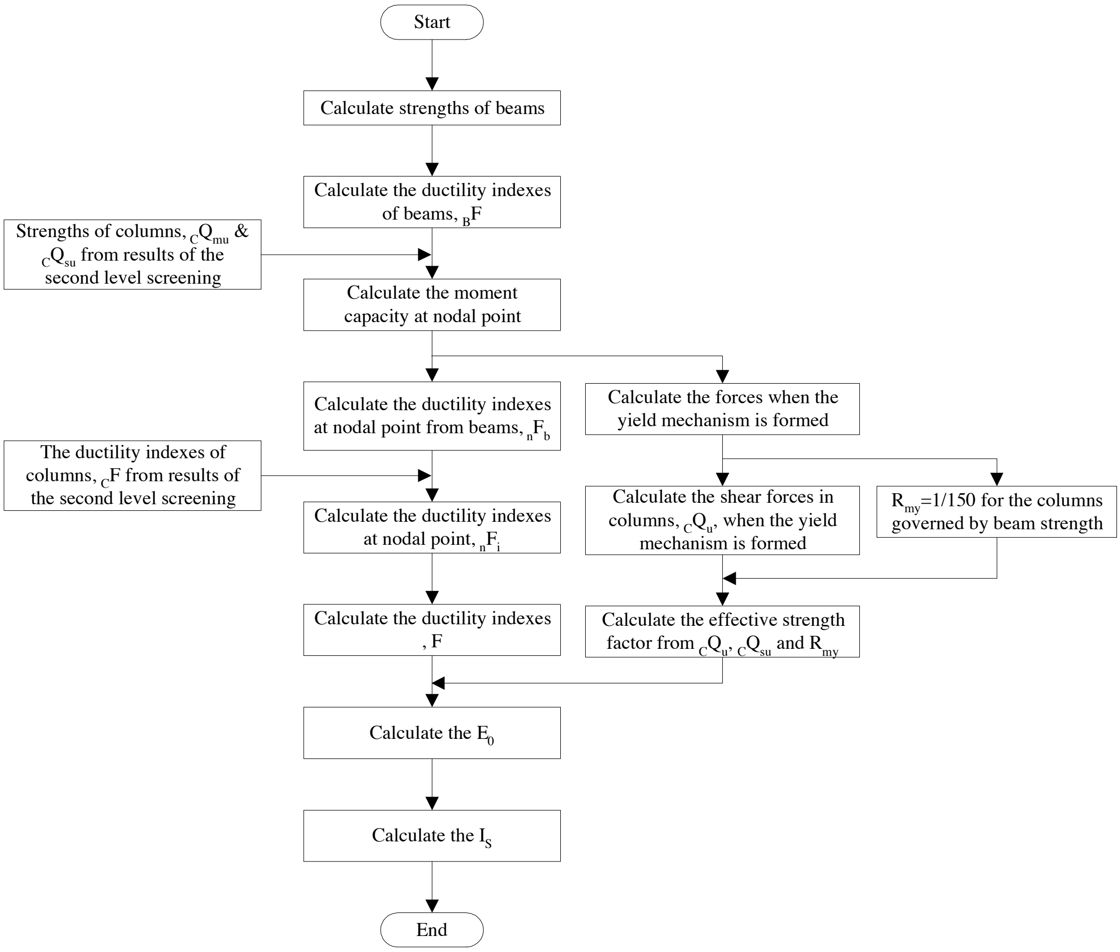


Fig. 1.1.A-9 Flowchart for the third level screening

Here, the calculation procedure with hoop spacing of 100mm is described as an example. Generally speaking, a seismic capacity may be influenced by the loading direction of shear force. However, in the example, a case of left-to-right loading is shown.

5.1 Strength of members

Eq. (A4-2) in the supplementary provisions of the current Standard is applied to the calculation of the ultimate flexural strength of the beams, which have standing wall and/or hanging wall on tensile side. Eq. (A4-1) in the supplementary provisions of the current Standard is applied to the calculation of the ultimate flexural strength of the beams, which do not have any standing or hanging wall, or have them on the compressive side. Eq. (A4-5) in the supplementary provisions of the current Standard is applied to the calculation of the ultimate shear strength. Since the purpose of the example is to show the calculation procedures, the gravity loads in the beams are neglected here.

The values calculated in the second level screening are used for the strengths of columns. The varied axial force during an earthquake is not considered.

The shear force of column, if it is governed by the beam strength, is derived from the moment capacity at nodal points when beams yield.

(1) Ultimate flexural strength

The ultimate flexural strengths of the 2~3G$_1$ in the *Y1* frame are calculated as follows.

(a) When the tensile force is acting in the bottom side of the beam

The ultimate flexural strength is calculated with Eq. (A4-1) in the supplementary provisions of the current Standard, where the height of the beam + the height of the standing wall is applied to the total height of the member.

$$d = 600 + 1100 - 50 = 1650\ (mm)$$

$$a_t = 4 \times 387 = 1548\ (mm^2)$$

$$\sigma_y = 343\ (N/mm^2)$$

$$\begin{aligned} M_{u1} &= 0.9 \cdot a_t \cdot \sigma_y \cdot d \\ &= 0.9 \times 1548 \times 343 \times 1650 \\ &= 788481540(N \cdot mm) = 788.5(kN \cdot m) \end{aligned}$$

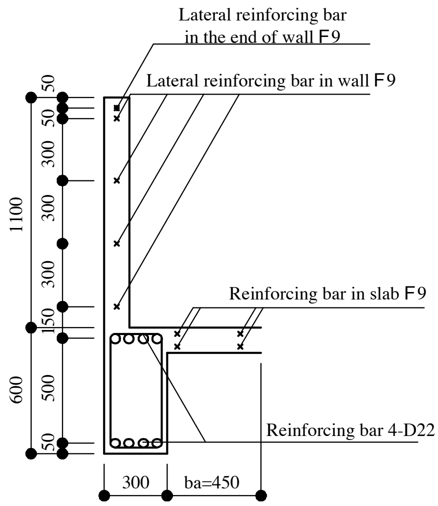


Fig. 1.1.A-10 Cross section of 2~3G$_1$

(b) When the tensile force is acting in the top side of the beam

The effective width of the slab is calculated according to the AIJ Standard for Structural Calculation of Reinforced Concrete Structures (AIJ: Architectural Institute of Japan). The reinforcing bars of slab in the effective width are able to be counted. The bar arrangement in the wall is assumed as shown in Fig. 1.1.A-10. The ultimate flexural strength is calculated with Eq. (A4-2) in the supplementary provisions of the current Standard. The strain of concrete at the compressive strength ${}_c\varepsilon_B$ and the yield strain of reinforcing bars ${}_s\varepsilon_y$ are assumed as 2000μ and 1667μ respectively.

The effective depth, d_e, is assumed as the distance between the centroid of tensile reinforcing bars and bottom of the beam (outermost of the compressive region).

$$d_e = \frac{550 \times (4 \times 387 + 4 \times 64) + 700 \times 64 + 1000 \times 64 + 1300 \times 64 + 1600 \times 64 + 1650 \times 133}{4 \times 387 + 8 \times 64 + 133} = 686.8(mm)$$

$$x_{nb} = \frac{{}_c\varepsilon_B}{{}_c\varepsilon_B + {}_s\varepsilon_y} d_e = \frac{2000}{2000 + 1667} \times 686.8 = 374.6(mm)$$

$$\begin{aligned} 0.85 F_c \cdot t \cdot x_{nb} / \sigma_y - \Sigma a_t' \frac{\sigma_y'}{\sigma_y} &= 0.85 \times 17.7 \times 300 \times 374.6 / 343 - (8 \times 64 + 133) \times \frac{294}{343} \\ &= 4376.5(mm^2) \end{aligned}$$

$$a_{te} = a_t + a_t'\left(\frac{\sigma_y'}{\sigma_y}\right) = 4 \times 387 + (8 \times 64 + 133) \times \left(\frac{294}{343}\right) = 2100.9(mm^2)$$

$$< 0.85F_c \cdot t \cdot x_{nb} / \sigma_y - \Sigma a_t' \frac{\sigma_y'}{\sigma_y} (= 4376.5)$$

$$x_n = \frac{a_{te} \cdot \sigma_y}{0.85 \cdot F_c \cdot t} = \frac{2100.9 \times 343}{0.85 \times 17.7 \times 300} = 159.7(mm)$$

$$M_{u2} = a_{te} \cdot \sigma_y (d_e - 0.5x_n) = 2100.9 \times 343 \times (686.8 - 0.5 \times 159.7)$$

$$= 437373450(N \cdot mm) = 437.4(kN \cdot m)$$

(c) Shear force at the moment capacity Q_{mu}

The clear span length of the G_1 beam ℓ_0 is calculated as follows;

$$\ell_0 = 4500 - 600 = 3900(mm) = 3.9(m)$$

Therefore,

$$Q_{mu} = (M_{u1} + M_{u2}) / \ell_0 = (788.5 + 437.4) / 3.9 = 314.3(kN)$$

(2) Ultimate shear strength

The ultimate shear strength is calculated with Eq. (A4-5) in the supplementary provisions of the current Standard, neglecting the effective width of the slab. In the direction where the standing/hanging wall carries compression force, the equivalent rectangular sections to the standing/hanging wall and beam section are applied to the equation. In the direction where the standing/hanging wall carries tensile force, they are neglected and Eq. (A4-4a)) or Eq. (A4-5) is used. The ultimate shear strength of the beam is estimated as the average of the strength in both directions.

The ultimate shear strengths of the 2~3G_1 in the *Y1* frame are calculated as follows.

The height of the standing wall, L' : $L' = 1100(mm)$

Total height, L : $L = 1700(mm)$

Dimension of the beam, $b \times D = 300 \times 600(mm)$

Total sectional area, ΣA: $\Sigma A = L' \times t + b \times D = 1100 \times 120 + 600 \times 300 = 312000(mm^2)$

$b_e = \Sigma A / L = 312000 / 1700 = 183.5(mm)$

Shear reinforcement ratio of beam, (p_w) : $p_w = \dfrac{2 \times 64}{300 \times 300} = 1.42 \times 10^{-3}$

Shear reinforcement ratio within the wall panel, (p_s) : $p_s = \dfrac{64}{120 \times 300} = 1.78 \times 10^{-3}$

$$p_{we} = \frac{p_w \times b \times D + p_s \times t \times L'}{\Sigma A} = \frac{1.42 \times 300 \times 600 + 1.78 \times 120 \times 1100}{312000} \times 10^{-3} = 1.57 \times 10^{-3}$$

(a) When the tensile force is acting in the bottom side of the beam Q_{su1}

$$p_{t1} = \frac{a_t}{b \cdot D} = \frac{4 \times 387}{300 \times 600} = 0.860(\%)$$

$$M / Q \cdot d_e = \frac{l_0}{2} \Big/ d_e = \frac{3900}{2} \Big/ 1650 = 1.18$$

$$Q_{su1} = \left\{ \frac{0.053 p_t^{0.23} (18 + F_c)}{M / (Q \cdot d_e) + 0.12} + 0.85 \sqrt{p_{we} \cdot \sigma_{wy}} \right\} \cdot b_e \cdot j_e$$

$$= \left\{ \frac{0.053 \times 0.860^{0.23} \times (18 + 17.7)}{1.18 + 0.12} + 0.85 \sqrt{1.57 \times 10^{-3} \times 294} \right\} \times 183.5 \times 1650 \times \frac{7}{8}$$

$$= (1.406 + 0.578) \times 183.5 \times 1650 \times \frac{7}{8} = 525439(N) = 525.4(kN)$$

(b) When the tensile force is acting in the top side of the beam Q_{su2}

$$p_{t2} = \frac{a_t}{b \cdot d} = \frac{4 \times 387}{300 \times 550} \times 100 = 0.938(\%)$$

$$M / Q \cdot d = \frac{l_0}{2} \Big/ d = \frac{3900}{2} \Big/ 550 = 3.54 \rightarrow 3.0$$

$$Q_{su2} = \left\{ \frac{0.053 p_t^{0.23} (18 + 17.7)}{M / (Q \cdot d) + 0.12} + 0.85 \sqrt{p_w \cdot \sigma_{wy}} \right\} \cdot b \cdot j$$

$$= \left\{ \frac{0.053 \times 0.938^{0.23} \times (18 + 17.7)}{3.0 + 0.12} + 0.85 \sqrt{1.42 \times 10^{-3} \times 294} \right\} \times 300 \times 550 \times \frac{7}{8}$$

$$= (0.598 + 0.549) \times 300 \times 550 \times \frac{7}{8} = 165568(N) = 165.6(kN)$$

(c) Ultimate shear strength Q_{su}

The ultimate shear strength is calculated as the average of the ultimate shear strength of Q_{su1} and Q_{su2}.

$$Q_{su} = (Q_{su1} + Q_{su2}) / 2 = (525.4 + 165.6) / 2 = 345.5(kN)$$

The strengths of other members were are calculated as well. The calculation result is listed as follows.

Table 1.1.A-22 Strength of beams

Story	Location	Member sign	Ultimate flexural strength (kN·m)		Q_{mu}(kN)	Ultimate shear strength(kN)		Q_{su}(kN)	Q_u(kN)	Failure mode
			M_{u1} (Tensile in bottom)	M_{u2} (Compression in bottom)		Q_{su1} (Tensile in bottom)	Q_{su2} (Compression in bottom)			
R	Y3	RG_2 (W/ hanging wall)	196.6	322.0	129.6	118.6	225.5	172.0	129.6	Flexural
	Y2	RG_1	197.1	271.6	120.2	160.1	160.1	160.1	120.2	Flexural
	Y1	RG_1	197.1	234.4	110.6	160.1	160.1	160.1	110.6	Flexural
4	Y3	$4G_2$ (W/ standing and hanging wall)	566.9	516.7	270.9	412.9	225.5	319.2	270.9	Flexural
	Y2	$4G_1$	197.1	271.6	120.2	160.1	160.1	160.1	120.2	Flexural
	Y1	$4G_1$ (W/ standing wall)	591.4	382.7	249.8	501.6	160.1	330.8	249.8	Flexural
3	Y3	$3G_2$ (W/ standing and hanging wall)	566.9	516.7	270.9	412.9	225.5	319.2	270.9	Flexural
	Y2	$3G_1$	262.8	337.3	153.9	165.6	165.6	165.6	153.9	Flexural
	Y1	$3G_1$ (W/ standing wall)	788.5	437.4	314.3	525.4	165.6	345.5	314.3	Flexural
2	Y3	$2G_2$ (W/ standing and hanging wall)	566.9	516.7	270.9	412.9	225.5	319.2	270.9	Flexural
	Y2	$2G_1$	262.8	337.3	153.9	165.6	165.6	165.6	153.9	Flexural
	Y1	$2G_1$ (W/ standing wall)	788.5	437.4	314.3	525.4	165.6	345.5	314.3	Flexural
1	Y3	FG (W/ standing wall)	595.4	624.6	305.0	803.3	384.9	594.1	305.0	Flexural
	Y2	FG	303.0	457.4	195.0	389.7	389.7	389.7	195.0	Flexural
	Y1	FG (W/ standing wall)	595.4	624.6	312.8	814.7	389.7	602.2	312.8	Flexural

5.2 Moment capacity at nodal point

(1) Moment capacity of beam at nodal point

The failure modes of all beams are evaluated as flexural failure as shown in Table 1.1.A-22. The moment capacity at nodal point when the yield hinge is formed at the face of column is calculated here. As mentioned earlier, the gravity loads in the beams are neglected here.

The moment capacity of the $4G_2$ beam is calculated as follows.

(The direction where the tension acts in the bottom end of beam)

$$566.9(kN\cdot m)+270.9(kN)\times 0.25(m)$$
$$=634.6(kN\cdot m)$$

(The direction where the tension acts in the upper end of beam)

$$516.7(kN \cdot m) + 270.9(kN) \times 0.25(m)$$
$$= 584.4(kN \cdot m)$$

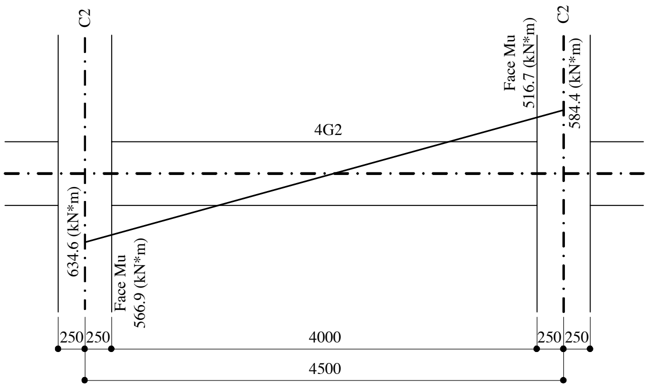


Fig. 1.1.A-11 Moment capacity at nodal point of the $4G_2$ beam

The moment capacities of other beams at nodal point are also calculated as well. The calculated result is listed in Table 1.1.A-23.

(2) Moment capacity of column at nodal point

The moment capacity calculated in the second level screening (Table 1.1.A-7) is applied. The failure modes of columns are shear failure and flexural failure as listed in the table. In case that the column fails in shear, the moment at nodal point when it fails in shear is calculated. In case that the column fails in flexure, the moment capacity at nodal point when the yield hinges are formed at upper and bottom ends of the column is calculated.

The moment capacity of the $1C_2$ column is calculated as follows.

The failure mode of the $1C_2$ column is classified into the extremely brittle column.

(Upper end)

$$282.3(kN) \times 1.3(m) = 367.0(kN \cdot m)$$

(Bottom end)

$$282.3(kN) \times 2.2(m) = 621.1(kN \cdot m)$$

The moment capacities of other columns at nodal point are also calculated as well. The calculated result is listed in Table 1.1.A-24.

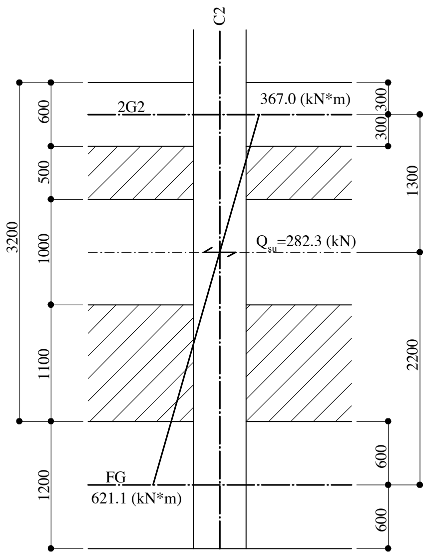


Fig. 1.1.A-12 Moment capacity at nodal point of the 1C$_2$ column

Table 1.1.A-23 Moment capacities of beams at nodal point (KN-M)

Story	Location	Tension in bottom end	Tension in upper end
R	Y3	229.0	354.4
	Y2	233.2	307.7
	Y1	230.3	267.6
4	Y3	634.6	584.4
	Y2	233.2	307.7
	Y1	666.3	457.6
3	Y3	634.6	584.4
	Y2	309.0	383.5
	Y1	882.8	531.7
2	Y3	634.6	584.4
	Y2	309.0	383.5
	Y1	882.8	531.7
1	Y3	671.7	700.9
	Y2	361.5	515.9
	Y1	689.2	718.4

Table 1.1.A-24 Moment capacities of columns at nodal point (KN-M)

Story	Location	Upper end	Bottom end
4	Y3	310.1	453.2
	Y2	406.8	406.8
	Y1	442.5	906.0
3	Y3	342.4	500.4
	Y2	489.9	489.9
	Y1	494.1	1011.7
2	Y3	361.7	528.6
	Y2	563.2	563.2
	Y1	511.9	1048.1
1	Y3	367.0	621.1
	Y2	626.7	744.2
	Y1	529.7	1235.9

5.3 Failure mode of nodal point and forces when the yield mechanism is formed

(1) Failure mode of nodal point

At each nodal point, the summation of the moment capacities of the left and right beams and that of the upper and lower columns are compared. The lower value of summation governs the failure mode at nodal point.

The calculation procedure for the nodal point on the 4th floor in the *Y1* frame is shown below as an example.

The summation of moment capacities of beams, $\sum M_b$;

$$\Sigma M_b = 666.3 + 457.6 = 1123.9(kN \cdot m)$$

The summation of moment capacities of columns, $\sum M_c$;

$$\sum M_c = 906.0(4C_1 \ bottom \ end) + 494.1(3C_1 \ upper \ end) = 1400.1(kN \cdot m)$$

Since $\sum M_b < \sum M_c$, the failure mode of the nodal point is evaluated as beam failure.

(2) Forces when the yield mechanism is formed

The forces when the yield mechanism is formed are calculated as follows; If the failure mode at the nodal point is beam failure, the $\sum M_b$ is equally divided into the upper and bottom column. If it is column failure, the $\sum M_c$ is equally divided into the left and right beam. However, the divided moment force can not exceed the moment capacity of beam and column at the nodal point.

The calculation procedure for the nodal point on the 4th floor in the *Y1* frame is shown below as an example. Since the failure mode at the nodal point is the beam failure, the $\sum M_b$ is equally divided into the upper and bottom column.

$$\frac{1}{2}\Sigma M_b = \frac{1}{2} \times 1123.9 = 562.0(kN \cdot m)$$

Since the moment force equally divided into the $3C_1$ upper end exceeds the moment capacity of the column, the moment capacity of the column is 494.1 (kN·m) when the yield mechanism is formed. Therefore, the moment force for the $4C_1$ bottom end is 1123.9-494.1=629.8 (kN·m).

The moment forces for other members when the yield mechanism is formed are calculated as well. The result is shown in Fig. 1.1.A-13

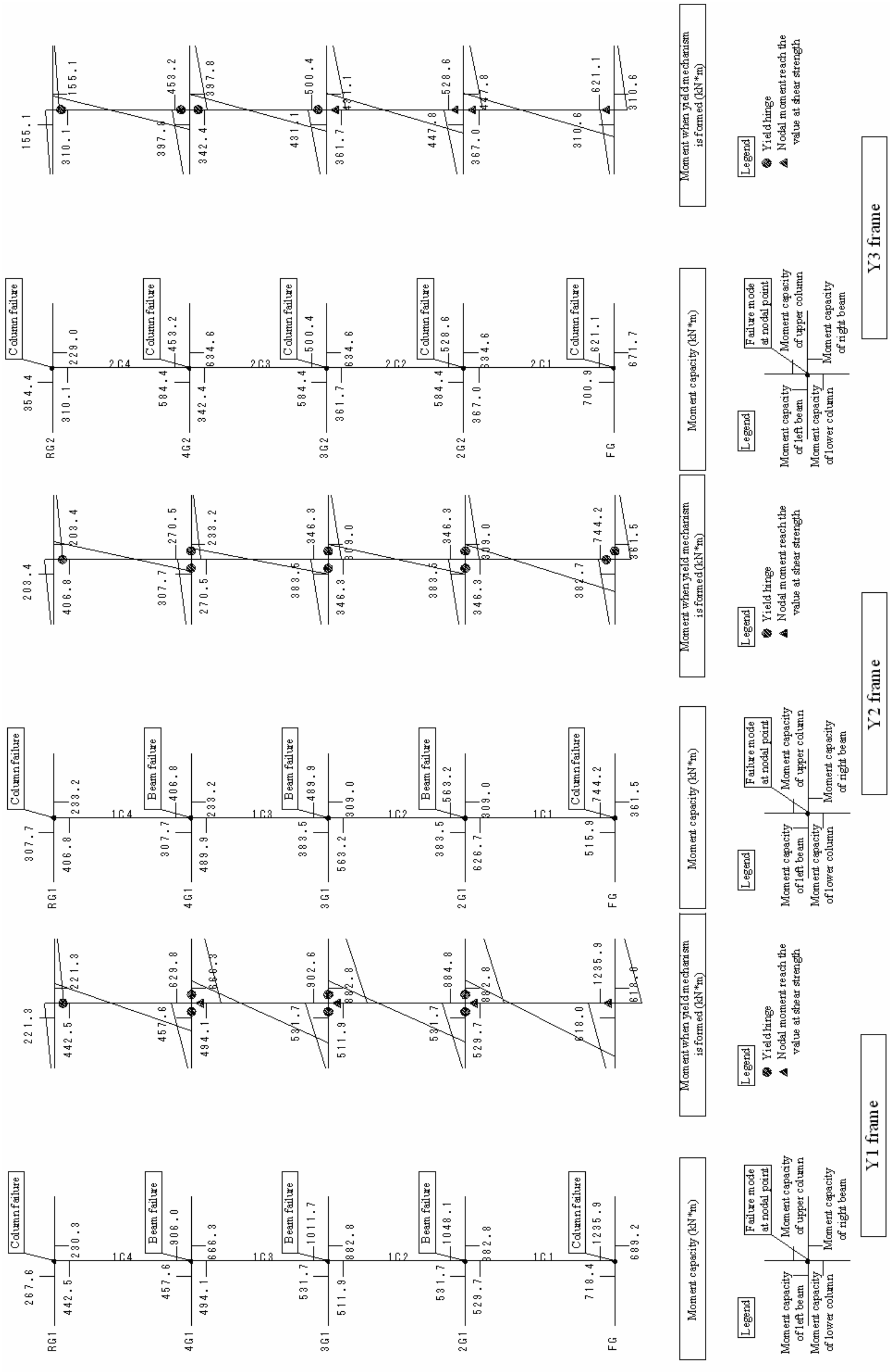


Fig. 1.1.A-13 Moment capacity at nodal point and force when the yield mechanism is formed

5.4 Shear force in ultimate state and failure mode of column

(1) Shear force in ultimate state of column $_cQ_u$

The shear force at ultimate state is calculated by dividing the sum of moment capacities estimated in the section 5.3 at upper and lower nodal points of the column by its height.

