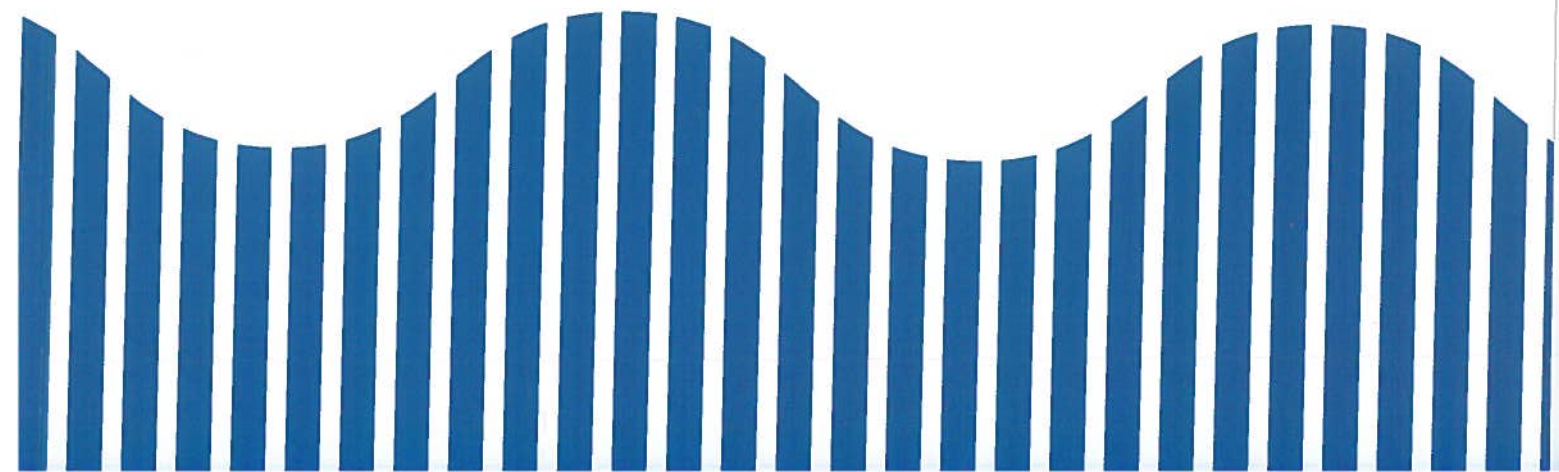




MANUAL FOR  
**SEISMIC  
RETROFIT DESIGN**  
OF EXISTING REINFORCED **CONCRETE**  
BUILDINGS

Public Works Department



# **MANUAL FOR SEISMIC RETROFIT DESIGN OF EXISTING REINFORCED CONCRETE BUILDINGS**

**PUBLIC WORKS DEPARTMENT**

**PREPARED UNDER**

**PROJECT FOR CAPACITY DEVELOPMENT ON NATURAL DISASTER RESISTANT TECHNIQUES  
OF CONSTRUCTION AND RETROFITTING FOR PUBLIC BUILDINGS (CNCRP)**

**A TECHNICAL COOPERATION PROJECT BETWEEN PWD AND JICA**

**2015**

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## Foreword

Bangladesh is a disaster prone country. The country is frequently affected by floods, cyclones and cyclone induced storm surges and tornados. The country is also under threat of moderate to strong earthquakes due to the geographical position. Bangladesh is close to one of the most tectonically active regions in the world. It is situated where three tectonic plates namely the Indian plate, the Eurasian plate and the Burmese plate met. Bangladesh, over the last two hundred and fifty years, had experienced eight major earthquakes of magnitude over 7.0. Among those earthquakes, two earthquakes namely Bengal Earthquake of 1885 and Srimongol Earthquake of 1918 had their epicenter within the country. Due to its proximity to the plate boundaries, active faults and track records of historical damaging earthquakes in and around Bangladesh, probability of occurrence of strong earthquake is high.

The risks of loss of life and damage to property due to earthquake are almost entirely associated with manmade structures. Because earthquake doesn't kill people, buildings do. The rapid urbanization of several cities especially Dhaka, Chittagong and Sylhet during the last 25 years with most of the buildings being non-engineered is a big concern.

Public Works Department (PWD) with a history of over 150 years is the Government Department which owns almost all the public buildings of the country in connection with construction and maintenance. The department inherits the legacy from British India through Pakistan period to present independent Bangladesh. A major portion of the huge building stock is unreinforced brick masonry buildings with low concrete strength, inadequate column section and non ductile RC framed structures. The Bangladesh National Building Code (BNBC) was formulated in 1993 and enacted in 2006. PWD has been following American Concrete Institute (ACI) code till 1993 and the BNBC subsequently for structural design purpose. But strict adherence to the code especially the seismic provisions came into practice very recently. As a result, a staggering number of existing buildings do not meet the seismic demand and capacity requirements of the current BNBC 2015 (Final Draft, July 2015)..

The Government of Bangladesh has taken a strong stand with disaster risk reduction. Government's success in certain areas of disaster risk mitigation such as flood, cyclone is acclaimed by the world and taken as role model in many countries. In case of earthquake disaster, the country is not sufficiently prepared to reduce the risk. The main reason is that earthquake is not a frequent phenomenon in Bangladesh. The country had experienced the last devastating earthquake in 1897 (The Great Indian Earthquake with magnitude 8.9). In the Standing Order on Disaster (SOD) of the Government, PWD is entrusted with the task to promote seismic resistant building and to retrofit public buildings which are vulnerable to earthquake.

Due to the lack of technical know-how, PWD could not undertake projects for retrofitting. To overcome this deficiency, PWD has undertaken a project with the technical cooperation of JICA titled "Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings (CNCRP)". The main purpose of the four year long project is to enrich the technical knowledge and working capacity of the engineers of PWD for seismic assessment, retrofitting design and construction of existing RC framed public buildings.

One of the outputs of this project is to develop 6 (six) individual manuals and guidelines as stated under for future references:

1. Manual for Seismic Evaluation of Existing Reinforced Concrete Buildings
2. Manual for Seismic Retrofit Design of Existing Reinforced Concrete Buildings
3. Manual for Retrofit Construction and Supervision of Reinforced Concrete Buildings
4. Guidelines for Quality Control of Design and Construction of Reinforced Concrete Buildings
5. Manual for Seismic Design of Reinforced Concrete Buildings

## 6. Manual for Vulnerability Assessment and Damage Prediction of Reinforced Concrete Buildings against Non Seismic Hazards

As stated earlier, many existing buildings do not meet the seismic demand and capacity requirements of the current BNBC 2015. The need for retrofitting may arise from one or more of the following reasons:

- (a) Violation of Bangladesh National Building Code in structural design and construction process.
- (b) Subsequent updating of Building Code.
- (c) Deterioration due to aging and unexpected natural and human created hazards.
- (d) Modification of existing structure.
- (e) Change in use of building.

The series of manuals and guidelines are the outcome of four year long experiences of CNCRP project. The engineers of PWD with technical assistance of the JICA experts tried to adapt the Japanese retrofit technology to local construction conditions and practices. Seismic retrofitting is a specialized type of job. The professionals and practicing engineers are requested to go through the manuals carefully and apply their engineering judgments before application.

The current edition of the manuals and guidelines are a modest beginning. Extensive research on local conditions such as construction materials, techniques, and practices in the light of local seismicity are necessary to upgrade the manuals. We, as professionals, believe that manuals are only a guide or outline and it is the expert who will have to take the final decision about actual extent of work to be done. We expect feedback from all quarters to enrich the future editions of the manuals.

The current Bangladesh National Building Code (BNBC 2015) does not contain any provisions of seismic evaluation and retrofit design. Throughout the project duration, the engineers of PWD studied the Japanese standard, guidelines and technical manuals for seismic evaluation and retrofit design of RC buildings. As judgment is very important in assessing vulnerability of a building, the Japanese method gives emphasis on critical observations and hand calculations. There are many factors and assumptions to be taken based on local construction circumstances. This “Manual for Seismic Retrofit Design of Existing Reinforced Concrete Buildings” has been prepared to supplement the English version of the original Japanese Standards, Guidelines and Technical Manual titled “Standard, Guidelines and Technical Manual for Seismic Evaluation and Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001”, published by The Japan Building Disaster Prevention Association (JBDPA).

We deeply acknowledge the Editorial Advisory Board consisting of respected members from Japan and Bangladesh for their valuable contribution. The authors from JICA expert team needs special mention for formulating the manuals. We also thank all the CNCRP team members for their hard work which eventually helped in publishing these manuals and guidelines. Finally I want to thank the Government of Japan and JICA for their whole hearted support and cooperation in all phases of the project CNCRP.

**Engr. Md. Kabir Ahmed Bhuiyan**

Chief Engineer

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## PREFACE

Bangladesh is located in a tectonically active region close to the plate boundaries of the Indian plate and the Eurasian plate to its north and east. Based on seismicity, Bangladesh is divided into three seismic zones, as per Bangladesh National Building Code (BNBC), 1993. The BNBC 1993 was adopted in 2006 under Building Construction Act, 1952. Most of the buildings constructed before adoption of BNBC 1993 is either non-engineered or designed without considering seismic load. The present construction scenario is not very encouraging either. Under these circumstances large numbers of buildings both public and private, in the urban areas needs structural assessment and retrofitting if found vulnerable.

The concept and practice of Japanese Standard of Seismic Evaluation and Guidelines of Retrofit Design for existing RC buildings has been studied and applied in Bangladesh through the PWD-JICA technical cooperation project CNCRP.

The Japanese Standard and Guidelines for Seismic Evaluation and Retrofit of Existing Reinforced Concrete Buildings, prepared in 1977 has been applied in many buildings in Japan successfully. The Guideline was revised in 1990 and 2001.

Following concise book of Standard and Guidelines translated in English under one cover was published in 2001:

**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**

Translated by: Building Research Institute, Published by: The Japan Building Disaster Prevention Association (JBDPA).

This English version 2001 covers the main portion of the Japanese Standard and Guidelines only.

This “**Manual for Seismic Retrofit Design of Existing Reinforced Concrete Buildings**” has been prepared to supplement the Japanese Standard and Guidelines mentioned above incorporating the seismic load of BNBC 2015 (Final Draft, July 2015). Effort has been taken to incorporate the design and construction practices of Bangladesh in the manual as much as possible. This manual will be used together with the “**Manual for Seismic Evaluation of Existing Reinforced Concrete Buildings**” prepared under CNCRP project.

Seismic retrofit in Japan has been disseminated after the Hyogo Ken Nanbu (Kobe) Earthquake 1995, together with the act on promotion of Seismic Retrofitting of Existing Buildings. More than 50,000 existing public school buildings have been retrofitted as of 2011.

It is expected that seismic performance of existing RC buildings will be improved through application of this Manual for Seismic Retrofit Design and the building damage risk in Bangladesh will be mitigated.

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## **ACKNOWLEDGEMENT**

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## NOTATIONS

Notations used in this manual are same as (i) “Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001” and (ii) “Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001” The notations are provided in two sections (Notation A and Notation B) referring to the relevant page numbers of those two manuals for understanding and classification of the users of this manual.

### NOTATION A.

	Page no.*
$A_c$ : Projected area of concrete cone failure surface of a single anchor ( $\text{mm}^2$ ).	2-47
$a$ : Shear span; distance between the beam face at the column top and the point of lateral force from the in filled wall.	2-13
$a_l$ : Gross sectional area of longitudinal reinforcing bars of column in tension side ( $\text{mm}^2$ ).	2-18
$a_t$ : Cross sectional area of tensile longitudinal reinforcement ( $\text{mm}^2$ ).	2-23
$a_{t2}$ : Cross sectional area of tensile reinforcement in the jacketing part of column.	2-24
$a_s$ : Cross sectional area of one set of reinforcement ( $\text{mm}^2$ ).	2-36
$a_s$ : Cross sectional area of stud ( $\text{mm}^2$ ).	2-37
$a_e$ : Minimal cross section area of expansion anchor ( $\text{mm}^2$ ).	2-47
$a_o$ : Effective cross section area of threaded steel bar or nominal cross section area of anchorage bar ( $\text{mm}^2$ ).	2-47
$a_e$ : Cross section area of expansion anchor at concrete interface, or cross section area of bonded anchorage bar ( $\text{mm}^2$ ).	2-47
$b$ : Width of column (mm).	2-18, 23, 29
$b_e$ : Effective width of columns resisting against the direct shear force considering the connected members in the orthogonal direction.	2-13
$b_e$ : $a, b$ (mm)	2-18
$b_2$ : Width of column after jacketing (mm).	2-24
$b_2$ : Column width after strengthening.	2-26
$b_s$ : Width of steel strap	2-26
$D$ : Depth of column (mm).	2-18, 23, 29
$D$ : Depth of columns resisting against the direct shear force.	2-13
$D_2$ : Depth of column after jacketing (mm)	2-24
$D_a$ : Diameter of drilled hole of existing concrete structure (mm).	2-47
$d_2$ : Effective depth of the retrofitted column (mm).	2-24
$d_a$ : Diameter of anchor; nominal diameter of anchorage bar for bonded anchor or diameter of sleeve of expansion anchor (mm).	2-47
$d_b$ : Nominal diameter of steel bar threaded into expansion anchor (mm).	2-47
$d_e$ : Distance between the center of the tensile reinforcing bars and the extreme fiber of wing wall in compression side (mm).	2-18
$E$ : Young's modulus of steel ( $\text{N/mm}^2$ ).	2-37
$E_c$ : Young's modulus of existing concrete ( $\text{N/mm}^2$ ).	2-47
$E_{fd}$ : Specified Young's modulus of carbon fiber sheet. A value indicated in the Table 3.3.6-1 can be used.	2-29

\*Page no. of “Japanese Guidelines for Seismic Retrofit of Existing RC Buildings, 2001”

	Page no.*
$E_c$ : Young's modulus calculated based on $\sigma_B$ . The test value can be used when measured during compression test.	2-49
$F$ : Ductility index	2-10
$F$ : Specified strength of steel (N/mm <sup>2</sup> ).	2-37
$F_c$ : Compressive strength of existing concrete (N/mm <sup>2</sup> )	2-8
$F_c$ : Specified design strength of concrete (N/mm <sup>2</sup> ).	2-10
$F_c$ : Specified compressive strength of existing concrete (N/mm <sup>2</sup> ).	2-48
$F_{cw}$ : Concrete strength of the installed wall panels (N/mm <sup>2</sup> )	2-12
$F_c$ : Specified design strength of concrete for existing structure (N/mm <sup>2</sup> ).	2-18
$F_{cl}$ : Specified concrete strength of existing structures (N/mm <sup>2</sup> ).	2-13
$F_{cl}$ : Compressive strength of concrete for existing structures (N/mm <sup>2</sup> ).	2-23
$F_{cl}$ : Specified design strength of concrete for wing wall (N/mm <sup>2</sup> ).	2-18
$f_{cr}$ : Limit compressive stress (N/mm <sup>2</sup> ).	2-37
$g$ : Distance between tensile and compressive longitudinal reinforcement of existing column (mm).	2-24
$g^2$ : $g$ for jacketing part of the column (mm).	2-24
$h'$ : Eq. (11) in the Standard.	2-18
$h'$ : Height of injected mortar (mm).	2-36
$j$ : Distance between the centroids of tension and compression forces; A value of 0.8D can be used.	2-29
$j_e$ : $7d_e/8$ (mm).	2-18
$K_{min}$ : $0.34/(0.52 + a/D)$	2-13
$l$ : Clear span of wall (mm)	2-10
$l'$ : Clear span of the installed wall panel (mm)	2-12
$l$ : Depth of drilled hole or embedment length of bonded anchor (mm).	2-47
$l_e$ : Effective embedment length of an anchor (mm).	2-47
$l_a$ : Length of expansion anchor (mm).	2-47
$l_1$ : Embedment length of expansion anchor to the existing concrete structure (mm).	2-47
$l_2$ : Exposed length of expansion anchor from the connection surface (mm).	2-47
$l_d$ : Full length of connection bar or anchorage bar (mm).	2-47
$l_n$ : Effective anchorage length of added wall (mm).	2-47
$M_u$ : Flexural strength	2-18, 23
$M/Q$ : It can be $h'$ of column on which the wing wall is installed.	2-18
$M/Q$ : It shall be obtained by detailed calculation referring to the section 3.2.2 (2) of the Standard.	2-24
$M/Q$ : 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.	2-29
$N$ : Axial force of column (N).	2-18, 23
$p_w$ : Wall reinforcement ratio	2-12
$p_g$ : Ratio of $a_g$ (gross cross section area of longitudinal reinforcing bars of a column concerned) to $b_e \cdot D$ .	2-13
$p_{we}$ : Refer to the explanation of eq.3.2.5-3	2-18
$p_w$ : Hoop ratio in the existing columns (N/mm <sup>2</sup> ).	2-18
$p_{sh}$ : Lateral reinforcement ratio of installed wing wall.	2-18
$p_{te}$ : $100a_t/(b_e \cdot d_e)$ ( $a_t$ : gross sectional area of tensile reinforcement of the column with installed wing wall).	2-18

\*Page no. of "Japanese Guidelines for Seismic Retrofit of Existing RC Buildings, 2001"

$p_{t2}$	: Tensile reinforcement ratio calculated by using the increased cross section of jacketed column (%).	2-24
$p_w$	: Shear reinforcement ratio of the existing column calculated by the increased cross section of jacketed column (decimal).	2-24
$p_{w2}$	: 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.	2-24
$p_{w2}$	: Equivalent hoop ratio of steel plate jacketing. Upper limit of total hoop ratio shall be 0.012.	2-26
$p_{w2}$	: Equivalent hoop ratio of steel plate, the same as Eq. (3.3.5-1).	2-27
$p_t$	: Tensile reinforcement ratio of existing column (%).	2-29
$p_w$	: Shear reinforcement ratio of existing column (decimal).	2-29
$p_{wf}$	: Shear reinforcement ratio of carbon fiber sheet (decimal).	2-29
$p_s$	: Reinforcement ratio.	2-36
$Q_D$	: design shear force	2-10
$Q_j$	: Sum of the shear strengths of connectors underneath the beam	2-12
$Q_c$	: Smaller value of the other column between the shear force at the yielding and shear strength	2-12
$Q_{wu}$	: Ultimate shear strength of wall calculated from Eq. (A2.1-2)	2-11
$WQ_{su}$	: Shear strength of shear walls	2-12
$WQ'_{su}$	: Shear strength of infilled shear panel (only for the panel part in the clear height and width)	2-12
$pQ_c$	: Direct shear strength at the top of a column	2-12
$WQ'_{su}$	: Shear strength of infilled shear panel	2-12
$pQ_c$	: Direct shear strength of column	2-13
$Q_{mu}$	: Shear force at flexural strength	2-18
$Q_{su}$	: Shear strength	2-18
$Q_u$	: Load carrying capacity of column with precast concrete wing wall	2-19
$Q_T$	: Shear force contributed by the diagonal compression brace which models a wing wall as shown in Figure 3.2.5-2	2-19
$Q_c$	: Shear force contributed by the existing columns	2-19
$Q_{su}$	: Shear strength of column	2-24
$Q_{su}$	: Shear strength of column	2-29
$Q_{su}$	: Shear strength of beam.	2-40
$Q_{mu}$	: Shear force at the flexural capacity of beam.	2-40
$Q_a$	: Shear capacity of an anchor (N).	2-48
$Q_{a1}$	: Shear capacity of an anchor determined by steel strength (N).	2-48
$Q_{a2}$	: Shear capacity of an anchor determined by bearing strength of concrete (N).	2-48
$q_{ds}$	: Shear strength contributed by each stud ((N) for one stud).	2-37
$S_D$	: Irregularity index	2-8
$T_a$	: Tensile capacity of an anchor (N).	2-47
$T_{a1}$	: Tensile capacity of an anchor determined by yielding of steel material (N).	2-47
$T_{a2}$	: Tensile capacity of an anchor determined by concrete cone failure (N).	2-47
$T_{a3}$	: Tensile capacity of an anchor determined by bond failure (N).	2-47
$t$	: Wall thickness of installed wing wall (mm)	2-18
$t$	: Thickness of steel plate.	2-26
$t_W$	: Wall thickness (mm)	2-10, 12
$X_s$	: Interval of reinforcement (mm).	2-36

\*Page no. of "Japanese Guidelines for Seismic Retrofit of Existing RC Buildings, 2001"

	Page no.*
$x_s$ : Spacing of steel strap	2-26
$\alpha$ : Reduction factor in consideration of the deflection condition to allow for load bearing contribution of column(s).	2-12
$\alpha$ : Safety factor for flexural failure.	2-40
$\alpha_e$ : Refer to the explanation of eq. 3.2.5-2.	2-18
$\beta$ : 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).	2-11
$\Phi$ : Reduction factor (= 0.8).	2-18
$\eta H$ : Limit of axial force ratio of column after jacketing.	2-27
$\eta H_o$ : Limit of axial force ratio of column before jacketing, 0.5 for 100 mm or less in hoop spacing, 0.4 for others.	2-27
$\tau W$ : Average shear stress of wall panel (N/mm <sup>2</sup> )	2-10
$\tau$ : Average shear stress of wall (N/mm <sup>2</sup> )	2-8
$\tau D$ : Values provided in Table 3.1.5-1	2-10
$A$ : Limit slenderness ratio $A = \sqrt{(\pi^2 \cdot E) / (0.6F)}$	2-37
$\lambda$ : Effective aspect ratio.	2-37
$\varepsilon_{fd}$ : Effective strain of carbon fiber sheet at shear failure. A value of 0.7% can be used.	2-29
$\sigma_y$ : Yield strength of longitudinal reinforcing bars of a column.	2-13
$\sigma_o$ : $N/(eb \cdot D)$ , where $N$ is an axial force of the column at ultimate mechanism, positive value means compression force.	2-13
$\sigma_{sy}$ : Yield strength (N/mm <sup>2</sup> ) of lateral reinforcement of installed wing wall.	2-18
$\sigma_y$ : Yield strength of longitudinal reinforcing bars of column (N/mm <sup>2</sup> ).	2-18
$\sigma_{oe}$ : Refer to the explanation of eq. 3.2.5-3	2-18
$\sigma$ : $p_g \cdot \sigma_y \cdot \sigma_o$	2-13
$\sigma_y$ : Yield strength of the wall reinforcing bar (N/mm <sup>2</sup> )	2-12
$\sigma_y$ : Yield strength of reinforcing bar (N/mm <sup>2</sup> ). The strength $\sigma_y$ shall be 294 N/mm <sup>2</sup> for round bars, and (specified yield strength + 49 N/mm <sup>2</sup> ) for deformed bars.	2-23
$\sigma_{y2}$ : Yield strength of tensile reinforcement in the jacketing part of column (N/mm <sup>2</sup> ). The strength $\sigma_{y2}$ shall be 294 N/mm <sup>2</sup> for round bars, and (specified yield strength + 49 N/mm <sup>2</sup> ) for deformed bars.	2-24
$\sigma_{wy}$ : Yield strength of shear reinforcement in the existing column (N/mm <sup>2</sup> ).	2-24
$\sigma_{wy2}$ : Yield strength of shear reinforcement in the jacketing column (N/mm <sup>2</sup> ). The strength $f_{wy}$ and $f_{wy2}$ shall be 294 N/mm <sup>2</sup> for round bars, and (specified yield strength + 49 N/mm <sup>2</sup> ) for deformed bars.	2-24
$\sigma_{wy2}$ : Yield strength of steel plate for jacketing (N/mm <sup>2</sup> ).	2-27
$\sigma_{wy}$ : Yield strength of shear reinforcement of existing column (N/mm <sup>2</sup> ).	2-29
$\sigma_{fd}$ : $\min \{E_{fd} \cdot \varepsilon_{fd}, (2/3) \cdot \sigma_f\}$ , tensile strength of carbon fiber sheet for shear design.	2-29
$\sigma_f$ : Specified tensile strength of carbon fiber sheet. A value indicated in Table 3.3.6-1 can be used.	2-29
$\sigma_o$ : Axial compressive stress. The value shall not be more than 7.8 N/mm <sup>2</sup> .	2-29
$\sigma_{max}$ : Tensile strength of stud, equal to or less than 400 (N/mm <sup>2</sup> ).	2-37
$\sigma_B$ : Compressive strength of existing concrete (N/mm <sup>2</sup> ).	2-47
$\sigma_y$ : Specified yield strength of steel bar (N/mm <sup>2</sup> ).	2-48
$m\sigma_y$ : Yield strength of expansion anchor (N/mm <sup>2</sup> ).	2-48
$\sigma_B$ : 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 $F_c$ , $\sigma_B$ shall be determined according to the Standard.	2-49

\*Page no. of "Japanese Guidelines for Seismic Retrofit of Existing RC Buildings, 2001"

		Page no. *
$\tau_o$	: Refer to the explanation of eq. 3.1.5-5	2-13
$\tau$	: Shear strength of anchor (N/mm <sup>2</sup> ).	2-48
$\tau_a$	: Bond strength of bonded anchor against pull-out force (N/mm <sup>2</sup> ).	2-48
$\tau_o$	: Basic bond strength of bonded anchor (N/mm <sup>2</sup> ).	2-48
<b>*Page no. of “Japanese Guidelines for Seismic Retrofit of Existing RC Buildings, 2001”</b>		

## NOTATION B.

		Page no. **
$A_c$	: Total cross-sectional area of columns (mm <sup>2</sup> ) in the storey concerned, where the areas of boundary columns in the walls with one or two boundary columns shall be neglected in calculation.	1-16
$a_g$	: Total cross sectional area of reinforcing bars (mm <sup>2</sup> ).	1-44
$a_h$	: Cross sectional area of a pair of the lateral reinforcement in shear wall	1-51
$A_i$	: Vertical distribution shape of lateral seismic force.	1-75
$A_{SC}$	: Total cross-sectional area of extremely short columns in the story concerned (mm <sup>2</sup> ).	1-16
$a_t$	: Total cross sectional area of tensile reinforcing bars in column (mm <sup>2</sup> ).	1-43
$a_t$	: Cross sectional area of tensile reinforcing bars of the boundary column in the tension side of wall.	1-50 1-51
$a_t$	: Cross sectional area of tensile reinforcing bars in the beam (mm <sup>2</sup> ).	1-57
$a_t$	: Cross sectional area of tensile reinforcing bars in the beam in case that the partial slit is in compression side (mm <sup>2</sup> ).	1-58
$\Sigma a_{wv}$	: Total vertical reinforcing bars in the shear wall (mm <sup>2</sup> ).	1-50
$A_{W1}$	: Total cross-sectional area of walls with two boundary columns in the storey and effective to the direction concerned (mm <sup>2</sup> ).	1-15
$A_{W2}$	: Total cross-sectional area of walls with one boundary column in the storey and effective to the direction concerned (mm <sup>2</sup> ).	1-16
$A_{W3}$	: Total cross-sectional area of walls without columns in the storey and effective to the storey concerned (mm <sup>2</sup> ).	1-16
$\Sigma A$	: Sum of cross sectional areas of column and wing wall and wall (mm <sup>2</sup> ).	1-51, 55, 56
$b$	: Column and Beam width (mm).	1-44, 59
$b_e$	: Equivalent thickness of the wall (mm),	1-51
$b_e$	: Beam width of the equivalent rectangular shaped beam.. (mm).	1-60
${}_bF_i$	: Ductility index of the beam on the left and the right sides of the node calculated according to the item (d).	1-24
${}_bM$	: Contribution of the boundary beam to the overturning moment resistance of the wall at the level of story concerned.	1-23
${}_bM_{ui}$	: Nodal moment at the ultimate strengths of the beams on the left and the right sides of the node.	1-24
${}_bQ_{su}$	: Shear strength of the beam	1-24

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	Page no.**
${}_bQ_{mu}$ : Shear force at the flexural failure of the beam, considering the effect of the shear force $Q_o$ due to gravity load.	1-24
$C_o$ : Base shear coefficient (greater than 1.0).	1-75
$C_1$ : Strength index $C$ of the first group (with small $F$ index).	1-12
$C_2$ : Strength index $C$ of the second group (with medium $F$ index).	1-12
$C_3$ : Strength index $C$ of the third group (with large $F$ index).	1-12
$C_c$ : Strength index of the columns, except for the extremely short columns.	1-10
${}_cQ_{mu}$ : Shear force at the ultimate flexural strength of the column.	1-45, 47, 73
${}_cQ_{su}$ : Ultimate shear strength of the column.	1-45, 47, 73
${}_cR_{30}$ : Standard drift angle of the column (measured in the clear height of column), 1/30.	1-45
${}_cR_{50}$ : Standard drift angle of the column (measured in the clear height of column), 1/50.	1-47
${}_cR_{150}$ : Standard drift angle of the column (measured in the clear height of column), 1/150.	1-48
${}_cR_{250}$ : Standard drift angle of the column (measured in the clear height of column), 1/250.	1-47
${}_cR_{mp}$ : Plastic drift angle of the column (measured in the clear height of column).	1-45
${}_cR_{mu}$ : Drift angle at the ultimate flexural strength of column (measured in the clear height of column).	1-45
${}_cR_{my}$ : Yield drift angle of column (measured in clear height of column).	1-45, 48
$C_{SC}$ : Strength index of the extremely short columns.	1-10
$C_{TU}$ : Cumulative strength index at the ultimate deformation of structure.	1-40
$C_W$ : Strength index of the walls.	1-10
${}_c\alpha$ : Effective strength factor of the column.	1-47
${}_c\varepsilon_B$ : Compressive strain at the concrete strength.	1-58
${}_c\tau_{mu}$ : Shearing unit stress at the flexural strength of column.	1-73
${}_c\tau_u$ : Shearing unit stress at the ultimate state of columns..	1-47
$D$ : Column and Beam depth.	1-9
$D_c$ : Column depth.	1-17
$D_s$ : Deformability and damping factor of structure.	1-75
$d$ : Effective depth of column and beam.	1-44, 56
$d_b$ : Diameter of the flexural reinforcing bar of the column.	1-47
$d_e$ : Distance from the center of the tensile reinforcing bars to the extreme fiber of the wing /standing/hanging wall in the compressive side (mm).	1-55, 58, 59
$F_{es}$ : Shape factor to take the effect of vertical stiffness unbalance and eccentricity into account.	1-75
$F_c$ : Compressive strength of concrete (N/mm <sup>2</sup> ), which may be taken as the specified design concrete strength	1-16, 44
$F_{SC}$ : Ductility index of the extremely short columns	1-10
$F_W$ : Ductility index of the walls	1-10
$G$ : Ground index	1-39
$h$ : Storey height.	1-52
$h_0$ : Clear height of column.	1-9,2 1,53

**\*\*Page no. of "Japanese Standard for Seismic Evaluation of Existing RC Buildings, 2001"**

$H_o$	: Standard height of the column from the bottom of the upper floor beam to the surface of the lower floor slab.	1-21, 48
$h'$	: The height from the floor level concerned to the top of the beam whose flexural reinforcement is counted into $\sum a_{lg}$ .	1-51
$h_{CO}$	: Inflection height calculated for columns.	1-17, 53
$h_{CW0}$	: Inflection height calculated for walls.	1-56
$h_i$	: Opening height.	1-52
$h_s$	: Standing or hanging wall height (mm).	1-58
$h_W$	: Height from the floor level concerned to the top of the multi-story wall.	1-53
$h_{W0}$	: Inflection height calculated as walls with two boundary columns.	1-18, 53
$i$	: Number of the storey for evaluation, where the first story is numbered as 1 and the top storey as $n$ .	1-10
$j$	: Distance between centroids of tension and compression forces.	1-44, 47, 59, 73
$j_e$	: Distance between the centroids of the tension and compression portions.	1-51, 55, 60
$L$	: Standard or averaged length of spans in the direction concerned.	1-17, 49, 53
$L'$	: Total length including length of wing walls.	1-53, 55
$L_j$	: Wall length in unit portion.	1-35
$L_W$	: Length of the wing wall (total length of the wing walls in case they locate at both sides of a column).	1-17, 49
$l$	: Total wall length including length of columns.	1-51
$l_i$	: Opening length.	1-52
$l_W$	: Distance between the centers of the boundary columns of the wall (mm).	1-50
$M/Q$	: Shear span length.	1-44, 51, 59
$N$	: Number of the inspected stories.	1-30
$N$	: Axial force (N).	1-43
$N$	: Total axial force in the boundary columns attached to the wall.	1-50
$n$	: Number of stories of a building.	1-10
${}_nF_b$	: Ductility index of the node determined from the beams.	1-23
${}_nF_c$	: Ductility index of the column above and below the node.	1-23
${}_nF_i$	: Ductility index of the node at the top or the bottom of the column.	1-23
${}_nM_{ui}$	: Nodal moment at the top or the bottom of the column at the failure mechanism.	1-23
$N_{max}$	: Axial compressive strength.	1-43
$N_{min}$	: Axial tensile strength.	1-43
$N_s$	: Additional axial force of column due to earthquakes.	1-46, 73
$p_s$	: Shear reinforcement ratio of the wall.	1-60

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	Page No. **
$P_{se}$ : Equivalent lateral reinforcement ratio of wall.	1-51
$p_{sh}$ : Horizontal shear reinforcement ratio of the wing wall.	1-55
$p_t$ : Tensile reinforcement ratio (%).	1-44, 59
$p_{te}$ : Equivalent tensile reinforcement ratio of wall (%).	1-51, 55
$p_w$ : Shear reinforcement ratio.	1-44, 55, 59, 60
$Q_{(F1)}$ : Shear force at the deformation capacity $R_1$ of a column in the second and higher groups.	1-13
$Q_{mu}$ : Shear force at flexural yielding of a column in the second and higher groups.	1-13
$Q_{su}$ : Shear strength of a column in the second and higher groups.	1-13
$Q_u$ : Ultimate lateral load-carrying capacity of the vertical members in the storey concerned.	1-18
$Q_{ud}$ : Seismic demand force for each storey.	1-75
$Q_{un}$ : Calculated capacity of structure.	1-75
$R_{250}$ : Standard inter-storey drift angle, $R_{250}=1/250$ .	1-20, 45
$R_{mu}$ : Inter-storey drift angle at the ultimate deformation capacity in flexural failure of the column member.	1-21
$R_{my}$ : Yield inter-storey drift angle.	1-13, 47
$R_{su}$ : Inter-storey drift angle at the ultimate deformation capacity in shear failure of the column member.	1-13, 20
$R_t$ : Coefficient for response in term of period and soil condition.	1-75
$R_y$ : Yield deformation in terms of inter-storey deformation angle.	1-20
$s$ : Spacing of hoops/ties	1-45
$s\sigma_{wy}$ : Yield strength of shear reinforcing-bars ( $N/mm^2$ ).	1-44
$s\varepsilon_y$ : Yield strain of the flexural reinforcing-bar in the beam.	1-58
$t$ : Wall thickness of wing wall in the compression side (mm).	1-58
$t_s$ : Remaining concrete thickness of the partial slit (mm).	1-58
$U$ : Usage index.	1-39
$W$ : Total weight of the story and above.	1-75
${}_wM$ : Moment resistance of the wall at the level of the storey concerned.	1-23
${}_wQ_{mu}$ : Shear force at flexural strength of the wall.	1-20, 22, 72
${}_wQ_{ru}$ : Shear force at uplift strength of the wall.	1-22
${}_wQ_{su}$ : Ultimate shear strength of the wall.	1-20, 22
$Z$ : Zone index.	1-39, 75
$\alpha_1$ : Effective strength factor of the columns at the ultimate deformation of the walls.	1-10
$\alpha_2$ : Effective strength factor of the walls at the ultimate deformation of the extremely short columns.	1-10
$\alpha_3$ : Effective strength factor of the columns at the ultimate deformation of the extremely short columns.	1-10
$\alpha_j$ : Effective strength factor in the $j$ -the group at the ultimate deformation $R_1$ corresponding to the first group (ductility index of $F_1$ ).	1-12

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	Page No. **
$\alpha_m$ : Effective strength factor of a flexural column.	1-13
$\alpha_s$ : Effective strength factor of a shear column.	1-13
$\beta$ : Wing wall length in compressive side divided by D.	1-55
$\gamma$ : Factor on the precision in calculation of the uplift strength of the wall.	1-22
$\mu$ : Ductility factor.	1-72
$\Sigma A$ : Cross sectional area of the wall with column.	1-51, 55
$\Sigma_c M_{ui}$ : Sum of the nodal moments at the ultimate strengths of the columns in the upper and the lower stories.	1-23
$\Sigma_b M_{ui}$ : Sum of the nodal moments at the ultimate strengths of the beams on the left and the right sides.	1-23
$\Sigma A_f$ : Total floor area supported by the story concerned ( $m^2$ ).	1-16
$\Sigma W$ : Total weight (dead load plus live load for seismic calculation) supported by the storey concerned.	1-16, 18
$\sigma_B$ : Compressive strength of concrete for evaluation	
$\sigma_{sy}$ : Yield strength of horizontal shear reinforcing-bars in the wing wall ( $N/mm^2$ ).	1-50, 51,5 5
$\sigma_y$ : Yield strength of tensile reinforcing bars ( $N/mm^2$ ).	1-44, 57,5 8
$\sigma_{wy}$ : Yield strength of shear reinforcing-bars in the column ( $N/mm^2$ ).	1-55, 50, 59
$\sigma_0$ : Axial stress in column ( $N/mm^2$ ).	1-44
$\sigma_{0e}$ : Axial stress in wall.	1-51, 55
$\tau_c$ : Shearing unit stress at the ultimate state of columns.	1-15
$\tau_{SC}$ : Shearing unit stress at the ultimate state of extremely short columns.	1-15
$\tau_{w1}$ : Shearing unit stress at the ultimate state of walls with two boundary columns.	1-15
$\tau_{w2}$ : Shearing unit stress at the e ultimate state of walls with two boundary columns.	1-15
$\tau_{w3}$ : Shearing unit stress at the ultimate state of walls without columns.	1-15

**\*\*Page no. of "Japanese Standard for Seismic Evaluation of Existing RC Buildings, 2001"**



## CHAPTER 1. GENERAL

### 1.1 GENERAL POLICY

#### 1.1.1 Scope of the Manual

This manual is for seismic retrofit design of existing Reinforced Concrete (RC) buildings. This manual has been prepared to supplement the following standard and guidelines, incorporating the characteristics of RC buildings in Bangladesh and seismic design load of Bangladesh National Building Code (BNBC).

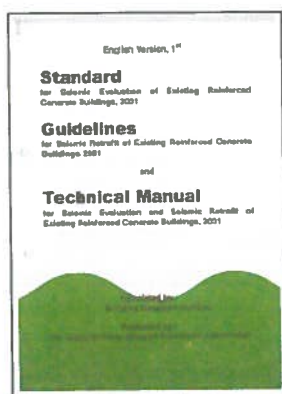
For items not covered in this manual, refer to following standard and guidelines.

(1) “*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*” translated by the Building Research Institute, Japan and published by Japanese Building Disaster Prevention Association.

For seismic evaluation of existing RC buildings, refer to,

(2) *Manual for Seismic Evaluation of Existing RC Buildings by CNCRP*

The guidelines for seismic evaluation and retrofit design of existing RC buildings are not covered in the BNBC (Bangladesh National Building Code) 1993. BNBC 2015 also does not cover. This manual has been introduced for seismic retrofit design of existing RC buildings. This manual has been prepared to supplement the Japanese Standard and Guidelines mentioned above. This Japanese Standard and Guidelines written in English is based on original Japanese Standard and Guidelines, translator’s note, and the lists of the references.



(a) English Version (2001)



(b) Guidelines for Seismic Retrofit Design (2001) in Japanese



(c) Standard of Seismic Evaluation (2001) in Japanese

**Figure 1.1.1 Japanese Standard and Guidelines for Seismic Assessment and Retrofit Design of RC Buildings.**

### 1.1.2 Definition of Technical Terms

Refer to the Section 1.3 Definitions for the terminology of the Japanese Standard.

### 1.1.3 Level of Screening of Seismic Evaluation

The degree of simplification in calculating the indices  $I_s$  (seismic index) of structures, three screening levels are provided from the first level (simple level) to the third level (detailed level) of screening. In this manual, 2<sup>nd</sup> level screening procedure is mainly used and is recommended for the retrofit design.

### 1.1.4 Outline of Seismic Evaluation and Seismic Retrofit Design

**(1) Concept:** Seismic Evaluation is a method by which seismic capacity of an existing RC building is evaluated. This evaluation could be done by manual calculation. Seismic Index of structure  $I_s$  shows its seismic performance level.

#### (2) Levels of seismic screening

There are 3 levels of seismic screening method,

- **1<sup>st</sup> level screening** is a simple calculation for the seismic capacity.
- **2<sup>nd</sup> level screening**, column collapse mechanism is usually assumed.

It is noted that column sizes of buildings in Bangladesh are small and concrete volume of beam column joints are also small compared with those of Japan. This may cause the collapse of beam-column joint preceding the collapse of column. Careful investigation will be required for the assumption of column collapse in Bangladesh.

- **3<sup>rd</sup> level screening**, beam and column collapse mechanism is considered, but calculation is too complex.

#### (3) Methodology

- a) Building Survey
- b) Classification of column (flexural or shear failure column, refer to table 1.5.2 of section 1.5) and walls
- c) Grouping of columns and walls
- d) The seismic index of structure  $I_s$  shall be calculated by following equation at each storey 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.

$$I_s = E_o \cdot S_D \cdot T$$

Where:

$E_o$  = Basic seismic index of structure.

$S_D$  = Irregularity index.

$T$  = Time index.

- e) Seismic Demand Index of Structure  $I_{so}$

- f) Judgment

Seismic Index of Structure  $I_s$  is compared with Seismic Demand Index of Structure  $I_{so}$ .

If  $I_s \geq I_{so}$  then the seismic performance of the building is satisfactory.

In case of Seismic Retrofit Design, concept of retrofit design is studied first considering the strength and ductility of the existing structure. Structural strengthening elements such as column jacketing, RC wing wall, RC shear wall, steel brace frame and others are provided so that  $I_s$  after retrofit exceeds  $I_{so}$ .

g)  $C \cdot F$  relation curve

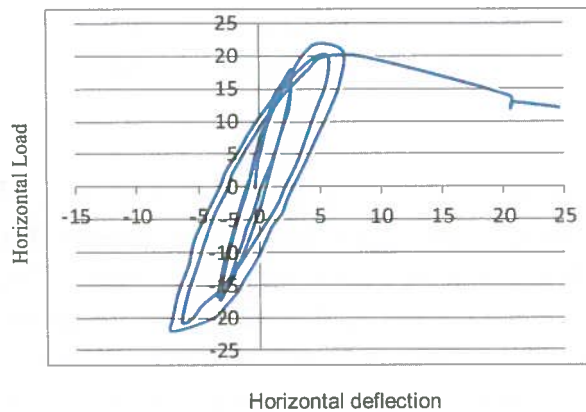
Seismic index of structure  $I_s$  shows seismic performance of a building.  $I_s$  is proportional to  $C \cdot F$  [strength index ( $C$ )  $\times$  ductility index ( $F$ )].

$$I_s \propto C \cdot F$$

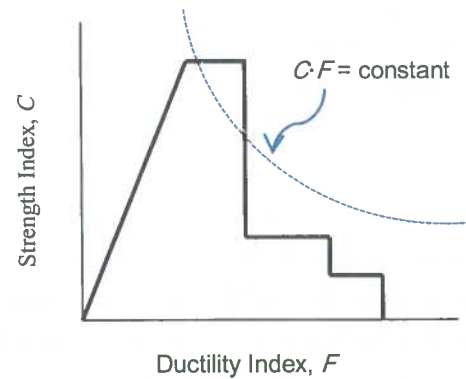
The equation of " $C \cdot F = \text{constant}$ " is expressed by a hyperbolic curve in the  $C \cdot F$  relation figure, so  $I_{so}$  is also expressed by a hyperbolic curve.

Then  $C \cdot F$  relation is compared with  $I_{so}$  or  $\frac{I_{so}}{S_d \cdot T}$

$I_s$  is evaluated for each direction and each storey. Strength and ductility is evaluated for each vertical member, in case of 2<sup>nd</sup> level screening. Then  $C \cdot F$  relation of a floor in each direction can be prepared through the summation of all vertical members of that floor. Static load-deflection curve of a frame is shown in Figure 1.1.2 (a), this is an example of a structural test result. In case of seismic evaluation and retrofit design, simplified multi-linear lines express the performance of a building. Vertical axis  $C$  and horizontal axis  $F$  is non-dimensional.