Shear force of the column on the 2nd floor in the *Y1* frame is calculated as follows.

The moment force at nodal point of the upper end of the column is 511.9 (kN-m), and that of the bottom end of the column is 884.8 (kN-m). The story height is 3.2(m). Therefore,

$$_cQ_u = \frac{511.9 + 884.8}{3.2} = 436.4(kN)$$

(2) Failure mode of the column

The failure mode is evaluated according to Fig. 2.3.1-2 in the appendix 2 of the current Standard (see the translators' note 1 below).

The failure mode of the column on the 2nd floor in the *Y1* frame is evaluated as an example below. The all beams connected to the nodal points of the upper and bottom ends of the column yield. Therefore, the failure mode of the column is evaluated as the "column governed by flexural beam".

The shear force in ultimate state and failure modes of other columns are evaluated as well. The results are listed in Table 1.1.A-25.

Translators' Note 1 --

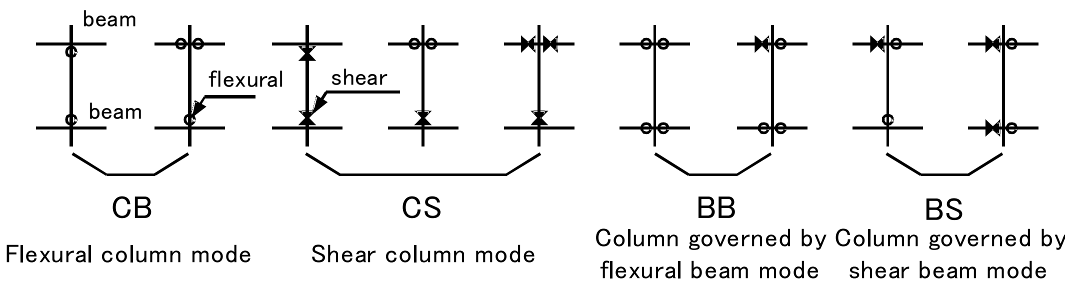


Figure TN.1 Failure mode evaluation in the third level screening method
(quoted from Figure 2.3.1-2 on page 278 of the current Standard of Japanese version)

-- **End of Translators' Note 1**

Table 1.1.A-25 Shear force at ultimate state $_cQ_u$ and failure mode

Story	Location	$_cQ_u(kN)$	Failure mode
4	Y3	238.5	Flexural column
	Y2	211.6	Flexural column
	Y1	335.2	Flexural column
3	Y3	263.4	Flexural column
	Y2	192.8	Column governed by flexural beam
	Y1	436.4	Column governed by flexural beam
2	Y3	278.2	Extremely brittle column
	Y2	216.5	Column governed by flexural beam
	Y1	436.4	Column governed by flexural beam
1	Y3	282.3	Extremely brittle column
	Y2	311.7	Flexural column
	Y1	504.5	Shear column

5.5 Ductility index *F*

The procedures to calculate the *F* index for the third level screening is shown below.

1) The ultimate flexural strength and the ductility index for each column itself $_nF_c$ calculated for the second level screening are applied to the third level screening.

2) The ductility index for each beam $_bF$ is calculated according to the strength margin of the beam for shear failure.

3) The ductility index for each node $_nF_b$ is calculated based on the ultimate flexural strength and the $_bF$ of beams around the nodal point.

4) The $_nF_b$ or $_nF_c$ is applied to the ductility index of the node $_nF_i$ considering the moment force ratio of beam and column around the node in the ultimate state. In the example, the modified ductility index for column itself $_nF'_c$ is calculated as the product of the $_nF_c$ and the strength margin of its ultimate shear force to the shear force when the yield mechanism is formed.

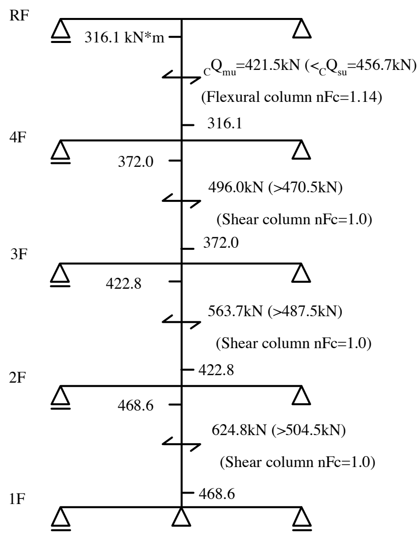


Figure 1.1.A-14 $_nF_c$ for columns in the *Y1* frame

5) The *F* index is decided according to the moment force at both ends of the column when yield mechanism is formed, the $_nF_i$ and the failure mode at the nodal point.

The calculation procedure for the *Y1* frame is shown as an example.

(1) Ductility index for columns

The ductility index of the column itself $_nF_c$ is calculated independently for upper and lower columns of the nodal point, and the value calculated for the second level screening method according to the section 3.2.3 (3)(c)-(f) of the current Standard is applied. The ultimate flexural strength at the face of the beam, the shear force at the flexural yielding, the ultimate shear strength, and the ${}_nF_c$ for the columns are shown in Fig. 1.1.A-14.

(2) Ductility index for beams

The ductility indices ${}_bF$ for beams are calculated with the equation (Eq. (26) in the section 3.2.3 (3)(d) of the current Standard) shown below according to the strength margin for shear failure.

$$As\ for\ {}_bQ_{su}/{}_bQ_{mu} \le 0.9:\ {}_bF = 1.5$$

$$As\ for\ {}_bQ_{su}/{}_bQ_{mu} \ge 1.2:\ {}_bF = 3.5$$

The linear interpolation method is used for the value of ${}_bQ_{su}/{}_bQ_{mu}$ in between.

In case that the beam has standing and/or hanging wall, the ${}_bF$ of 1.5 is applied in the example.

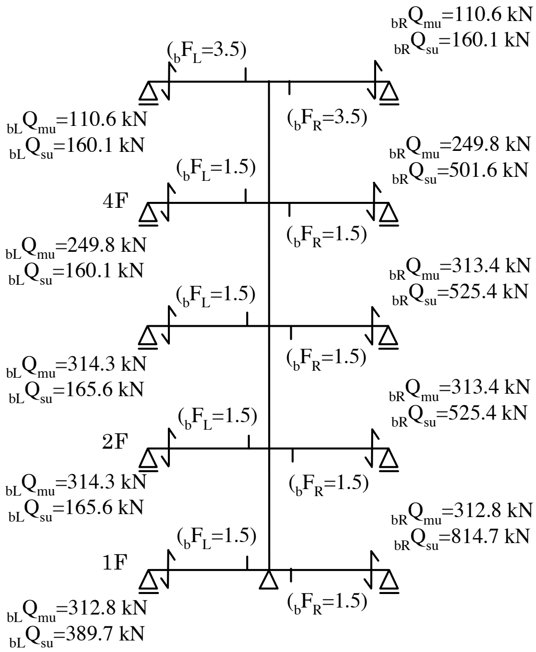


Fig. 1.1.A-15 ${}_bF$ for beams in the *Y1* frame

(The beam on the roof)

Since the beams on the left and right sides are assumed to be all the same, the calculation procedure for one beam is shown as follows.

$${}_bQ_{su} = 160.1kN,\ {}_bQ_{mu} = 110.6kN$$

$$Since\ {}_bQ_{su}/{}_bQ_{mu} = 160.1/110.6 = 1.45 > 1.2,\ {}_bF = 3.5$$

(The beams on the 1st to the 4th floor)

Since all beams have standing walls, the ${}_bF$ of 1.5 is applied to all beams.

(3) Ductility index for nodal points governed by beam strength

The ductility index for nodal point governed by beam strength ${}_nF_b$ is calculated as the weighted average value of the ductility index for beams ${}_bF$ connected to the nodal point as shown below (Eq. (25) in the section 3.2.3 (4)(c)(iii) of the current Standard). The weighting factor is calculated based on the ultimate flexural strength of beams.

$${}_nF_b = \Sigma({}_bq_i \times {}_bF_i)$$

$${}_bq_i = {}_bM_{ui} / \Sigma_b M_{ui}$$

where:

$_bF_i$ = The ductility index for left and right beams of the nodal point.

$_bM_{ui}$ = The ultimate flexural strength of beams at the nodal point.

(The nodal point on the roof floor)

Since the ductility indices for the left and right beams are all the same ($_bF = 3.5$),

Left beam

$$_bq_{Left} = {}_bM_{uLeft}/\Sigma_bM_{ui} = 267.6/(267.6+230.3) = 0.537$$

Right beam

$$_bq_{Right} = {}_bM_{uRight}/\Sigma_bM_{ui} = 230.3/(267.6+230.3) = 0.463$$

$$_nF_b = \Sigma(_bq_i \times {}_bF_i) = 0.537 \times 3.5 + 0.463 \times 3.5 = 3.5$$

* The calculation procedure is shown here. However, since the ductility indices for left and right beams are all the same, the $_nF_b$ is equal to the $_bF$ without calculating the $_bq$.

(As for the nodal points on the 4th to 1st floor)

Since the ductility indices for the left and right beams are all the same ($_bF = 1.5$), $_nF_b = {}_bF = 1.5$

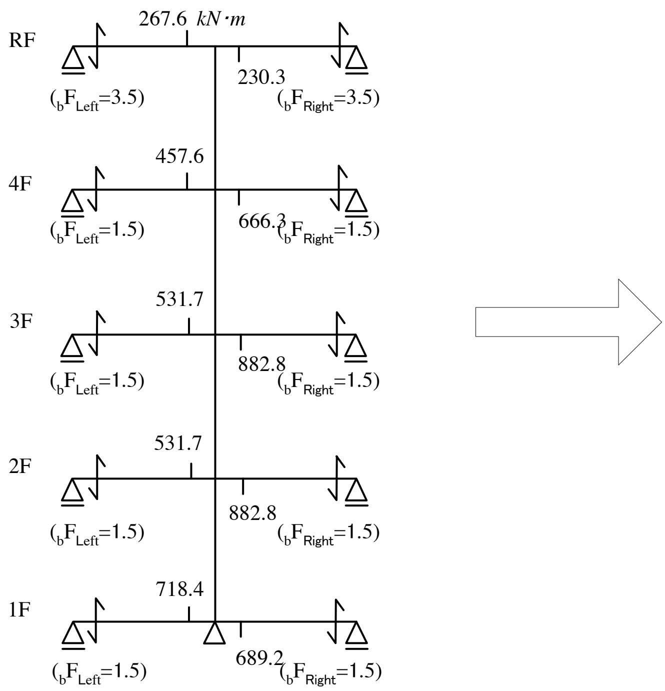


Fig. 1.1.A-16 Moment capacity of beams at nodal points and $_bF$ governed by the beam strength in the *Y1* frame

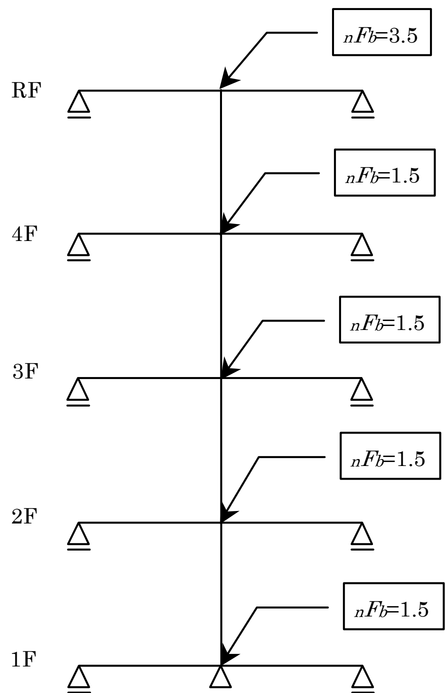


Fig. 1.1.A-17 $_nF_b$ at nodal points governed by the beam strength in the *Y1* frame

(4) Ductility index for nodal point

The ductility index for the nodal point ${}_nF_i$ is calculated with the equation shown below (Eq. (24) in the section 3.2.3 (4) (c) (ii) of the current Standard) according to the ultimate flexural strength of beams and columns connected to the nodal point, the ductility index for the nodal point governed by beam strength, and the ductility index for the column itself. In the example, if the behavior of the nodal point is governed by the beam strength and the margin of column strength is less than 40%, the ductility index for column ${}_nF_c$ is modified to ${}_nF'_c$ by multiplying the strength margin of its ultimate shear force to the shear force when the yield mechanism is formed in order to take the energy dissipation in beams into account.

In case of $\Sigma\,{}_cM_{ui}/\Sigma\,{}_bM_{ui} \geq 1.4$;	The ductility index governed by beam strength ${}_nF_b$ is applied. (${}_nF_i = {}_nF_b$)
In case of $\Sigma\,{}_cM_{ui}/\Sigma\,{}_bM_{ui} \leq 1.0$;	The ductility index for column itself ${}_nF_c$ is applied. (${}_nF_i = {}_nF_c$)

The interpolation method can be used for the value of $\Sigma\,{}_cM_{ui}/\Sigma\,{}_bM_{ui}$ in between based on the beam ductility index ${}_nF_b$ and the modified column ductility index ${}_nF'_c$.

where:

$\Sigma\,{}_cM_{ui}$ = The sum of ultimate flexural strengths of upper and lower columns of the nodal point.

$\Sigma\,{}_bM_{ui}$ = The sum of ultimate flexural strengths of left and right beams of the nodal point.

${}_nF_b$ = The ductility index for the nodal point governed by the beam strength calculated in the section (3).

${}_nF'_c$ = The modified ductility index for column itself calculated from the equation below.

$${}_nF'_c = {}_nF_c \times \frac{\min({}_cQ_{mu},\ {}_cQ_{su})}{{}_cQ_u} \leq 3.2$$

However, if the ${}_nF'_c$ is greater than the ductility index governed by the beams connected to the nodal point of the top or bottom of column, the ${}_nF'_c$ should be $\min({}_nF_{bTop},\ \underline{{}_nF_{bBottom}})$.

${}_nF_c$ = The ductility index for column itself calculated for the second level screening.

${}_cQ_{mu}$ = The shear force at the ultimate flexural strength.

${}_cQ_{su}$ = The ultimate shear strength.

${}_cQ_u$ = The shear force when the yield mechanism is formed.

(a) Modified ductility index for column itself $_nF'_c$

(The column on the 4th floor)

Since $_nF_c = 1.14$, $\min\left({}_cQ_{mu}, {}_cQ_{su}\right) = 421.5$ kN・m, $_cQ_u = 335.2$ kN・m (See Figs. 1.1.A-18, -19), $_nF'_c = 1.14 \times (421.5/335.2) = 1.43 < 3.2$.

Since $_nF'_c = 1.43 < \min\left({}_nF_{bTop}, \underline{{}_nF_{bBottom}}\right) = 1.5$, $_nF'_c = 1.43$.

The calculated ductility indices for other stories are shown in Fig. 1.1.A-19.

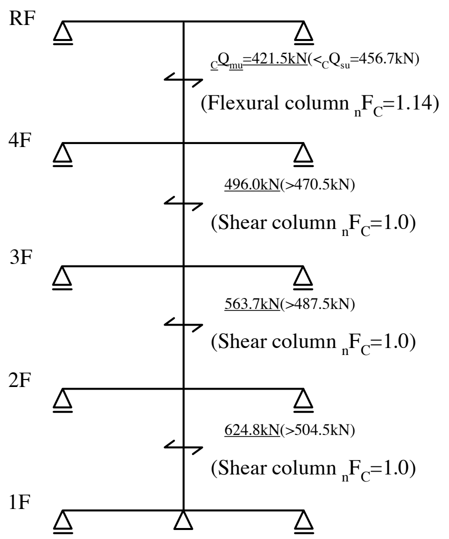


Fig. 1.1.A-18 Not modified $_nF_c$ in the *Y1* frame

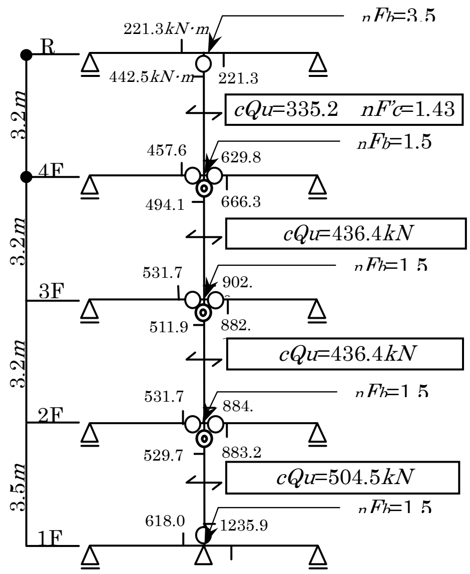


Fig. 1.1.A-19 Modified $_nF'_c$ in the *Y1* frame

(b) Ductility index for nodal point $_nF_i$

(Column on the 4th floor)

-Top end

$$\Sigma_c M_{ui} = 442.5(kN \cdot m)$$

$$\Sigma_b M_{ui} = 267.6 + 230.3 = 497.9(kN \cdot m)$$

$$\Sigma_c M_{ui} / \Sigma_b M_{ui} < 1.0$$

Since $\Sigma_c M_{ui} / \Sigma_b M_{ui} < 1.0$, the column is expected to fail. Therefore, the modified ductility index for column itself $_nF'_c$ is applied to the ductility index at nodal point $_nF_i$. Thus, the ductility index for the top of column is calculated as $_nF_{4Top} = {}_nF'_c = 1.43$. (column failure)

-Bottom end

$$\Sigma_c M_{ui} = 906.0 + 494.1 = 1400.1(kN \cdot m)$$

$$\Sigma_b M_{ui} = 457.6 + 666.3 = 1123.9(kN \cdot m)$$

$$\Sigma_c M_{ui} / \Sigma_b M_{ui} = 1400.1/1123.9 = 1.24$$

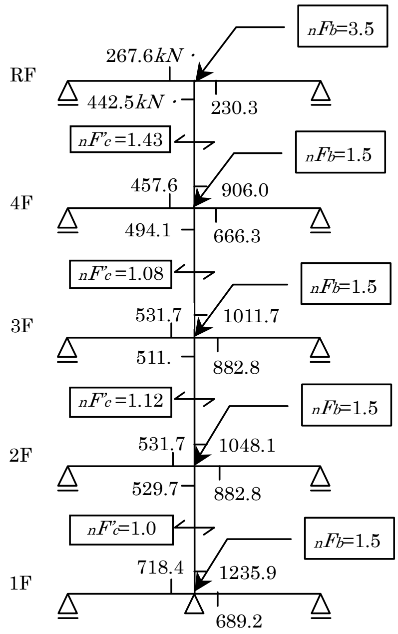


Fig. 1.1.A-20 Moment capacity of beams and columns in the *Y2* frame and $_nF_b$ by beams and $_nF_c$ by columns

Since $1.0 < \Sigma_c M_{ui} / \Sigma_b M_{ui} < 1.4$, the strength of the column is not strong enough compared to the strength of beam (strength ratio <1.4), and the beam failure is not assured. Therefore, the ductility index for the nodal point $_nF_i$ takes intermediate value between the beam ductility index $_nF_b$ and the modified column ductility index $_nF'_c$. Thus, the ductility index for the bottom of column on the 4th floor is calculated using the linear interpolation method as below.

$$_nF_{4Bottom} = \frac{_nF_b - {}_nF'_c}{1.4 - 1.0}\left(\frac{\Sigma_c M_{ui}}{\Sigma_b M_{ui}} - 1.0\right) + {}_nF'_c = \frac{1.5 - 1.43}{1.4 - 1.0} \times (1.24 - 1.0) + 1.43 = 1.47$$

(beam failure)

(Column on the 3rd floor)

-Top end

$$\Sigma_c M_{ui} = 906.0 + 494.1 = 1400.1(kN \cdot m)$$

$$\Sigma_b M_{ui} = 457.6 + 666.3 = 1123.9(kN \cdot m)$$

$$\Sigma_c M_{ui} / \Sigma_b M_{ui} = 1400.1/1123.9 = 1.24$$

Since $1.0 < \Sigma_c M_{ui} / \Sigma_b M_{ui} < 1.4$, the linear interpolation method is applied.

$$_nF_{3Top} = \frac{1.5 - 1.08}{1.4 - 1.0} \times (1.24 - 1.0) + 1.08 = 1.33$$ (beam failure)

-Bottom end

$$\Sigma_c M_{ui} = 1011.7 + 511.9 = 1523.6(kN \cdot m)$$

$$\Sigma_b M_{ui} = 531.7 + 882.8 = 1414.5(kN \cdot m)$$

$$\Sigma_c M_{ui} / \Sigma_b M_{ui} = 1523.6/1414.5 = 1.077$$

Since $1.0 < \Sigma_c M_{ui} / \Sigma_b M_{ui} < 1.4$, the linear interpolation method is applied.

$${}_nF_{3Bottom} = \frac{1.5-1.08}{1.4-1.0} \times (1.077-1.0) + 1.08 = 1.16 \qquad \text{(beam failure)}$$

(Column on the 2nd floor)

-Top end

$$\Sigma_c M_{ui} = 1011.7 + 511.9 = 1523.6(kN \cdot m)$$

$$\Sigma_b M_{ui} = 531.7 + 882.8 = 1414.5(kN \cdot m)$$

$$\Sigma_c M_{ui} / \Sigma_b M_{ui} = 1523.6/1414.5 = 1.077$$

Since $1.0 < \Sigma_c M_{ui} / \Sigma_b M_{ui} < 1.4$, the linear interpolation method is applied.

$${}_nF_{2Top} = \frac{1.5-1.12}{1.4-1.0} \times (1.077-1.0) + 1.12 = 1.19 \qquad \text{(beam failure)}$$

-Bottom end

$$\Sigma_c M_{ui} = 1048.1 + 529.7 = 1577.8(kN \cdot m)$$

$$\Sigma_b M_{ui} = 531.7 + 882.8 = 1414.5(kN \cdot m)$$

$$\Sigma_c M_{ui} / \Sigma_b M_{ui} = 1577.8/1414.5 = 1.116$$

Since $1.0 < \Sigma_c M_{ui} / \Sigma_b M_{ui} < 1.4$, the linear interpolation method is applied.

$${}_nF_{2Bottom} = \frac{1.5-1.12}{1.4-1.0} \times (1.116-1.0) + 1.12 = 1.23 \qquad \text{(beam failure)}$$

(Column on the 1st floor)

-top end

$$\Sigma_c M_{ui} = 1048.1 + 529.7 = 1577.8(kN \cdot m)$$

$$\Sigma_b M_{ui} = 531.7 + 882.8 = 1414.5(kN \cdot m)$$

$$\Sigma_c M_{ui} / \Sigma_b M_{ui} = 1577.8/1414.5 = 1.116$$

Since $1.0 < \Sigma_c M_{ui} / \Sigma_b M_{ui} < 1.4$, the linear interpolation method is applied.

$${}_nF_{1Top} = \frac{1.5-1.0}{1.4-1.0} \times (1.116-1.0) + 1.0 = 1.14 \qquad \text{(beam failure)}$$

-Bottom end

$$\Sigma_c M_{ui} = 1235.9(kN \cdot m)$$

$$\Sigma_b M_{ui} = 718.4 + 689.2 = 1407.6(kN \cdot m)$$

$$\Sigma_c M_{ui} / \Sigma_b M_{ui} = 1235.9/1407.6 = 0.878$$

Since $\Sigma_c M_{ui} / \Sigma_b M_{ui} < 1.0$, the modified ductility index for the column itself ${}_nF'_c$ is applied to the ${}_nF_i$. Thus, the ductility index for the bottom of the column on the 1st floor is ${}_nF_{1Bottom} = {}_nF'_c = 1.0$ (column failure).