(a) Static repeated load-deflection curve of a frame  
(An experiment of a retrofitted frame)



(b) Strength index and Ductility index relation expressed by multi-linear lines (Retrofit design)

**Figure 1.1.2 Difference of Expression Showing the Behavior of Frames**

## 1.2 SCOPE OF APPLICATION

Following scope of application for existing RC buildings is covered in this manual:

1) Type of structure

Reinforced concrete (RC) frame structure is covered by this manual. Flat plate (slab) RC structure is out of scope in principle.

2) Number of stories

Mid to low-rise buildings with 6 storeys and less are covered generally.

3) Concrete strength

Buildings with concrete strength not less than 9.0 N/mm<sup>2</sup> are covered.

Note: Brick wall masonry building is out of scope and is not covered by this manual.

### 1.2.1 Type of Structure

This manual covers the seismic retrofit of existing RC frame structure with in-filled brick walls mainly. Other structures such as steel structure and brick masonry structure are not covered by this manual.

#### a) Flat plate (slab) structure

There are a lot of RC flat plate buildings of without RC core walls. It will be a vulnerable structure, since horizontal stiffness and strength is low. Proper layout of seismic resistant elements in plan and in elevation shall be considered, and floor slabs shall be evaluated to transfer seismic load. Retrofit design of flat plate structure is out of scope in principle, and careful evaluation and retrofit design is required separately.

#### b) Building without floor beams partially and without grade beams

There are existing RC buildings without floor beams partially and without grade beams. Horizontal strength of a column without floor beams at top and bottom will be almost zero. Horizontal load carrying capacity of a building shall be reduced incorporating this condition. In case of buildings without grade beams, it is required to evaluate effective column length at ground floor level considering connection condition to foundation footings for evaluation of column strength.

### 1.2.2 Number of Stories

Applicable limit of number of storey is 6 generally. In case of higher storey buildings, flexural behavior or axial deflection of columns increases. It will be allowed to apply higher buildings by incorporating the change of axial force of columns by seismic load. In this manual, applied maximum number of storey will be 6 storied generally, and this condition will cover most of exiting RC buildings in Bangladesh. In case of a building with 7 stories and above, flexural deflection behavior of a building against seismic load is to be evaluated and axial force change is to be considered when evaluating flexural strength of columns.

### 1.2.3 Concrete Strength

Brick chips aggregate have been used for concrete work of existing old RC buildings in Bangladesh. The concrete strength with brick chips is low, and it has been observed that actual concrete strength in some cases are found to be less than  $9\text{N/mm}^2$ , according to the result of concrete core sampling test. It will be defined that concrete strength with not more than  $13.5\text{N/mm}^2$  is low strength concrete. Low strength concrete is not covered by the “Japanese Seismic Evaluation and Retrofit Design Guideline” generally. However there is “A report by the special research committee for low strength concrete” issued by the Chugoku branch of Japan Concrete Institute in February 2009. This special report covers low strength concrete up to  $9.0\text{N/mm}^2$ . It suggests providing reduction coefficient for low strength concrete to evaluate shear strength of columns. Buildings with low strength concrete less than  $9\text{N/mm}^2$  are out of scope in this manual. It will be required to evaluate separately in detail for buildings with concrete strength less than  $9.0\text{N/mm}^2$ .

## 1.3 DEFINITIONS

The terminology used in this manual, unless specified otherwise, conform to the “Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001 (the Japanese Standard).”

The following are the terminologies generally used in the manual, along with definitions. Special terminology related to post-installed anchors is defined separately in Section 3.5.

### 1.3.1 Terminology Relating to Retrofit and Restoration

#### **Repair**

Restoring structural performance to its pre-damage state by mending the damaged parts of a building

#### **Strengthening**

Improving structural performance by reinforcing the weak or lacking parts. Herein, this generally refers to improvement of seismic performance by any means. For buildings damaged by an earthquake, this denotes elevating seismic performance above the pre-damage level.

#### **Restoration**

Refurbishing damaged buildings and parts thereof by repairing and strengthening to make them fit to be used again.

#### **Retrofit**

General term for restoration, strengthening, and repair.

### 1.3.2 Terminology Relating to Seismic Evaluation

#### **Seismic Index of Structure $I_s$**

An index representing the seismic performance of a structure in continuous quantity; the greater the value, the higher the seismic performance

#### **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 Index of Structure $E_o$**

An index representing the basic seismic performance of a building, evaluated as a product (multiplication) of the strength and the deformation performance.

#### **Strength index $C$**

An index representing the sum of the horizontal lateral-load carrying capacity of vertical members like columns and RC walls, divided by the weight of a building above the appropriate storey.

#### **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  $E_o$  in consideration of unbalance in stiffness distribution and/or irregularity in structural plan and elevation of a building. This is evaluated using a demerit point system with a perfect point of 1.0 for a general building and 1.2 for a building with one or more basement.

#### **Time Index $T$**

An index evaluating the effect of structural defects such as cracking, deformation, and aging on the seismic performance of a building. This is evaluated using a demerit point system with a perfect point of 1.0.

### 1.3.3 Terminology Relating to Strengthening (Retrofit) Methods

#### **Steel Bracing**

A structure that combines existing RC frame and new steel members by strengthening the existing RC frames with steel braces or steel panels.

#### **Strengthening joint**

A boundary portion that RC frames and new steel member combined to transfer the strength. A joint is either directly joined using such means as bolts and welding or indirectly joined using grout mortar in order to realize integration.

#### **Insertion of RC wall**

A general term for seismic walls that are shaped by cast-in-place concrete within existing frames or the addition of precast concrete walls.

### 1.3.4 Terminology Relating to Strengthening (Retrofit) Construction

#### Post-installed anchor method

A general term for a method that drills into the existing concrete to add seismic walls or wing walls and embed anchor bars or anchors themselves. Types include the post-installed adhesive (chemical) anchor method and the post-installed metal anchor method.

#### Post-installed adhesive (chemical) anchor method

A type of post-installed anchor method that attaches and bonds resin such as epoxy-acrylate resin to the drilled holes of the existing concrete in order to fix anchor bars.

#### Post-installed metal anchor method

A type of post-installed method that drills into existing concrete and fixes the metal anchor by means of an expansion mechanism.

#### Split preventing re-bar

A reinforcing bar that resists the fracture of concrete of the inserted walls along the anchor bars or anchors them. Spiral-shaped, hoop-shaped, and ladder-shaped bars are used.

### 1.3.5 Main Notations Related to Seismic Evaluation are as Follows.

$C$	: strength index	$M_u$	: ultimate flexural strength
$C_T$	: cumulative strength index	$Q_{su}$	: ultimate shear strength
$C_{TU}$	: cumulative strength index of ultimate state	$Q_{mu}$	: ultimate shear of flexural yielding
$E_s$	: basic seismic demand index of structure	$R$	: storey drift angle
$E_o$	: basic seismic index of structure	$R_{mp}$	: plastic story drift angle of flexural yielding
$F$	: ductility index	$R_{mu}$	: ultimate story drift angle of flexural yielding
$I_s$	: seismic index of structure	$R_{my}$	: storey drift angle of flexural yielding
$I_N$	: seismic index of non-structural elements	$R_{su}$	: ultimate story drift angle of shear failure
$I_{so}$	: seismic demand index of structure	$R_y$	: storey drift angle of yielding
$S_D$	: irregularity index (configuration factor)	$R_{150}$	: standard storey drift angle for $F = 1.27$
$T$	: time index (deterioration factor)	$R_{250}$	: standard storey drift angle for $F = 1.0$
$U$	: usage index (importance factor)	$R_{30}$	: standard storey drift angle for $F = 3.2$
$Z$	: zone index	$R_{50}$	: standard storey drift angle for $F = 2.6$
		$\alpha$	: effective strength factor

## 1.4 CONCEPT OF RETROFIT DESIGN

### 1.4.1 General

There are several basic retrofit design methods for existing RC buildings;

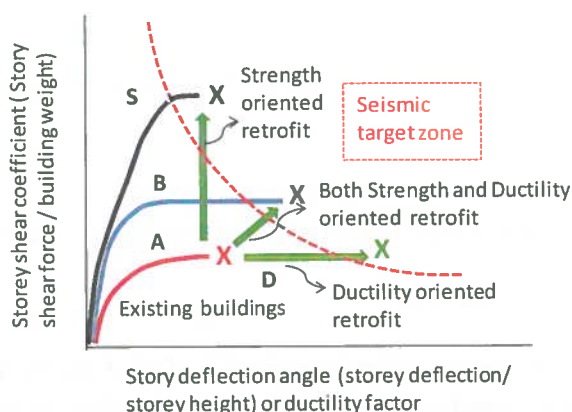
1. Strength oriented retrofit method
2. Ductility oriented retrofit method
3. Both strength and ductility retrofit method
4. Modification of irregularity
5. Reduction of building weight

It is suggested to select the method incorporating the characteristics of existing building and the concept of retrofit design. Classification of seismic improvement by retrofit method is shown in Chapter 2.1.

The concept of retrofit design of an existing RC building is shown in Figure 1.4.1. Vertical axis is horizontal strength at ground floor divided by building weight, which is base shear coefficient, and the value is non dimensional. Horizontal axis is storey deflection angle (ductility factor), which is storey deflection divided by storey height, and the value is non dimensional. The curve A of the Figure 1.4.1 is a typical existing RC building, strength and ductility is not enough. There are three retrofit methods which are as follows.

1. Strength oriented retrofit method (curve S).
2. Ductility oriented retrofit method (curve D).
3. Both strength and ductility retrofit method (curve B).

Right upper side of hyperbolic curve of the Figure is expressed as “Seismic target Zone”, which is Target Zone. In case the curve of a building reaches the target zone, then it is judged that the building is acceptable.



**Figure 1.4.1 Load and Deflection Curves and Concept of Retrofit**

In this manual, retrofit method such as base isolation system that reduces seismic load is not covered. A concept of this retrofit is shown at Supplement B7 for information only.

## 1.5 SEISMIC INDEX OF STRUCTURE $I_s$

### 1.5.1 General

As shown in the Standard, Seismic Index of Structure  $I_s$  shall be calculated by Equation (1) at each storey and in each horizontal direction of a building.

$$I_s = E_o \cdot S_D \cdot T \quad (1) \text{ of the J. Standard}$$

Where:

$E_o$  = Basic seismic index of structure (defined in 3.2 of the J. Standard)

$S_D$  = Irregularity index (defined in 3.3 of the J. Standard)

$T$  = Time index (defined in 3.4 of the J. Standard)

Seismic Index of Structure  $I_s$  is expressed by Equation (1).  $S_D$  and  $T$  is 1.0 generally.  $I_s$  is equal to  $E_o$ , in case  $S_D$  and  $T$  is 1.0.

**(1) 1<sup>st</sup> Level Screening****Table 1.5.1 Classification of Vertical Members in the First Level Screening Procedure (From Japanese Standard)**

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
RC wall	RC walls including those without boundary columns

Note:  $h_o$  : Column clear height $D$  : Column depth**Basic Seismic Index of Structure**

$$E_o = \frac{n+1}{n+i} (C_w + \alpha_1 \cdot C_c) \cdot F_w \quad (2) \text{ of the J. Standard}$$

$$E_o = \frac{n+1}{n+i} (C_{sc} + \alpha_2 C_w + \alpha_3 C_c) \cdot F_{sc} \quad (3) \text{ of the J. Standard}$$

Where;

 $n$  = Number of stories of a building. $i$  = Number of the storey for evaluation, where the first storey is numbered as 1 and the top storey as  $n$ . $\frac{n+1}{n+i}$  = The storey-shear modification factor. $C_w$  = Strength index of the RC walls, calculated by Equation (7) of the J. standard. $C_c$  = Strength index of the columns, calculated by Eq. (8) of the J. standard, except for the extremely short columns. $C_{sc}$  = Strength index of the extremely short columns, calculated by Equation (9) of the Japanese standard. $\alpha_1$  = 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  $C_w \cong 0$ . $\alpha_2$  = Effective strength factor of the walls at the ultimate deformation of the extremely short columns, which may be taken as 0.7. $\alpha_3$  = 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 RC 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 extremely short columns, which may be taken as 0.8.**(2) 2<sup>nd</sup> Level Screening****Table 1.5.2 Classification of Vertical Members Based on Failure Modes in the 2nd Level Screening Procedure from the Japanese Standard**

Vertical member	Definition
RC shear wall	RC walls whose shear failure precede flexural yielding
RC flexural wall	RC 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  $\alpha_j$  may be taken as given in Table 3 of the Japanese Standard. (a) Ductility-dominant basic seismic index of structure Equation (4),

$$E_o = \frac{n+1}{n+i} \sqrt{E_1^2 + E_2^2 + E_3^2} \quad (4) \text{ of the J. Standard}$$

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 Equation (5), of the J. Standard

$$E_o = \frac{n+1}{n+i} \left[ C_1 + \sum_j \alpha_j C_j \right] \cdot F_1 \quad (5) \text{ of the J. Standard}$$

Where

$n$  = Number of stories of a building.

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

$\frac{n+1}{n+i}$  = Storey shear modification factor.

$\alpha_j$  = Effective strength factor in the  $j$ -the group at the ultimate deformation  $R_i$  corresponding to the first group (ductility index of  $F_1$ ), given in Table 3 of the J. Standard.

Followings are the notes of Table 3 of the J. Standard.

$\alpha_s$  = Effective strength factor of a shear column, calculated by

$$\alpha_s = \frac{Q_{(F1)}}{Q_{SU}} = \alpha_m \times \frac{Q_{mu}}{Q_{SU}} \leq 1.0$$

$\alpha_m$  = Effective strength factor of a flexural column, calculated by

$$\alpha_m = \frac{Q_{(F1)}}{Q_{mu}} = 0.3 + 0.7 \times \frac{R_1}{R_{my}}$$

$R_{my}$  = Drift angle at flexural yielding, calculated by Equation (A1.3-1) in the Supplementary provisions 1 of the Japanese Standard. Generally  $R_{my} = {}_cR_{my} = {}_cR_{150}$ , standard drift angle of the column (measured in the clear height of column), 1/150

$R_{su}$  = Drift angle at shear strength, calculated by Equation (A1.2-11) in the Supplementary provisions 1 of the Japanese Standard. Generally  $R_{su} = R_{250}$ , standard inter-drift angle,  $\frac{1}{250}$ .

$Q_{(F1)}$  = 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) of the Standard.

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

The basic seismic index of structure  $E_o$  shall be taken as the larger one from Equation (4) and (5).

**(c) Calculation of strength index  $C$**

$$C = \frac{Q_u}{\Sigma W} \quad (12) \text{ of the Japanese standard}$$

Where:

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

$\Sigma W$  = The weight of the building including live load for seismic calculation supported by the storey concerned. So, load factor is excluded.

**(d) Calculation of ductility index  $F$**

The ductility index of a vertical member in the 2<sup>nd</sup> level screening procedure shall be calculated as in Art. 3.2.3 of the Japanese Standard.

**(3) 3<sup>rd</sup> level screening procedure**

Refer to the Japanese Standard.

## 1.6 SEISMIC DEMAND INDEX OF STRUCTURE $I_{so}$

### 1.6.1 Basic Principles

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

(2) Seismic safety of structure shall be judged by Equation (37) of J. Standard

$$I_s \geq I_{so}$$

Where:

$I_s$  = Seismic index of structure

$I_{so}$  = Seismic demand index of structure

If Equation (37) of J. Standard 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.

### 1.6.2 Seismic Demand Index $I_{so}$

(1) The seismic demand index of structure  $I_{so}$  should be calculated by Equation (38) of Japanese Standard regardless of the story in the building,

$$I_{so} = E_s \cdot Z \cdot G \cdot U$$

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 Equation (37) of Japanese Standard in the second and the third level screening procedure and assessed to be “Safe” Equation (39) of Japanese Standard shall also be satisfied.

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

Where:

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

$S_D$  = Irregularity Index.

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

The formula of J. Standard to calculate the seismic demand index  $I_{so}$  of structure is not applicable for Bangladesh because of difference in seismicity and building construction condition. A procedure to calculate the seismic demand index of structure for Bangladesh is proposed below based on study and experiment conducted under CNCRP Project (PWD-JICA technical cooperation project). (Detailed study report is attached as suppliment-1).

As far as the **1<sup>st</sup> level screening**, this will not be used as final judgment in Bangladesh, since background of building construction is different.

As far as the **3<sup>rd</sup> level screening**, this is complicated method and it takes time, and has not been mentioned in detail in this manual.

As a result, **2<sup>nd</sup> level screening** is practical and reliable, and is used for seismic evaluation and seismic retrofit design.

### 1.6.3 Seismic Demand Index, $I_{so}$ (Proposed for Bangladesh)

Proposed Seismic Demand Index of Structure  $I_{so}$  shall be by following equations,

(1) In case of the 2<sup>nd</sup> and 3<sup>rd</sup> level screening method

$$I_{so} = 0.8 \times \frac{2}{3} \times Z \cdot I \cdot C_S \quad (1.1)$$

Where,

$Z$  : Seismic zone coefficient, as defined in Section 2.5.4.2 of BNBC 2015.

$I$  : Structure importance factor, as defined in Section 2.5.5.1 of BNBC 2015.

$C_S$  : Normalized acceleration response spectrum, which is a function of structure (building) period and soil type (site class) as defined by Equations 6.2.35a to 6.2.35d of BNBC 2015.

(2) In addition, following condition is required.

$$C_{TU} \cdot S_D \geq 0.4 \times \frac{2}{3} \times Z \cdot I \cdot C_S \quad (1.3)$$

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

**Proposed  $I_{SO}$  ( $I_{SO}$  Based on BNBC 2015)**

1<sup>st</sup> level screening method is not used for the judgment of safety. 2<sup>nd</sup> level screening method which is practical is used.

In case of 2<sup>nd</sup> and 3<sup>rd</sup> level screening method, Proposed Seismic Demand Index of Structure  $I_{SO}$  is expressed by following equation.

$$I_{SO} = 0.80 \times \frac{2}{3} \times Z \cdot I \cdot C \quad (1.1)$$

Where,

$Z$  : Seismic zone coefficient, as defined in Section 2.5.4.2 of BNBC 2015

$I$  : Structure importance factor, as defined in Section 2.5.5.1 of BNBC 2015

$C_s$  : Normalized acceleration response spectrum, which is a function of structure (building) period and soil type (site class) as defined by Equations 6.2.35a to 6.2.35d of BNBC 2015

Example: considering medium height RC buildings,  $I_{SO}$  for 2<sup>nd</sup> and 3<sup>rd</sup> level screening is calculated as follows,

Zone 2 (Dhaka), soil type  $SC$ ,  $I_{SO} = 0.80 \times 0.38 = 0.30$  ( $Z=0.2, I=1.0, C_s=2.875$ )

soil type  $SD$ ,  $I_{SO} = 0.80 \times 0.45 = 0.36$  ( $Z=0.2, I=1.0, C_s=3.375$ )

Zone 4 (Sylhet), soil type  $SC$ ,  $I_{SO} = 0.80 \times 0.69 = 0.55$  ( $Z=0.36, I=1.0, C_s=2.875$ )

soil type  $SD$ ,  $I_{SO} = 0.80 \times 0.81 = 0.65$  ( $Z=0.36, I=1.0, C_s=3.375$ )

Refer to the Supplement A1, for more information. As shown in the Supplement A1, time-history response analysis of a frame with restoring force characteristics of degrading tri-linear model applying artificial earthquake waves corresponding to BNBC 2015 was done. The result was compared with the elastic response, and then the factor 0.80 was introduced for the setting of seismic demand index  $I_{SO}$ .

 **$I_{SO}$  based on BNBC 93**

Proposed Seismic Demand Index of Structure  $I_{SO}$  shall be by following equation.

In case of the 2<sup>nd</sup> and/or 3<sup>rd</sup> level screening method is applied,

$$I_{SO} = 0.80 \times Z \cdot C \cdot I \quad (1.2)$$

Where:

$Z$  : Seismic zone coefficient given in Table 6.2.22 of BNBC 93

$I$  : Structure importance coefficient given in Table 6.2.23 of BNBC 93

$C$  : Numerical coefficient given by the relation,  $C = 1.25S/T^{2/3}$ ,

$S$  : Site coefficient for soil characteristics,  $T$ : Fundamental period of vibration in seconds, the value of  $C$  need not exceed 2.75.

The proposed Seismic Demand Index of Structure  $I_{SO}$  to meet the condition of BNBC 93 is derived using same concept of that of BNBC 2015.

Example: In case of 2<sup>nd</sup> and 3<sup>rd</sup> level screening of  $I_{SO}$  for medium height typical RC buildings is calculated as follows,

Zone 2 (Dhaka),  $I_{SO} = 0.80 \times 0.41 = 0.33$  ( $Z=0.15, I=1.0, C=2.75$ )

Zone 3 (Sylhet),  $I_{SO} = 0.80 \times 0.69 = 0.55$  ( $Z=0.25, I=1.0, C=2.75$ )

**Strength Requirement**

$$C_{TU} \cdot S_D \geq 0.4 \times \frac{2}{3} Z \cdot I \cdot C_S \quad (1.3)$$

This requirement is shown to provide minimum strength frames at ultimate deformation and to control maximum Ductility index,  $F$  which is 2.0.

Where:  $C_{TU}$  = Cumulative strength index at the ultimate deformation of structure.  $S_D$  = Irregularity index.