(5) Ductility index for column governed by beam strength

The ductility index for column governed by beam strength F_{ave} is calculated with the equation shown below (Eq. (22) in the section 3.2.3 (4)(c)(i) of the current Standard) according to the moment forces at the top and bottom of column when the yield mechanism is formed, and the ductility index for the nodal point. The forces when the yield mechanism is formed and the ductility indices for nodal points in the $Y1$ frame are shown in Fig. 1.1.A-21. (in the figure, the ○ mark indicates the yield hinge location and the ◎ mark indicates that the moment force at the nodal point reachs the moment force corresponding to the ultimate shear strength.)

$$F_{ave} = \Sigma\left({}_nq_i \times {}_nF_i\right)$$

$${}_nq_i = {}_nM_{ui} / \Sigma_n M_{ui}$$

where:

${}_nF_i$ = The ductility index for the nodal point at the top or bottom of the column.

${}_nM_{ui}$ = The moment force when the yield mechanism is formed.

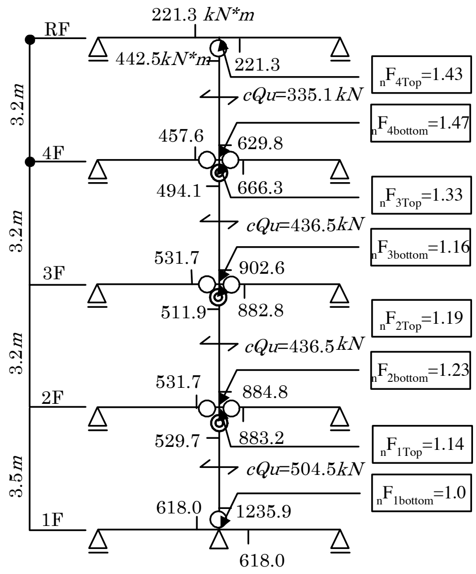


Fig. 1.1.A-21 Ultimate shear strength of column and ductility index for nodal points ${}_nF_i$ in the $Y1$ frame

(Column on the 4th floor)

The sum of the moment forces at the nodal points of the top and bottom of the column when the yield mechanism is formed is calculated as follows.

$$\Sigma_n M_{u4} = 442.5 + 629.8 = 1072.3 kN \cdot m$$

--Top of column

The ductility index for the nodal point, ${}_nF_{4Top} = 1.43$

Moment force when the yield mechanism is formed, ${}_nM_{u4Top} = 442.5 kN \cdot m$

$${}_nq_{4Top} = {}_nM_{u4Top} / \Sigma_n M_{u4} = 442.5/1072.3 = 0.413$$

--Bottom of column

The ductility index for the nodal point, ${}_nF_{4Bottom} = 1.47$

Moment force when the yield mechanism is formed, ${}_nM_{u4Bottom} = 629.8 kN \cdot m$

$${}_nq_{4Bottom} = {}_nM_{u4Bottom} / \Sigma_n M_{u4} = 629.8/1072.3 = 0.587$$

Thus, the ductility index F_{ave} is calculated as follow.

$$F_{ave} = \Sigma\left({}_nq_i \times {}_nF_i\right) = {}_nq_{4Top} \times {}_nF_{4Top} + {}_nq_{4Bottom} \times {}_nF_{4Bottom} = 0.413 \times 1.43 + 0.587 \times 1.47 = 1.45$$

(Column on the 3rd floor)

The sum of the moment forces at the nodal points of the top and bottom of the column when the yield mechanism is formed is calculated as follows.

$$\Sigma_n M_{u3} = 494.1 + 902.6 = 1396.7 kN \cdot m$$

--Top of column

The ductility index for the nodal point, ${}_nF_{3Top} = 1.33$

Moment force when the yield mechanism is formed, ${}_nM_{u3Top} = 494.1 kN \cdot m$

$${}_nq_{3Top} = {}_nM_{u3Top} / \Sigma_n M_{u3} = 494.1/1396.7 = 0.354$$

--Bottom of column

The ductility index for the nodal point, ${}_nF_{3Bottom} = 1.16$

Moment force when the yield mechanism is formed, ${}_nM_{u3Bottom} = 902.6 kN \cdot m$

$${}_nq_{3Bottom} = {}_nM_{u3Bottom} / \Sigma_n M_{u3} = 902.6/1396.7 = 0.646$$

Thus, the ductility index F_{ave} is calculated as follow.

$$F_{ave} = \Sigma\left({}_nq_i \times {}_nF_i\right) = {}_nq_{3Top} \times {}_nF_{3Top} + {}_nq_{3Bottom} \times {}_nF_{3Bottom} = 0.354 \times 1.33 + 0.646 \times 1.16 = 1.22$$

(Column on the 2nd floor)

The sum of the moment forces at the nodal points of the top and bottom of the column when the yield mechanism is formed is calculated as follows.

$$\Sigma_n M_{u2} = 512 + 885 = 1379 kN \cdot m$$

--Top of column

The ductility index for the nodal point, ${}_nF_{2Top} = 1.19$

Moment force when the yield mechanism is formed, ${}_nM_{u2Top} = 512 kN \cdot m$

$${}_nq_{2Top} = {}_nM_{u2Top} / \Sigma_n M_{u2} = 512/1397 = 0.367$$

--Bottom of column

The ductility index for the nodal point, ${}_nF_{2Bottom} = 1.23$

Moment force when the yield mechanism is formed, ${}_nM_{u2Bottom} = 885 kN \cdot m$

$${}_nq_{2Bottom} = {}_nM_{u2Bottom} / \Sigma_n M_{u2} = 885/1397 = 0.633$$

Thus, the ductility index F_{ave} is calculated as follow.

$$F_{ave} = \Sigma\left({}_nq_i \times {}_nF_i\right) = {}_nq_{2Top} \times {}_nF_{2Top} + {}_nq_{2Bottom} \times {}_nF_{2Bottom} = 0.367 \times 1.19 + 0.633 \times 1.23 = 1.22$$

(Column on the 1st floor)

The sum of the moment forces at the nodal points of the top and bottom of the column when the yield mechanism is formed is calculated as follows.

$$\Sigma_n M_{u1} = 529.7 + 1235.9 = 1765.6 kN \cdot m$$

--Top of column

The ductility index for the nodal point, ${}_nF_{1Top} = 1.14$

Moment force when the yield mechanism is formed, ${}_nM_{u1Top} = 529.7 kN \cdot m$

$${}_nq_{1Top} = {}^{n}M_{u1Top} \big/ \Sigma_n M_{u1} = 529.7 \big/ 1765.6 = 0.300$$

--Bottom of column

The ductility index for the nodal point, ${}_nF_{1Bottom} = 1.0$

Moment force when the yield mechanism is formed, ${}_nM_{u1Bottom} = 1235.9 kN \cdot m$

$${}_nq_{1Bottom} = {}^{n}M_{u1Bottom} \big/ \Sigma_n M_{u1} = 1235.9 \big/ 1765.6 = 0.700$$

Thus, the ductility index F_{ave} is calculated as follow.

$$F_{ave} = \Sigma\left({}_nq_i \times {}_nF_i\right) = {}_nq_{1Top} \times {}_nF_{1Top} + {}_nq_{1Bottom} \times {}_nF_{1Bottom} = 0.300 \times 1.14 + 0.700 \times 1.0 = 1.04$$

The ductility indices F_{ave} for other columns are also calculated in the same way and listed in Table 1.1.A-26.

Table 1.1.A-26 Ductility index F_{ave} according to the third level screening

Story	Frame	Failure mode	${}_nF'_c$		${}_bF$		${}_nF_b$	${}_nF_i$		F_{ave}
					Left	Right		Applied*1	${}_nF_i$	
4	Y3	CB	1.0	Top	1.5	1.5	1.5	${}_nF'_c$	1.0	1.0
				Bottom	1.5	1.5	1.5	${}_nF'_c$	1.0	
	Y2	CB	3.2	Top	3.5	3.5	3.5	${}_nF'_c$	3.2	3.32
				Bottom	3.5	3.5	3.5	${}_nF_b$	3.5	
	Y1	CB	1.43	Top	3.5	3.5	3.5	${}_nF'_c$	1.43	1.45
				Bottom	1.5	1.5	1.5	*ave*	1.47	
3	Y3	CB	1.0	Top	1.5	1.5	1.5	${}_nF'_c$	1.0	1.0
				Bottom	1.5	1.5	1.5	${}_nF'_c$	1.0	
	Y2	BB	2.59*	Top	3.5	3.5	3.5	${}_nF_b$	3.5	2.99
				Bottom	2.59	2.59	2.59	${}_nF_b$	2.59	
	Y1	BB	1.08	Top	1.5	1.5	1.5	*ave*	1.33	1.22
				Bottom	1.5	1.5	1.5	*ave*	1.16	
2	Y3	CSS	0.8	Top	1.5	1.5	1.5	${}_nF'_c$	0.8	0.8
				Bottom	1.5	1.5	1.5	${}_nF'_c$	0.8	
	Y2	BB	2.59*	Top	2.59	2.59	2.59	${}_nF_b$	2.59	2.59
				Bottom	2.59	2.59	2.59	${}_nF_b$	2.59	
	Y1	BB	1.12	Top	1.5	1.5	1.5	*ave*	1.19	1.22
				Bottom	1.5	1.5	1.5	*ave*	1.23	
1	Y3	CSS	0.8	Top	1.5	1.5	1.5	${}_nF'_c$	0.8	0.8
				Bottom	1.5	1.5	1.5	${}_nF'_c$	0.8	
	Y2	CB	2.34	Top	2.59	2.59	2.59	${}_nF_b$	2.59	2.42
				Bottom	3.5	3.5	3.5	${}_nF'_c$	2.34	
	Y1	CS	1.0	Top	1.5	1.5	1.5	*ave*	1.14	1.04
				Bottom	1.5	1.5	1.5	${}_nF'_c$	1.0	

ave : the value linearly interpolated with ${}_nF'_c$ and ${}_nF_b$

* mark indicates that ${}_nF'_c$ is calculated as $\min\left({}_nF_{bTop}, {}_nF_{bBottom}\right)$, since the ductility index for the column itself modified according to the strength margin of its ultimate shear force to the shear force when the yield mechanism is formed is greater than the ductility index for the nodal point governed by the beam strength connected to the top or bottom of the column.

5.6 Basic seismic index of structure E_0

(1) Effective strength factor

The calculation method for the effective strength factor is the same as that in the second level screening. Here, the R_{my} is assumed as 1/150 according to the comment in the section 3.2.3 of the current Standard, which states "the relationship between lateral restoring force and deflection angle of the column is similar to the relationship of long column (with long clear height) in case that beam fails prior to column". On the other hand, it is assumed that the R_{my} takes the average value of the R_{my} for the top and bottom ends of the column with the failure mode such as weak beam at the top of column and weak column at the bottom of column.

The calculation procedures of the third floor are shown below as an example.

--*Y1* column

$_CQ_{su} = 471(kN)$, $_CQ_u = 436(kN)$, $F = 1.22$

Weak column at the top : $R_{my} = 1/250$

Weak beam at the bottom : $R_{my} = 1/150$

The R_{my} is considered as the average value of the R_{my} for the top and bottom ends of the column.

$$R_{my} = \frac{1/250 + 1/150}{2} = 1/188$$

--*Y2* column

$_CQ_{su} = 374(kN)$, $_CQ_u = 193(kN)$, $F = 2.99$, $R_{my} = 1/150$, column governed by the beam strength (weak beam)

--*Y3* column

$_CQ_{su} = 274(kN)$, $_CQ_u = {}_CQ_{mu} = 263(kN)$, $F = 1.0$, $R_{my} = 1/250$, flexural column (weak column)

(a) *Y1* column (*F*=1.22, R_{my}=1/188)

(In case of the first group with $F = 1.0$ (*Y3* column))

According to Table 3 in the section 3.2.1 (2)(b) of the current Standard, the effective strength factor for the *Y1* column corresponding to the F_1 of 1.0 (*Y3* column) is calculated as α_m of 0.83 as follows.

$$\alpha_m = 0.3 + 0.7 \cdot \frac{R_1}{R_{my}} = 0.3 + 0.7 \times \frac{1/250}{1/188} = 0.83$$

(b) *Y2* column

(In case of the first group with *F*=1.0 (*Y3* column))

According to Table 3 in the section 3.2.1 (2)(b) of the current Standard, the effective strength factor for the *Y2* column corresponding to the F_1 of 1.0 (*Y3* column) is calculated as α_m of 0.72.

(In case of the first group with *F*=1.22 (*Y1* column))

According to Table 3 in the section 3.2.1 (2)(b) of the current Standard, the effective strength factor α_m for the *Y2* column corresponding to the F_1 of 1.0 (*Y1* column) is calculated as follows.

$$\alpha_m = 0.3 + 0.7 \cdot \frac{R_1}{R_{my}}$$

Here, the R_1 of the first group with the F_1 of 1.22 is calculated with Eq. (15) in the section 3.2.3 (3)(d) of the current Standard.

$$F = 1.0 + 0.27 \frac{R_{mu} - R_{250}}{R_y - R_{250}} \qquad \text{Eq. (15) in the section 3.2.3 (3)(d)}$$

$R_y = 1/150$ and $R_{mu} = R_1$ are applied to the equation, then

$$R_1 = \frac{F - 1.0}{0.27}\left(R_{150} - R_{250}\right) + R_{250}$$

is derived. Therefore, R_1 can be calculated as follows.

$$R_1 = \frac{1.22 - 1.0}{0.27}\left(\frac{1}{150} - \frac{1}{250}\right) + \frac{1}{250} = \frac{1}{162}$$

Since the R_{my} for the *Y2* column is 1/150, the effective strength factor is calculated as follows.

$$\alpha_{m2} = 0.3 + 0.7 \cdot \frac{R_1}{R_{my}} = 0.3 + 0.7 \times \frac{1/162}{1/150} = 0.95$$

Therefore, the effective strength factor for the *Y2* column corresponding to the F_1 of 1.22 is calculated as 0.95.

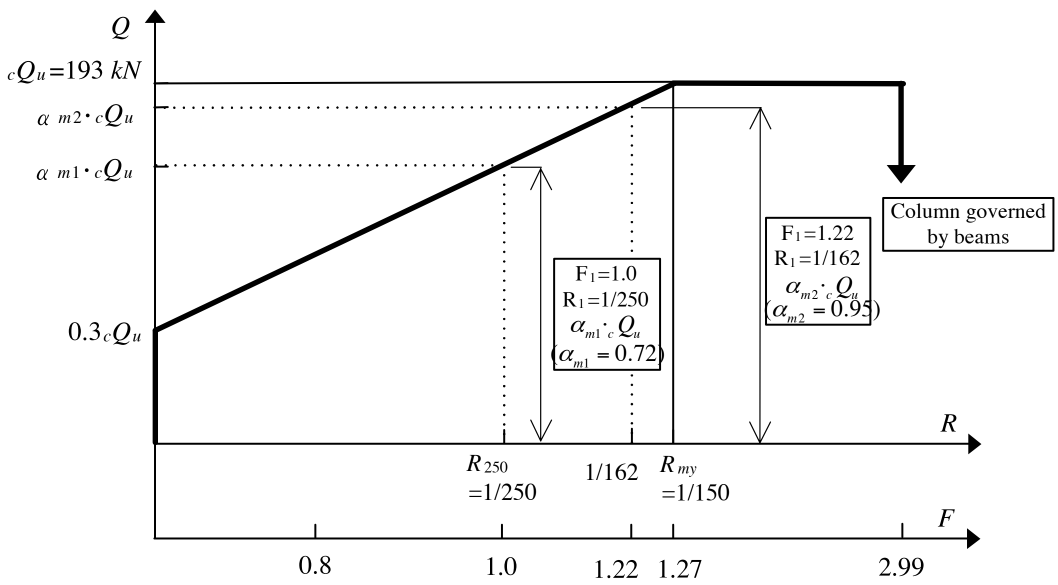


Fig. 1.1.A-22 Force-deformation relationship of *Y2* column (3RD floor)

The factors for the other stories are also calculated in the same way and listed in Table 1.1.A-27.

Table 1.1.A-27 Effective strength factor

Story	Frame	R_{my}	$_CQ_{mu}(tf)$	$_CQ_{su}(tf)$	$_CQ_u(tf)$	F	First group			
							F_1=0.8	F_1=1.0	1.0<F_1<1.27	1.27≦F_1
4	Y3	1/250	238	270	238	1.0	—	1.0	—	—
	Y2	1/150	254	353	212	3.32	—	0.72	—	1.0
	Y1	1/188	421	457	335	1.45	—	0.83	—	1.0
3	Y3	1/250	263	274	263	1.0	—	1.0	—	—
	Y2	1/150	306	374	193	2.99	—	0.72	0.95	1.0
	Y1	1/188	496	471	436	1.22	—	0.83	1.0	—
2	Y3	1/250	287	278	278	0.8	1.0	—	—	—
	Y2	1/150	352	395	217	2.59	0.51	—	0.95	1.0
	Y1	1/188	564	487	436	1.22	0.56	—	1.0	—
1	Y3	1/250	310	282	282	0.8	1.0	—	—	—
	Y2	1/150	392	416	312	2.42	0.51	—	0.76	1.0
	Y1	1/250	625	504	504	1.04	0.81	—	1.0	—

(2) Basic seismic index of structure E_0

The E_0 index is calculated in the same way as the second level screening method with the effective strength factor α calculated in the previous section. The results are listed below.

Table 1.1.A-28 *C*, *F* indices and effective strength factor for column

Story	Frame (X2)	ΣW (kN)	Q_u (kN)	C	F	Effective strength factor for the first group, α_i			
						F_1=0.8	F_1=1.0	1.0<F_1<1.27	1.27≦F_1
4	Y3	529.6	238	0.450	1.00	—	1.0	—	—
	Y2		212	0.400	3.32	—	0.72	—	1.0
	Y1		335	0.632	1.45	—	0.83	—	1.0
3	Y3	1059.1	263	0.249	1.00	—	1.0	—	—
	Y2		193	0.182	2.99	—	0.72	0.95	1.0
	Y1		436	0.412	1.22	—	0.83	1.0	—
2	Y3	1588.7	278	0.175	0.80	1.0	—	—	—
	Y2		217	0.136	2.59	0.51	—	0.95	1.0
	Y1		436	0.275	1.22	0.56	—	1.0	—
1	Y3	2118.2	282	0.133	0.80	1.0	—	—	—
	Y2		312	0.147	2.42	0.51	—	0.76	1.0
	Y1		504	0.238	1.04	0.81	—	1.0	—

Table 1.1.A-29 E_0 index

Story	$\frac{n+1}{n+i}$	Eq. (5): $(C_1+\sum\alpha_i\cdot C_i)\times F_i$ 1st group F_1	1st group C_1	2nd group α_2	2nd group C_2	3rd group α_3	3rd group C_3	E_{01}
4	0.625	1.0	0.450	0.83	0.632	0.72	0.400	0.79
		1.45	0.632	1.0	0.400	—	—	0.94
		3.32	0.400	—	—	—	—	0.83
3	0.714	1.0	0.249	0.83	0.412	0.72	0.182	0.52
		1.22	0.412	0.95	0.182	—	—	0.51
		2.99	0.182	—	—	—	—	0.39
2	0.833	0.8	0.175	0.56	0.275	0.51	0.136	0.27
		1.22	0.275	0.95	0.136	—	—	0.41
		2.59	0.136	—	—	—		0.29
1	1.0	0.8	0.133	0.81	0.238	0.51	0.147	0.32
		1.04	0.238	0.76	0.147	—	—	0.36
		2.42	0.147	—	—	—	—	0.36

Story	Group	Eq.(4): $\sqrt{(C_1\cdot F_1)^2+(C_2\cdot F_2)^2+(C_3\cdot F_3)^2}$ 1st group C_1	1st group F_1	2nd group C_2	2nd group F_2	3rd group C_3	3rd group F_3	E_{02}
4	2	0.450	1.0	0.632 0.400	1.45	—	—	0.98
	3	0.450	1.0	0.632	1.45	0.400	3.32	1.05
3	2	0.249	1.0	0.412 ***0.173**	1.22	—	—	0.54
	3	0.249	1.0	0.412	1.22	0.182	2.99	0.56
2	2	0.275	1.22	0.136	2.59	—	—	0.41
	3	—	—	—	—	—	—	—
1	2	0.238	1.04	0.147	2.42	—	—	0.43
	3	—	—	—	—	—	—	—

The effective strength factor is 0.95 in case that the members with *F* of 2.99 are included in the same group as the members with *F* of 1.22 according to Table 1.1.A-28. In that case, the *C* index for the members with *F* of 2.99 is calculated as 0.173 ($C=0.182\times0.95=0.173$).

5.7 Seismic index of structure I_S

The calculation procedure for seismic index of structure I_S is the same as that in the second level screening. The procedure is that the I_S is calculated using the maximum E_0 index in case that the calculated $C_{TU}\cdot S_D$ is greater than or equal to $0.3Z\cdot G\cdot U$ ($C_{TU}\cdot S_D\geq 0.3Z\cdot G\cdot U$), since each story is assumed to have no second-class prime elements. The irregularity index and time index are also assumed as 1.0.

(1) $C_{TU}\times S_D$ index

The calculation procedure for the $C_{TU}\times S_D$ is the same as that in the second level screening. The calculation results are listed in Table 1.1.A-30.

Table 1.1.A-30 Calculation for C_{TU}

Story	$\frac{n+1}{n+i}$	Max of *F*	C_{TU}	$C_{TU}\cdot S_D$	Result	E_0 Index Eq. (4)	E_0 Index Eq. (5)
4	0.625	3.32	0.250	0.250	NG	1.05	0.83
		1.45	0.575	0.575	OK	0.98	0.94
		1.0	0.746	0.746	OK	—	0.79
3	0.714	2.99	0.130	0.130	NG	0.56	0.39
		1.22	0.424	0.424	OK	0.54	0.51
		1.0	0.483	0.483	OK	—	0.52
2	0.833	2.59	0.113	0.113	NG	0.41	0.29
		1.22	0.342	0.342	OK	—	0.41
		0.8	0.320	0.320	OK	—	0.27
1	1.0	2.42	0.147	0.147	NG	0.43	0.36
		1.04	0.344	0.344	OK	—	0.36
		0.8	0.400	0.400	OK	—	0.32

From the results, the group of the maximum F index of each story cannot be applied due to the limitation of $C_{TU} \times S_D$.

(2) I_S index

The I_S index is calculated using the E_0 index in case that $C_{TU} \cdot S_D$ is greater than or equal to $0.3Z \cdot G \cdot U$. Here, the S_D and T indices are assumed to be 1.0. The I_S is calculated with the Eq. of $I_S = E_0 \cdot S_D \cdot T$.

--4th story

The E_0 index calculated with Eq. (4) using two groups of which the F indices are 1.0 and 1.45 is applied. Therefore,

$$E_0 = 0.98 \qquad I_S = 0.98 \times 1.0 \times 1.0 = 0.98$$

--3rd story

The E_0 index calculated with Eq. (4) using two groups of which the F indices are 1.0 and 1.22 is applied. Therefore,

$$E_0 = 0.54 \qquad I_S = 0.54 \times 1.0 \times 1.0 = 0.54$$

--2nd story

The E_0 index calculated with Eq. (5) using the F index of 1.22 is applied. Therefore,

$$E_0 = 0.41 \qquad I_S = 0.41 \times 1.0 \times 1.0 = 0.41$$

--1st story

The E_0 index calculated with Eq. (5) using the F index of 1.04 is applied. Therefore,

$$E_0 = 0.36 \qquad I_S = 0.36 \times 1.0 \times 1.0 = 0.36$$

6. Background Data

6.1 Relationship between the *F* index, ductility factor, and margin for shear failure of flexural column

The relationship among the *F* index, the ductility factor, and the strength margin for shear failure of flexural column, which are calculated according to the current and previous Standard for the sake of comparison, is shown in Fig. 1.1.A-23. The right side of the figure shows the relationship between the strength margin for shear failure and the ductility factor of the member. The left side of the figure shows the relationship between the ductility factor and the *F* index. The condition of member for the calculation is shown in the figure. As an example, the calculated *F* index of *Y2-X2* column on the 2^{nd} floor of the example building in this chapter (hoop spacing of 100mm) is also superimposed.

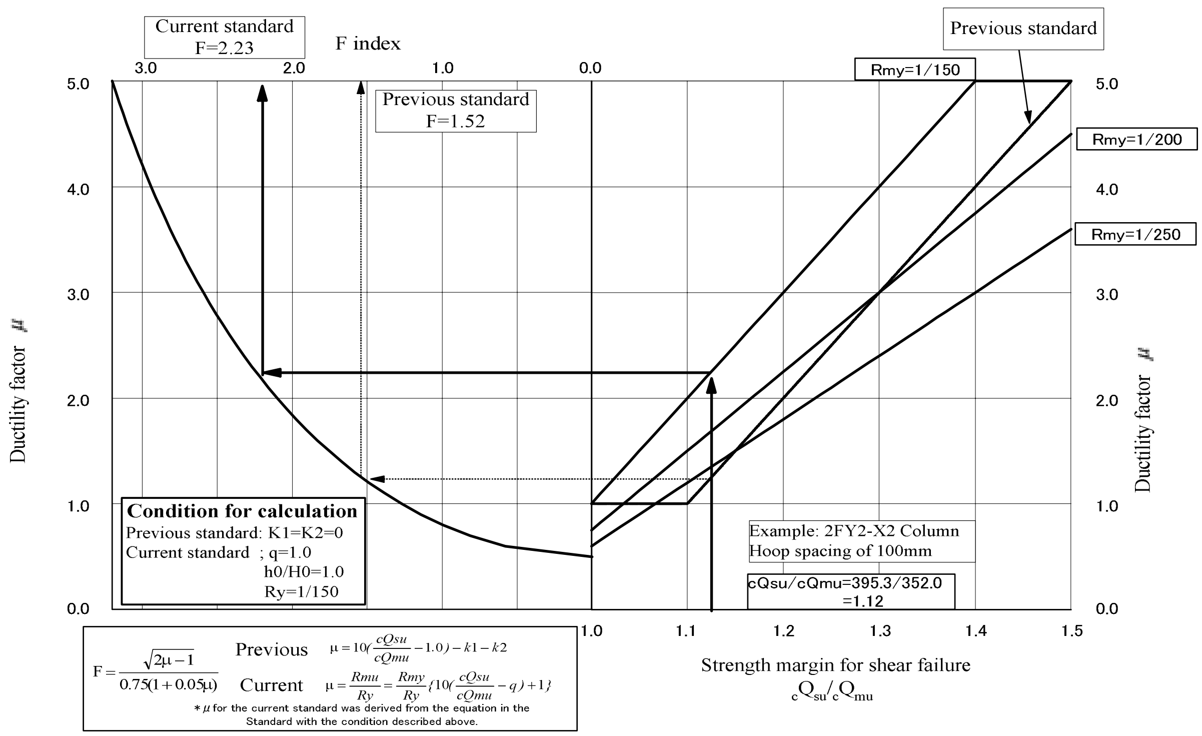


Fig. 1.1.A-23 Relationship between the *F* index, ductility factor, and strength margin for shear failure of flexural column

The current Standard takes the inter-story deflection angle at the flexural yielding of the column R_{my} into account for the calculation of the ductility factor, while the previous Standard does not take into. This is the difference between the Standards. On the condition for calculation, it can be seen in the figure that the ductility factor of flexural column with the R_{my} of 1/150 and the *F* index calculated by the current Standard are always greater than that calculated by the previous Standard. However, in accordance with the R_{my} getting smaller, the ductility factor calculated by the current Standard is preferably smaller than that by the previous Standard in some case. Moreover, since the ductility factor for flexural column μ is always greater or equal to 1.0 in the previous Standard, the *F* index for flexural column is always calculated as greater than or equal to 1.27, if certain conditions are satisfied. On the other hand, the *F* index for flexural column varies from 1.0 to 3.2 by the current Standard. Therefore the *F* index by the current Standard can be smaller than that by the previous Standard in the area of a small R_{my}, if the strength margin for shear failure is relatively small.

6.2 Scope of the shear column where its *F* is greater than 1.0

According to the current Standard, the *F* index for shear column can be also calculated based on the strength margin for shear failure and the aspect ratio of the member in the same manner as flexural column, and the maximum of the *F* index for shear column is defined as 1.27.

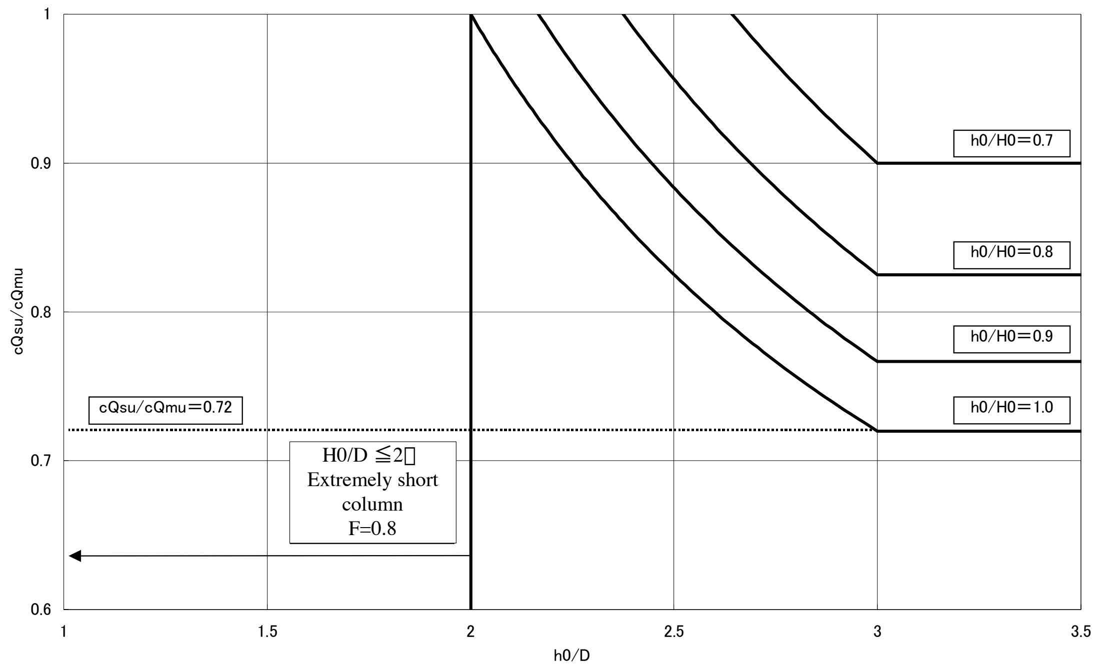


Fig. 1.1.A-24 Scope of the shear column where its *F* is greater than 1.0

The condition of the shear column with the *F* index of greater than 1.0 of is shown in Fig. 1.1.A-24. The axis of abscissa represents the h_0/D, and the axis of ordinate represents the strength margin for shear failure. The curves with different h_0/H_0 are shown in the figure. If the member has a strength margin for shear failure of which values are greater than that at the intersecting point of h_0/D and h_0/H_0 curve in the figure, the *F* index for the column is greater than 1.0. For example, if the column with h_0/D of 3.0 and h_0/H_0 of 1.0 has the strength margin for shear failure of 0.72, the *F* index for the shear column is calculated as greater than 1.0. On the other hand, if the column with h_0/D of 3.0 and h_0/H_0 of 0.7 has the strength margin for shear failure of less than 0.90, the calculated *F* index is less than 1.0. In addition, if h_0/H_0 is less than 0.6, the calculated *F* index is always less than 1.0 regardless of the h_0/D and the strength margin for shear failure.

6.3 E_0 index calculated with Eq. (4) in case of the structure with shear columns only

The E_0 index calculated with Eq. (4) of the current and previous Standards in case of the structure that has only shear columns is shown in Fig. 1.1.A-25. The axes of abscissa and ordinate represent the E_0 index for the first group ($= C_1 \times F_1$) and for the second group ($= C_2 \times F_2$), respectively.

Since the F index for the shear column is constantly 1.0 and is grouped into one group according to the previous Standard, the E_0 index (E_{0old}) by the previous Standard shows quadrant ($E_{0old} = \sqrt{(C_{1old} \times 1.0)^2}$).

The C and F indices for the first and second groups calculated by the current Standard are referred to as C_{1new}, F_{1new}, C_{2new}, F_{2new}, hereafter respectively. The E_0 indices calculated by the current and previous Standards are the same when the C_{2new} is equal to zero. With the equation of $C_{1new} + C_{2new} = C_{1old}$, the E_0 index (E_{0new}) by the current Standard can be calculated as a linear line of which y-intercept is $C_{1old} \times F_{2new}$ and x-intercept is $C_{1old} \times F_{1new}$.

It is obvious in the figure that the E_0 index calculated by the current Standard is less than that by the previous Standard generally, although the value by the current Standard becomes greater than that by the previous Standard, in case that $C_2 \times F_2$ is enough greater than $C_1 \times F_1$ ($C_2 \times F_2 >> C_1 \times F_1$). The E_0 index calculated by the current Standard can be $\sqrt{2}/2$ (0.7) times as much as the value by the previous Standard in case that F_2 is close to 1.0 and $C_2 \times F_2$ is nearly equal to $C_1 \times F_1$.

If Eq. (5) is applied to the calculation of the E_0 index, the E_0 index calculated by the current Standard can be less than that by the previous Standard, even if the value calculated with Eq. (4) of the current Standard is greater than that with Eq(4) of the previous Standard in case of $C_2 \times F_2 >> C_1 \times F_1$, since the C_2 index is multiplied by the effective strength factor α.

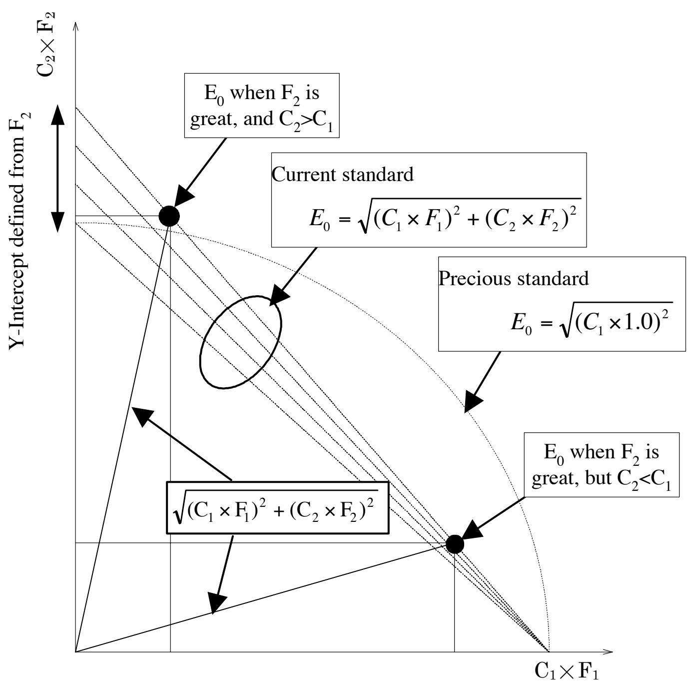


Fig. 1.1.A-25 *E₀* index in case of the structure with shear columns only

B Case Example-1 (School Building)

1. Outline of the structure

1.1 Usage and construction year

Usage : Elementary school building

Construction year : 1970

1.2 Floor area and structural type

Building area : 752.0 m^2

Total floor area : 2345.0 m^2

Classification of structure : Reinforced concrete frame structure

1.3 Material preservation

Design drawings, structural drawings, structural calculation, soil investigation report, all exist

1.4 Trouble record

None

1.5 Repair record

None

1.6 Others

Main frame in the *X* direction : frame structure with hanging and standing wall

Main frame in the *Y* direction : frame structure with continuous shear wall

(Some walls do not continue down to the ground, existing soft story column)

2. Evaluation of seismic capacity

2.1 Evaluation policy

(1) Standard reffered to :

Standard for seismic evaluation of existing reinforced concrete building, 2001

(2) The screening level : the second level screening

(3) Modeling in the evaluation

1. The sustained load is applied to the axial force in columns (varied axial force due to lateral force is also considered for soft story columns)
2. Ai distribution shape is applied to the lateral external force distribution shape in the longitudinal and transverse directions.
3. Yielding hinge of column is assumed to locate on the face of beam. If beam has hanging and/or standing wall, the location is assumed on the face of the wall.
4. The weight of pent house is added to the weight of the third floor.

(4) Basic seismic demand index I_{SO}

The I_{SO} index is calculated as follow.

$$I_{SO} = Es \times Z \times G \times U = 0.7 \times 1.0 \times 1.0 \times 1.0 = 0.7$$

2.2 Floor area, weight, and material properties

(1) Floor area and weight

Table 1.1.B-1

Story	Floor area A_f(m^2)	Total floor area $\sum A_f$ (m^2)	Weight W(kN)	Total weight $\sum W$ (kN)	Unit weight (kN/m^2) w ($\sum W/\sum A_f$)
PH	63.50	63.50	1281	1281	20.2
3	775.02	838.52	10582	11863	13.6 (14.1)
2	775.02	1613.54	10421	22285	13.4 (13.8)
1	731.18	2344.72	11904	33475	15.3 (14.3)

(2) Material strength

--Concrete

Design strength at original : F_c=17.7 N/mm^2

Material test results : F_c =16.1 N/mm^2 (Minimum)

Applied strength for the evaluation : F_c =15.7 N/mm^2

--Steel

Main bar (SD30) : $\sigma = 343$ N/mm^2

Hoop (SD24) : $\sigma = 294$ N/mm^2

Reinforcing bar in wall (SD24) : $\sigma = 294$ N/mm^2

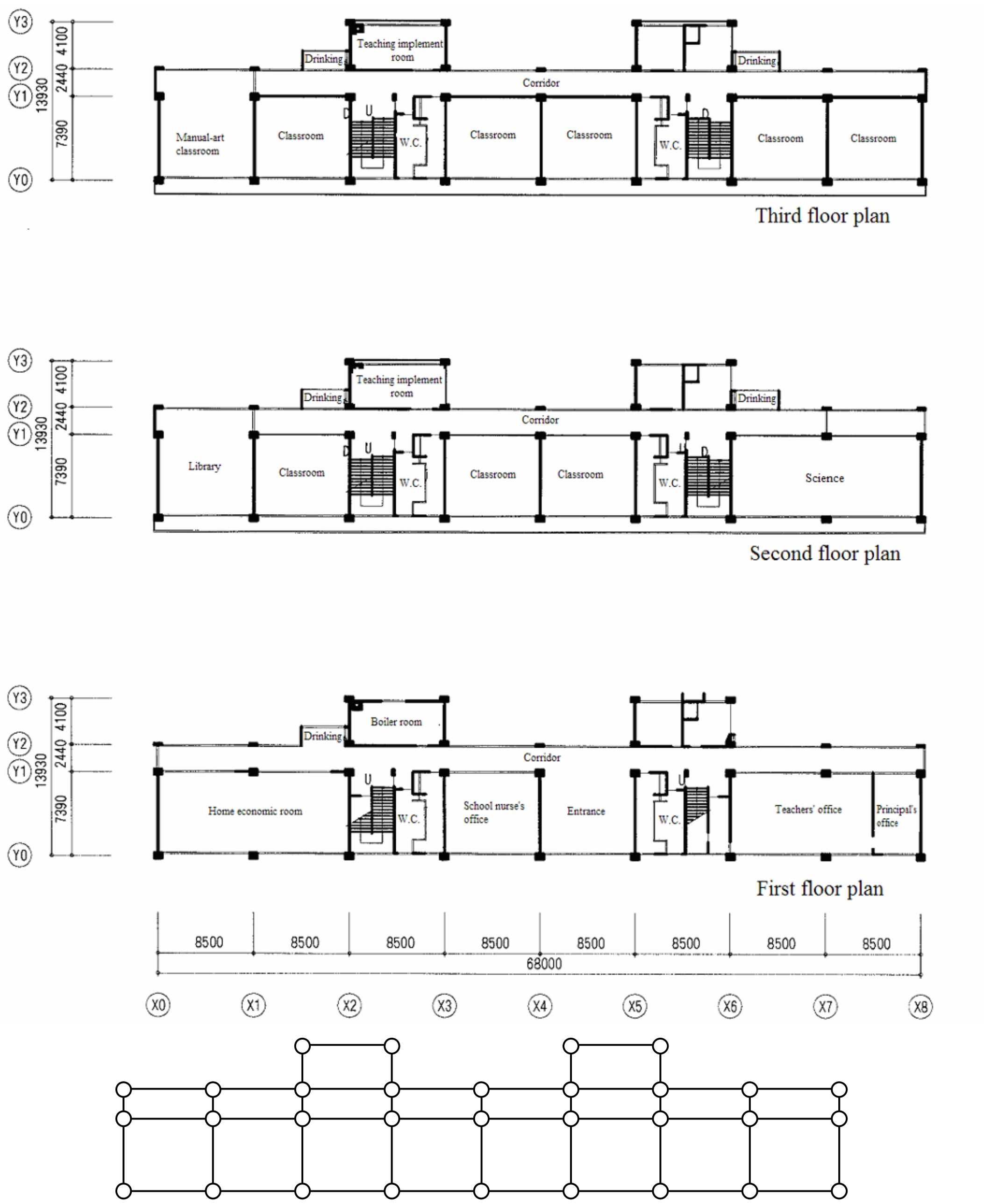
2.3 Outline of site investigation

Table 1.1.B-2 Concrete strength from material test (core sampling)

Story	Compressive strength (N/mm^2)	Average $\overline{X}$ (N/mm^2)	Standard deviateon σ (N/ mm^2)	Compressive strength σ_B(N/ mm^2)	Applied strength (N/ mm^2)	Carbonation depth (cm)
3	17.2 16.2 19.5	17.6	1.7	16.8	15.7	0.20 1.00 1.80
2	17.9 15.7 16.5	16.7	1.2	16.1	15.7	0.40 2.50 2.00
1	19.0 17.8 17.2	18.0	1.0	17.5	15.7	0.10 0.00 0.00

Average : $\overline{X}$, Standard deviation : σ , Compression strength : $\sigma_B = \overline{X} - \sigma/2$

2.4 Existing plan drawings

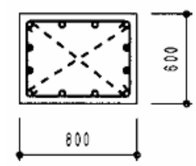
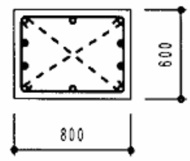
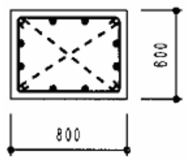
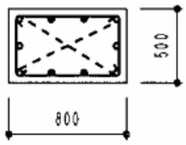
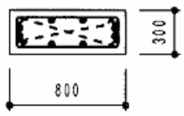
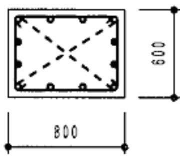
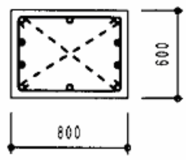
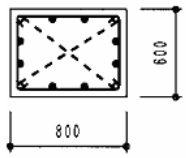
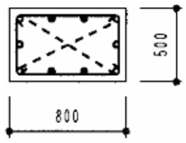
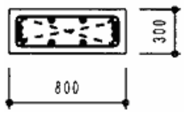
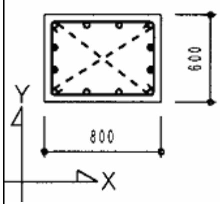
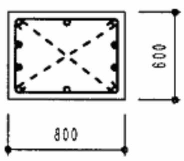
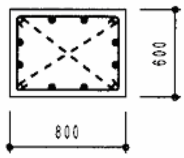
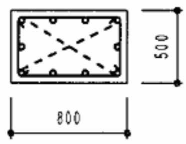
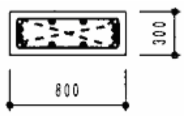


First floor columns key-plan

Fig. 1.1.B-1 Plan view and first floor columns key-plan

2.5 Column and wall lists

Column list

Remark	C21	C22	C23	C29	C27
3^{rd} floor					
$b \times D$	600×800	600×800	600×800	500×800	300×800
Main bar	8-D25+4-D19	6-D25+4-D19	4-D25+8-D19	6-D25+4-D19	4-D25+8-D19
Hoop	$9\phi - @200$	$9\phi - @200$	$9\phi - @200$	$9\phi - @200$	$9\phi - @200$
Remark	C11	C12	C13	C19	C17
2^{nd} floor					
$b \times D$	600×800	600×800	600×800	500×800	300×800
Main bar	8-D25+4-D19	6-D25+4-D19	4-D25+8-D19	6-D25+4-D19	4-D25+8-D19
Hoop	$9\phi - @200$	$9\phi - @200$	$9\phi - @200$	$9\phi - @200$	$9\phi - @200$
Remark	C1	C2	C3	C9	C7
1^{st} floor					
$b \times D$	600×800	600×800	600×800	500×800	300×800
Main bar	8-D25+4-D19	6-D25+4-D19	4-D25+8-D19	10-D25	4-D25+8-D19
Hoop	$9\phi - @200$	$9\phi - @200$	$9\phi - @200$	$9\phi - @200$	$9\phi - @200$

Wall list

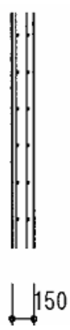
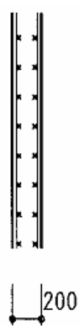
	W12	W15	W20
Dimension			
Vertical reinforcement	$9\phi - @200$	$2-9\phi - @200$	$2-13\phi - @200$
Horizontal reinforcement	$9\phi - @200$	$2-9\phi - @200$	$2-13\phi - @200$
Reinforcement at ends	$1-13\phi$	$2-13\phi$	$2-16\phi$
Reinforcement around opening	$1-13\phi$	$2-13\phi$	$2-16\phi$

Fig. 1.1.B-2 Column and wall lists

2.6 Framing elevation

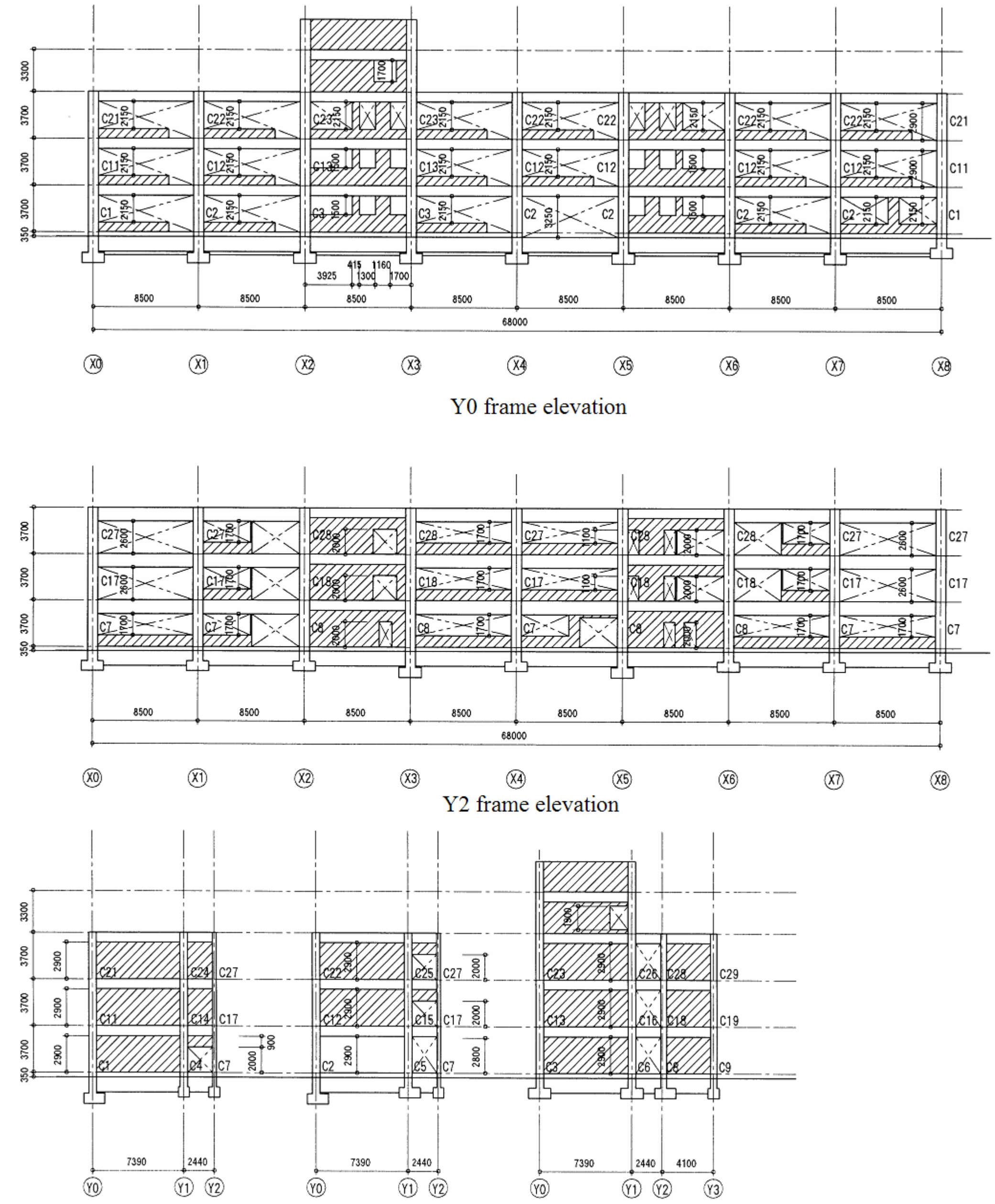


Fig. 1.1.B-3 Framing elevations

2.7 Irregularity index and time index

Table 1.1.B-3 Irregularity index S_D

			Gi(Grade)				
			1.0	0.9	0.8	*R1i*	*R2i*
Horizontal balance	a	Regularity	Regular a1	Nearly regular a2	Irregular a3	1.0	0.5
	b	Aspect ratio	b≦5	5＜b≦8	8＜b	0.5	0.25
	c	Constriction	0.8≦c	0.5≦c＜0.8	c＜0.5	0.5	0.25
	d	Expansion joint	1/100≦d	1/200≦d＜ 1/100	D＜1/200	0.5	0.25
	e	Volt	e≦0.1	5＜e≦8	0.3＜e	0.5	0.25
	f	Eccentric volt	f1≦0.4 & f2≦0.1	f1≦0.4 & 0.1＜f2≦0.3	0.4＜f1 or 0.3＜f2	0.25	0
	g						
vertical balance	h	Underground floor	1.0≦h	0.5≦h＜１.0	h＜0.5	0.5	0.5
	i	Story height regularity	0.8≦I	0.7≦I＜0.8	i＜0.7	0.5	0.25
	j	Soft story	No soft story	Soft story	Eccentric soft story	1.0	1.0
	k						
Eccentricity	l	Eccentricity (X direction)	1≦0.1	0.1＜ l ≦0.15	0.15＜l		1.0
	m	Eccentricity (Y direction)	1≦0.1	0.1＜ l ≦0.15	0.15＜l		1.0
Stiffness	n	(Stiffness/mass)Ratio (X direction)	n≦1.3	1.3＜n≦1.7	1.7＜n		1.0
	o	(Stiffness/mass)Ratio (Y direction)	n≦1.3	1.3＜n≦1.7	1.7＜n		1.0

$$
\begin{aligned}
S_{D2} = &\left[\{1-(1-1)\times 0.5\}\right]\times\left[\{1-(1-0.9)\times 0.25\}\right]\times\left[\{1-(1-1)\times 0.25\}\right]\times\left[\{1-(1-0.9)\times 0.25\}\right]\\
&\times\left[\{1-(1-1)\times 0.25\}\right]\times\left[\{1-(1-1)\times 0.0\}\right]\times\left[\{1.1-(1-0.8)\times 0.5\}\right]\times\left[\{1-(1-1)\times 0.25\}\right]\\
&\times\left[\{1-(1-1)\times 1.0\}\right]\times\left[\{1-(1-1)\times 1.0\}\right]\times\left[\{1-(1-1)\times 1.0\}\right]=0.95
\end{aligned}
$$

Time index (*T*): Since the calculation procedure has not been revised, the table to calculate the index is ignored. *T*=0.93 (see translators' note 2 shown below).