**1.7 TARGET PERFORMANCE AND JUDGMENT**

The target of seismic performance of a building after retrofit should be decided by considering the existing condition and usage of the building, cost of retrofitting and compliance of BNBC. Seismic index of structure  ${}_R I_s$  after the retrofit satisfies the following equation.

$${}_R I_s \geq \alpha \times I_{SO} \quad \text{The guideline (Exp. 1.2-1)}$$

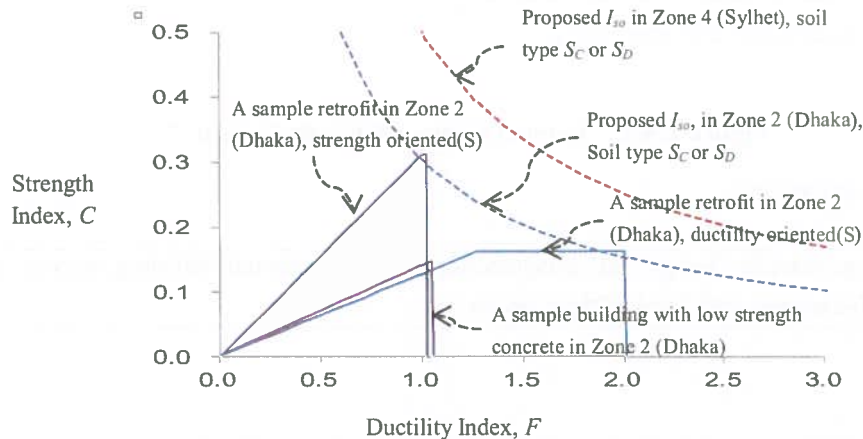
Where;  $\alpha$  = Design and construction coefficient (1.0~1.2)

$I_{SO}$  = Seismic demand index of structure, as shown in Section 1-6

In case that seismic index of structure  $I_s$  is equal or greater than seismic demand index  $I_{SO}$ , then the seismic performance of the building is judged equivalent to a building designed and constructed based on BNBC 2015.

$\alpha$  = design and construction coefficient (1.0~1.2) is introduced. In case that the assessment of retrofit designs and/or retrofit construction is difficult, it is suggested to use  $\alpha$  with the value of more than 1.0. This value will be re-considered after the accumulation of design and construction of retrofit buildings in Bangladesh near future.  $C$  (Strength index) and  $F$  (Ductility index) relation can show the basic seismic performance of a building. For example, as shown in Figure 1.7.1, (A) A sample building with low strength concrete, which is no ductile, in Zone 2 (Dhaka), say  $I_s = 0.1$ . After retrofit by strength oriented method like (S),  $I_s$  will be increased more than 0.30 ( $C \cdot F = 0.30 \times 1.0$ ) for soil type SC, and 0.36 for soil type SD. After retrofit by ductility oriented method like (D),  $I_s$  will be more than 0.30 ( $C \cdot F = 0.15 \times 2.0$ ) for soil type SC and 0.36 for soil type  $S_D$ . Assuming  $S_D$  in this example. It is emphasized that the seismic performance of a building is expressed by the multiplication of  $C$  (strength index) and  $F$  (ductility index) at each storey and in each horizontal direction of a building. The function of  $C \cdot F = \text{constant}$  shows hyperbolic curve as shown in Figure 1.7.1.

As far as the safety against vertical load, this manual doesn't cover and shall be evaluated at the same time based on BNBC.



**Figure 1.7.1 Examples of the Change of Seismic Performance of Buildings**

## 1.8 PROCESS OF RETROFIT DESIGN

Retrofit design shall be done in the sequence of retrofit plan, basic design, detail design and assessment of retrofit. This process shall be continued until the seismic index of structure  $I_s$  after the retrofit satisfies the seismic demand index of structure  $I_{so}$ .

As stated in Section 1.4 of the Guidelines, the process of retrofit plan, basic design, detail design and assessment of retrofit will be required as shown in Figure 1.8.1. This process shall be repeated. Until the seismic index of structure  $I_s$  after the retrofit satisfies the seismic demand index of structure  $I_{so}$ . As far as the layout of retrofit members, architectural and M/E requirement will be coordinated.

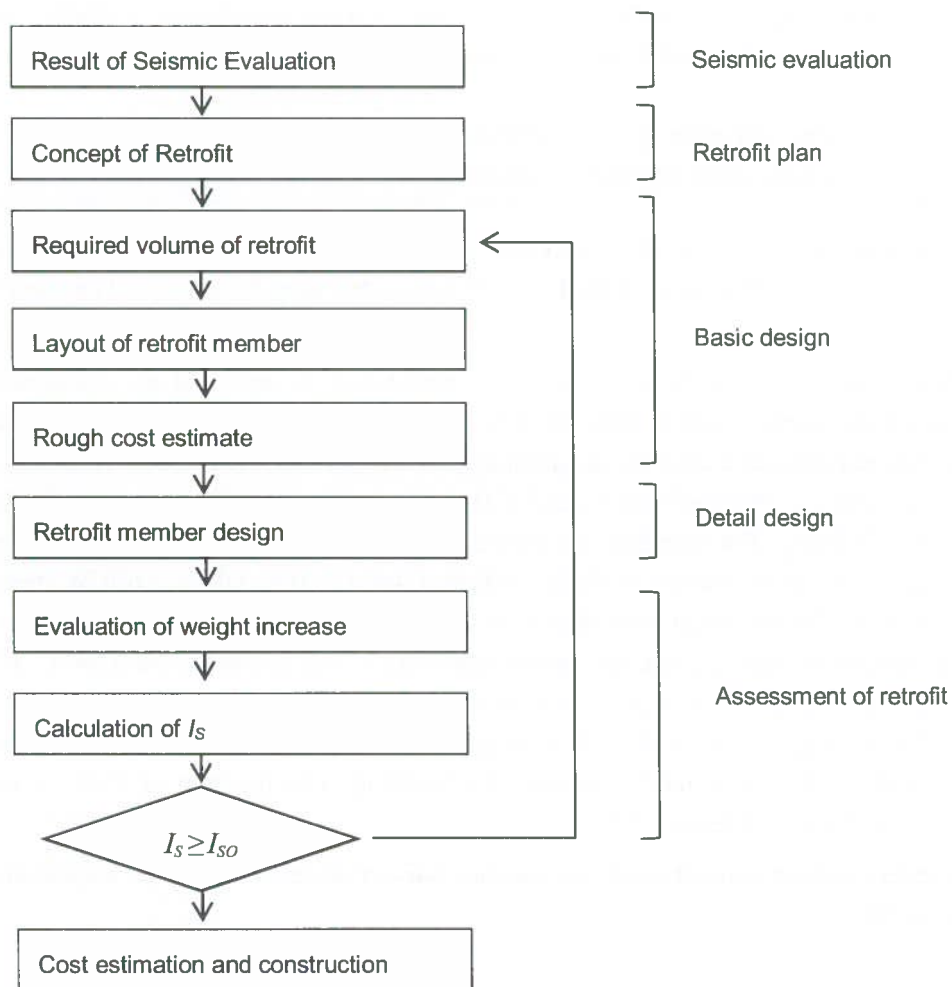
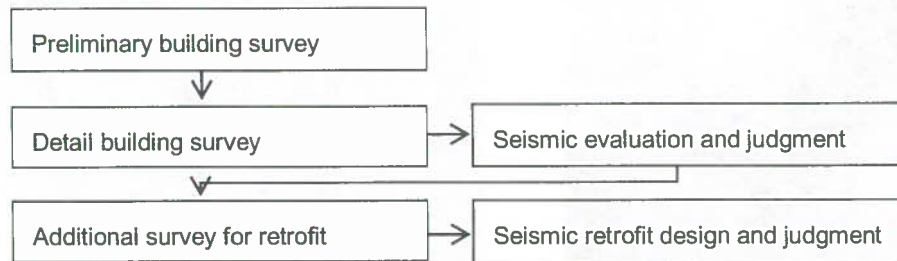


Figure 1.8.1 Typical Flow of Retrofit Design

## 1.9 BUILDING SURVEY

In order to develop retrofit design and construction plan, additional building survey shall be done together with the discussion with the building owner.

Standard process of building detail survey, seismic evaluation and retrofit design is shown in Figure 1.9.1. Detail building survey to prepare structural and architectural as-built drawings shall be required prior to the process of seismic assessment, since existing drawings are not necessarily show as built conditions. Material tests including concrete core and re-bar sampling shall also be required for seismic evaluation and retrofit design.



**Figure 1.9.1 Standard Technical Flow of Retrofit Design**

**Table 1.9.1 Contents of Building Survey**

Type of Survey	Objectives	Survey items
Preliminary survey	Visual observation to understand the characteristics of the building. To study the contents of detail survey. To study the applicability of the evaluation standard and retrofit design guideline.	General survey of the building, such as number of story, floor area, usage, year of construction, structural systems etc. Any deviation from approved drawing. Rebound hammer test on concrete surface.
Detail survey of the building with design drawings	To get necessary data/ information to execute seismic evaluation and retrofit design. To survey the dimensions of structural frames, arrangement of re-bars, configuration of column ties and concrete strength etc. To calculate the time index and irregularity index.	Usage and equipment/ live loads. The dimensions of building frames and reinforcing bars, arrangement of bars etc. Concrete core sampling test and re-bar tensile test. Structural cracks, deformations, aging by carbonation test. Any settlement of the building foundation, and soil investigation if required.
Additional survey for retrofit design	To develop retrofit design of members related to construction work. To discuss and get the consent of the building owner.	Coordination with the operation work. Detail dimensions of existing members. Locations of related mechanical piping and electrical cables to be converted or to be adjusted for the retrofit design development.

Note: 1) Non-destructive test such as re-bar detector will be used for the checking of re-bar arrangement of columns, but it is recommended to expose re-bars for a few columns, because re-bar detector cannot measure the size of re-bar accurately.



(a) Use of re-bar detector



(b) Position of re-bar



(c) Exposure of re-bar



(d) Concrete core sampling



(e) Sampled concrete core



(f) Compressive test (at HBRI)



(g) Non-shrink grout mortar and bonding agent for repair



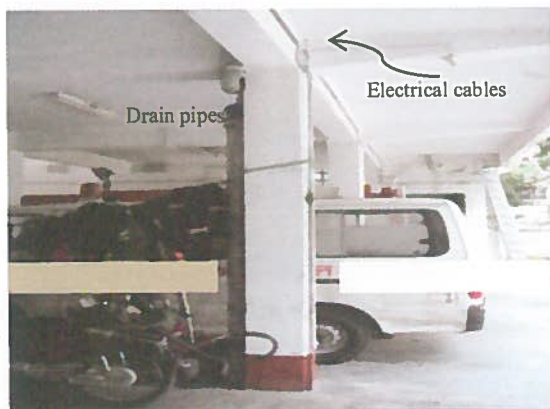
(h) Rebound test after the removal of plaster



(i) Sampling of re-bar for tensile test



(j) Exposure of foundation  
(Note: wearing helmet is suggested)



(k) Drain pipe and electrical cables on a column



(l) Shear strength test of brick wall  
(Note: wearing helmet is suggested)

**Figure 1.9.2 Various Survey and Test at a Building Site**

## 1.10 CONSTRUCTION OF RETROFIT WORK

The construction including materials of retrofit work shall conform to the Manual for retrofit construction work by CNCRP and related provisions of BNBC.

- 1) The construction including materials of retrofit work often involves inexperienced and nonstandard items compared with the construction of a new building. Experiments in Japan prove that the load-carrying capacity and deformation capacity of the members those have been strengthened largely depend on the detail and the precision of bond between the existing member and the original member. The quality control of construction is particularly important in the seismic retrofit works.
- 2) As far as the sample construction of retrofit methods are concerned, test work was done by CNCRP, refer to Supplement A10, Summary of Test Work (sample construction of retrofit work). Total 10 retrofit methods have been introduced.
- 3) Following Figure 1.10.1 is the fabrication of retrofitted specimen of the structural experiment 2013 by CNCRP (b), (d) and actual construction work (a), (c), (e), (f). Important details for joint of members are shown. Scale of the specimen at the experiment is approximately 1/3 of the actual member size. Chemical anchor was used as a post installed anchor. Gun for epoxy grouting should be used at site. Ladder type re-bar was installed at the perimeter of RC wall and of steel framed brace to prevent the split failure of filled mortar. Pressured non-shrink grout mortar was used to prevent the occurrence of any gap at the top of RC wall, and the perimeter of steel framed brace.
- 4) Strength of post-installed anchor depends on workmanship at the site, so on-site test of post-installed anchor is required.

(1) Insertion of new RC wall



(a) Re-bar and concreting work

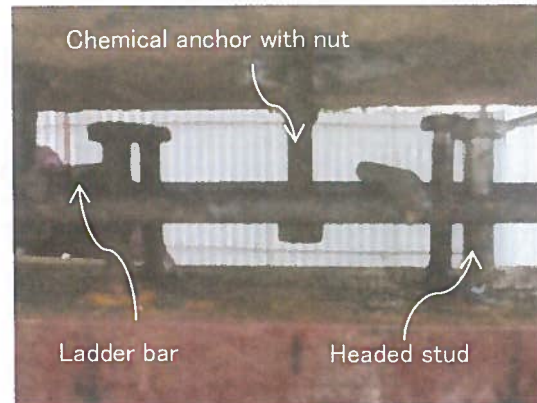


(b) Non-shrink grout mortar work done at the top of wall portion

(2) Providing steel framed brace



(c) Post installed anchor work



(d) Detail of joint between steel frame and RC beam



(e) Erected steel framed brace



(f) Pressured non-shrink mortar grouting at the top in progress

Figure 1.10.1 Construction of retrofit work

## 1.11 SUGGESTED MODIFICATION OF JAPANESE STANDARD FOR BANGLADESH

The Summary of modifications of Japanese Standard /Guidelines for its application in Bangladesh is shown in Table 1.11.1

Modifications of Japanese standard and guidelines for its application in Bangladesh are summarized in Table 1.11.1, with respect to A: General, B: Ductility index, C: Strength index, D: Irregularity index, E: Quality management of retrofit work and F: Others. It is noted that proposed numerical values have been considered based on the present best knowledge, but are tentative values and it will need further research/experiment for the verification/ modification in Bangladesh.

**Table 1.11.1 Suggested Modifications of Japanese Standard for its Application in Bangladesh**

(\*) denotes numerical values shown are tentative suggestion and needs further research/experiment for verification/ modification in Bangladesh.

Item	Japan	Bangladesh
Title	The Standard for Seismic Evaluation of Existing RC Buildings, Guidelines for Seismic Retrofit Design of Existing RC Buildings 2001 (JBDPA)	Seismic Evaluation Manual and Seismic Retrofit Design Manual of Existing RC Buildings (CNCRP)
<b>A: General</b>		
1. Status	"The Standard" and "Guidelines"	Technical "Recommendations"
2. Level of screening	1 <sup>st</sup> , 2 <sup>nd</sup> and 3 <sup>rd</sup> level screening method. 2 <sup>nd</sup> level is mainly used.	1 <sup>st</sup> , 2 <sup>nd</sup> and 3 <sup>rd</sup> level screening method. <b>2<sup>nd</sup> level screening method</b> is applied, which is suitable and practical for buildings. 1st level screening method is not used for the judgment. (Retrofit, Chap.1.1)
3. Existing buildings	Min. strength is secured by the building law at construction. Strength of concrete core = Average – standard deviation/ 2, 100mm diameter in general.	Many buildings are not following BNBC93, which became mandatory in 2006. <b>Detail building survey</b> is required. (Retrofit, Chap.1.9) (*)Strength of concrete core: (No change) Core strength is generally lower than that of cylinder, and strength of tested value divided by 0.85 may be used, minimum 50 mm diameter in general for columns. Ref. ACI 437 and 214
4. Application: Concrete strength	Concrete strength $F_c$ , not less than $13.5 \text{ N/mm}^2$ (Not low strength concrete)	Concrete strength $F_c$ , not less than <b><math>9.0 \text{ N/mm}^2</math></b> . Reduction factor $K_r$ is used for column shear strength in case of concrete strength lower than $13.5 \text{ N/mm}^2$ . (Chap.1.2)
5. Seismic index of structure, $I_s$	$I_s = E_o \times S_D \times T$ $E_o \propto \frac{n+1}{n+i} \times C \times F$	$I_s = E_o \times S_D \times T$ $E_o \propto \frac{n+1}{n+i} \times C \times F$ (No change)

Item	Japan	Bangladesh
6. Seismic demand index of structure, $I_{so}$	<p>Seismic demand index of structure <math>I_{so}</math>,</p> <p>(1) <math>I_{so}</math></p> $I_{so} = E_s \cdot Z \cdot G \cdot U$ <p><math>E_s</math> = Basic seismic demand index of structure</p> <p><math>E_s = 0.8</math>, for 1<sup>st</sup> level screening</p> <p><math>E_s = 0.6</math>, for 2nd level screening</p> <p><math>E_s = 0.6</math>, for 3rd level screening</p> <p><math>Z</math> = Seismic zone index</p> <p><math>G</math> = Ground index</p> <p><math>U</math> = Usage index</p> <p>(2) <math>C_{TU} \cdot S_D \geq 0.3 \cdot Z \cdot G \cdot U</math></p> <p><math>C_{TU}</math> = Cumulative strength index at the ultimate deformation of structure.</p> <p><math>S_D</math> = Irregularity index.</p> <p>(Example)</p> <p>Midrise RC in Tokyo,</p> $I_{so} = 0.6 \times 1.0 \times 1.0 \times 1.0 = 0.6$ <p>(<math>Z = G = U = 1.0</math>)</p>	<p>(*) Proposed <math>I_{so}</math> for 2<sup>nd</sup> and 3<sup>rd</sup> level screening,</p> $I_{so} = 0.8 \times \frac{2}{3} \cdot Z \cdot I \cdot C_s$ <p>(80% of elastic response shear force coefficient)</p> <p><math>Z</math> : Seismic zone coefficient, as defined in Section 2.5.4.2 of BNBC2015</p> <p><math>I</math> : Structure importance factor</p> <p><math>C_s</math> : Normalized acceleration response spectrum, which is a function of structure (building) period and soil type (site class)</p> <p>(Example)</p> <p>Zone 2 (Dhaka), medium height RC buildings</p> <p>soil SC, <math>I_{so} = 0.8 \times 0.38 = 0.30</math> (<math>Z=0.2</math>, <math>I=1.0</math>, <math>C_s=2.875</math>)</p> <p>soil SD, <math>I_{so} = 0.8 \times 0.45 = 0.36</math> (<math>Z=0.2</math>, <math>I=1.0</math>, <math>C_s=3.375</math>)</p> <p>Zone 4 (Sylhet), medium height RC buildings</p> <p>soil SC, <math>I_{so} = 0.8 \times 0.69 = 0.55</math> (<math>Z=0.36</math>, <math>I=1.0</math>, <math>C_s=2.875</math>)</p> <p>soil SD, <math>I_{so} = 0.8 \times 0.81 = 0.65</math> (<math>Z=0.36</math>, <math>I=1.0</math>, <math>C_s=3.375</math>)</p> $C_{TU} \cdot S_D \geq 0.4 \times \frac{2}{3} \times Z \cdot I \cdot C_s$ <p><math>C_{TU}</math> = Cumulative strength index at the ultimate deformation of structure. <math>S_D</math> = Irregularity index.</p> <p>(Chap. 1.6 and Supplement A1)</p>
7. Yield shear force coefficient and ductility ratio	$C_Y / C_E = 0.75 \cdot (1 + 0.05 \cdot \mu) / \sqrt{(2 \times \mu - 1)}$ <p>(equ.3.2.3-2 of the Standard) is used.</p> <p>This is modified “Energy constant principle”.</p>	<p>(No change) Same equation, which is modified “Energy constant principle”, is applied.</p> <p>Note: BNBC follows displacement constant principle. (Retrofit, Supplement A1 and A4)</p>
8. Evaluation of Ductility index $F$	$R_{mu} = R_{my} + R_{mu}$ $= R_{my} \cdot (1 + (10 \cdot Q_{su} / c \cdot Q_{mu} - 1.1))$ $\leq R_{max}, R_{30} \quad (A1.2.2 \& 3)$ $F = \frac{\sqrt{2R_{mu} / R_y} - 1}{0.75 \cdot (1 + 0.05R_{mu} / R_y)} \quad (16)$	<p>(No change) (Retrofit, Supplement A 4)</p> <p><math>Q = 1.1</math>, in case interval of hoop, <math>S &gt; 100\text{mm}</math>.</p>
9. Retrofit design policy	<p>Strength oriented, ductility oriented, or the combination will be selected.</p>	<p>Strength and stiffness oriented retrofit is recommended. (Ductility oriented retrofit allows large deformation and many columns are required to be retrofitted, beam column joints are not easy to retrofit technically and many in fill brick walls are required to be retrofitted. This method will not be recommended.) (Retrofit, Chap. 2.1)</p>

Item	Japan	Bangladesh
<b>B: Ductility index <math>F</math> related</b>		
1. Upper limit of deformation capacity 1.1 Column axial force ratio $N/(b \cdot D \cdot F_c)$ and Ductility index, $F$	Column tie interval $> 100\text{mm}$ , $N/(b \cdot D \cdot F_c) > 0.4$ , $F = 1.0$ Column tie interval $\leq 100\text{mm}$ , $N/(b \cdot D \cdot F_c) > 0.5$ , $F = 1.0$ $N/(b \cdot D \cdot F_c) \leq 0.6$ is applicable range. $b, D$ : Column width and depth $F_c$ : concrete strength $N$ : Axial force without load factor ( $N/(b \cdot D \cdot F_c) < 0.4$ is common.)	(*) <b>Ordinary concrete:</b> ( $\geq 13.5 \text{ N/mm}^2$ ) In case that shear reinforcement of column by BNBC, 2-10mm@ 150mm is satisfied or $P_w$ (shear reinforcement ratio) $\geq 0.2\%$ <b><math>0.4 &lt; N/(b \cdot D \cdot F_c) &lt; 0.55</math>, <math>F = 1.27</math></b> (aim $R = 1/150$ ), $N/(b \cdot D \cdot F_c) \geq 0.55$ , $F = 1.0$ (aim $R = 1/150$ ). In case that shear reinforcement is other cases, $N/(b \cdot D \cdot F_c) \geq 0.4$ , $F = 1.0$ (aim $R = 1/150$ ). For both cases $N/(b \cdot D \cdot F_c) \geq 0.8$ , $F = 0.8$ , (aim $R = 1/500$ ). This needs further consideration. (*) <b>Low strength concrete:</b> ( $< 13.5 \text{ N/mm}^2$ ) In case that shear reinforcement 2-10mm@ 200mm is satisfied or $P_w \geq 0.15\%$ <b><math>0.4 &lt; N/(b \cdot D \cdot F_c) &lt; 0.6</math>, <math>F = 1.27</math></b> (aim $R = 1/150$ ), $N/(b \cdot D \cdot F_c) \geq 0.6$ , $F = 1.0$ (aim $R = 1/250$ ). In case that shear reinforcement is other cases, $0.4 < N/(b \cdot D \cdot F_c)$ , $F = 1.0$ (aim $R = 1/250$ ). For both cases $N/(b \cdot D \cdot F_c) \geq 0.8$ , $F = 0.8$ , (aim $R = 1/500$ ). This needs further consideration. (Considering the test result by CNCRP in 2012 and 2013 (Ultimate deformation capacity of $R = 1/100$ for $N/(b \cdot D \cdot F_c) = 0.68$ and engineering judgment (Retrofit, Supplement A3) Column axial capacity at long term is checked by BNBC (Factored load), $P_u = 0.85\Phi [0.85 f'_c \times A_c + f_y A_{st}]$ Average of dead and live load factor is 1.45~1.5.
1.2 Shear stress	$cR_{\max}(s) = cR_{250}$ for $c \tau_u / F_c > 0.2$ (A1.2-7)	(*) (No change) (Retrofit, Supplement A4) It needs further consideration related to BNBC.
1.3 Tensile re-bar ratio	$cR_{\max}(t) = cR_{250}$ for $p_t > 1.0\%$ (A1.2-8)	(*) $cR_{\max}(t) = cR_{250}$ for $p_t > 1.3\%$ , $F = 1.0$ (Evaluation, Supplement 4, Retrofit, Supplement A4)
1.4 Interval tie (hoop)	$(cR_{\max}(b) = cR_{250})$ for $s/d_b > 8$ (A1.2-9)	(*)(No change) (Retrofit, Supplement A4) It needs further consideration related to BNBC. Requirement of $(s/d_b \geq 6)$ is shown for Special moment frame of ACI 318.
1.5 Clear span and depth ratio	$cR_{\max}(h) = cR_{250}$ for $h_o/D \leq 2.0$ (A1.2-10)	(*) No concrete standing wall, Evaluation of brick standing wall, see next item. (Retrofit, Supplement A4)