Translators' Note 2 --

Since this is the first English version of the current Standard, the table to calculate time index is shown below.

Item / Degree / Portion / Range		Structural cracking and deflection			Deterioration and aging		
		a	b	c	a	b	c
Portion	Range	1. Cracking caused by uneven settlement. 2. Shear or inclined cracking in beams, walls, and columns, observed evidently.	1. Deflection of a slab or beam, affecting on the function of non-structural element. 2. Same as left but not observed from some distance. 3. Same as above but can be observed from some distance.	1. Minute structural cracking not corresponding to the items a or b. 2. Deflection of a slab or beam, not corresponding to the item a or b.	1. Cracking by concrete expansion due to the rust of reinforcing bar. 2. Rust of reinforcing bar. 3. Cracking caused by a fire disaster. 4. Deterioration of concrete caused by chemicals.	1. Seep of the rust of reinforcing bar due to rain water or water leak. 2. Neutratization to the depth of reinforcing bar or equivalent aging. 3. Spalling off of finishing materials.	1. Blemish of concrete due to rain water, water leak, and chemicals. 2. Deterioration or slight spalling off of a finishing material.
I Slab including sub-beam	1) 1/3 or more of total area of floor slab.	0.017	0.005	0.001	0.017	0.005	0.001
	2) 1/3~1/9	0.006	0.002	0	0.006	0.002	0
	3) 1/9 or less	$\boxed{0.002}$	$\boxed{0.001}$	$\boxed{0}$	$\boxed{0.002}$	$\boxed{0.001}$	$\boxed{0}$
	4) 0 remark)	0	0	0	0	0	0
II Beam	1) 1/3 or more of total members in the each evaluating direction	0.05	0.015	0.004	0.05	0.015	0.004

	2) 1/3~1/9	0.017	0.005	0.001	0.017	0.005	0.001
	3) 1/9 or less	$\boxed{0.006}$	$\boxed{0.002}$	$\boxed{0}$	$\boxed{0.006}$	$\boxed{0.002}$	$\boxed{0}$
	4) 0 remark)	0	0	0	0	0	0
III Wall & Column	1) 1/3 or more of total number of members	0.15	0.045	0.011	0.15	0.045	0.011
	2) 1/3~1/9	0.05	0.015	0.004	0.05	0.015	0.004
	3) 1/9 or less	0.017	0.005	0.001	0.017	0.005	0.001
	4) 0 remark)	$\boxed{0}$	$\boxed{0}$	$\boxed{0}$	$\boxed{0}$	$\boxed{0}$	$\boxed{0}$
Subtracted points	Subtotal	0.025	0.008	0.001	0.025	0008	0.001
Total	Ground total	$p_1 =$ 0.034			$p_2 =$ 0.034		
Time index for the 1st to 3rd story : $Ti = (1 - p_1) \times (1 - p_2) = (1\text{-}0.034) \times (1\text{-}0.034) = 0.933 \rightarrow 0.93$							

-- **End of Translators' Note 2**

2.8 Second-class prime element evaluation

The evaluation results in the *X* direction are shown here.

(1) Since all extremely brittle columns have shear wall in orthogonal direction, they are not second-class prime elements, whose failure leads to collapse.

(2) Evaluation of the column with the *F* index of 1.0 (Shear and flexural column)

a) Evaluation of the first floor in the *X* direction is carried out (other floors are ignored).

b) The residual axial strength of the second-class prime elements is not evaluated here, although they needs to be evaluated separately.

Table 1.1.B-4

Direction	Story	Location	Evaluation condition	Second-class prime element
X	1	X0-Y0	Structural wall in the orthogonal direction	○
		X1-Y0	Based on local circumstances	×
		X7-Y0	Based on local circumstances	×
		X1-Y1	Based on local circumstances	×
		X2-Y1	Structural wall in the orthogonal direction	○
		X3-Y1	Structural wall in the orthogonal direction	○
		X5-Y1	Structural wall in the orthogonal direction	○
		X0-Y2	Vertical load can be carried to surrounding members by beams or walls	○
		X1-Y2	Vertical load cannot be carried to surrounding members by beams or walls	×
		X4-Y2	From the calculation shown below	×
		X7-Y2	Vertical load cannot be carried to surrounding members by beams or walls	×
		X8-Y2	Vertical load cannot be carried to surrounding members by beams or walls	×
		X5-Y3	Structural wall in the orthogonal direction	○
		X6-Y3	Vertical load can be carried to surrounding members by beams or walls	○

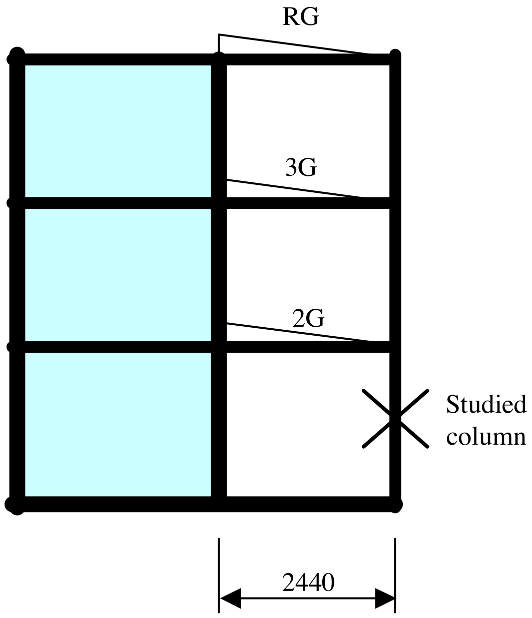
(3) Calculation example of the second-class prime element (load redistribution)

Studied column : (X4,Y2) on the 1st floor

a) Appropriate model for the load redistribution is assumed.

b) The beams in the corridor side is studied as cantilever, since the sectional shape of the outer most column in the corridor side is planular and its ultimate flexural strength is small.

c) The effect of slab reinforcing bar is ignored to make the calculation simple.



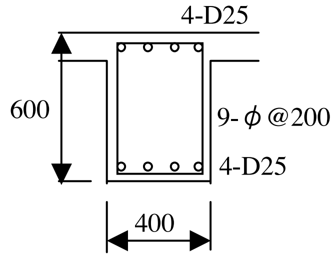


Fig. 1.1.B-4

$$N_1 = 743kN$$

$$at = 4 \times 5.07 = 20.28cm^2$$

$$P_t = 0.92\%$$

$$P_w = 0.0016$$

$$M/(Q \cdot d) = (2 \times 244)/(2 \times 55) = 4.44$$

Ultimate shear strength of each beam;

$$_RQ_{mu} = 0.9 \times 2028 \times 343 \times 550 / 2440 \times 10^{-3} = 141.1kN$$

$$_RQ_{su} = \left\{ \frac{0.053 \times 0.92^{0.23}(18+15.7)}{3.00+0.12} + 0.85\sqrt{0.0016 \times 294} \right\} \times 400 \times 480 \times 10^{-3} = 219.6kN$$

$$_3Q_{mu} = 141.1kN \qquad _3Q_{su} = 219.6kN \qquad _2Q_{mu} = 141.1kN \qquad _2Q_{su} = 219.6kN$$

$$\sum_2^R Q = 141.1 + 141.1 + 141.1 = 423.3kN \quad < \quad N_1 = 743kN \qquad \text{NG}$$

Therefore, this column is classified to the second-class prime element.

2.9 Study on the soft story column

The *X1* frame on the 1st floor is studied since it has the soft story columns.

(1) Result of the second level screening

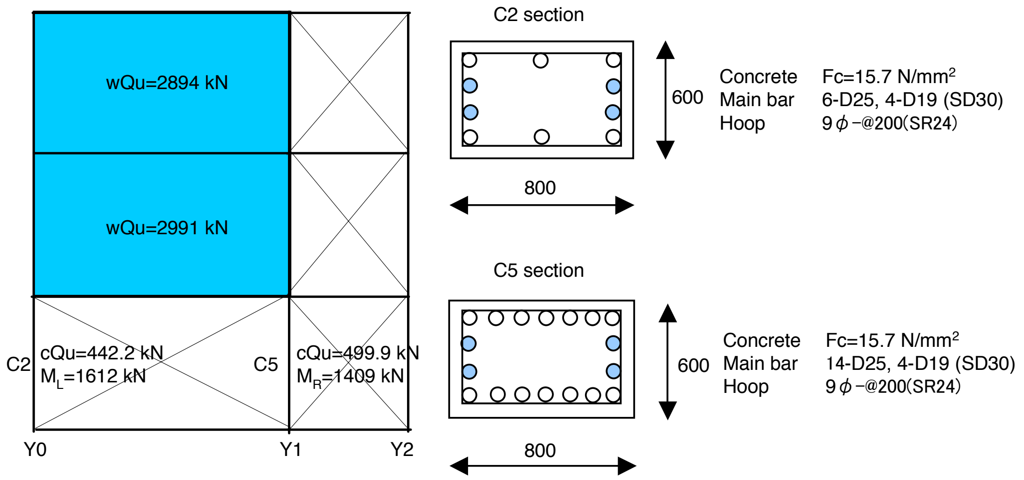


Fig. 1.1.B-5

(2) Calculation of the axial force

Axial forces when the shear walls in the upper stories fails and totally overturned failure occurs in the soft story, are evaluated.

(i) When the shear walls in the upper stories fail in shear simultaneously

(The external force distribution should be assumed appropriately. Here, the lateral load-carrying capacities at second and third stories calculated for the second level screening are used for the distribution of story shear forces.)

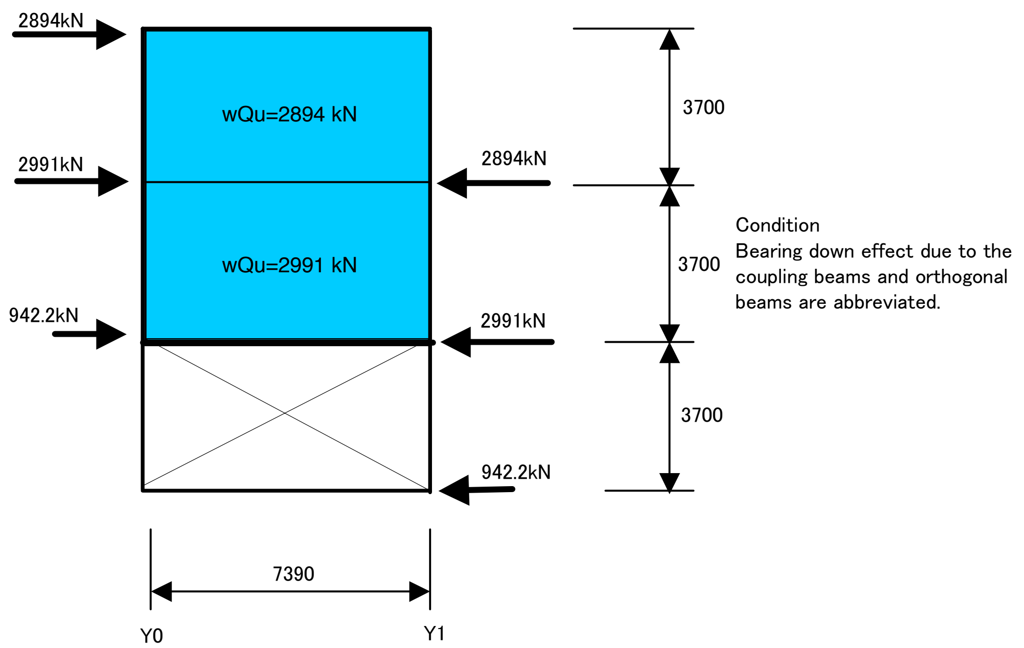


Fig. 1.1.B-6

The axial force acting in *Y0* column on the 1st floor;

$$N_{Y0} = \{2894\times11.1+(2991-2894)\times7.4-(2894-942.2)\times3.7\}/7.39+1612 = 3467+1612$$
$$= 5079kN$$

The axial force acting in *Y1* column on the 1st floor;

$$N_{Y1} = \{2894\times11.1+(2991-2894)\times7.4-(2894-942.2)\times3.7\}/7.39+1409 = 3467+1409$$
$$= 4876kN$$

(ii) When the tensile column yield in the axial direction

$$N = a_g \cdot \sigma_y + ({}_1N_L + {}_1N_R)$$
$$N_{Y0} = (14\times507+4\times285)\times343\times10^{-3}+(1612+1409) = 5846kN$$
$$N_{Y1} = (6\times507+4\times285)\times343\times10^{-3}+(1612+1409) = 4455kN$$

(iii) Demand axial force for the soft story column

The smaller force calculated in (i) and (ii) is applied for the demand axial force.

$$N_{Y0} = 5079kN$$
$$N_{Y1} = 4455kN$$

(3) Study on the second-class prime element (*Y0* column on the 1st floor is studied. *Y1* column is not studied since it has orthogonal shear wall)

(a) Failure mode of the column under the axial force of N_S

$$0.4b\cdot D\cdot F_c = 0.4\times600\times800\times15.7\times10^{-3} = 3014kN$$
$$N_{max} = 600\times800\times15.7\times10^{-3}+(6\times507+4\times285)\times343\times10^{-3} = 8970kN$$
$$M_u = \{0.8a_t\cdot\sigma_y\cdot D+0.12b\cdot D^2\cdot F_c\}\left(\frac{N_{max}-N}{N_{max}-0.4bDF_c}\right)$$
$$M_u = \{0.8\times3\times507\times343\times600\times10^{-6}+0.12\times800\times600^2\times15.7\times10^{-6}\}\times\left(\frac{8970-5079}{8970-3014}\right)$$
$$= 163.4+360.9 = 524.3kN\cdot m$$
$$Q_{mu} = 2M_u/h_0 = 2\times524.3/2.9 = 361.6kN$$
$$Q_{su} = \left\langle\frac{0.053p_t^{0.23}(18+F_c)}{M/(Q\cdot d)+0.12}+0.85\sqrt{P_w\cdot{}_s\sigma_{wy}}+0.1\sigma_0\right\rangle\cdot b\cdot j$$
$$= \left\langle\frac{0.053\times0.317^{0.23}(18+15.7)}{2.64+0.12}+0.85\sqrt{0.0008\times294}+0.1\times8.0\right\rangle\times800\times480\times10^{-3}$$
$$= 656.3kN$$

The column is the flexural column, since $Q_{mu} = 361.6kN < Q_{su} = 656.3kN$

(b) Failure mode of the column under the balanced axial force (*N=0.4bDF*$_c$)

$$M_u = 0.8a_t \cdot \sigma_y D + 0.5N \cdot D\left(1 - \frac{N}{bDF_c}\right)$$

$$= 0.8 \times 3 \times 507 \times 343 \times 600 \times 10^{-6} + 0.5 \times 3014000 \times 600 \times (1 - 0.4) \times 10^{-6}$$

$$= 250.4 + 542.5 = 792.9kN \cdot m$$

$$Q_{mu} = 2M_u / h_0 = 2 \times 792.9 / 2.9 = 546.8kN$$

$$Q_{su} = \left\langle \frac{0.053 p_t^{0.23} (18 + F_c)}{M/(Q \cdot d) + 0.12} + 0.85\sqrt{P_w \cdot {}_s\sigma_{wy}} + 0.1\sigma_0 \right\rangle \cdot b \cdot j$$

$$= \left\langle \frac{0.053 \times 0.317^{0.23} (18 + 15.7)}{2.64 + 0.12} + 0.85\sqrt{0.00106 \times 343} + 0.1 \times 6.3 \right\rangle \times 800 \times 480 \times 10^{-3}$$

$$= 591.0kN$$

The column is the flexural column, since $Q_{mu} = 546.8kN < Q_{su} = 591.0kN$

(c) Study on the compressive axial force ratio η

$$\eta_{max} = N_s / (bDF_c) = 5079 / (800 \times 600 \times 15.7 \times 10^{-3}) = 0.674 > \eta_u = 0.4(Hoop\square 9\phi\ @\ 200)$$

Although the column is the flexural column, it is classified into the second-class prime element, since the compressive axial force ratio is greater than 0.4.

(4) Re-evaluation of the I_S index

(The "re-evaluation" means not to adjust the I_S for the whole structure but to judge if the soft story column needs to be strengthened or not.)

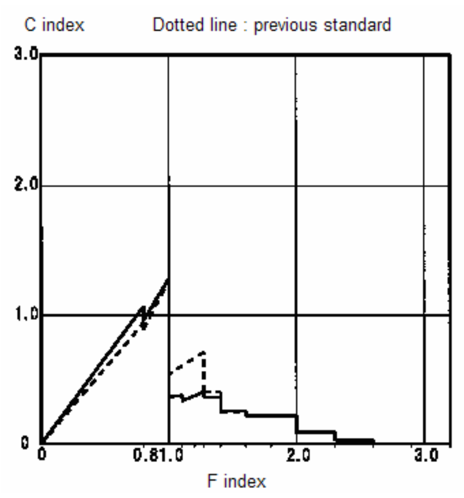
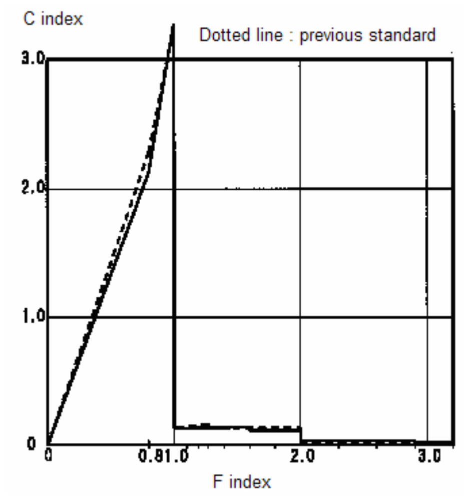
$$\eta_{max} = 0.676$$

$$\eta_u = 0.4$$

$$I_{S-re} = 0.96 \times \left(\frac{0.4}{0.676}\right)^2 = 0.34 < I_{SO}$$

Therefore, the soft story column needs to be strengthened.

2.10 *C-F* relationships

<table>
<tr><th></th><th>X direction</th><th>Y direction</th></tr>
<tr><td>3rd story</td><td>
</td><td>
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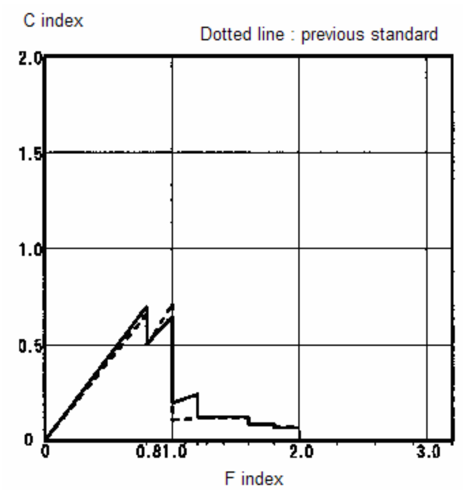
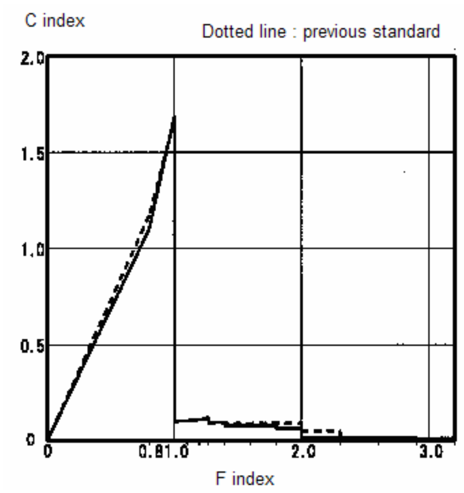
<tr><td>2nd story</td><td>
</td><td>
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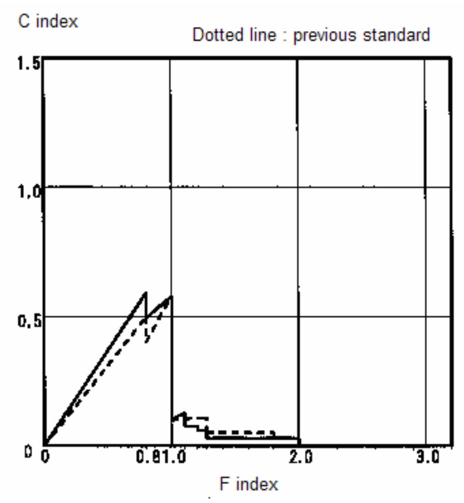
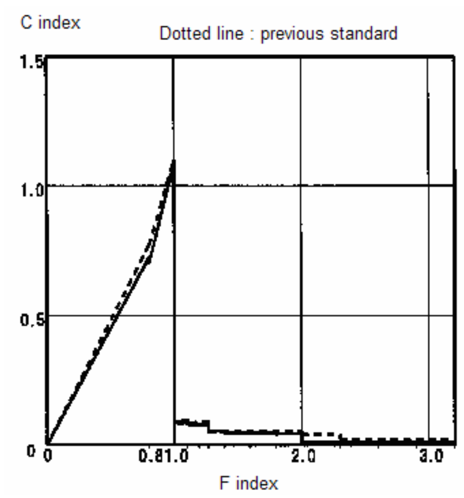
<tr><td>1st story</td><td>
</td><td>
</td></tr>
</table>

Fig. 1.1.B-7 *C-F* relationships

2.11 Failure mode

The original values of Q are calculated in terms of the units of gravitational system. Then the calculated values are multiplied by 10 to change the unit to SI. Therefore, it can be 2% greater than the accurate value. The calculated value is almost the same as the results with the material properties of F_c of 16 N/mm^2, main bar of 350 N/mm^2, and hoop and reinforcing bar in walls of 300 N/mm^2. The values of F indices are independent of the revision of the unit and Standard version.