Item	Japan	Bangladesh
2. Short column (Shear failure) caused by standing wall and Ductility index, $F$	Evaluate Ductility index considering short column caused by RC standing wall. Shear failure column, $F = 1.0$ (1/250) Extremely brittle column, $F = 0.8$ (1/500) Or provide structural slit on RC standing wall to prevent short column.	(*) If the shear failure of short column due to brick standing wall has not been studied, upper limit of column $F$ , <b>Ordinary concrete</b> , $F = 1.5$ (aim of storey deflection angle, $R=1/124$ ) ~ <b>1.75</b> (1/100) <b>Low strength concrete</b> ( $F_c < 13.5 \text{ N/mm}^2$ ), $F = 1.27$ ( $R=1/150$ ) ~ <b>1.5</b> (1/124) In case that shear reinforcement ratio $P_w$ of column is less than 0.2%, the use of smaller $F$ of above is used. (According to structural test by CNCRP in 2012 and 2013, short column due to brick standing wall will cause shear failure at storey deflection angle of 1/100 and more generally in case of low strength concrete. Suggested $F$ value is slightly overestimated incorporating the horizontal strength increase due to brick standing wall.) (Supplement A2)
3. Beam column joint and Ductility Index $F$	No specific requirement. (Damage of beam column joint is very rare in Japan because of relatively larger column size.)	(*) If no safety has been confirmed for the beam column joint, upper limit of column $F$ , following is used. <b>Ordinary concrete</b> , max. $F = 1.75$ (aim of storey deflection angle, $R=1/100$ ) <b>Low strength concrete</b> , max. $F = 1.5$ ( $R=1/124$ ) (According to trial calculation, shear failure occur at beam column joint, in case of low strength concrete. On the other hand, it is said that shear failure of beam column joint occur at 1/100 of column deflection angle generally.) (Retrofit, Supplement A7)
4. 90 degree hook of column tie and Ductility Index, $F$	Reduction of Ductility index of column, $F$ compared with 135 degree hook is suggested.	(*) Reduction of Ductility index of column, $F$ compared with 135 degree hook is used. Proposed one idea is to reduce shear reinforcement ratio to " $0.5 \times P_w$ ", in case of 90 degree hook. (Retrofit, Supplement A4)
5. Retrofit by steel framed brace for existing low strength concrete frame	Out of scope for low strength concrete	(*) Existing concrete strength lower than $13.5 \text{ N/mm}^2$ , and not less than $9.0 \text{ N/mm}^2$ , following is used. Standard indirect connection method : $F=1.50$ or less. Epoxy resin connection method : $F=1.27$ or less (Retrofit, Chap. 3.3)
<b>C: Strength index <math>C</math> related</b>		
1. Plain main-bar and low strength concrete	No specific requirement.	(*) It is used to reduce 20% of flexural strength of column tentatively, in case of low strength concrete, of which bond stress is low. (Retrofit, Chap. 3.4)
2. Deformed bar, shear span column depth ratio $M/(Q \cdot D)$	No specific requirement.	(*) It is required to reduce the strength in case of the combination of deformed bar, low strength concrete and shear span column depth ratio $M/(Q \cdot D) < 3.0$ , considering the bond failure. (Evaluation, Supplement 5)
3. Short anchor length of beam main bar at external column	No specific requirement.	(*) If anchor length of beam main bar at an external column is supposed, it is used to reduce 25% and max. 50% (for thin depth column such as 250mm) of flexural strength of the column by 2 <sup>nd</sup> level screening. (Chap. 3.4). Similar condition of $F$ of B3 is applied.

Item	Japan	Bangladesh
4. Concrete strength of jacketed column	Concrete strength of existing column portion is used for the strength evaluation of jacketed column.	(*) It might be acceptable to use average concrete strength of existing and additional portion proportional to its section area for the calculation of flexural strength of column. (Existing concrete is generally low strength and size of existing column is generally small.) Need further study and investigation.
<b>D: Irregularity index <math>S_D</math> related</b>		
1. Piloti (Soft storey)	Evaluation of Piloti (soft storey) related to RC wall is required.	(*) Evaluation of Piloti (soft storey) related to brick wall is required. (Retrofit, Chapter 2.2)
<b>E: Quality management of retrofit work</b>		
1. Quality control	Retrofit design guidelines, Chapter 4, Construction of retrofit work	Separate manual, Construction control manual of Retrofit work. (Retrofit, Chap. 1. 10)
<b>F: Others</b>		
1. Aim of storey deflection angle	Experiment/ analysis in Japan. Storey deflection angle at yield of standard column, approx. 1/150 Ultimate deflection angle of RC shear wall, approx. 1/250 Ultimate deflection angle of extremely brittle column, approx. 1/500	(*) No change. Result of experiment by CNCRP on 2012 and 2013 is shown below for information. (Retrofit, Supplement A2) Storey deflection angle at yield of standard column, approx. 1/100 Storey deflection angle at max. strength with high axial force ratio (low strength concrete), approx. 1/100 Storey deflection angle at shear failure of RC wall, approx. 1/200 Storey deflection angle of max. strength of steel framed brace, approx. 1/200 Further accumulation of related data is required.

Note: ACI 437R-03 (Strength Evaluation of Existing Concrete Buildings), the Sec 5.1.1

ACI 318R-14 (Commentary on Building Code Requirements for Structural Concrete)

ACI 214.4R-03 (Guide for Obtaining Cores and Interpreting Compression Strength Result), Sec 6.2

## CHAPTER 2. PLANNING AND BASIC DESIGN

### 2.1 PLANNING OF RETROFIT

#### 2.1.1 General

Basic policy to meet the demand seismic performance by improving strength and/or ductility of the building is defined. In addition, optimum retrofit method that meets demand performance is selected. An overall study is conducted at the planning stage considering the building function after retrofit and workability of retrofit construction as well as performance upgrading by seismic retrofit.

#### 2.1.2 Classification of Retrofit Method

Retrofit method and system for the seismic improvement is classified as shown in Figure 2.1.1 Strength improvement or ductility improvement is adopted for the retrofit generally. Avoidance of damage concentration is also important for the plan of retrofit. Reduction of seismic force such as Base isolation and Vibration control has not been covered in this manual, but related information is shown in “Supplement B7” for information only.

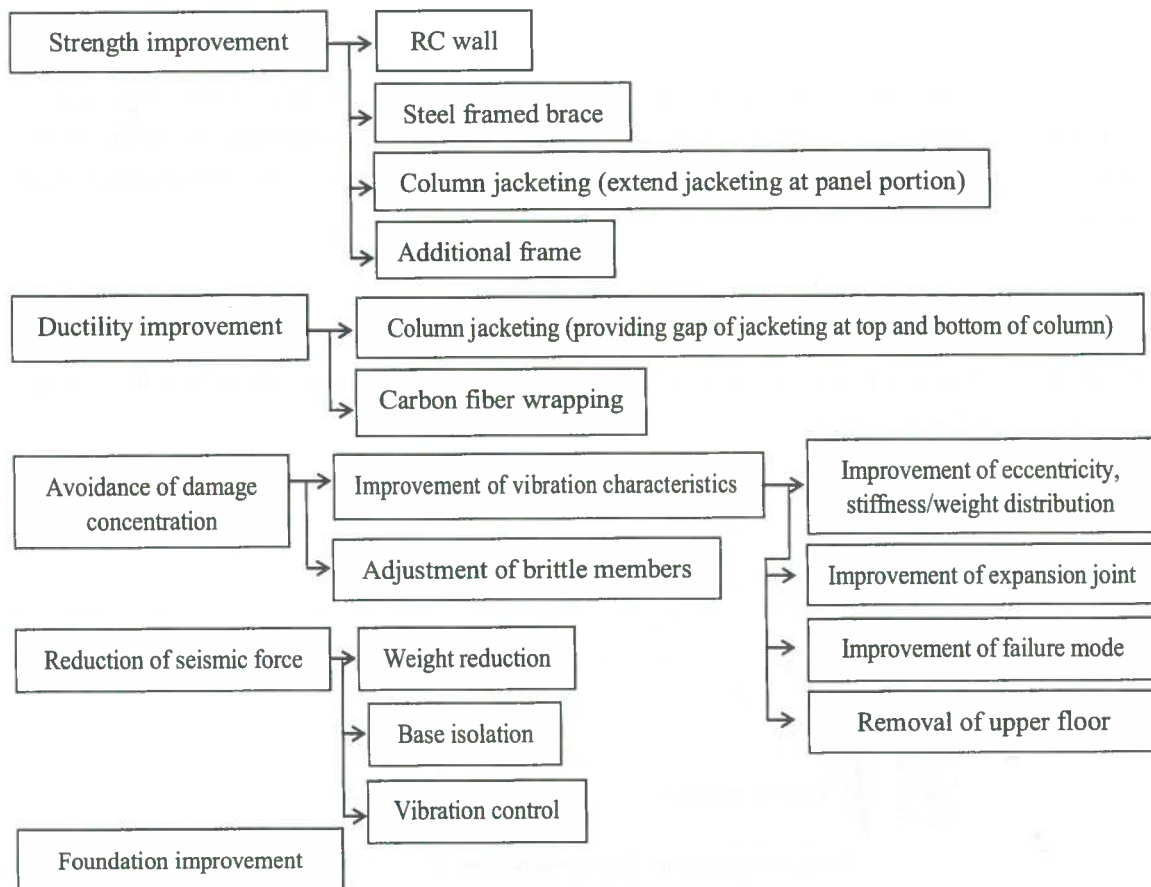


Figure 2.1.1 Classification of Retrofit Method/ System for the Seismic Improvement

### 2.1.3 Plan of Retrofit

Following factors are considered for planning of retrofit:

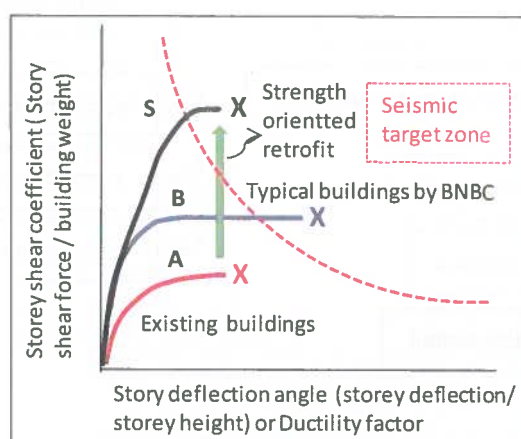
1. Defective storey based on the result of seismic evaluation, especially the strength index  $C$  and ductility index  $F$  of each storey and each direction, and irregularity index  $S_D$ .
2. Defining demand performance
3. Limitation of retrofit such as characteristics of existing structure, building function, usage, cost, time and ease of construction
4. Effective retrofit method to get reasonable performance
5. Estimation of required volume of retrofit
6. Layout of to be retrofitted members
7.  $I_s$  value and  $C$  value and their smooth vertical distribution after the retrofit.
8. Impact on existing foundation

### 2.1.4 Recommendation

Generally it can be said that both strength and ductility of existing RC buildings in Bangladesh are not enough. To improve the ductility of a building, almost all columns will be required to retrofit at lower floors. It will allow large deflection of frames and may cause damage of brick walls, non-structural elements.

In this manual, it is recommended to apply strength oriented retrofit and to reduce storey deflection of frames as shown in Figure 2.1.2, which will also reduce the in-plane damage of brick walls, non-structural elements. Strength improvement methods such as providing RC wall, steel framed brace (indirect connection type) and column jacketing are introduced in Chapter 3.

If ductility oriented retrofit is applied, almost all columns at lower storey may be required to be retrofitted. The possibility of shear failure at beam column joint is high at large deflection. On the other hand retrofit method at beam column joint is practically difficult. In addition, big deflection of in-filled brick wall in plane will require some countermeasures.



**Figure 2.1.2 Recommended Retrofit Plan for Existing RC Buildings**

## 2.2 BASIC DESIGN

### 2.2.1 General

The required seismic performance of retrofit is defined by difference between the demand performance and the performance of existing building. Arrangement of the retrofit elements is 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 is to be considered.

### 2.2.2 Required Quantity of Retrofit

In case of strength oriented structure, required strength of retrofit is estimated as shown in Figure 2.2.1, the shortage of strength at intended ductility index is calculated as follows.

$$\Delta Q_i = \Delta C_i \times \Sigma W_i = \frac{n+i}{n+1} \times \frac{1}{F'} \times \left( \frac{{}_R I_S}{S'_D \cdot T'} - \frac{I_{Si}}{S_D \cdot T} \right) \times \Sigma W_i$$

Where,  $\Delta C_i$  : Shortage of Strength Index at  $i$ th storey

$\Delta Q_i$  : Shortage of Shear Strength at  $i$ th storey

$\Sigma W_i$  : Total building weight supported by  $i$ th storey

$\frac{n+i}{n+1}$  : Reciprocal of storey modification factor

$F'$  : Intended Ductility Index for retrofit

${}_R I_S$  : Target Seismic Index of Structure for retrofit

$I_{Si}$  : Seismic Index of Structure at  $i$ th storey before retrofit (Equation (5) of the “J. Standard”)

$S_D, S'_D$  : Irregularity Index before and after retrofit

$T, T'$  : Time Index before and after retrofit

Source; “Handbook of earthquake resisting design of structure”, JSCA, (in Japanese)

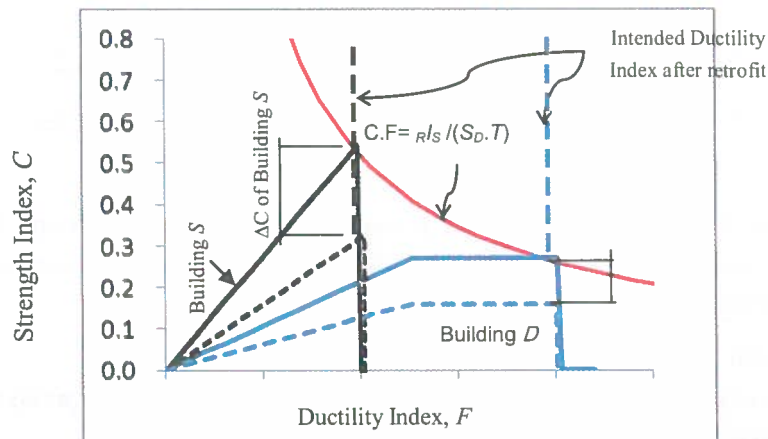


Figure 2.2.1 Estimation of Required Strength of Sample Buildings

**A Sample Table for required strength calculation:**

Required horizontal strength  $Q_R$  for retrofit, at intended Ductility index  $F$  after the retrofit is calculated as shown in Table 2.1.1

$$E_o = \left( \frac{n+1}{n+i} \right) \cdot C \cdot F$$

$$I_s = E_o \cdot S_D \cdot T = \left( \frac{n+1}{n+i} \right) \cdot C \cdot F \cdot S_D \cdot T \quad (1) \text{ The Japanese Standard}$$

After retrofit,  $C \cdot F = \left( \frac{n+i}{n+1} \right) \cdot \frac{I_s}{S_D \cdot T}$ ,  $C = \frac{Q}{\Sigma W}$

Then,  $Q_R = \left( \frac{n+i}{n+1} \right) \cdot \frac{I_{SO}}{(F \cdot S_D \cdot T)} \cdot \Sigma W - \left( \frac{n+i}{n+1} \right) \text{original } C \text{ (at intended } F) \cdot \Sigma W$

In case of ground floor,  $Q_R = \frac{I_{SO}}{(F \cdot S_D \cdot T)} \cdot \Sigma W - \text{original } C \text{ (at intended } F) \cdot \Sigma W$

**Table 2.2.1 Table of Calculation for Required Horizontal Strength**

	$\Sigma W$ , Weight (kN), load factor is not used.	$\frac{n+i}{n+1}$	Design shear coefficient, $\frac{n+i}{n+1} \times \frac{I_{SO}}{F \cdot S_D \cdot T}$  $I_{SO}$ , $F$ , $S_D$ (after retrofit)	Design shear strength $Q_D$ , after retrofit, $\frac{n+i}{n+1} \times \frac{I_{SO}}{F \cdot S_D \cdot T} \times \Sigma W_i$ (kN)  (1)	Original strength, $\frac{n+i}{n+1} \cdot C(atF) \cdot \Sigma W_i$ (kN)  (2)		Required additional strength, $Q_R$ (1) - (2) (kN)
					$Q_o$ (X,Y)	C (X,Y)	
4							
3							
2							
1							

**2.2.3 Required Amount of Retrofit**

Number of RC walls or Steel brace with frame is required to be estimated. Rough shear strength of one set of RC wall and one set of steel bracing is calculated with reasonable assumption as follows, to get some idea of retrofitting.

**(1) Retrofit by RC wall**

Assuming the increase of shear strength by the installation of the RC wall, required rough total length of shear wall is calculated as follows.

$$L_i = \frac{\Delta Q_i}{\tau_u \cdot t_w} \quad (2.2)$$

Where,  $L_i$ : Required total length of additional RC wall at  $i$ th storey

$\Delta Q_i$ : Required shear strength for retrofit at  $i$ th storey

$t_w$ : Thickness of RC wall for retrofit

$\tau_u$ : Shear stress of shear wall at ultimate stage

An example of supposed shear strength of RC wall is shown in Table 2.2.2 The configuration is shown in Figure 2.2.2 In this example, wall clear length is supposed to be 5,500mm and concrete strength is  $18\text{N/mm}^2$ . For the detail of calculation, refer to Section 3.1 of the Guideline. Average shear stress  $\tau$  ( $\text{N/mm}^2$ ) of wall panel is shown for information. Ductility Index  $F$  of RC wall with shear failure is 1.0.

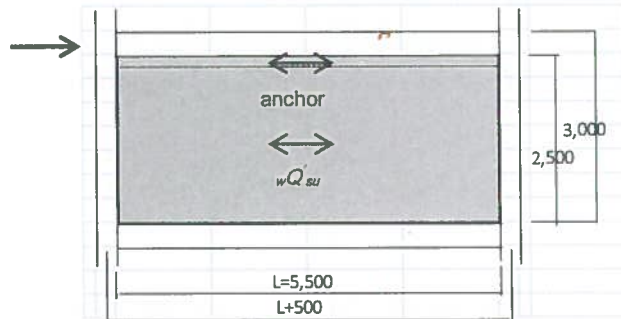


Figure 2.2.2 RC Wall for Retrofit

Anchor  $Q'_u$ : shear strength of post installed anchor, refer to Chapter 3.5.

$wQ'_{su}$ : Shear strength of RC wall, refer to Chapter 3.2.

Table 2.2.2 Shear Strength of RC Wall Panel by Retrofit

Wall thickness (mm)	Shear strength of anchor (kN)		Wall re-bar, for vertical and horizontal re-bar	$p_w$	Shear strength of wall panel (kN)			$\tau$ ( $\text{N/mm}^2$ ) = $wQ_{su} / (t_w \cdot L)$
	Post installed anchor	anchor $Q_u$ (kN)			$Q_1$ (kN) $= t \cdot L \cdot P_w \cdot \sigma_y$	$Q_2$ (kN) $= t \cdot L \left( \frac{F_{cw}}{20} + 0.5 p_w \sigma_y \right)$	$wQ_{su} = \max(Q_1, Q_2)$	
$t_w = 160$	$\Phi 16\text{mm}@150$	1650	$\Phi 8\text{mm}@150\text{W}$	0.00419	1474	1520	1520	1.74
	$\Phi 20\text{mm}@200$	1930	$\Phi 10\text{mm}@200\text{W}$	0.00523	1842	1713	1842	2.09
$t_w = 200$	$\Phi 20\text{mm}@150$	2060	$\Phi 10\text{mm}@150\text{W}$	0.00522	2300	2140	2300	2.09 (1.87)
	$\Phi 22\text{mm}@175$	2680	$\Phi 10\text{mm}@175\text{W}$	0.00447	1970	1975	1975	1.79

Note: 1)  $L = 5500\text{mm}$ , clear length of wall.

2) Concrete strength of wall,  $F_c = 18\text{N/mm}^2$ , and existing members,  $18\text{N/mm}^2$ .

3) Re-bar,  $\sigma_y = 400\text{N/mm}^2$

4) Non-shrink mortar will be provided at the top of wall with split preventing re-bar.

In case that existing concrete strength is  $14\text{N/mm}^2$ , following post installed anchor is recommended, based on the calculation of Section 3. 5.

Table 2.2.3 Shear Strength of Post Installed Anchor (in Case Concrete Strength is  $14\text{N/mm}^2$ )

Wall thickness (mm)	Post installed anchor	anchor $Q_u$ (kN)
$t_w = 160$	$\Phi 20\text{mm}@200$	1610
	$\Phi 20\text{mm}@150$	2170
$t_w = 200$	$\Phi 20\text{mm}@150$	2170
	$\Phi 22\text{mm}@175$	2260

It is noted that shear strength of RC wall will be decided by the post installed anchor at the connection in case of low strength concrete of existing structure.

Shear strength of in-filled shear panel (only for the panel in the clear width) by the J. Guidelines.

Column strength is excluded in this calculation.

$$wQ'_{su} = \max(p_w \cdot \sigma_y, F_{cw} / 20 + 0.5 \cdot p_w \cdot \sigma_y) \cdot t_w \cdot \ell' \quad (3.1.5-4) \quad \text{The J. Guidelines}$$

Where:

$p_w, \sigma_y$  = wall reinforcement ratio and yield strength of the wall reinforcing bar (N/mm<sup>2</sup>)

$F_{cw}$  = Concrete strength of the installed panels (N/mm<sup>2</sup>)

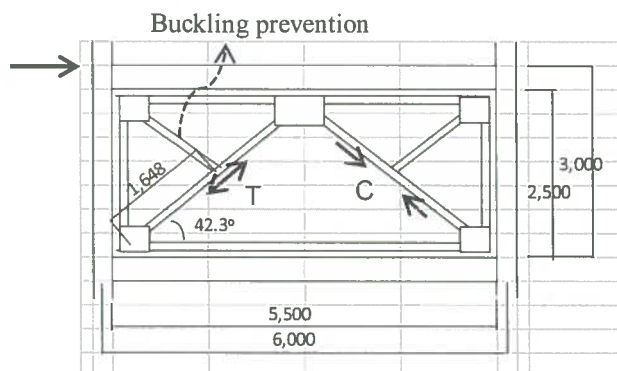
$t_w, \ell'$  = Wall thickness and clear span of installed wall panel (mm)

## 2) Steel framed brace

Supposing the increase of the shear strength by providing steel brace with frame, required number of steel brace is roughly calculated as follows.

$$N_i = \frac{\Delta Q_i}{Q_{bu}} \quad (2.3)$$

Where:  $\Delta Q_i$  : Required shear strength for retrofit,  $Q_{bu}$ : Increased shear strength of one set of steel brace  
As far as the member of steel bracing is concerned, the use of H section member, such as H-150 × 150 × 7 × 10 and more is recommended in the Section 3.4 of the Guidelines. The alternative is proposed in this manual to use the channel section or combination of two pieces of angle section. Unless proper method and quality of butt welding is ensured, the connection of gusset plate type should be applied at the joint using fillet welding only. Dimension of typical frame is shown below (Figure 2.2.3). For detail calculation of steel framed brace, refer to Section 3.3 of this manual.



C, T: Compressive and tensile strength of brace member respectively, refer to Chapter 3.3.

Figure 2.2.3 An Example of Steel Framed Brace

Table 2.2.4 Shear Strength of Steel Framed Brace

Steel bracing	Post installed anchor (kN)	Headed stud (kN)	C: compression strength (kN)	T: Tensile strength (kN)	Shear strength (kN) = (C + T) · cos θ
2L-100 × 100 × 10 (C-200 × 100 × 10)	φ16mm@150 (Q = 1,650 kN)	2- φ 16mm @ 150	1,030	1,040	1,530
2L-100 × 100 × 13 (C-200 × 100 × 13)	φ19mm@150 (Q = 2,330 kN)	2- φ 16mm @ 150	1,320	1,320	1,950
2L-130 × 130 × 12 (C-260 × 130 × 12)	Φ22mm@175 (Q = 2,680 kN)	2- φ 19mm @ 175	1,760	1,630	2,500

Note: 1) Yield stress,  $\sigma_y = 345\text{N/mm}^2$  or equivalent material.

2) Effective area of 80% by bolting holes was supposed for the calculation of tension strength.

3) Concrete strength of existing members,  $F_c = 18\text{N/mm}^2$ .

4) Non-shrink grout mortar is provided at the perimeter, with split prevention re-bar.

It is noted that shear strength of steel bracing will be decided by the post installed anchor at the connection in case of low strength concrete, which is similar to RC wall.

Ductility index  $F$  of Steel braced frame is in the range of 1.5~2.0 generally.

### 2.2.4 Non-Structural Brick Wall

It is well known that existing brick walls, which are non-structural elements, affect the behavior of the structure. But structural behavior is not well known and the quantitative evaluation is still not well established.

Existing brick wall will affect the stiffness and strength of the frame. This may have positive and/or negative impact on the structure.

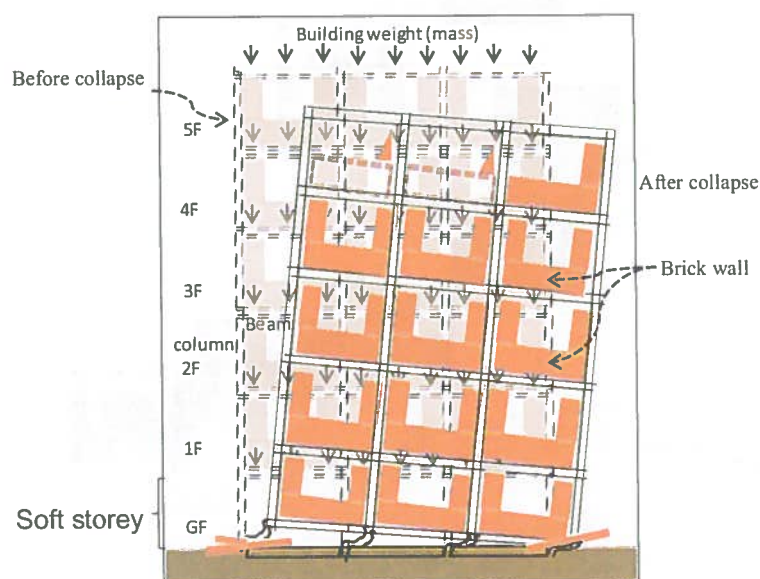
#### Short column

Short column is caused by brick standing wall, and this may cause brittle failure of columns. However strength oriented retrofit will reduce the response deflection and will reduce this influence of short columns.

#### Soft storey (pilotis)

An important aspect for retrofit related to brick wall is a building with the soft story, or pilotis floor. If the ground floor is a car parking with no brick walls and 1<sup>st</sup> floor and above are for residential or office use with plenty of infill walls, then the ground floor will be judged as the soft story. In case of an earthquake, damage will be concentrated at the soft story. Schematic picture of collapse of a soft story is shown in Figure 2.2.4 In this case, it is recommended strongly to retrofit the soft storey.

For quantitative assessment it is suggested to do execute time history response analysis based on restoring force characteristics of RC frames incorporating assumed increase of stiffness and strength by brick walls.



**Figure 2.2.4 Schematic Picture of Collapse of a Soft Storey (Pilotis)**

#### Eccentricity

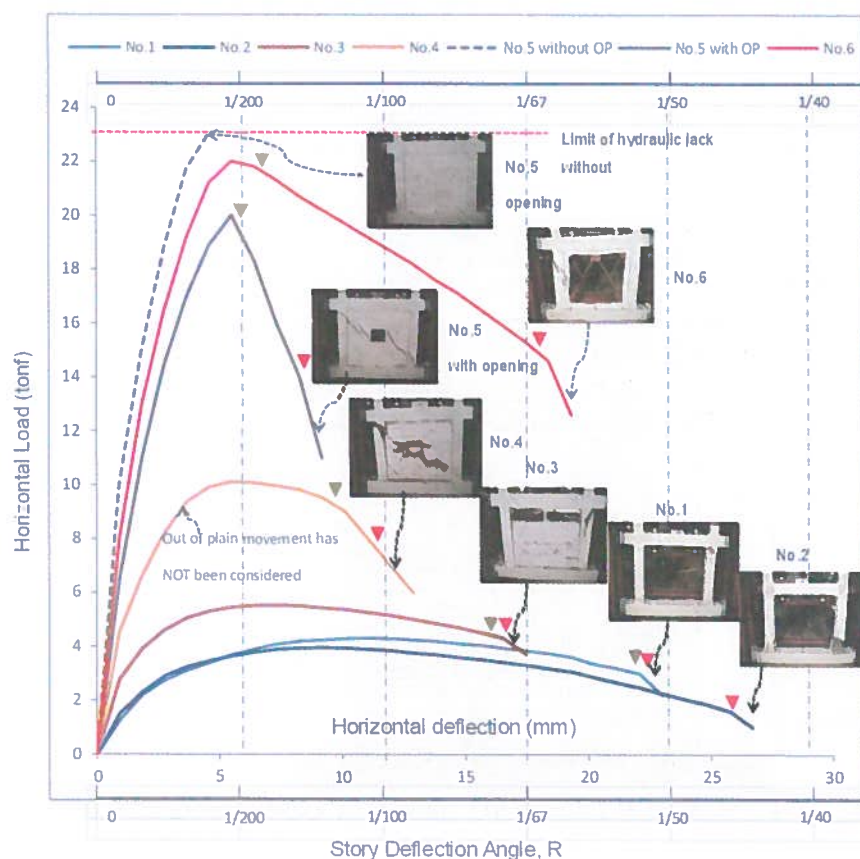
In case of eccentricity caused by brick wall adequate, retrofit measures should be taken to prevent the effect of eccentricity.

### 2.2.5 Extraction from the Structural Experiment 2013 by CNCRP

Structural experiments on retrofitted members were done by CNCRP in 2013. The summary of the experiment is shown in the attached Supplement 2. Simplified monotonic load- deflection curve is shown in Figure 2.2.5 and 2.2.6 Low strength concrete ( $F_c = 10.6\text{N/mm}^2$ ) with column axial force ratio ( $N/b \cdot D \cdot F_c$ ) of 0.68 was used. Poor shear reinforcement was provided in the columns.

#### i) Retrofit by RC wall and steel framed brace

By providing RC wall (specimen no.5) or steel framed brace (specimen no.6), shear strength of a frame can be increased by retrofitting. After shear failure of the wall /steel framed brace the column could not support vertical load any longer due to shear failure, which is brittle in nature. The reason of this failure was due to little shear reinforcement and high axial force ratio of the weak column. It will be recommended to provide column jacketing together with RC wall or steel braced frame to increase deformability of columns. In case existing columns are not so weak, column jacketing will not be required to install RC wall or steel framed brace.



Note: Marking: ▼ denotes a point of "Drop in vertical strength".

▼ denotes a point of "Shear failure".

R: Story deflection angle = Horizontal deflection (mm) / Story height (1,175mm)

N: Axial force (N)

$F_c$ : Concrete strength ( $\text{N/mm}^2$ )

b: Column width (mm)

D: Column depth (mm)

1 tonf = 2, 205 lbf = 9.8 kN

1 Mpa = 1  $\text{N/mm}^2$

1  $\text{N/mm}^2$  = 145 psi

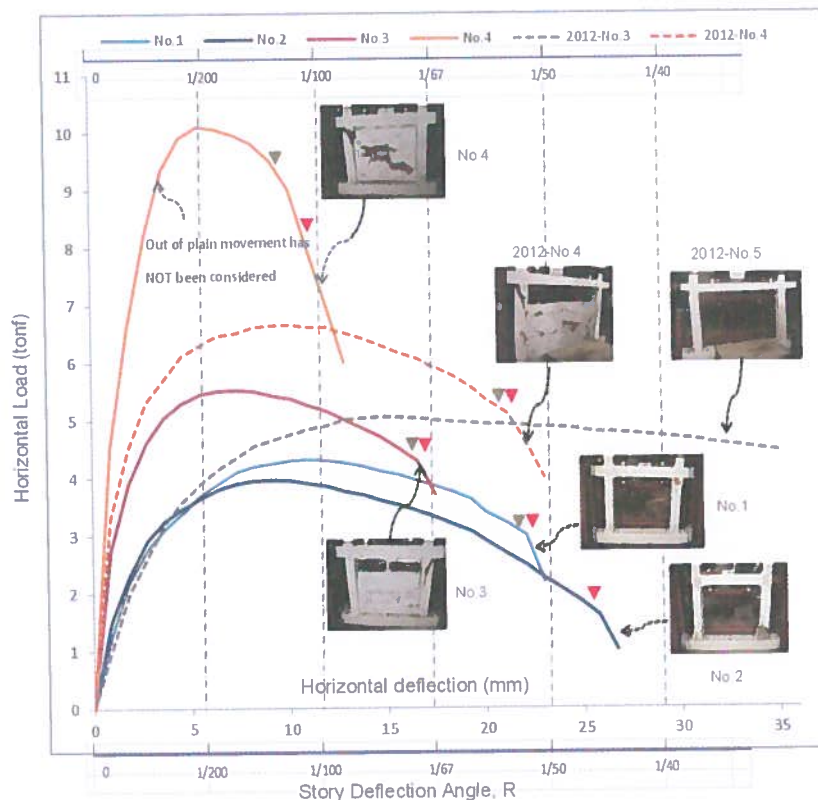
**Figure 2.2.5 Simplified Load-Deflection Curve with Retrofitted Specimens**

## ii) Non-structural brick wall

Specimen with brick standing wall of No.3 (2013) and 2012 No.4 show the reduction of deformation and increase of strength. Shear failure of column occurred and the drop of vertical strength occurred at the same time. Specimen No.4 with brick-wall and without opening showed the increase of strength but deformability was reduced. In this case, out of plane movement of brick wall has not been considered. It will be effective to control the storey deflection angle of RC frame within 1/150 in case of low strength concrete, and within 1/100 in case of ordinary concrete, to prevent the in-plane failure of non-structural brick wall.

Axial force: Specimen No.1~No.4,  $N/(b \cdot D \cdot F_c) = 0.68$ , ( $F_c = 10.6 \text{ N/mm}^2$ ,  $N = 163 \text{ kN}$ )

Specimen 2012-No.4, 5,  $N/(b \cdot D \cdot F_c) = 0.44$  ( $F_c = 10.6 \text{ N/mm}^2$ ,  $N = 163 \text{ kN}$ )



Note: Marking: ▼ denotes a point of “Drop in vertical strength”.

▼ denotes a point of “Shear failure”

$N$ : Axial force (N)

$F_c$ : Concrete strength ( $\text{N/mm}^2$ )

$b$ : Column width (mm)

$D$ : Column depth (mm)

1 tonf = 2, 205 lbf = 9.8 kN

1 Mpa = 1  $\text{N/mm}^2$

1  $\text{N/mm}^2$  = 145 psi

**Figure 2.2.6 Simplified Load-Deflection Curve with Brick Wall Specimens**



## CHAPTER 3. RETROFIT DESIGN OF MEMBERS AND FRAMES

### 3.1 INTRODUCTION

Retrofit design of members and frames are done based on the structural characteristics by the assessment and the target of the improvement of strength, ductility and irregularity of the building. Following methods are recommended to apply generally.

#### **RC wall, Steel framed brace and RC jacketing for column**

Retrofit method and system are introduced in Chapter 2. Followings are typical methods of retrofit. Retrofit design of members and frames are done subject to the conditions, such as required strength, ductility and irregularity.

- 1) Installing RC wall
- 2) Installing RC wing wall
- 3) Column (a) RC jacketing (b) Steel plate jacketing (c) Carbon fiber wrapping
- 4) Installing Steel framed brace
- 5) Beam strengthening
- 6) Foundation strengthening
- 7) Non-structural elements (components)
- 8) Post-installed anchor

It is to be noted that above method 1), 2) and 4) are used to increase strength generally. Method 3) (a) and 3) (b) are used for both strength and ductility increase. Method 3) (c) is used to increase ductility.

It is recommended to increase strength of members and frames of buildings as stated in Chapter 2. In this manual, 1) RC wall, 4) Steel framed brace (in-direct connection method), 3) RC jacketing for column, and 8) post-installed anchor are introduced. Refer to the Japanese Guidelines for methods not covered in this manual.

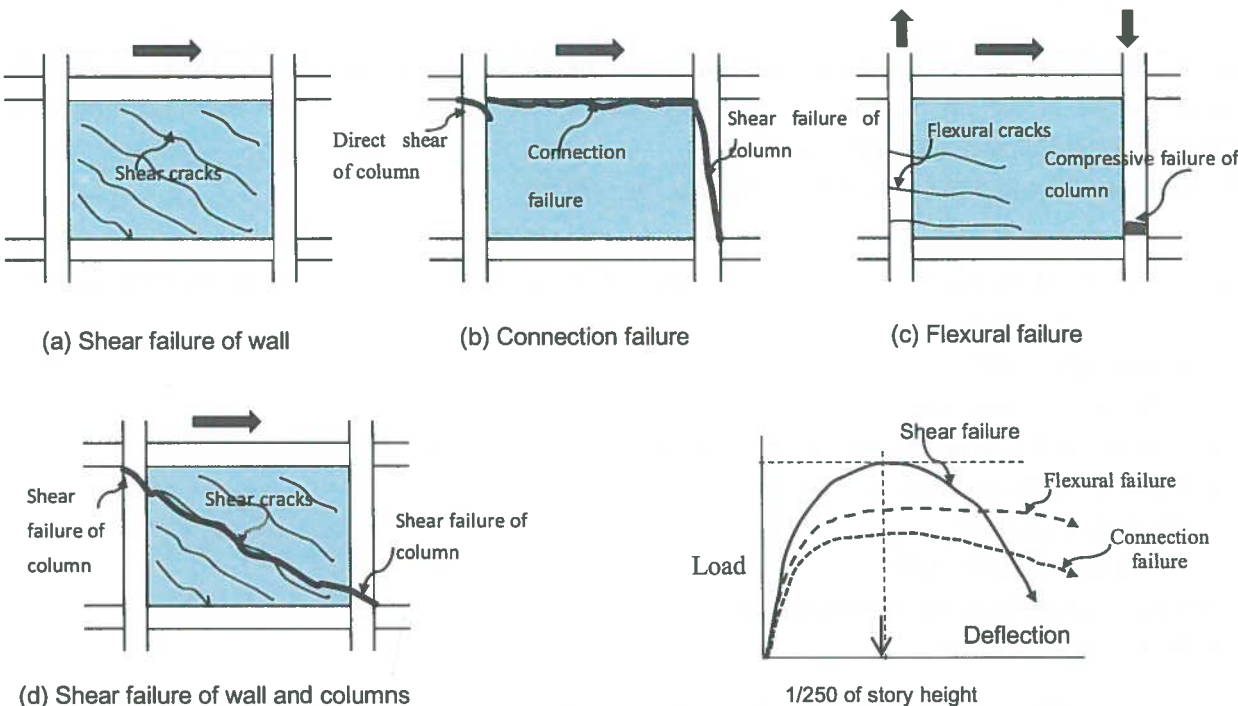
### 3.2 INSTALLING RC WALLS

#### 3.2.1 Outline

Retrofit by installing shear wall is to install new RC wall and/or replace existing brick wall with RC wall, in order to increase horizontal load carrying capacity of existing building. This is a strength oriented retrofit. Post installed anchors are used for the connection of new RC wall with existing RC frames. Non-shrink grout mortar is used at the top of new RC wall generally. Re-bar preventing split of concrete is also used.

### 1) Failure mode of RC wall

Following failure modes of Figure 3.2.1 are considered with respect to strength and ductility of RC wall and columns. Ductility index is evaluated based on the type of failure. Connection failure is not desirable because of its brittle nature.



**Figure 3.2.1 General Behavior of RC Wall**

Source: "Handbook of Earthquake Resistant Design of Structure", JSCA, (in Japanese)  
 Figure (d) is added from structural experiment 2013 by CNCRP.

It is noted that failure mode of (d) shear failure of wall and column has occurred because of weak column. (See the structural experiment by CNCRP 2013). This is very brittle failure without ductility. Storey deflection angle at shear failure of RC wall is evaluated as approximately 1/250 in Japan, but according to the same structural experiment by CNCRP this was approximately 1/200, because of slender size of columns. (Refer to Supplement A2).

### 3.2.2 Target Performance

Installations of shear walls are designed so that the capacity of retrofitted building meets the demand capacity. Strength is evaluated considering the failure mode of shear wall, boundary frame or uplift strength of wall. Expected strength of in-filled shear wall is  $\tau = 0.25 F_c$  ( $\tau$  is the average shear stress of wall in clear span of columns,  $F_c$  is compressive strength of existing concrete) in case of walls without opening. Different ductility is expected due to the failure mode. Ductility Index  $F$  is set as follows. In case of mid-rise buildings, shear failure mode and flexural failure mode are generally considered.

- |                              |               |
|------------------------------|---------------|
| (i) Shear failure mode       | ..... 1.0     |
| (ii) Flexure failure mode    | ..... 1.0~2.0 |
| (iii) Foundation uplift mode | ..... 1.0~3.0 |

#### (1) Shear wall

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

**(2) Flexural wall**

The ductility index of a flexural wall should be calculated by Eq.(13) of the J. Standard 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  $F = 1.0$

If  ${}_wQ_{su}/{}_wQ_{mu} \geq 1.3$  then  $F = 2.0$  (in case of wall without columns,  $F = 1.5$ ) (13) of J. Standard

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 of J. Standard

${}_wQ_{mu}$  = Shear force at the flexural strength of the wall, calculated according to the item 3.2.2 (2) (c) of the J. Standard. The inflection point of wall is estimated.

Flexural strength of wall should be calculated based on Supplementary Eq. A2.1.1 of J. Standard.

$${}_wM_u = a_t \cdot \sigma_{sy} \cdot l_w + 0.5 \sum (a_{wy} \cdot \sigma_{wy}) \cdot l_w + 0.5N \cdot l_w \text{ (N} \cdot \text{mm)} \quad (\text{A2.1-1 of J. Standard})$$

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

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

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

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

**3.2.3 Ultimate Strength of Column****(1) Ultimate flexural strength**

(a) The ultimate flexural strength of column shall be calculated with Equation (A1.1-1 of J. Standard).

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

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

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

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

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

$$M_u = 0.8a_t \cdot \sigma_y \cdot D + 0.4N \cdot D \quad (\text{A1.1-1 of J Standard})$$

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 ( $\text{mm}^2$ ).

$a_g$  = Total cross sectional area of reinforcing bars ( $\text{mm}^2$ ).