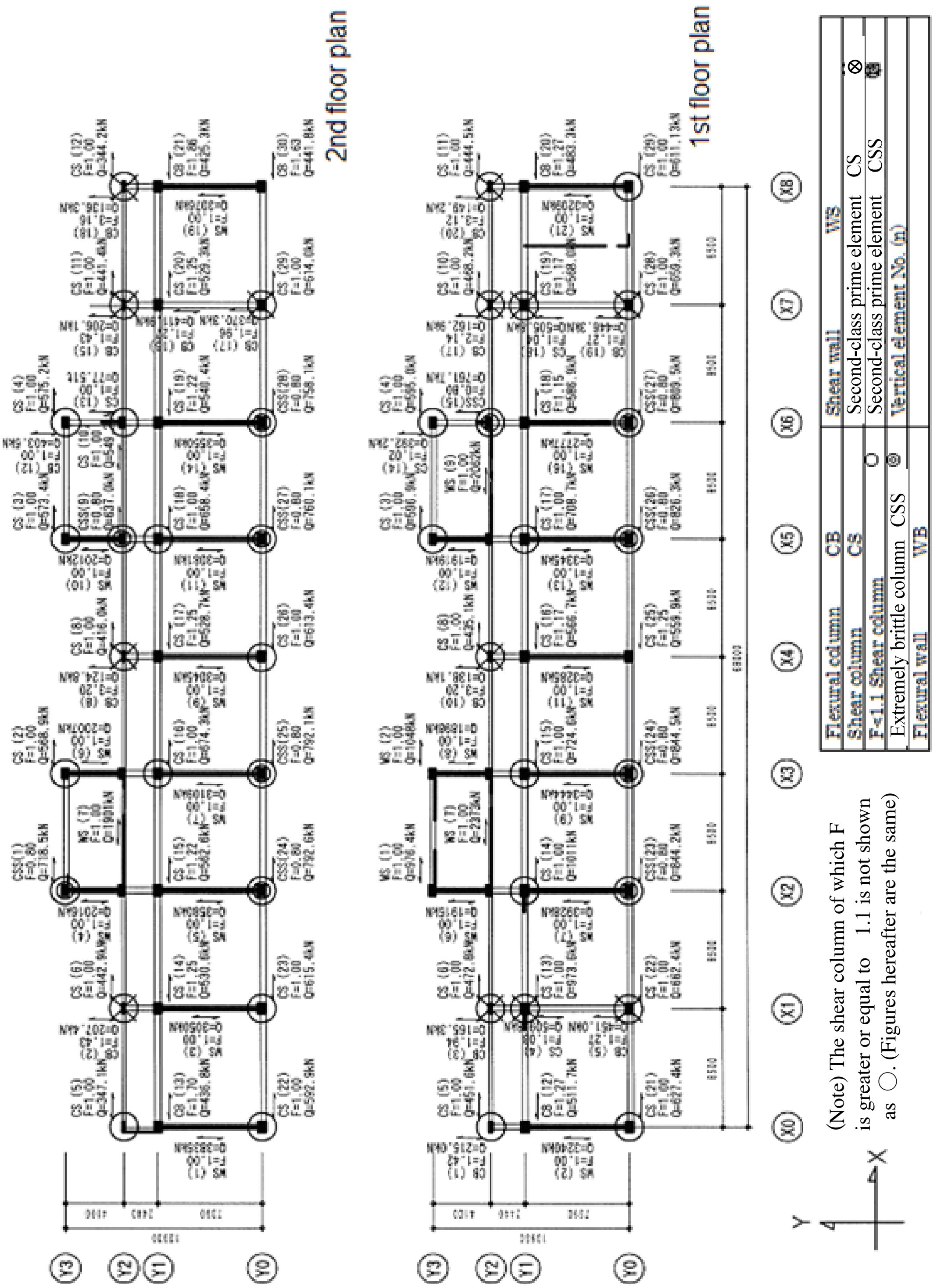


Fig. 1.1.B-8 Failure mode (current Standard)

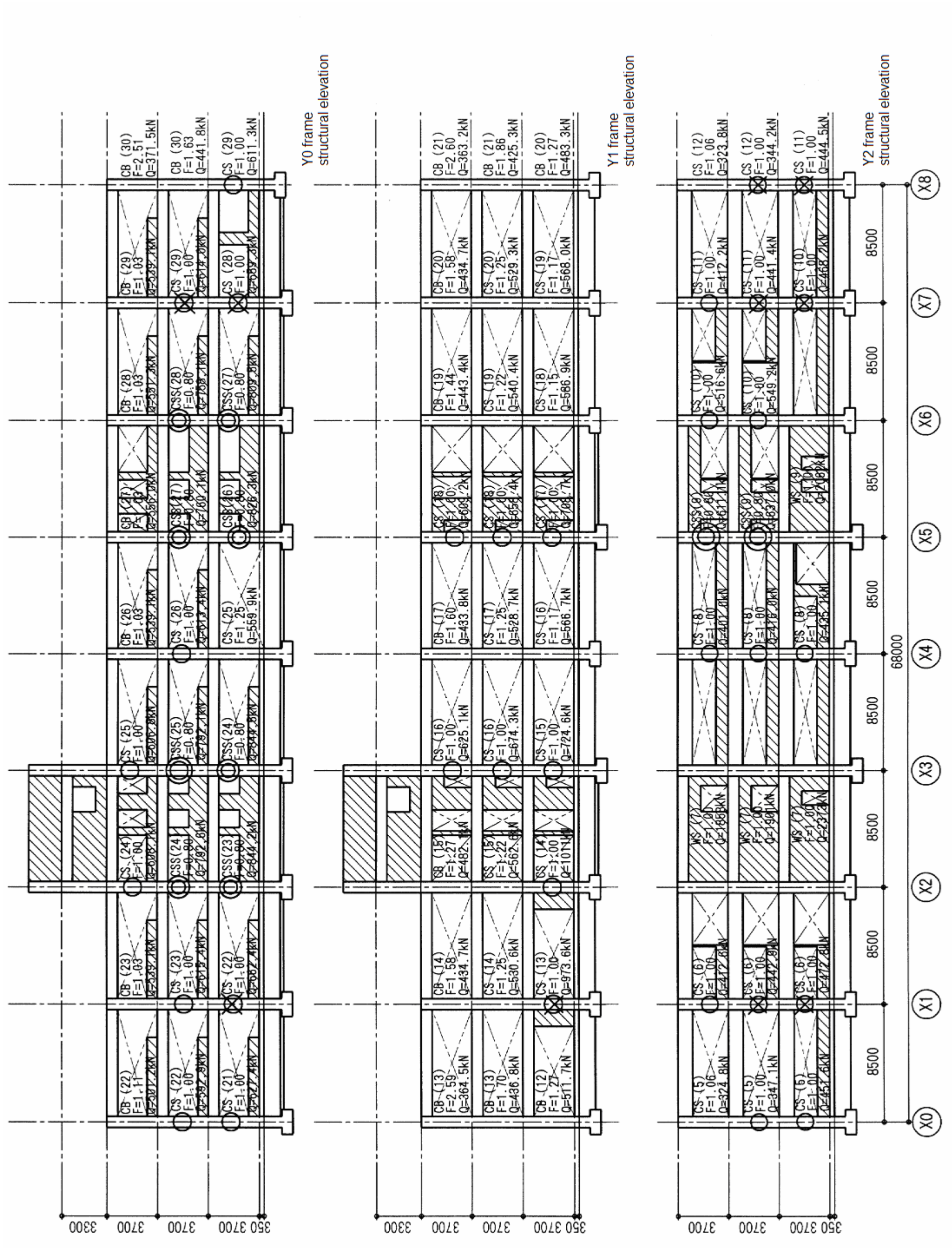


Fig. 1.1.B-9 Failure mode (current Standard)

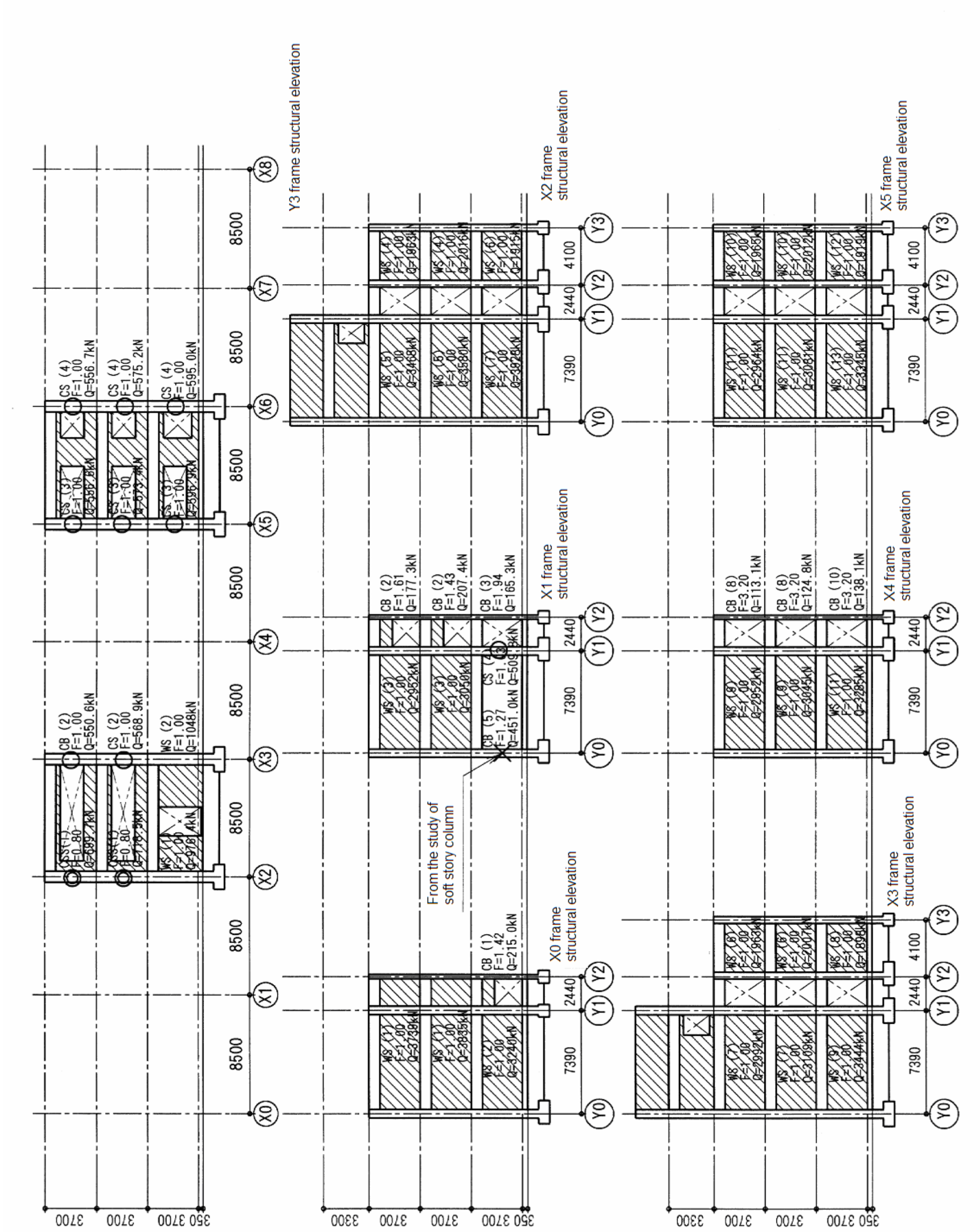


Fig. 1.1.B-10 Failure mode (current Standard)

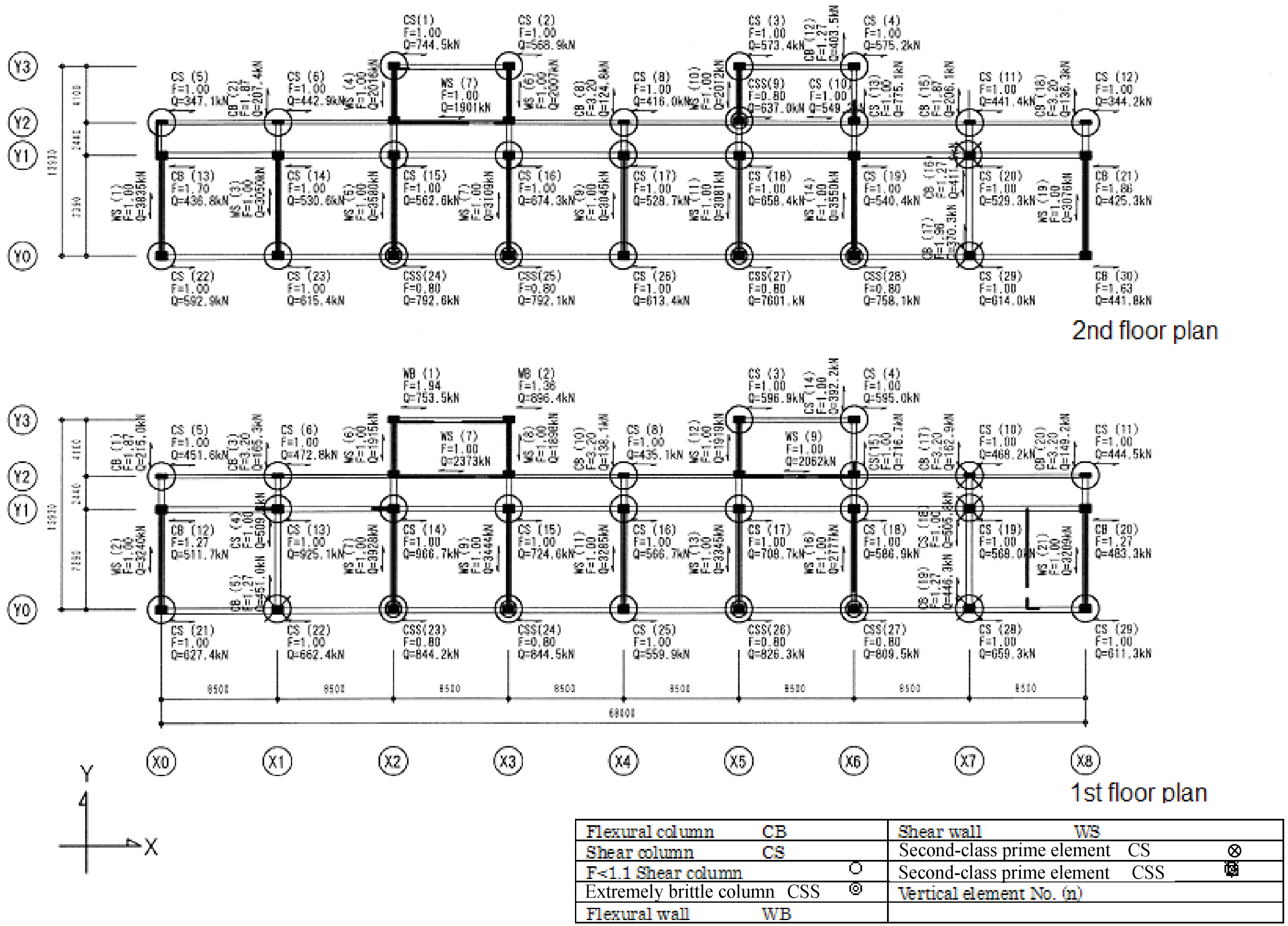


Fig. 1.1.B-11 Failure mode (previous Standard)

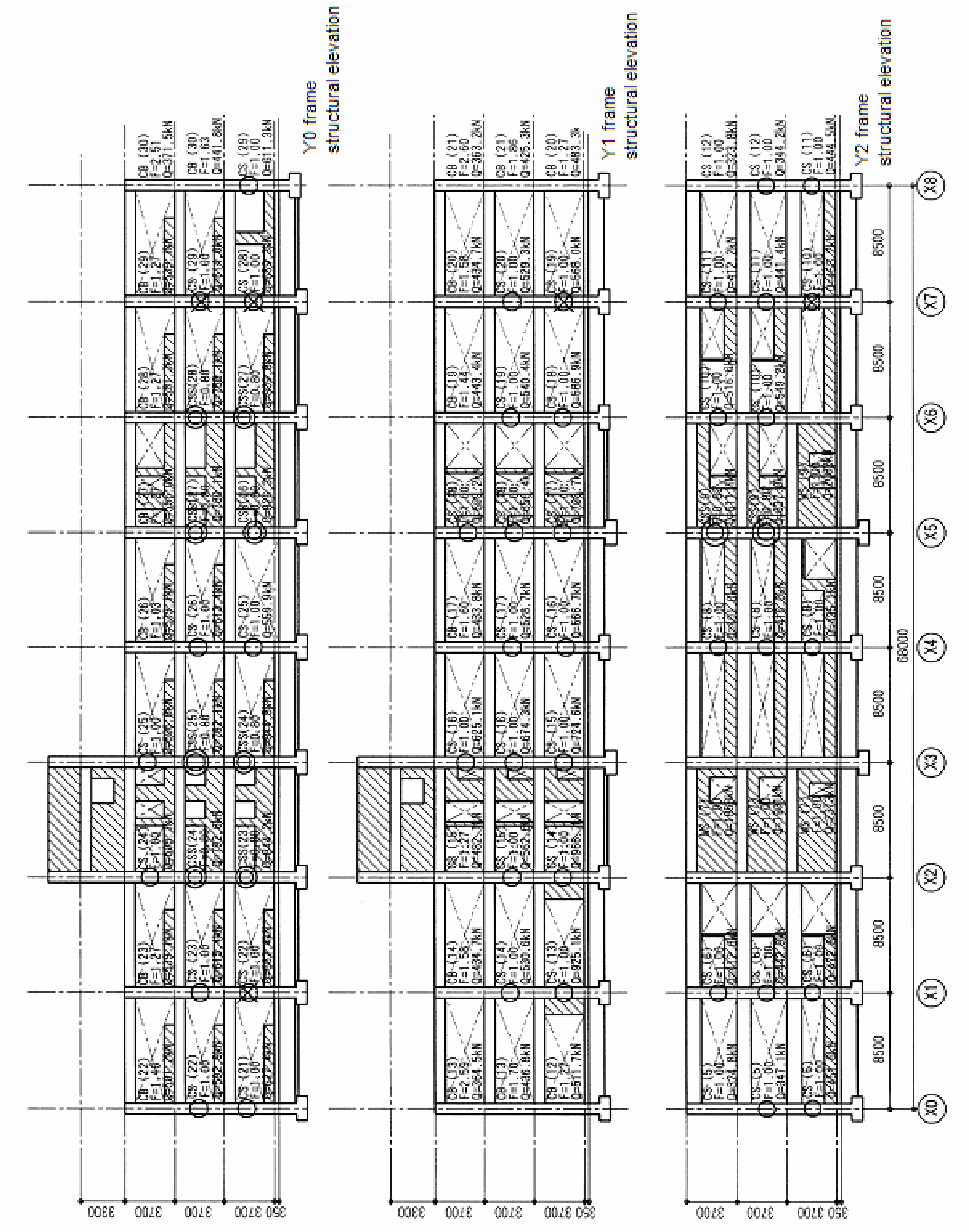


Fig. 1.1.B-12 Failure mode (previous Standard)

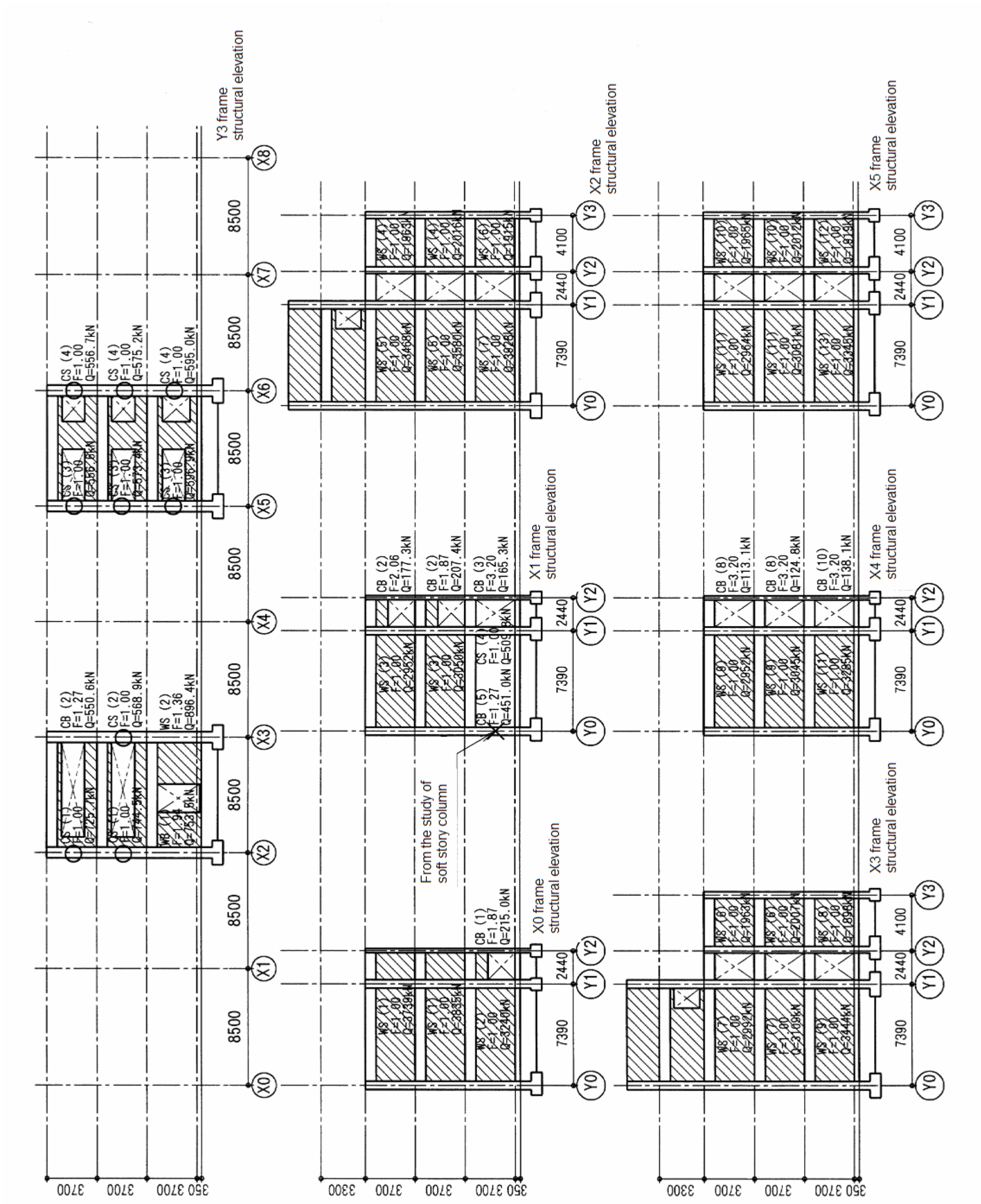


Fig. 1.1.B-13 Failure mode (previous Standard)

2.12 Seismic capacity evaluation result sheet

Table 1.1.B-5 Seismic capacity evaluation result (current Standard)

Results of the second level screening									
Name of Build.(××× Elementary school) Construction year (1970) Address (××Prefecture ×× city ××)									
Evaluated Engineer (××× Structural design office ××) Date of evaluation (Year ××/××/××)									
Seismic demand index Iso = Es x Z x G x U = 0.70									
Direction	Story	C	F	E0	SD	T	IS	CT x SD	Result
X	3	0.11 1.29	0.80 1.00	(0.63) [$\boxed{0.95}$]	0.95	0.93	(0.56) [$\boxed{0.84}$]	(0.75) [$\boxed{0.90}$]	O K
	2	0.20 0.45 0.20	0.80 1.00 1.20	(0.49) [$\boxed{0.56}$]	0.95	0.93	(0.43) [$\boxed{0.49}$]	(0.69) [$\boxed{0.56}$]	N G
	1	0.10 0.47 0.13	0.80 1.00 1.10	(0.48) [$\boxed{0.58}$]	0.95	0.93	(0.34) [$\boxed{0.51}$]	(0.45) [$\boxed{0.55}$]	N G

( ): Index considering the extremely brittle columns
Ai distribution shape is used for lateral external force distribution shape
[ ]: Index considering the members with F of 1.0 (using Eq. (5))

Results of the second level screening									
Name of Build.(××× Elementary school) Construction year (1970) Address (××Prefecture ×× city ××)									
Evaluated Engineer (××× Structural design office ××) Date of evaluation (Year ××/××/××)									
Seismic demand index Iso = Es x Z x G x U = 0.70									
Direction	Story	C	F	E0	SD	T	IS	CT x SD	Result
Y	3	3.28	1.00	[$\boxed{2.42}$]	0.95	0.93	[$\boxed{2.14}$]	[$\boxed{2.30}$]	O K
	2	1.69	1.00	[$\boxed{1.47}$]	0.95	0.93	[$\boxed{1.30}$]	[$\boxed{1.39}$]	O K
	1	0.02 1.08	0.80 1.00	(0.82) [$\boxed{1.08}$]	0.95	0.93	0.34* (0.58) [$\boxed{0.96}$]	(0.97) [$\boxed{1.03}$]	N G O K

( ): Index considering the extremely brittle columns *Considering the soft story column
[ ]: Index considering the members with F of 1.0 (using Eq. (5))

Table 1.1.B-6 Seismic capacity evaluation result (previous Standard)

Results of the second level screening

Name of Build.(××× Elementary school) Construction year (1970) Address (××Prefecture ×× city ××)

Evaluated Engineer (××× Structural design office ××) Date of evaluation (Year ××/××/××)

Seismic demand index Iso = Es x Z x G x U = 0.70

Direction	Story	C	F	E0	SD	T	IS	CTxSD	Result
X	3	0.05 1.27	0.80 1.00	(0.56) [$\boxed{0.94}$]	0.95	0.93	(0.49) [$\boxed{0.83}$]	(0.66) [$\boxed{0.89}$]	OK
	2	0.16 0.71	0.80 1.00	(0.46) [$\boxed{0.62}$]	0.95	0.93	(0.41) [$\boxed{0.55}$]	(0.55) [$\boxed{0.59}$]	NG
	1	0.10 0.58	0.80 1.00	(0.40) [$\boxed{0.58}$]	0.95	0.93	(0.36) [$\boxed{0.51}$]	(0.48) [$\boxed{0.55}$]	NG

( ): Index considering the extremely brittle columns

Ai distribution shape is used for lateral external force distribution shape

[ ]: Index considering the members with F of 1.0 (using Eq. (5))

Results of the second level screening

Name of Build.(××× Elementary school) Construction year (1970) Address (××Prefecture ×× city ××)

Evaluated Engineer (××× Structural design office ××) Date of evaluation (Year ××/××/××)

Seismic demand index Iso = Es x Z x G x U = 0.70

Direction	Story	C	F	E0	SD	T	IS	CTxSD	Result
Y	3	3.25	1.00	[$\boxed{2.40}$]	0.95	0.93	[$\boxed{2.13}$]	[$\boxed{2.28}$]	OK
	2	1.67	1.00	[$\boxed{1.47}$]	0.95	0.93	[$\boxed{1.29}$]	[$\boxed{1.38}$]	OK
	1	1.08	1.00	[$\boxed{1.08}$]	0.95	0.93	[$\boxed{0.98}$]	[$\boxed{1.05}$]	OK

Ai distribution shape is used for lateral external force distribution shape

[ ]: Index considering the members with F of 1.0 (using Eq. (5))

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(Translators) The background information of the standard for seismic evaluation of existing reinforced concrete buildings, 2001, and the guidelines for seismic retrofit of existing reinforced concrete buildings, 2001 is cited in the commentary in the Japanese version. As the English version (1st) is prepared for the provisions of the standard and the guidelines, the lists of the references are attached here for the extend user of this English version.