$b$  = Column width (mm).

$D$  = Column depth (mm).

$\sigma_y$  = Yield strength of reinforcing bars ( $\text{N/mm}^2$ ).

$F_c$  = Compressive strength of concrete ( $\text{N/mm}^2$ ).

(b) The multi layered reinforcement shall be considered in using Equation (A1.1-1 of J. Standard).

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

**(2) Ultimate shear strength**

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

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

Where:

- $p_t$  = Tensile reinforcement ratio (%).
- $p_w$  = Shear reinforcement ratio,  $p_w = 0.012$  for  $p_w \geq 0.012$ .
- $\sigma_{wy}$  = Yield strength of shear reinforcing bars (N/mm<sup>2</sup>).
- $\sigma_o$  = Axial stress in column (N/mm<sup>2</sup>).
- $d$  = Effective depth of column.  $D-50\text{mm}$  may be applied.
- $\frac{M}{Q}$  = Shear span length. Default value is  $\frac{h_o}{2}$ .
- $h_o$  = Clear height of 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 Equation (A1.1-2). And if the value of  $\sigma_o$  is greater than  $8\text{N/mm}^2$ , the value of  $\sigma_o$  shall be  $8\text{N/mm}^2$  in using Equation (A1.1-2).

**3.2.4 Strength of in-Filled RC Wall****(1) Shear strength of wall ( ${}_wQ_{su}$ )**

Shear strength of wall as a united wall is calculated. In addition to this, shear strength is calculated from the comparison of failure mode of shear wall panel and connection as follows.

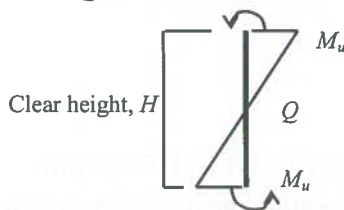
$${}_wQ_{su} = \min. \{Q_{su}, {}_wQ'_{su} + 2 \cdot \alpha \cdot Q_c, Q_j + {}_pQ_c + \alpha \cdot Q_c\} \quad \text{The J. Guidelines (modified 3.1.5-3)}$$

Where,

- ${}_wQ_{su}$  = Shear strength of shear walls.
- ${}_wQ'_{su}$  = Shear strength of infilled shear panel (only for the panel part in the clear height and width).
- $Q_j$  = Sum of the shear strength of connectors underneath the beam.
- ${}_pQ_c$  = 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.
- $\alpha$  = 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

$Q_{su}$  = Shear strength as a united shear wall with columns

$Q_c$  is calculated as follows,



Shear force at the flexural yielding,  $Q = 2 \cdot M_u / H$ , and compared with shear strength of column  $Q_u$ , smaller value is  $Q_c$

## (2) Shear strength as a united shear wall with columns

$$Q_{su} = \left\{ \frac{0.053 p_{te}^{0.23} (18 + F_c)}{\frac{M}{(Q \cdot l)} + 0.12} + 0.85 \sqrt{p_{se} \cdot \sigma_{wy}} + 0.1 \sigma_{0e} \right\} \cdot b_e \cdot j_e \quad \text{for } 1 \leq \frac{M}{(Q \cdot l)} \leq 3 \quad (\text{N}) \quad \text{The J. Standard (A2.1-2)}$$

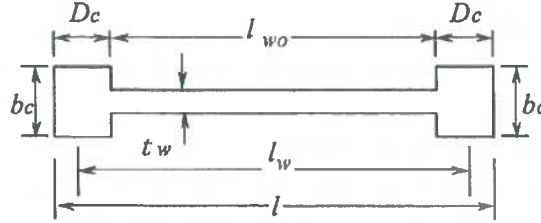


Figure 3.2.2 Wall with Boundary Columns (Ref. The J. Standard Fig. A2.1-1)

Where:

 $p_{te} = 100a_t / (b_e \cdot l)$  : 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. $b_e = \Sigma A / l$  : Equivalent thickness of the wall. $\Sigma A$  = 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. $\sigma_{wy}$  = Yield strength of the lateral reinforcing bar. $\sigma_{0e} = N / (b_e \cdot l)$ : Axial stress. The  $\sigma_{0e}$  shall be not greater than  $8 \text{ N/mm}^2$ . $j_e$  = 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.

## (3) Shear strength of in-filled shear wall panel (only for the panel in the clear height and width)

$${}_w Q_{su} = \max (p_w \cdot {}_w \sigma_y, F_{cw} / 20 + 0.5 \cdot p_w \cdot {}_w \sigma_y) \cdot t_w \cdot l' \quad (3.1.5-4) \text{ the J. Guidelines}$$

Where:

 $p_w, {}_w \sigma_y$  = Wall reinforcement ratio and yield strength of the wall reinforcing bar ( $\text{N/mm}^2$ ) $F_{cw}$  = Concrete strength of the installed panels ( $\text{N/mm}^2$ ) $t_w l_w$  = Wall thickness and clear span of installed wall panel (mm)

**(4) Direct shear strength of columns**

$${}_pQ_c = K_{min} \cdot \tau_o \cdot b_e \cdot D$$

The J. Guidelines (3.1.5-5)

Where:

$$K_{min} = 0.34 / (0.52 + a/D).$$

$$\tau_o = 0.98 + 0.1F_{cl} + 0.85\sigma \text{ in case } 0 \leq \sigma \leq 0.33 F_{cl} - 2.75$$

$$0.22 F_{cl} + 0.49\sigma \text{ in case } 0.33 F_{cl} - 2.75 < \sigma \leq 0.66 F_{cl}$$

$$0.66 F_{cl} \text{ in case } 0.66 F_{cl} < \sigma$$

$b_e$  = 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.

$a_i$  = Shear span; distance between the beam face at the column top and the point of lateral force from the infilled wall.  $a = D/3$  is used generally.

$F_{cl}$  = Specified concrete strength of existing structures (N/mm<sup>2</sup>).

$$\sigma = p_g \cdot \sigma_y + \sigma_o.$$

$p_g$  = Ratio of  $a_g$  (gross cross section area of longitudinal reinforcing bars of a column concerned) to  $b_e \cdot D$ .

$\sigma_y$  = Yield strength of longitudinal reinforcing bars of a column.

$\sigma_o$  =  $N/(b_e \cdot D)$ , where  $N$  is an axial force of the column at ultimate mechanism, positive value means compression force.

**(5) Structural detail**

- Ladder type re-bars or equivalent shall be provided at anchor area to prevent split failure of wall panel.
- Concrete strength for strengthening shall not be less than existing concrete.
- Wall thickness of not less than  $t_w = 160$  mm with double layer reinforcement is recommended.
- Wall reinforcement ratio  $p_w$  and connecting re-bar (such as ladder re-bar) ratio of not less than 0.4% is required.

**3.2.5 Example on In-filled RC Wall**

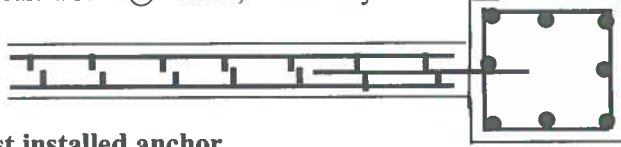
Shear strength of in-filled RC wall is calculated as follows.

**(1) Design Data:**

- No opening is assumed in inserted shear wall.
- Centre to centre span of column = 6000 mm
- Column size = 500 mm × 500 mm
- Story height = 3000 mm
- Depth of floor beam = 500 mm
- Main rebar of column = 8 –  $\phi 25$  mm deformed bar and tie spacing =  $\phi 10$  mm @ 150 mm
- Thickness of inserted shear wall = 160 mm
- Reinforcement of shear wall = Double layer  $\phi 8$  mm @ 150 mm in each direction
- Existing axial force on each column,  $N = 1750$  kN

**Wall**

Thickness: 160mm

Re-bar:  $\Phi 8\text{mm}@150\text{mm}$ , Double layer**Post installed anchor**Anchor:  $\Phi 19\text{mm}@150\text{mm}$  at perimeter**Column**Size: 500mm  $\times$  500mmMain bar: 8-  $\Phi 25\text{mm}$  deformed barTie (shear reinforcement):  $\Phi 10\text{mm}@150\text{mm}$ Existing axial force on each column,  $N = 1750 \text{ kN}$ **Figure 3.2.3 Column and Wall Section****(2) Material Data:**

## i. Existing column:

Concrete strength:  $F_c = 14 \text{ N/mm}^2$ Young's modulus:  $E_c = 17,580 \text{ N/mm}^2$ Yield strength of re-bar:  $\sigma_y = 275 \text{ N/mm}^2$ 

## ii. Inserted shear wall:

Concrete strength:  $F_c = 18 \text{ N/mm}^2$  (Allowable lowest for retrofit work)Young's modulus:  $E_c = 19,930 \text{ N/mm}^2$ Yield strength of re-bar:  $\sigma_y = 400 \text{ N/mm}^2$ Design strength of non-shrink grout mortar:  $F_m = 30 \text{ N/mm}^2$ 

Shear strength (or design) of in-filled RC wall (inserted shear wall) is calculated as follows:

The strength of shear wall shall be minimum between wall-column failure and joint-column failure. But this strength shall not be more than monolithic shear wall.

$${}_wQ_{su} = \min \{ {}_wQ'_{su} + 2 \cdot \alpha \cdot Q_c, \quad Q_j + {}_pQ_c + \alpha \cdot Q_c \}$$

**(3) Strength of existing column,****(a) Shear force of column at ultimate flexural strength**Column width,  $b = 500 \text{ mm}$ Column depth,  $D = 500 \text{ mm}$ Now,  $0.4b \cdot D \cdot F_c = 0.4 \times 500 \times 500 \times 14$ 

$$= 1,400 \times 10^3 \text{ N} < 1,750 \text{ kN (axial force on column)}$$

Axial compressive strength,  $N_{max} = a_g \cdot \sigma_y + b \cdot D \cdot F_c$ 

$$= 8 \times 490.6 \times 275 + 500 \times 500 \times 14$$

$$= 1,079 \times 10^3 + 3,500 \times 10^3 = 4,579 \times 10^3 \text{ N}$$

Ultimate flexural strength of column =  ${}_cM_u$ 

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

$$= (0.8 \times 3 \times 490.6 \times 275 \times 500 + 0.12 \times 500 \times 500^2 \times 14) \times \frac{4579 \times 10^3 - 1750 \times 10^3}{4579 \times 10^3 - 1400 \times 10^3}$$

$$= (161.8 \times 10^6 + 210 \times 10^6) \times 0.889$$

$$= 330 \times 10^6 \text{ Nmm}$$

Where,

 $a_t$  = Total cross sectional area of tensile reinforcing bar =  $3 \times 490.6 \text{ mm}^2$ Therefore, shear force at ultimate flexural strength,  ${}_cQ_{mu}$

$$= 2 \times \frac{c M_u}{h_o} \quad (\text{where, } h_o = \text{Clear height} = 2500 \text{ mm})$$

$$= 2 \times 330 \times \frac{10^6}{2500}$$

$$= 264 \times 10^3 \text{ N}$$

(b) Ultimate shear strength of column due to shear reinforcement

$$c Q_{su} = \left\{ \frac{0.53 p_t^{0.23} (18 + F_c)}{M / (Q \cdot d) + 0.12} + 0.85 \sqrt{p_w \cdot s \sigma_{wy}} + 0.1 \sigma_o \right\} \cdot b \cdot j \quad (\text{A 1.1-2 J. Standard})$$

Where,

$p_t$  = Tensile reinforcement ratio in %

$$= \frac{a_t \times 100}{(b \cdot D)} = \frac{3 \times 490.6 \times 100}{(500 \times 500)} = 0.588\%$$

$p_w$  = Shear reinforcement ratio

$$= \frac{a_w}{s \times b} = \frac{2 \times 78.5}{150 \times 500} = 0.00209 \quad [a_w = \text{area of shear reinforcement and } s = \text{tie spacing}]$$

For 90° hook, shear reinforcement ratio =  $0.5 p_w = 0.5 \times 0.00209 = 0.00104$ .

$s \sigma_{wy}$  = Yield strength of shear reinforcing bar =  $275 \text{ N/mm}^2$

$$\sigma_o = \text{Axial stress in column} = \frac{1750 \times 10^3}{500 \times 500} = 7 \text{ N/mm}^2$$

$$\text{Check, } \frac{M}{Q \cdot d} = 2.5 < 3.0, \quad \left( 1.0 \leq \frac{M}{Q \cdot d} \leq 3.0 \right)$$

$$\begin{aligned} \text{So, } c Q_{su} &= \left( 0.053 \times 0.588^{0.23} \times \frac{18+14}{2.62} + 0.85 \sqrt{(0.00104 \times 275)} + 0.7 \right) \times 500 \times 0.8 \times 500 \\ &= (0.573 + 0.455 + 0.7) \times 500 \times 0.8 \times 500 \\ &= 345.6 \times 10^3 \text{ N} \end{aligned}$$

So, shear strength of boundary column,  $Q_c$  = minimum of  $c Q_{mu}$  and  $c Q_{su} = 264 \times 10^3 \text{ N}$

(c) Direct shear strength of column

$$p Q_c = K_{min} \cdot \tau_o \cdot b \cdot D$$

Where,

$$\begin{aligned} K_{min} &= \frac{0.34}{0.52 + \frac{a}{D}} \\ &= \frac{0.34}{0.52 + \frac{1}{3}} = 0.40 \end{aligned}$$

$$a = \text{Shear span} = \frac{D}{3} \quad (\text{commonly assumed})$$

Now check  $\sigma$  to calculate  $\tau_o$

$$\sigma = p_g \cdot \sigma_y + \sigma_o$$

Where,

$p_g$  = Column main rebar ratio

$$= \frac{a_g}{b \cdot D} = \frac{490.6 \times 8}{500 \times 500} = 0.0157$$

$\sigma_y$  = Yield strength of main rebar of column = 275 N/mm<sup>2</sup>

$\sigma_o$  = Axial stress of column

$$= \frac{N}{b \cdot D} = \frac{1750 \times 10^3}{500 \times 500} = 7.0 \text{ N/mm}^2$$

Therefore,

$$\sigma = p_g \cdot \sigma_y + \sigma_o$$

$$= 0.0157 \times 275 + 7.0 = 11.31 \text{ N/mm}^2, \text{ which is greater than } 0.66 F_{cl} \text{ (i.e. } 9.24 \text{ N/mm}^2)$$

$$\text{So, } \tau_o = 0.66 \cdot F_{cl} = 9.24 \text{ N/mm}^2$$

$$\text{And, } p_{Qc} = K_{\min} \cdot \tau_o \cdot b \cdot D$$

$$= 0.40 \times 9.24 \times 500 \times 500 = 924 \times 10^3 \text{ N} > Q_c \text{ [So, direct shear strength of column will not govern]}$$

#### (4) Shear strength of shear wall

(a) Shear strength of monolithic shear wall with boundary column is calculated as follows:

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

Check applicability of this equation:

In case of no detail analysis deflection height ( $\frac{M}{Q}$ ) may be considered as  $\frac{h_w}{2}$  i.e (height of shear wall/2)

$$\text{Now, check for } \frac{M}{Q \cdot l} = \frac{h_w}{2l}$$

$$= \frac{3000}{2 \times 6000} = 0.25 < 1.0, \left( 1.0 \leq \frac{M}{Q \cdot l} \leq 3.0 \right)$$

$$\text{Minimum value of } \frac{M}{Q \cdot l} = 1$$

In equation,

$b_e$  = Equivalent width of shear wall

$$= \frac{\sum A}{l} = \frac{160 \times 5500 + 500 \times 500 \times 2}{6500} = 212.3 \text{ mm}$$

$l$  = Total wall length = 6000 + 500 = 6500 mm

$p_{te}$  = Equivalent tensile reinforcement ratio in %

$$= 100 \times \frac{a_g}{\sum A}$$

$$= \frac{100 \times 490.6 \times 8}{212.3 \times 6500} = 0.284\%$$

$p_{se}$  = Equivalent lateral reinforcement ratio in %

$$= \frac{a_h}{b_e \cdot s}$$

$a_h$  = Area of horizontal reinforcement in shear wall

$s$  = Spacing of horizontal reinforcement in shear wall

$$\text{Now, } P_{sc} \cdot \sigma_y = \frac{2 \times 50.2}{150 \times 212.3} \times 400 = 1.261$$

$\sigma_{oe}$  = Axial stress

$$= \frac{N}{b_e \cdot l}$$

$$= \frac{2 \times 1750000}{212.3 \times 6500}$$

$$= 2.53 \text{ N/mm}^2 < 8 \text{ N/mm}^2$$

$J_e$  = Distance between centroid of tension and compression forces =  $0.8 \times l$

$$\begin{aligned} \text{So, } Q_{su} &= \left( 0.053 \times 0.284^{0.23} \times \frac{18+18}{1+0.12} + 0.85 \sqrt{(1.261)} + 0.1 \times 2.53 \right) \times 212.3 \times 0.8 \times 6500 \\ &= (1.28 + 0.954 + 0.253) \times 1103960 \\ &= 2,745 \times 10^3 \text{ N} \end{aligned}$$

(b) Shear strength of in-filled wall panel only

Strength of in-filled wall panel ( ${}_w Q'_{su}$ ) is calculated as

$${}_w Q'_{su} = \max \left( p_w \cdot {}_w \sigma_y, \frac{F_{cw}}{20} + 0.5 \cdot p_w \cdot {}_w \sigma_y \right) \cdot \tau_w \cdot l' \quad (\text{J. Guidelines 3.1.5-4})$$

Where,

$p_w, {}_w \sigma_y$  = Wall reinforcement ratio and yield strength of the wall reinforcing bar (N/mm<sup>2</sup>)

$F_{cw}$  = Concrete strength of the installed wall panels (N/mm<sup>2</sup>)

$\tau_w \cdot l'$  = Wall thickness and its span of the installed wall panel (mm).

$$p_w = 2 \times \frac{50.2}{160 \times 150} = 0.0042$$

For wall thickness = 160mm and double layer rebar 8mm @150 in each direction

$$\begin{aligned} \text{Now, } {}_w Q'_{su} &= \max \left( p_w \cdot {}_w \sigma_y, \frac{F_{cw}}{20} + 0.5 \cdot p_w \cdot {}_w \sigma_y \right) \cdot \tau_w \cdot l' \\ &= \max \left( 0.0042 \times 400, \frac{18}{20} + 0.5 \times 0.0042 \times 400 \right) \times 160 \times 5500 \\ &= \max (1.68, 1.74) \times 160 \times 5,500 = 1531 \times 10^3 \text{ N} \end{aligned}$$

Wall strength including column strength,

$$\begin{aligned} {}_w Q'_{su} + 2 \cdot \alpha \cdot Q_c &= 1531 \times 10^3 + 2 \times 0.7 \times 264 \times 10^3 \quad [\alpha = \text{Reduction factor considering deflection ability} = 0.7] \\ &= 1901 \times 10^3 \text{ N, which is less than the strength of monolithic shear wall } Q_{su} = 2745 \times 10^3 \text{ N} \end{aligned}$$

Therefore the capacity of shear wall = 1901 kN.

Calculation of ladder type rebar or spiral rebar is not shown here. Section 3.5.2 (3) may be followed for calculation.

**(5) Design of post installed anchor (Refer to Section 3.5.2)**

According to failure mode in-filled wall panel will fail at connection with upper beam. So, required number of anchor bolt for upper beam have to be calculated considering strength of wall panel only. Shear strength of bonded anchor,  $\phi 19\text{mm}$  @150 is supposed for strength calculation at upper and lower beam. Same spacing is generally provided at column.

Considering  $\phi 19\text{mm}$  bolt capacity of each anchor bolt will be minimum of following two calculations.

Strength of anchor material,  $Q_{a1} = 0.7 \cdot m \sigma_y \cdot s a_e = 0.7 \times 400 \times 283.4 = 79,348\text{N}$  (Refer to sec 3.5)

Bearing pressure strength of concrete,  $Q_{a2} = 0.4 \cdot \sqrt{E_c \cdot \sigma_B} \cdot a_e$

$$= 0.4 \sqrt{17580 \times 14} \times 283.4 = 56222 \text{ N (Equation 3.9.4-9) J. Guidelines}$$

So, capacity of each anchor bolt is,  $Q_a = 56.2 \text{ kN}$

If anchor bolt is provided @ 150 mm then total no of anchor bolt for upper beam = 35

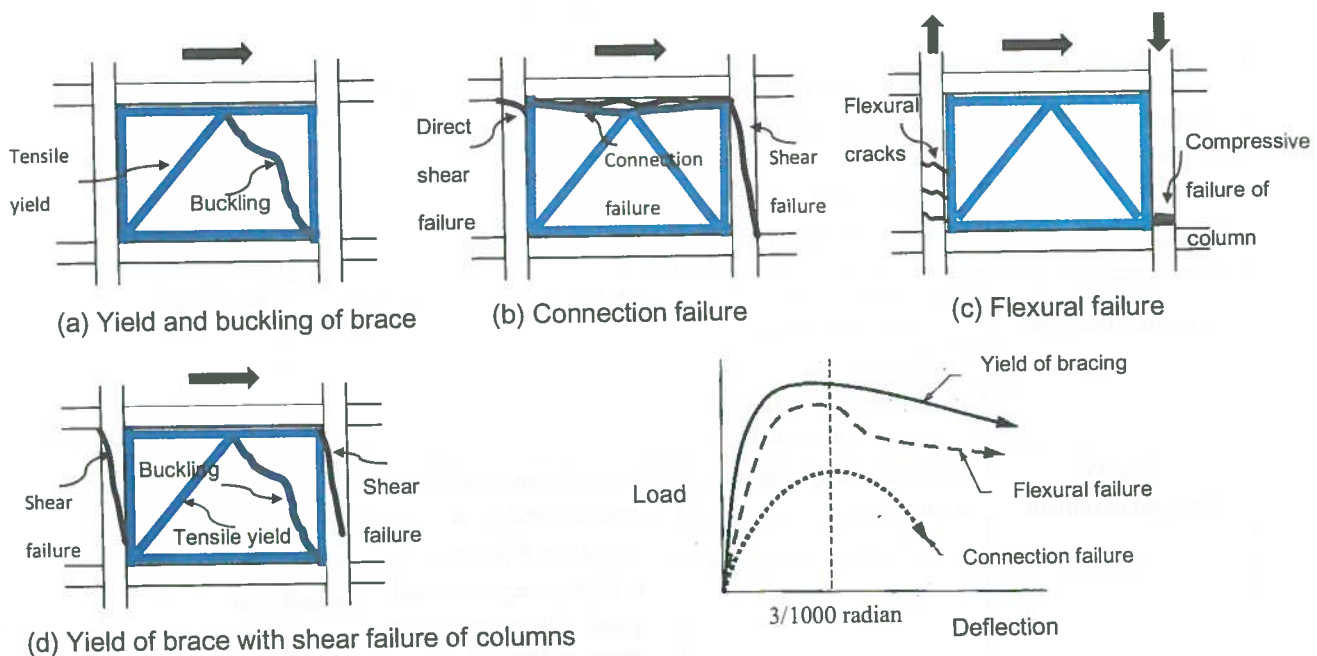
Strength at connection,

$$Q_{ja} = \text{Number of anchor} \times Q_a = 35 \times 56.2 = 1,968 \text{ kN} > {}_w Q_{su} = 1531 \text{ kN}$$

**3.3. STEEL FRAMED BRACE****3.3.1 Outline**

Failure modes of steel framed brace shall be evaluated with respect to strength and ductility. It is usual to provide steel frame at perimeter with brace. Slenderness ratio of steel brace is controlled. Detail of shear key such as post installed anchor and headed stud shall be studied. Indirect connection is used with non-shrink grout mortar as a standard method.