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Prefaces and Members Lists to the Japanese Editions

A Note on Publication (First Edition)

As the geological structure of the earth has been revealed, it has also become clear that the Japan Island is situated on an extremely unstable part of the Earth's crust. On the other hand, dramatic advances are also being achieved in earthquake prediction techniques, accompanied by much speculation about areas where gigantic earthquakes may be expected in the future.

While memories of the 1964 Niigata Earthquake, 1968 Tokachi-oki Earthquake, and other disasters which caused serious damage are still remaining in mind, various other parts of the world have also suffered a series of severe earthquakes in recent years, with heavy damage to buildings reported.

The Ministry of Construction therefore planned the establishment of a method of evaluating the seismic performance of existing buildings. As a first step, the Ministry decided to establish a Standard for Seismic Evaluation and Guidelines for Seismic Retrofit of Existing RC Buildings as a project for fiscal year 1976, and entrusted their preparation to this organization.

To demonstrate the answering to this trust and showing its profound respect for this appropriate and timely action, the Japan Special Building Safety Center* immediately asked Prof. Hajime Umemura of the University of Tokyo, who is an authority in the field, to serve as committee Chairman, and Associate Prof. Tsuneo Okada of the Institute of Industrial Science, University of Tokyo and Dr. Masaya Hirosawa of the Building Research Institute (BRI), Ministry of Construction to chair the Sub-committee on the Standard for Seismic Evaluation and Sub-committee on the Guidelines for Seismic Retrofit, respectively. As shown in the appendix, a committee made up of persons of experience and leading academic authorities was organized to undertake this project.

Although this work was originally expected to be extremely difficult, the remarkable results were achieved in the short period of 9 months.

This was the result of the unstinting efforts of all Committee members and particularly the members of the Sub-committees. Here, I would like to express my heartfelt appreciation to all the members of the Committee and my profound thanks to the members of the Building Guidance Division, Housing Bureau, Ministry of Construction for their active guidance throughout this work.

I believe that these results will undoubtedly contribute not only to improvement in the seismic performance of buildings in Japan, but also to earthquake engineering worldwide.

"Provide for the worst, and the best will take care of itself" is an iron rule which is good in all times. With this in mind, I hope that all those concerned will make full use of this Standard for Seismic Evaluation and the Guidelines for Seismic Retrofit of Existing Buildings as a deterrent to disaster in the unfortunate event of an earthquake.

March 1977

Keiji Horii, President

The Japan Special Building Safety Center

* current The Japan Building Disaster Prevention Association

Preface (First Edition)

In Japan, widespread adoption of medium- and low-rise reinforced concrete (RC) buildings as a type of earthquake-resistant / fire-resistant structure began following the 1923 Great Kanto Earthquake. During the same period, Dr. Toshikata Sano advocated the seismic coefficient method, which skillfully grasped the effect of seismic motion on buildings, as a method of seismic design. With increasing acceptance of seismic design methods based on the seismic coefficient method, Japan subsequently constructed many buildings with high levels of earthquake resistance, even when compared with world standards. However, a number of RC buildings based on seismic design suffered unexpected damage in the great earthquakes which followed the Great Kanto Earthquake, including the 1948 Fukui Earthquake, 1964 Niigata Earthquake, 1968 Tokachi-oki Earthquake, 1975 Oitaken-chubu Earthquake, and others, indicating that designs based solely on the provisions of the conventional seismic coefficient method had resulted in buildings with structural systems which did not adequately guarantee safety.

Recent advances in earthquake engineering have made it possible to construct super-high rise buildings. Considering past earthquake damage, with the benefit of this new knowledge, it is clear that buildings designed using the same seismic design method display a wide range of seismic performance, ranging from buildings with excellent seismic performance to a small number whose safety is problematic.

In particular, the remarkable damage suffered by low-rise RC structures in the 1968 Tokachi-oki Earthquake encouraged a variety of research on seismic design methods for RC structures which consider dynamic behavior during earthquakes. Some of these results have already been incorporated in the Enforcement Order of the Building Standard Law, Standard for Structural Calculation of Reinforced Concrete Structures (Architectural Institute of Japan), and other guidelines for practical use.

Thus, going hand in hand with academic progress, the lessons of past disasters have been put to good use, as seen in advances in seismic design methods. At present, however, it cannot be said that adequate study has been given to the earthquake safety of existing buildings which were constructed without the benefit of this experience.

Given this situation, the Building Guidance Division, Housing Bureau, Ministry of Construction, drew up plans to create a Standard for Seismic Evaluation and Guidelines for Seismic Retrofit of Existing Buildings. The actual preparation of the standard and the guidelines was entrusted to the Japan Special Building Safety Center*. A Committee on the Standard for Seismic Evaluation and the Guidelines for Seismic Retrofit of Existing Buildings was organized in the Center, and among existing building, focused its work on medium- and low-rise RC buildings. Two sub-committees were created to prepare drafts of the standard and the guidelines, these being the Sub-committee on the Standard for Seismic Evaluation and the Sub-committee on the Guidelines for Seismic Retrofit.

This report summarizes the discussions in the committee, based on the drafts prepared by the Sub-committees, and consists of three separate volumes, as follows.

1. Standard for Seismic Evaluation of Existing RC buildings with Commentary
2. Guidelines for Seismic Retrofit of Existing RC Buildings with Commentary
3. Technical Manual for the Standard for Seismic Evaluation and the Guidelines for Seismic Retrofit of Existing RC Buildings

Vol. 1, Standard for Seismic Evaluation with Commentary, presents a method of evaluating seismic capacity which assigns points to the seismic performance of superstructure, including the structural parts and non-structural part of the building, respectively. In obtaining indexes of seismic performance which consider strength, deformation capacity, failure mode, and earthquake response, priority is given to simplicity. Therefore, the Standard was prepared to enable application based on the structural calculations specified in seismic design methods in common use, or simpler calculations, and engineering judgments.

It should be noted that the Standard does not specify particular criteria for seismic judgments on the necessity of remediation based on the results of evaluations by this method. This philosophy was adopted because seismic judgments on buildings should not be based solely on the seismic performance of the building superstructure, but should also consider other essential conditions such as the relationship between the building and soil, the use and importance of the building, and the risk of earthquake. However, Vol. 3 contains examples of application to buildings which were damaged and undamaged in the 1968 Tokachi-oki Earthquake, as well as standard values for the borderline between damaged / undamaged buildings in the same earthquake, and describes a basic policy for seismic judgments.

Vol. 2, Guidelines for Seismic Retrofit with Commentary, presents methods of strengthening buildings which show low seismic index values in seismic evaluations and gives concrete guidelines for the strengthening design and construction.

Vol. 3 describes methods of concrete application of the aforementioned Standard for Seismic Evaluation and Guidelines for Seismic Retrofit. Because this report was completed in a short period of approximately 9 months, many points still require full study. However, we believe that it has achieved the distinctive feature of presenting a simple, integrated method from evaluation to strengthening, focusing on the seismic safety of existing buildings.

In the seismic design of medium- and low-rise reinforced concrete buildings to date, there seems to have been a tendency to follow the stipulated procedures mechanically, without adequately considering what level of seismic performance the finished building will actually possess. Although the Standard for Evaluation and the Guidelines for Retrofitting Design presented here are basically intended for existing buildings, they can also be applied to new buildings at the point in time when the structural design is complete. We therefore hope that the Standard and the Guidelines will be actively used not only with existing buildings, but also with newly designed buildings.

In closing, I would like to express my deep appreciation to the Housing Bureau of the Ministry of Construction, which planned this project, the Japan Special Building Safety Center, which was responsible for the work, and the members of the Committee, which conducted deliberations. In particular, I would like to thank all those concerned in the Sub-committees for undertaking the preparation of the draft Standard and Guidelines with such energy and compiling the results in such a short period of time.

March 1977

Hajime Umemura, Chairman

Committee on the Standard for Seismic Evaluation and the Guidelines for Seismic Retrofit of Existing Buildings

* current The Japan Building Disaster Prevention Association

Foreword (First Edition, 1977)

This handbook, which is entitled Standard for Evaluation of Seismic Capacity of Existing RC Buildings, proposes a method in which the seismic performance of existing reinforced concrete buildings is expressed in continuous quantities. It consists of the main text of the Standard and a commentary.

Recent years have seen considerable research on seismic evaluation of existing buildings using various approaches to the evaluation of seismic performance (earthquake resistance). However, in this standard, seismic evaluation in the broad sense is divided into the two processes of seismic evaluation and seismic judgment, which are defined as follows:

1) Seismic evaluation: Evaluation of the seismic performance of a building using a relative seismic index (continuous quantity).

2) Seismic judgment: Judgment of seismic performance using the seismic index obtained in the seismic evaluation as a base, and also considering various conditions such as the use, importance, and age of the building and other factors.

Of these, this Standard presents an approximated calculation method for the former, and has attempted to express the seismic performance of buildings in terms of two indexes, the seismic index of structure, I_S, and the seismic index of non-structural elements, I_N. Because the standard was completed in a short period of time, it may contain a number of points which are not fully developed. However, we will continue to study these issues, for example, by accumulating additional examples of application.

The following are distinctive features of this method. The Standard was created for the purpose of evaluating a large number of buildings in the shortest possible time. Therefore, while referring to other already-proposed seismic design and evaluation methods, we have tried to simplify the present method as much as possible without losing sight of the main points. For this purpose, we have created three methods which differ in the level of calculation method. These are called the first level screening method, second level screening method, and third level screening method. Because the first level screening method is the simplest of the three, the reliability of results will inevitably be lower than with the other two methods. In other words, lower level methods are intentionally simpler, but their reliability is also lower. Considering this, when the same building is evaluated using a low level and high level screening method, the seismic index value should increase as the level of the screening method increases. Although we intended to make this a distinctive feature of the Standard, it is clear from study in the present stage that the results do not necessarily show the desired tendency, depending on the properties of the building. We plan to improve this weakness through further study.

As an additional feature, the Standard also considers the quality of structural design which are difficult to evaluate only by a rough calculation method, the degree of deterioration in seismic performance over time, and other features, using a checklist method.

As mentioned previously, the evaluation Standard does not cover the seismic judgment. However, the results of application of the Standard to earthquake-damaged buildings are shown in the Technical Manual, which also examines the relationship between damage in past earthquakes and the magnitude of the seismic index according to the Standard. Users are invited to see this manual for details.

In concluding this Foreword, I would like to express my deep appreciation to all those who contributed to the preparation of the draft of this Standard, including Prof. Hajime Umemura, Chairman of the Committee, and all the committee members from whom we received valuable guidance and support throughout the project, the members of the Sub-committee, who were responsible for the hard work of preparing the draft, Messrs. Masayoshi Yoshida, Hiroyuki Uno, and Tamio Mori of the Building Guidance Division, Housing Bureau, Ministry of Construction, who provided useful guidance, and Messrs. Mikio Maeoka and Yoshinori Takahashi of the Japan Special Building Safety Center*, who were responsible for administration of the Sub-committee.

March 1977

Tsuneo Okada, Chairman

Sub-committee on the Standard for Seismic Evaluation

* current The Japan Building Disaster Prevention Association

Preface (First Edition, 1977)

The Guidelines for Seismic Retrofit describe items related to the design and execution of seismic retrofit for improvement of existing reinforced concrete (RC) buildings which are judged to have inadequate seismic safety. Although the Guidelines are basically intended for use as a set with “Standard for Seismic Evaluation of Existing R/C Buildings,” which is employed in judging the necessity of retrofit, parts concerning the performance evaluation of members to be strengthened and the execution of strengthening work may also be useful in more general applications.

As noted in the Foreword by Prof. Okada to the above-mentioned Standard, in recent years, there has been a strong need for evaluation of the seismic capacity of existing buildings, and a number of buildings have required seismic retrofit based on such evaluations. However, adequate materials on retrofit design methods and precautions when executing retrofit work have not necessarily been available.

To remedy this problem, in March 1976, the Ministry of Construction’s Government Buildings Department and the Building Research Institute (BRI) prepared an “Outline of Seismic Retrofit of Existing Buildings (Draft).” The present Guidelines were created referring to this Outline (Draft), and incorporate recent experimental data and other information, while also considering the relationship with the above-mentioned Standard.

The main content of the Guidelines consists of items on establishing strengthening targets, representative retrofit methods, performance evaluation methods for those retrofit methods, and items related to execution.

The content of the Guidelines, beginning with various retrofit methods, was prepared using the limited experimental data available to date, and some parts may require improvement based on future research. Therefore, for the reference of engineers who are responsible for retrofit design and execution, we have included an outline and list of the existing materials which were used as supporting data for the Guidelines.

In principle, the Guidelines are intended for application to buildings which have not been seriously damaged by earthquakes or other natural disasters, but may also serve as a useful reference when retrofitting stricken buildings.

In conclusion, I wish to express my sincere appreciation to Prof. Hajime Umemura, Chairman of the Committee, and all of the Committee members for their invaluable guidance and advice in preparing the draft of the Guidelines, the Sub-committee members who actually prepared the Guidelines, Messrs. Masayoshi Yoshida, Hiroyuki Uno, and Tamio Mori of the Building Guidance Division, Housing Bureau, Ministry of Construction, who provided useful advice, and Messrs. Mikio Maeoka and Yoshinori Takahashi of the Japan Special Building Safety Center, who were responsible for administration of the Sub-committees.

March 1977

Masaya Hirosawa, Chairman

Sub-committee on the Guidelines for Seismic Retrofit

On Publication of Revised Edition (1990 Rev.)

Japan is located in an earthquake-prone region of the globe and during its history has experienced serious disasters caused by a number of major earthquakes. Thus, ensuring the seismic safety of buildings is a matter of concern not only to construction engineers including the administrative authorities responsible for construction, but also the general public.

Reflecting the country's history of earthquake-related disasters, active studies and research on seismic technologies began at an early date in Japan. The results were reflected in the Building Standard Law, and more detailed measures were put in place, corresponding to the technical levels of the times. Accordingly, existing buildings which were constructed some years ago were designed in conformance with the standards available at the time and do not necessarily possess adequate seismic performance under today's standards.

From this viewpoint, seismic evaluation of existing buildings built prior to promulgation of the Standard for New Seismic Design Method in 1981 using more accurate evaluation standards, together with appropriate seismic retrofit where necessary, are extremely important for mitigating damage in the event of an earthquake.

In the First Edition of this work, in 1976, the Ministry of Construction entrusted the Japan Building Disaster Prevention Association* with preparation of a Standard for Seismic Evaluation / Guidelines for Seismic Retrofit of Existing RC Buildings. The results, which were compiled through the diligent efforts of the Committee (chaired by then-Prof. Hajime Umemura of the University of Tokyo), were published in April 1977.

During the period of more than 10 years which have passed since publication of the First Edition, we have accumulated records of seismic evaluation and seismic retrofit, as well as new research results, and have also collected information from studies of earthquake damage in Japan and various other parts of the world. In order to reflect this knowledge properly, we are publishing the results of deliberations in the Revising Committee (chaired by Prof. Emeritus Umemura, University of Tokyo), which extended over more than 2 years, in this Revised Edition.

Today, the importance of improved building safety and maintenance are strongly advocated, and there will also be an increasing need for seismic evaluation and seismic retrofit in the future. In this respect, it is my greatest hope that this Revised Edition will be actively used and will contribute to improving the earthquake resistance of buildings and minimizing damage in the unfortunate event of an earthquake.

In closing, I wish to express my heartfelt appreciation to Chairman Umemura, Subcommittee Chairman Okada, Working Groups Chairmen Murakami and Hirosawa, and all the committee members who participated in this work, and my profound thanks to those concerned at the Building Guidance Division, Housing Bureau, Ministry of Construction for their positive guidance throughout this project.

December 1990

Yoshihiro Maekawa, President

The Japan Building Disaster Prevention Association

* then The Japan Special Building Safety Center

Note on Revision (1990 Revision)

The First Edition of this work, the Standard for Seismic Evaluation and Guidelines for Seismic Retrofit of Existing RC Buildings were issued in 1977 in preparation for the revision of the Building Standard Law / Enforcement Order, which was promulgated in 1980 and took effect the next year. Following publication of the Standard and the Guidelines for Existing RC Buildings, similar standards / guidelines were also established for Steel Buildings (1979), Wooden Buildings (1979), and Steel Encased Reinforced Concrete Buildings (1986).

Subsequently, these standards / guidelines were frequently utilized in seismic countermeasures for existing buildings, and together with the so-called new seismic design method in the Building Standard Law / Enforcement Order, proved useful in securing earthquake resistance in buildings. Nevertheless, more than 10 years have now passed since the First Edition, and particularly in recent years, remarkable progress has been achieved in earthquake engineering / seismic technology. Thus, a revision incorporating the results of research and technical progress during this period was considered necessary. To address this need, the Japan Building Disaster Prevention Association established a Revising Committee on the Standard for Seismic Evaluation and Guidelines for Seismic Retrofit of Existing Buildings, and organized a Sub-committee on Reinforced Concrete Buildings, Sub-committee on Steel Buildings, and Sub-committee on Wooden Buildings in the Steering Committee to review these respective areas.

In publishing this Standard for Seismic Evaluation / Guidelines for Seismic Retrofit of Existing RC Buildings (1990 Rev.), I hope that this work will enjoy the same wide acceptance as the First Edition, and will be used to ensure the earthquake- resistance of buildings.

December 1990

Hajime Umemura, Chairman

Revising Committee on the Manual for Repair Technology of Earthquake-damaged Buildings and the Standard for Seismic Evaluation of Existing Buildings

Foreword to Revised Edition (1990 Rev.)

The Standard for Seismic Evaluation and Guidelines for Seismic Retrofit of Existing RC Buildings with Commentary were issued 13 years ago. This project was carried out by the Committee chaired by Prof. Hajime Umemura, with myself as Chairman of the Sub-committee on the Standard for Seismic Evaluation and Masaya Hirosawa as Chairman of the Sub-committee on the Guidelines for Seismic Retrofit, and reached publication within slightly more than six months. Although the Standard and the Guidelines were completed in this short period due to the tireless work of the Sub-committee members, at the same time, we were also fortunate that there was a growing accumulation of the basic research results necessary for seismic evaluation and seismic retrofitting design during the period.

In the 13 years since publication of the First Edition, the Standard / Guidelines have been adopted more widely than those involved in the drafting work could have expected. As the first such occasion, the Standard / Guidelines were used in the so-called Countermeasures for Tokai Earthquake (planning for anticipated earthquakes in the high-risk Pacific coast area of Japan). A computer program, SCREEN-Edition 2 was also published by the author and others and has been used in evaluations of more than 4,000 public reinforced concrete buildings in Shizuoka Prefecture (center of Tokai region). Of these, some 400 have already undergone retrofitting. This work has also been used in evaluations of numerous public and private buildings in other regions, in evaluation / retrofitting when existing buildings were expanded, and in evaluation / retrofitting of earthquake-damaged buildings. It is also widely used in other countries outside Japan, including Mexico, China, Armenia, and [the former] Yugoslavia.

Although this wide application has confirmed the usefulness of the Standard / Guidelines, on the other hand, there was also a feeling that a revision incorporating recent knowledge in the field had become necessary.

The policy for the present revision included the following main points.

1) Standard for Seismic Evaluation

(1) In normal cases, it should be possible to obtain substantially the same index values as with the method used to date.

(2) Based on experience obtained from examples of application to date, a commentary which will be useful in judgments by evaluation personnel should be added.

(3) More complete judgment values should be provided as reference values, and these should be incorporated in the body of the standard.

(4) Examples of application should be presented in an organized manner.

2) Guidelines for Seismic Retrofit

(1) More complete performance evaluation formulas for retrofitted buildings should be presented.

(2) Data on recent strengthening methods should be added.

(3) Examples of application should be presented in an organized manner.

For the revision work, a Working Group on the Standard for Seismic Evaluation chaired by Masaya Murakami and Working Group on the Guidelines for Seismic Retrofit chaired by Masaya Hirosawa were organized in the Sub-committee on Reinforced Concrete Buildings.

In concluding these remarks, I would like to express my appreciation to all the committee members concerned and to the Secretariat of the Japan Building Disaster Prevention Association.

December 1990

Tsuneo Okada, Chairman

Sub-committee on Reinforced Concrete Buildings

Introduction by Chairman of Working Group on the Standard for Seismic Evaluation (1990 Rev.)

The Standard for Seismic Evaluation / Guidelines for Seismic Retrofit of Existing RC Buildings with Commentary has been widely used since its first publication in 1977. However, more than 13 years have already passed since the First Edition.

During this period, great strides have been made in the field of earthquake engineering, and numerous results with applicability to the Standard can be noted. By examining a large number of examples of application of the Standard, we have also discovered points which require improvement, and seismic judgment methods which were not formally adopted in the original Standard have been established in various areas, referring to the 1978 Miyagiken-oki Earthquake and examples of application to damaged / undamaged buildings in the 1968 Tokachi-oki Earthquake. On the other hand, the Building Standard Law and its Enforcement Order were revised in 1980.

Based on this background, the Standard has been revised by incorporating recent results, while also considering compatibility with the Standard as it existed to date. However, it is conceivable that many points which require updating were left unrevised, and numerous inherent problems may also exist. We will therefore continue to collect examples of application and make further improvements.

The main revisions in this edition include the addition of judgment values for identifying buildings with seismic performance on the same order as buildings designed under the current Building Standard Law / Enforcement Order, a review of strength evaluation incorporating recent knowledge, and expansion of the numerical values and methods which can be adopted by the responsibility of the evaluator. On the other hand, there has been no change in the substance of the Standard. Users should note that judgment values have not been prepared for non-structural elements, following the practice to date, and should therefore refer to the Technical Manual. Users should also refer to this edition of the Technical Manual for an integrated method of judging the risk of falling and destruction of non-structural members, including equipment.

The new user should note that the Standard is to evaluate a large number of buildings in the shortest possible time and it includes 3 screening levels (1st, 2nd, 3rd level screening method), which are progressively more complex but offer increasingly higher levels of reliability, and the seismic reliability of buildings is expressed continuously by two types of index values, the seismic index of structure, I_S, and the seismic index of non-structural elements, I_N. We also advise new users to refer to the side notes in the Technical Manual, as we believe that the method can be mastered more quickly.

The expansion of numerical values and procedures which can be adopted at the discretion of the evaluator may make application of the Standard more difficult. However, we expect that good evaluation results can be obtained if the evaluator has a full understanding of the properties of the building. In this connection, please also refer to the Commentary and the Technical Manual.

Accompanying this revision, the Technical Manual also largely rewritten. Because new and revised topics include the aforementioned seismic performance of non-structural elements, seismic demand, materials on modeling, and a simplified third level screening method, we believe that even persons who are already familiar with the Standard will benefit from the Manual.

Finally, I would like to express my appreciation to the numerous persons who contributed to this revision, including Committee Chairman Hajime Umemura, Sub-committee Chairman Tsuneo Okada, and the members of the Sub-committee on Reinforced Concrete Buildings for their valuable guidance and advice, the members of the Working Groups involved in the revision work for their extremely hard work, Mr. Mitsuyoshi Takatsu of the Building Guidance Division, Housing Bureau, Ministry of Construction and Messrs. Yasunori Yamanaka and Mitsuaki Ohmae of the Building Disaster Countermeasure Division, Housing Bureau, Ministry of Construction, for their valuable advice, and Messrs. Akinobu Matsuo and Yoshitoku Takahashi of the Japan Building Disaster Prevention Association, who were responsible for administration of the Subcommittees.

December 1990

Masaya Murakami, Chairman

Working Group on the Standard for Seismic Evaluation

Introduction by Chairman of Working Group on the Guidelines for Seismic Retrofit (1990 Rev.)

These Guidelines for Seismic Retrofit are applicable to existing reinforced concrete buildings which are judged to have inadequate seismic safety, and describe items related to the design and execution of seismic retrofitting for improvement.

The Guidelines were originally published in 1977, together with the Standard for Seismic Evaluation of Existing RC Buildings, and were the first of their kind in the world. Although the Guidelines were subsequently utilized in seismic retrofitting design for existing public buildings and other structures in a number of prefectures and metropolitan areas, including Shizuoka Prefecture and Tokyo Metropolitan, after a lapse of 13 years, revision was required.

Japan's technical achievements in the field of structural seismic design in recent years have been truly remarkable, even when compared with the most advanced technologies in other countries. Related technical development has not been limited to the structures of newly constructed and existing buildings, but also extends to evaluation and strengthening of the structures of damaged buildings, as well as other fields such as the ground and non-structural members and building utilities.

As the occasion for present revised Guidelines, a Standard for Judgment of Earthquake Damage and a Manual for Repair Method for various types of structures and ground were prepared based on results obtained in a project called Development of Repair Technology for Buildings and Infrastructure Damaged by Earthquakes, which was carried out between 1981 and 1985 as a General Technology Development Project of the Ministry of Construction. The two above-mentioned works are being published by the Japan Building Disaster Prevention Association simultaneously with the revised Guidelines.

Thus, this Revised Edition includes buildings damaged by earthquakes as a new area for application of the Guidelines. Revisions and additions resulting from the accumulation of examples in the fields concerned and results of related research have also been included. In addition to the expanded scope of application, main points of this revision include new sections on retrofitting design and basic design, in which more complete examples of execution are provided, sections on strengthening by brace installation, which can be seen in many examples in recent years, chemical anchors, and revisions of yield strength formulas, etc. for anchors.

Reviewing major earthquakes which have caused damage in Japan and elsewhere since the First Edition of the Guidelines was published in 1977, those in Japan include the 1978 Miyagiken-oki Earthquake (M=7.4, 27 deaths) and 1983 Nihonkai-chubu Earthquake (M=7.7, 104 deaths), while serious quakes in foreign countries include the 1985 Michoacan, Mexico Earthquake (M=7.9, approx. 10,000 deaths), 1988 Spitak, USSR Earthquake (M=7.0, approx. 30,000 deaths), 1989 Loma Prieta, USA Earthquake (M=7.1, 62 deaths), and were followed in 1990 by a successive severe destructive earthquakes in Gilan, Iran (M=7.0-7.5, approx. 30,000 deaths) and the Luzon, Philippines (M=7.7, approx. 2,000 deaths). Many of the deaths in these earthquakes were caused by the collapse of existing buildings. In particular, the collapse of modern medium- and large-scale reinforced concrete buildings was an important cause of death in the Michoacan, Mexico Earthquake, the Spitak, USSR Earthquake, and the Luzon, Philippines Earthquake. Considering this, the problem of improving the earthquake resistance of existing buildings has become a major concern worldwide.

In Japan, the seismic safety of existing buildings is comparatively high by global standards. However, safety concerns still exist at a significant number of schools and public buildings.

Based on accumulated results, we have prepared a menu for selection of the most rational method of seismic retrofitting, in terms of both function and cost, and have incorporated related materials in the present revision.

In view of the conditions described above, we sincerely hope that these revised Guidelines will be widely used in improving the earthquake resistance of both existing buildings and damaged buildings.

In concluding this Introduction, I would like to express my appreciation to Chairman Hajime Umemura of the Revising Committee for his valuable guidance and constant support, Chairman Tsuneo Okada and the members of the Sub-committee on Reinforced Concrete Buildings, and the members of the Working Group who actually prepared the draft of the revision, to Mr. Mitsuyoshi Takatsu of the Building Guidance Division, Housing Bureau, Ministry of Construction and Messrs. Yasunori Yamanaka and Mitsuaki Ohmae of the Building Disaster Countermeasure Division, Housing Bureau, Ministry of Construction for their valuable advice, and to Messrs. Akinobu Matsuo and Yoshinori Takahashi of the Japan Building Disaster Prevention Association, who were responsible for administration of the Working Group.

December 1990

Masaya Hirosawa, Chairman

Working Group on the Guidelines for Seismic Retrofit

On Publication of the 2001 Revised Edition

Since the 1970s, the Japan Building Disaster Prevention Association has devoted itself to publishing and disseminating standards for seismic evaluation of existing buildings, guidelines for seismic retrofit, and related documents, and to promoting wider use of seismic evaluation / seismic retrofit through technical evaluations by Seismic Judgment Committees. During this period, damage to buildings in the 1978 Izu-Ohshima-kinkai Earthquake, 1978 Miyagiken-oki Earthquake, 1983 Nihonkai-chubu Earthquake, and others showed the necessity of seismic evaluation / seismic retrofit of buildings. However, until the 1995 Hanshin-Awaji Earthquake Disaster (Kobe Earthquake), seismic evaluation/seismic retrofitting was limited to parts of the Kanto and Tokai Regions (area surrounding Tokyo and the Pacific seaboard west of Tokyo), and had not been generally adopted nationwide.

One of the important lessons of the 1995 Hanshin-Awaji Earthquake Disaster was that seismic evaluation / seismic retrofit of existing buildings, and particularly buildings which were designed and constructed prior to the 1981 revision of the Building Standard Law and Enforcement Order is an essential condition for alleviating the effects of earthquakes. Based on this recognition, implementation of countermeasures began in December 1995, and included enforcement of the Law for Promotion of Seismic Retrofit of Buildings, which required seismic evaluation / seismic retrofit nationwide.

In 1977, this Association published a Standard for Seismic Evaluation and Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings in advance of similar standards for other types of structures. This was followed by a partial revision in 1990. However, as mentioned above, the years since the 1995 Hanshin-Awaji Earthquake Disaster have seen an increasing number of examples of application of the Standard and the Guidelines and considerable technical development. We therefore decided to prepare a new revision to incorporate this recent knowledge.

In preparing this revision, the Association created a Revising Committee on the Standard for Seismic Evaluation and the Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, chaired by Masaya Murakami, and a Draft Making Committee, chaired by Toshimi Kabeyasawa. Four Sub-committees were organized under these Committees to carry out the study, the Sub-committee on the Standard for Seismic Evaluation, also chaired by Toshimi Kabeyasawa, Sub-committee on the Guidelines for Seismic Retrofit, chaired by Takashi Kaminosono, Sub-committee on the Technical Manual for Adoption, chaired by Matsutaro Seki, and Sub-committee on the Non-structural Elements, chaired by Isao Sakamoto. Here, I wish to express my heartfelt appreciation for the efforts of the Committees and Sub-committees, thanks to which this major revision was completed in a remarkably short period of time.

The main points and outline of the revision are presented separately in the Preface by Chairman Murakami and in Forewords by Subcommittee Chairmen Kabeyasawa, Kaminosono and Seki. In short, however, it was understood that “this revision, while keeping the framework of the existing Standard and Guidelines, should incorporate new knowledge and aim at greater completeness to facilitate use by engineers.” As there should be no extreme differences in the results obtained by evaluation and retrofitting using the former Standard and Guidelines and results with the Revised Edition, for the time being, seismic evaluations and seismic retrofitting can be carried out using either this Revised Edition or the 1990 Edition.

In closing, I wish to express my thanks to all concerned at the Building Guidance Division, Housing Bureau, Ministry of Infrastructure, Land and Transport for supervising this project.

October 2001

Tsuneo Okada, President

The Japan Building Disaster Prevention Association

Preface

These Standard for Seismic Evaluation / Guidelines for Seismic Retrofit were originally created in 1977 by the Committee on the Standard for Seismic Evaluation and the Guidelines for Seismic Retrofit of Existing Buildings (Committee Chairman, the late Dr. Hajime Umemura; Chairman of Sub-committee on the Standard for Seismic Evaluation: Tsuneo Okada; Chairman of Sub-committee on the Guidelines for Seismic Retrofit: Masaya Hirosawa), in this organization's predecessor, the Japan Special Building Safety Center. The Standard and the Guidelines were revised in 1990 by the Revising Committee on the Manual for Repair Technology of Earthquake-damaged Buildings and the Standard for Evaluation of Seismic Capacity of Existing Buildings (Committee Chairman, the late Dr. Umemura; Chairman of Sub-committee on Reinforced Concrete Buildings: Tsuneo Okada; Secretary of the same Sub-committee: Masaya Hirosawa). This is the 2nd Revised Edition, following the revision of 1990.

After the 1990 Revision, many buildings were severely damaged in the 1995 Hanshin- Awaji Earthquake Disaster, which occurred in January 1995, resulting in a wide recognition of problems with the earthquake resistance of existing buildings. In response to this damage, Japan enacted the Law for Promotion of Seismic Retrofit of Buildings, which established a legal mandate for seismic evaluation and seismic retrofitting efforts. As a result of this legal obligation, and aided by the creation of a financial support system, seismic evaluation / seismic retrofit were rapidly adopted. A large number of engineers were also trained, and Seismic Judgment Committees were established in all regions of the country to review the appropriateness of evaluations of seismic capacity. These Seismic Judgment Committees were also given partial responsibility for confirming seismic evaluations of existing buildings.

On the other hand, more than 10 years have now passed since the 1990 Revision. This period has seen an ongoing accumulation of experimental data on member performance, progress in research which is conscious of evaluation and retrofitting methods, and attempts to apply new methods in examples of retrofitting.

Based on these circumstances, there was a heightened feeling that a new revision was needed, which would incorporate the views of members of Seismic Judgment Committees and engineers involved in evaluation/retrofitting while also including recent research results. This led to the present revision.

The original intention in preparing the First Edition of the Standard was to enable simple manual calculations, and in the process, to consider engineering judgments. As stated in the Preface to the First Edition, the standard should "enable application based on the structural calculations specified in seismic design methods in common use, or simpler calculations, and engineering judgments." This philosophy was also retained in the previous 1990 Revision.

This point was discussed in the SPRC Committee of the Association in developing the Computer Program for Seismic Evaluation of RC Buildings (SCREEN Edition 2) in 1978. A proposal to have engineers perform manual calculations and study output data at each step was accepted, but the opposite of this is closer to reality. In fact, only a very small number of structural engineers now make manual calculations. In view of the increased complexity and volume of calculations in the present revision, the use of computers is assumed. At the same time, however, we wish to emphasize the necessity of engineering judgments.

The above-mentioned computer program played a key role in subsequent application of the earlier versions of the evaluation standards. Therefore, ideas obtained in preparing the program are used in various calculations, such as strength calculations for members, in the present revision as well.

Accordingly, as this evaluation standard envisions the use of programs, in addition to continuity, the revision also considers compatibility with the former evaluation standard so as to achieve rationality. To avoid large differences due to programming, the necessity of assumptions was eliminated as much as possible. In this process, we attempted to enhance appropriateness and rationality as far as possible by incorporating recent knowledge, including research and experimental results. On the other hand, a perfect computer program used for any type of buildings can not be available, particularly for the third level screening method. Therefore, we advise engineers to include engineering judgments and study interim processes when engaged in evaluation / retrofitting work, and to make effective use of programs in part, based on an adequate understanding of the evaluation Standard and its intentions. From this viewpoint, we believe that the former Standard can also be used in evaluations in the future and the efforts to verify its compatibility with the present Standard will be required.

The composition of this Standard departs from the framework used in the former evaluation Standard, which comprised a main text and commentary, in that the Revised Edition is divided into four general parts, a main text, the related commentary, a supplementary text consisting mainly of calculation equations with commentary, and appendixes. The reason for separating the main text and supplement was to make it possible to revise equations, when necessary, by introducing new research results.

In the Guidelines for Seismic Retrofit, the chapter units remain unchanged. However, important points and points which require emphasis are presented as section or item units. Although no changes have been made in the general outline, new knowledge and methods have been included and the Guidelines have been reviewed while endeavoring to maintain compatibility with the current revision of the Evaluation Standard.

It should be noted that the design of strengthening members is performed in accordance with the Guidelines for Seismic Retrofit, but thereafter, the seismic performance of the building as a whole is evaluated based on the Standard for Seismic Evaluation, including engineering judgments, as in earlier editions. It is not assumed that seismic performance after retrofitting should be assessed using only computer programs, with no engineering judgments of any kind.

The Technical Manual which presents exercises using the seismic evaluation Standard, recent examples of retrofitting, etc., has been considerably revised. However, universal and essential items are presented without change.

In concluding this Preface, I wish to express my appreciation to the members of the Revising Committee for their comments in preparing this revision, the Draft Making Committee (Chairman: Toshimi Kabeyasawa), who carried out the actual revision work with such energy, the Sub-committee on the Standard for Seismic Evaluation (also chaired by Kabeyasawa), Sub-committee on the Guidelines for Seismic Retrofit (Chairman: Takashi Kaminosono), Sub-committee on the Technical Manual (Chairman: Matsutaro Seki), and Sub-committee on the Non-structural Elements (Chairman: Isao Sakamoto), and to the Secretariat of the Japan Building Disaster Prevention Association for their generous cooperation.

October 2001

Masaya Murakami, Chairman

Revising Committee on the Standard for Seismic Evaluation and the Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings

Foreword by Chairman of Sub-committee on the Standard for Seismic Evaluation

The 1977 Edition, the First Edition of this Standard describes the initial basic concept at the establishment as follows: "The drafting of this Standard was started tentatively based on the Standard for Judgment of Seismic Capacity of Existing Reinforced Concrete Buildings published by the Building Research Institute (BRI), Ministry of Construction in 1973, although the concepts and methods developed by the original studies by the committee as well as a number of other seismic design methods and seismic evaluation methods were reflected everywhere in the provisions. Although they are listed at the end as references, we wish to note that we have referred in particular to the references 2), 3), 4), 5), 6), 7), 8), and 9), in addition to the draft by the BRI, in preparing the substantial part of this standard." In the Revised Edition of 1990, the followings were revised: 1) the seismic demand index was newly introduced for the judgment on seismic safety, 2) the formulas for strength evaluation and others were reviewed incorporating new knowledge, and 3) the range of numerical values and methods were extended, which may be adopted by the judgment of the engineers, and so on. However, the 1990 revision basically followed the philosophy and assessment methods of the First Edition as they were, and substantial changes seemed to be unnecessary. It might be because relatively few major earthquakes occurred in Japan after the 1978 Miyagiken-oki Earthquake, which caused serious damages, and also because users of the Standard had been limited to a small number of experienced engineers so that the Standard had been put into practice flexibly with the judgment of the engineers.

However, in response to the extraordinary lessons from the 1995 Hanshin-Awaji Earthquake Disaster (Kobe Earthquake), the Japanese government enacted "the Law for Promotion of Seismic Retrofit of Buildings," in December, 1995. Also with the start of "the Five-Year Plan for Earthquake Disaster Emergency Project Plan (Ministry of Education)" and active efforts on earthquake disaster mitigation by the local government level, practical application of the Standard increased drastically in comparison with the time before then. The Standard was disseminated widely to structural engineers with little experience in practical seismic evaluation work, while some confusions were observed in the practical application. For example, some engineers were applying the method in automatic or inappropriate manner, or using the computer programs for seismic evaluation in the same way as the integrated structural design programs. On the other hand, based on the experiences through the application to a large number of practical cases of evaluation, problems in the methods of calculation and judgment were assembled and analyzed, by which a proposal of more generalized evaluation methods started anticipating.

At the beginning of revision work for this Edition (2001), the conventional calculation methods were reviewed extensively including the newly proposed formulas from recent research by comparing the correlations with experimental data so as to propose new or revised evaluation methods. It was drawn from the results of the review that the accumulation of new data was not much adequate to change the conventional methods, while on the other hand, it was also recognized that enforcing the details of the previous version would be necessary to dissolve the confusions described above. Therefore, the details of the evaluation and calculation methods or the expressions used in the provisions have been reviewed comprehensively, maintaining the continuity to the basic concepts and assessment framework in the past versions of the Standard. As a result, provisions and the evaluation methods have been revised as comprehensively as possible, in such parts, for example, where the applicability of the evaluation methods was not necessarily adequate, where the evaluation gave serious discontinuity as a result, and where the relative values of the evaluation results were apparently irrational. The commentaries were also enhanced or completed with notes on application and explanation on the judgment concepts and others.

The specific revised points in this version are listed below.

(1) Technical terms for seismic evaluation are clearly defined.

(2) The site inspection is clearly placed in and directly reflected to the seismic evaluation procedure.

(3) The discontinuities are eliminated from the methods for evaluating the cumulative strength and the ductility index.

(4) The relationship between the deformations of members (yield deformation, ultimate deformation) and the story drift is evaluated reasonably and reflected explicitly to the accumulation of member strengths and the evaluation of ductility indexes.

(5) The evaluation methods are provided definitely for the column with wing wall(s), the wall with a column, the pilotis columns and the columns supporting the wall above.

(6) The detailed judgment method for the second-class prime elements is clearly specified.

(7) The evaluation of the ductility indices in the third level screening procedure is revised totally in consideration of the structural failure modes.

(8) The relationships between the damages observed in recent major earthquakes and the seismic demand indices are described.

(9) The alternative evaluation methods are also provided in the appendix.

(10) The SI units are used.

As for the overall framework of the Standard, the site inspection is moved to the independent new Chapter 2. The evaluation or calculation equations for the strength and the ductility (deformation capacity) of members are also moved to the supplementary provisions, so that the new methods for evaluation based on the future research may easily be reflected to the provisions. Because the revision has resulted in the new hierarchical framework, it should be noted that some of the main provisions need referring to the corresponding supplementary rules.

October 2001

Toshimi Kabeyasawa,

Chairman of the Sub-committee on the Standard for Seismic Evaluation,

Revising Committee on the Standard for Seismic Evaluation and the Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings

Foreword by Chairman of the Sub-committee on the Guidelines for Seismic Retrofit

The First Edition of the Standard for Seismic Evaluation and the Guidelines for Seismic Retrofitting Design of Existing Reinforced Concrete Buildings was published in 1977. Based on subsequent experimental research and the actual results of retrofitting, including strengthening and other methods, a revised Edition of the Standard and the Guidelines were published in 1990. The following years saw steady growth in related research and examples of retrofitting. In particular, the number of examples of seismic evaluation and seismic retrofitting increased rapidly after the 1995 Hanshin-Awaji Earthquake Disaster, and new related research pursuing easily-executable retrofitting, retrofitting while the object building is in use, environment-friendly retrofitting, retrofitting using new technologies, and examples of retrofitting also increased. Thus, there has been remarkable progress in technology and the accumulation of materials in the last 10 or so years.

This revision was carried out with the aim of realizing efficient execution of reliable seismic retrofitting at a level corresponding to this technical progress and accumulated body of knowledge. The Guidelines contain research results and numerous examples of actual use and were prepared with the main emphasis on generally-applicable retrofitting methods. Evaluation of retrofitting results based on recent research and experience, as well as retrofitting methods which use construction methods now in general use, have also been added in this revision. The main description of the details of retrofitting methods, evaluation methods, and execution methods added in this Revised Edition are as follows.

1) Retrofitting by additional walls (reinforcement details for opening and infilling)

2) Methods utilizing column strengthening (strengthening by continuous fiber, details of column edge slit, bending strengthening details)

3) Retrofitting by steel frame-type frameworks (removing steel framework, and adding the steel plate panel)

4) Other retrofitting methods (strengthening by braces or frames on exterior frame)

5) Foundation retrofitting (strengthening by increasing number of piles)

6) Floating methods using PC steel bars

7) Execution of strengthening work (important items for execution and quality control)

8) Reference materials (strengthening techniques positively evaluated by the Japan Building Disaster Prevention Association)

The Building Standard Law was again revised in 1998, and concepts for performance-based seismic design have been generally established. However, there are cases in seismic retrofitting where it is difficult to satisfy strengthening targets due to functional restrictions on the building, and cases where local damage may occur in structural and non-structural members even with strengthening. Therefore, in deciding important items such as strengthening target values, consultation with the building owner is necessary and indispensable, and it is also necessary to inform the owner and others concerned regarding the expected behavior of the retrofitted building during earthquakes.

In principle, the unit system used in the revised Guidelines is the SI unit system.

October 2001

Takashi Kaminosono, Chairman of the
Sub-committee on the Guidelines for Seismic
Retrofit, Revising Committee on the Standard for
Seismic Evaluation and the Guidelines for Seismic
Retrofit of Existing Reinforced Concrete Buildings

Foreword by Chairman of the Sub-committee on the Technical Manual

This Technical Manual comprises a commentary on actual examples of evaluation and retrofit and various data and concepts which form the background of the Standard for Evaluation of Seismic Capacity and the Guidelines for Seismic Retrofitting Design and is intended to give users a deeper understanding of the content of the Standard and the Guidelines.

The content of the Technical Manual consists of three parts. The first part presents a commentary on the Standard for Seismic Evaluation with exercises, using three buildings as examples, namely, a moment resisting frame structure, a school building, and a building with a hall. In particular, in the description of the moment resisting frame structure, the Technical Manual provides an easily-understood commentary which allows the user to trace the evaluation process from first level screening to third level screening by manual calculations. With the remaining two building types, the process from seismic evaluation to retrofitting design is shown to enable the user to understand the total flow.

The second part of the Technical Manual contains reference materials for seismic judgments of structures and non-structural members and notes on modeling, etc. These items have been expanded by adding recent knowledge on the basic data and thinking for deciding judgment values and on modeling when making seismic evaluations.

The third part presents examples of five buildings where seismic retrofitting was actually performed using the Technical Manual for Adoption of the Guidelines for Seismic Retrofit. Because retrofitting of these buildings was carried out before revision of the Standard, the work was based on the 1990 Revised Edition of the Standard.

The main points of concrete revisions in the Technical Manual are as follows.

(1) The content of the Standard for Seismic Evaluation is described in a way which enables tracing by manual calculations.

(2) The entire process from seismic evaluation of a building to drafting of a retrofitting design is described. A comparison of the results of evaluation by the new Standard and former Standard (1990 Rev.) is also presented.

(3) The content of modeling for strength index and structural balance index calculations has been expanded.

(4) Buildings where retrofitting work was actually performed are discussed.

When a seismic evaluation is actually performed, some type of commercially-available computer program is normally used. In this case, it has frequently been pointed out that there is a reason for concern about proper execution of the seismic evaluation, as some evaluators tend to mechanically output results without adequately understanding the content of the program. However, due to the general complexity of the structural form and other features of existing buildings, wide-ranging, high-level engineering judgments are required when conducting a seismic evaluation. Because the Standard is ultimately a general guide, detailed rules for the use are left to those responsible for creating individual computer programs, but as a result, evaluation results strongly reflect the judgments of the program's author. From experience to date, it is known that differences in results increase in proportion to the complexity of the building. Against the background of these facts, an adequate understanding of the Standard by the evaluator is a necessary condition for more accurate seismic evaluations. For this reason as well, we hope that those concerned will make positive use of this revised Technical Manual.

October 2001

Matsutaro Seki, Chairman of the Sub-committee on the Technical Manual, Revising Committee on the Standard for Seismic Evaluation and the Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings

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chart index