Following failure modes of steel framed brace are considered as shown Figure 3.3.1. Connection failure is not recommended because of its brittle nature.



**Figure 3.3.1 General Behavior of Steel Framed Brace**

Source; "Handbook of earthquake resisting design of structure", JSCA, (in Japanese)

Figure (d) is added from structure experiment by CNCRP.

It is noted that failure mode of (d) Yield of bracing with shear failure of columns is occurred in case of weak column as shown in the structural experiment by CNCRP 2013. This is a very brittle failure with low ductility. Storey deflection angle at this failure is evaluated as approximately 1/330 in Japan, but according to the same structural experiment by CNCRP this was approximately 1/200, because of slender size of columns. (Refer to Supplement 2).

### 3.3.2 Target Performance

As stated in the Section 3.4.2 of the J. Guidelines, resistance system of steel framed brace is categorized from type 1 to type 4 as shown in Table 3.4.2-1 of the J. Guidelines. Ductility index of structures strengthened with steel framed brace is shown in table 3.4.2-2 of the J. Guidelines.

1) Flexural RC column or shear failure RC column, ductility index,  $1.5 \leq F \leq 2.0$

In case of low strength concrete of RC frame,  $F \leq 1.5$

2) Direct shear and connection failure dominant, ductility index,  $F$  is 1.0

3) Flexural yielding of RC frame, and rotation, refer to the Tables of the J. Guidelines.

Table 3.4.2-1 and Table 3.4.2-2 from the translated J. Guidelines, version 2011, shows failure mechanism of structure and Ductility index of structures respectively.

**Table 3.3.1 Failure Mechanisms of Structure Strengthened with Steel Frame**  
(Ref: Table 3.4.2-1 the J. Guidelines)

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 beam	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

**Table 3.3.2 Ductility Index of Structures Strengthened with Steel Framed Brace**

(Ref: 3.4.2-2 the J. Guidelines)

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 beam 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 $\frac{Q_{su2}}{(\gamma \cdot Q_{ru})} < 1.1$ , $F = 1.5$ In case, $\frac{Q_{su1}}{(\gamma \cdot Q_{ru})} > 1.1$ , and $\frac{Q_{su2}}{(\gamma \cdot Q_{ru})} > 1.1$ , and $\frac{Q_{mu}}{(\gamma \cdot Q_{ru})} > 1.21$ , 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 $\gamma$  : See the provisions in the section of uplift wall of the Standard

### 3.3.3 Design of Steel Framed Brace

#### 1) Shear strength

Shear strength of steel braced frame is calculated as follows.

$${}_sQ_{su} = \min({}_sQ_{Bu}, {}_sQ_{Bj})$$

(Commentary.3.4.5-1) The J. Guidelines

Where:

${}_sQ_{Bu}$  = Shear strength of a framed brace at brace yield

${}_sQ_{Bj}$  = Shear strength of a framed brace at connection failure

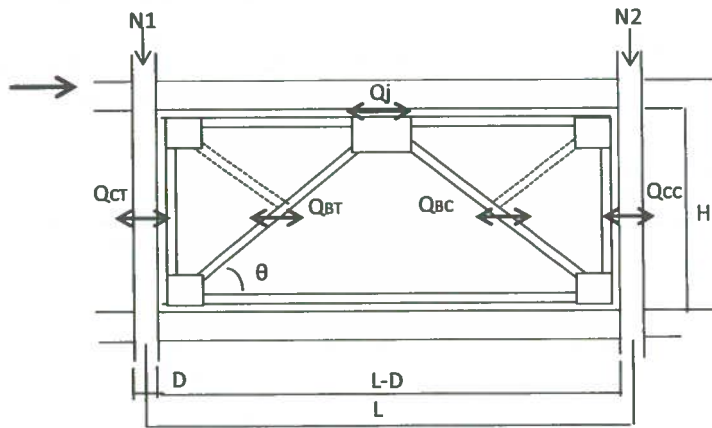
#### (a) Shear strength of a framed brace at brace yield ( ${}_sQ_{Bu}$ )

Shear strength of steel framed brace is calculated as shown in Figure 3.3.2

$${}_sQ_{Bu} = Q_{Bc} + Q_{Bt} + Q_{cc} + Q_{ct}$$

$$Q_{Bc} = f_{cr} \cdot A_B \cdot \cos\theta \quad (\text{horizontal component of compressive strength})$$

$$Q_{Bt} = F \cdot A_B \cdot \cos\theta \quad (\text{horizontal component of tensile strength})$$



**Figure 3.3.2 Resistance Mechanism of Steel Framed Brace**

Source; "Handbook of earthquake resisting design of structure", JSCA, (in Japanese)

Where;

$A_B$ : Section area of bracing member, effective section area is used in case of bolting connection ( $\text{mm}^2$ )

$Q_{cc}, Q_{ct}$ : ultimate shear strength of column at compression and tensile side respectively (N)

$F$ : Standard strength of steel (Yield strength,  $\text{N/mm}^2$ )

$f_{cr}$ : Limit compressive stress ( $\text{N/mm}^2$ ), calculated as follows,

$$f_{cr} = \{1 - 0.4(\lambda/\Lambda)^2\} \cdot F \quad \text{for } \lambda \leq \Lambda$$

The J. Guidelines (3.4.5-1)

$$f_{cr} = 0.6F/(\lambda/\Lambda)^2 \quad \text{for } \lambda > \Lambda$$

Where:

$f_{cr}$  = Limit compressive stress ( $\text{N/mm}^2$ ).

$$\Lambda = \text{Limit slenderness ratio} \quad \left( \Lambda = \sqrt{(\pi^2 \cdot E)/(0.6F)} \right)$$

$\lambda$  = Slenderness ratio

$F$  = Specified strength of steel ( $\text{N/mm}^2$ ).

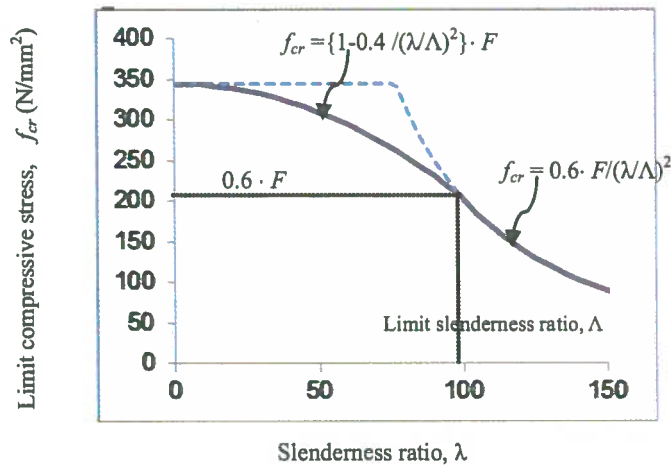
$E$  = Young's modulus of steel ( $\text{N/mm}^2$ ).

Note: 1) Above "Aspect ratio" will also be expressed by "Slenderness ratio".

Slenderness ratio  $\lambda$  is expressed by effective buckling length / radius of gyration of a member.

- 2) “ $F$  value” will be specified yield strength of steel ( $\text{N/mm}^2$ ).
- 3) Compressive strength is calculated by  $F_{cr}$  ( $\text{N/mm}^2$ ) multiplied by section area ( $\text{mm}^2$ ) of a compressive member.
- 4) In case of  $F = 345\text{N/mm}^2$ ,  $\Lambda$  will be  $\sqrt{\frac{\pi^2 \times 205000}{0.6 \times 345}} = 98.9$
- 5) Radius of gyration, refer to related design formula or a table of structural steel members.

Equation (3.4.5-1) of the Guidelines is shown by Figure 3.3.3 in case material  $F = 345\text{N/mm}^2$  (ASTM A-572).



**Figure 3.3.3 Limit Compressive Stress of Steel Bracing by Japanese Guidelines**

(Note: The idea of Euler's elastic buckling stress is the basis, and the range between  $0.6 \cdot F$  (yield stress) to  $F$  is approximated by a parabolic curve considering yield strength, unavoidable eccentricity and residual stress etc.)

**(b) Shear strength of a framed brace at connection failure ( ${}_sQ_{Bj}$ )**

$${}_sQ_{Bj} = {}_pQ_c + Q_j + Q_{cT} \quad (\text{Exp. 3.4.5-1}) \quad \text{The J. Guidelines}$$

Where:

${}_pQ_c$ : Direct shear strength of column at tensile side.

$Q_j$ : Shear strength at connection, and smaller value of post installed anchor and headed stud.

Shear strength of a stud is calculated as follows,

$$q_{ds} = 0.64 \cdot \sigma_{max} \cdot a_s$$

Where:

$\sigma_{max}$ : Tensile strength of stud, and is not more than  $400\text{N/mm}^2$

$a_s$ : Section area of stud ( $\text{mm}^2$ )

Headed stud of diameter 16mm and 19mm were tested, in the J. Guidelines. Tensile strength shall be tested where used. Same interval of post installed anchor and the headed stud will be used for the design.

## 2) Structural detail

- Slenderness ratio of steel brace shall not be more than 58 in principle.
- Minimum size of steel brace member should be H-150 (height)  $\times$  150 (width)  $\times$  7 (web)  $\times$  10 (flange) or equivalent stiffness members.
- Mortar connection area for steel frame shall follow,
  - Bond anchor with diameter 16mm, or expansion anchor with diameter 16mm with interval of not more than 250mm shall be used.
  - Headed stud with axis diameter of 16mm or 20mm shall be used, and interval shall be not more than 250mm.
  - Lap length of anchor and headed stud shall be more than half of anchor length and headed stud.
  - Strength of pressure grout mortar shall be not less than  $30\text{N/mm}^2$ .
  - Re-bar with not less than 0.4% for sprit prevention at mortar connection area is suggested.

### 3.3.4 Example Calculation of Steel Framed Brace

#### (1) Design condition

(Member dimension)

Span length : $L = 6000\text{mm}$	Clear span : $l = 5,500\text{mm}$
Story height : $H = 3,000$	Clear height : $h = 2,500\text{mm}$
Beam : $b \times D = 300\text{mm} \times 500\text{mm}$	Column : $b \times D = 500\text{mm} \times 500\text{mm}$
Clear dimension (gap) between steel and RC member, 150mm	
Existing axial force on each column, $N = 1750\text{kN}$	

(Material)

Strength of existing concrete: $\sigma_B = 14\text{N/mm}^2$	Young's modulus: $E_c = 17,580\text{N/mm}^2$
Yield strength of existing re-bar: $\sigma_y = 275\text{N/mm}^2$	Yield strength of steel: $F = 345\text{N/mm}^2$ (ASTM A572)
Design strength of pressured grout mortar: $\sigma_M = 30\text{N/mm}^2$	Tensile strength of stud: $\sigma_{max} = 400\text{N/mm}^2$
Yield strength of anchor re-bar: $\sigma_y = 400\text{N/mm}^2$	

(Assumed section)

Steel brace: C-200  $\times$  100  $\times$  10 (or 2L-100  $\times$  100  $\times$  10)  
 Frame: C-200  $\times$  100  $\times$  10 (or 2L-100  $\times$  100  $\times$  10)  
 A member for buckling prevention: C-200  $\times$  100  $\times$  10 (or 2L-100  $\times$  100  $\times$  10)

#### (2) Strength of existing column

Section detail; refer to the example calculation at Sec.3.2.4.

(a) Shear force of column decided from ultimate flexural strength

$${}_cM_u = M_u = (0.8a_t \cdot \sigma_y \cdot D + 0.12b \cdot D^2 \cdot F_c) \times ((N_{max} - N) / (N_{max} - 0.4b \cdot D \cdot F_c)) = 330 \times 10^6 \text{N}\cdot\text{mm}$$

$${}_cQ_{mu} = 2 \cdot {}_cM_u / h_o = 264 \times 10^3 \text{N}$$

(b) Shear strength

$${}_cQ_{su} = \left\{ \frac{0.053 p_t^{0.23} (18 + F_c)}{M / (Q \cdot d) + 0.12} + 0.85 \sqrt{p_w \cdot s \cdot \sigma_{wy}} + 0.1 \sigma_0 \right\} \cdot b \cdot j = 375 \times 10^3 \text{N}$$

(c) Direct shear strength

$${}_pQ_c = K_{min} \cdot \tau_o \cdot b \cdot D$$

$$= 0.40 \times 9.24 \times 500 \times 500 = 924 \times 10^3 \text{N}$$

**(3) Strength calculation of brace**

(a) Section, area = 3,800mm<sup>2</sup>, radius of gyration  $i_x = 80.2$  mm,  $i_y = 30.4$  mm

(b) Buckling length,  $l_{kx} = 3,720$  mm, (buckling length of brace for out-of-plane direction),

$l_{ky} = 1,860$  mm (length in-plane direction is reduced to half by providing buckling prevention member at center position)

(c) Slenderness ratio,  $\lambda_x = l_{kx}/i_x = 46.3 < 58$ ,

$\lambda_y = l_{ky}/i_y = 61.1 > 58$  This will be acceptable, since  $l_{ky}$  is reduced more considering in plane direction of gusset plate.

(d) Limit slenderness ratio, supposed yield strength of steel  $F = 345\text{N/mm}^2$ , (ASTM-A-572)

$$\Lambda = \sqrt{\frac{\pi^2 E}{0.6 \times 345}} = \sqrt{\frac{\pi^2 \times 205000}{0.6 \times 345}} = 98.9$$

(e) Compressive strength,  $N_c = f_{cr} \cdot A_B = 292 \times 3,800 = 1,109 \times 10^3\text{N}$

$$f_{cr} = \left\{ 1 - 0.4 \left( \frac{\lambda}{\Lambda} \right)^2 \right\} \cdot F = \left\{ 1 - 0.4 \left( \frac{61.1}{98.9} \right)^2 \right\} = 0.847 \times 345 = 292\text{N/mm}^2$$

(f) Tensile strength,  $N_o = F \cdot A_B = 345 \times 3,800 \times 0.8 = 1,048 \times 10^3\text{N}$  Effective area assumed to be 80% of full area considering bolt hole.

(g) Load carrying capacity of brace, horizontal component of axial strength of steel brace is calculated.

Inclination of steel brace is 42.3 degrees.  $sQ_u = (N_c + N_o) \cdot \cos \theta$

$$sQ_u = (1,109 + 1,048) \times 10^3 \times 0.739 = 1,595 \times 10^3\text{N}$$

(h) Average shear stress at mortar connection,  $\tau = sQ_u / A_M = 1,595 \times 10^3 / (200 \times 5,500) = 1.45\text{N/mm}^2$

(i) Strength of steel framed brace ( $sQ_{su1}$ )

$$sQ_{su1} = sQ_u \text{ (steel brace)} + Q_{cl} \text{ (left column)} + Q_{cc} \text{ (right column)} = 1,595 \times 10^3 + 2 \times 264 \times 10^3 = 2,123 \times 10^3\text{N}$$

**(4) Strength of frame at failure of headed stud ( $sQ_{su2}$ )**

Design force,  $sQ_u = 1,595\text{kN}$ , assume  $2 \times \phi 16\text{mm}@150\text{mm}$  ( $a_s = 201\text{mm}^2$ )

$$q_{ds} = 0.64 \cdot \sigma_{max} \cdot a_s = 0.64 \times 400 \times 201 = 51.4\text{kN/no.}$$

$$Q_{js} = 36\text{rows} \times 51.4 \times 2 = 3,700\text{kN}$$

Assume  $2 \times \phi 12\text{mm}@150\text{mm}$  ( $a = 113\text{mm}^2$ )

$$Q_{ds} = 28.9\text{kN}, Q_{js} = 2,080\text{kN}$$

$$sQ_{su2} = Q_{js} \text{ (Strength at joint)} + pQ_c \text{ (direct shear)} + Q_c \text{ (Shear)} = 2,080 + 924 + 264 = 3,268\text{kN}$$

**(5) Strength of frame at failure of bonded anchor ( $sQ_{su3}$ )**

Refer to section 3.5 for the notation of post installed anchor.

(a) Shear strength of one anchor ( $\phi 19\text{mm}$  anchor)

$$\tau_{a1} = 0.7 \cdot a\sigma_y = 280 < 294\text{N/mm}^2$$

$$q_{a1} = 0.7 \cdot a\sigma_y \cdot s a_e = 79\text{kN}$$

$$\tau_{a2} = 0.4 \cdot \sqrt{E_c \cdot \sigma_B} = 0.4 \sqrt{17580 \times 14} = 198\text{N/mm}^2 < 294\text{N/mm}^2$$

$$q_{a2} = 0.4 \cdot \sqrt{E_c \cdot \sigma_B} \cdot s a_e = 56 \text{ kN/no.}$$

(b) Strength of anchor

$$Q_{ja} = \frac{l_o \cdot Q_a}{s \cdot a} = \frac{5500 \times 56}{150} = 35 \times 56 = 1960 \text{ kN}$$

$$sQ_{su3} = Q_{ja} + pQ_c + Q_c = 1960 + 924 + 264 = 3,148 \text{ kN}$$

**(6) Load bearing capacity of steel framed brace**

Minimum of ( $sQ_{su1} = 2,123\text{kN}$ ,  $sQ_{su2} = 3,248\text{kN}$ ,  $sQ_{su3} = 3148\text{kN}$ )

i.e.  $sQ_{su} = 2123\text{kN}$

**(7) Evaluation of Ductility Index**

Resistance type of the steel framed brace is type I of the Table 3.3.1. Whichever smaller value of  $sQ_{su2}$  and  $sQ_{su3}$  is selected as the strength decided by the connection with direct shear of column and is used as  $sQ_{su2}$  for the Table 3.3.2 ( $sQ_{su3}$  or  $sQ_{su2}$ )/ $sQ_{su1} = 3148/2,123 = 1.48 > 1.1$ . Column is evaluated as flexural column. Then estimated ductility index of steel framed brace is  $F = 2.0$  from the Table 3.3.2.

It is noted that the ductility index of RC columns without steel framed brace is estimated as 1.27 separately. Refer to supplement A3, for  $N/b \cdot D \cdot F_c$  (axial force ratio, 0.5 in this case) of a column and the ductility index.

**3.4 COLUMN JACKETING****3.4.1 RC Jacketing****(1) Flexural strength of RC jacketed column**

(i) In case of upgrading strength

Flexural strength of RC jacketed columns to improve their flexural strength shall be calculated by the following equation.

When,  $N_{max} \geq N > 0.4 \cdot b_2 \cdot D_2 \cdot F_{cavg}$ ,

$$M_u = (a_t \cdot \sigma_y \cdot g + a_{t2} \cdot \sigma_{y2} \cdot g_2 + 0.12 \cdot b_2 \cdot D_2^2 \cdot F_{c1}) \cdot \frac{(N_{max} - N)}{(N_{max} - 0.4 \cdot b_2 \cdot D_2^2 \cdot F_{c1})} \quad \text{Newly added (3.3.4-2a)}$$

When  $0.4 \times b_2 \cdot D_2 \cdot F_{c1} \geq N \geq 0$

$$M_u = a_t \cdot \sigma_y \cdot g + a_{t2} \cdot \sigma_{y2} \cdot g_2 + 0.5 \cdot N \cdot D_2 \cdot \left(1 - \frac{N}{b_2 \cdot D_2 \cdot F_{c1}}\right) \quad (3.3.4-2) \quad \text{The J. Guidelines}$$

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.

$\sigma_{y2}$  = Yield strength of tensile reinforcement in the jacketing part of column ( $\text{N/mm}^2$ ).

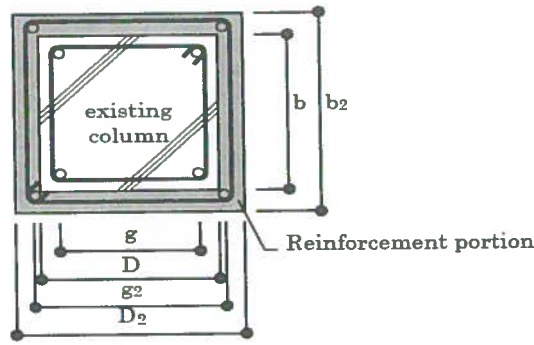
$b_2$  = Width of column after jacketing (mm).

$D_2$  = Depth of column after jacketing (mm).

$N_{max}$  = Axial compressive strength =  $a_t \cdot \sigma_y + a_{t2} \cdot \sigma_{y2} + b_2 \cdot D_2 \cdot F_{c1}$

$F_{c1}$  = Compressive strength of existing concrete ( $\text{N/mm}^2$ ).

In this manual, average strength per section area of existing and new concrete  $F_{cavg}$ , may be used instead of  $F_{c1}$ .



**Figure 3.4.1 Section of Column after Retrofit** (Ref: Fig. 3.3.4-2 The J. Guideline)

Above equation is the development of equation (A1.1-1) of the “J. Standard”. The introduction with theoretical assumption is shown in the Supplement A5.

Shear force of column decided from flexural strength is,

$${}_cQ_{mu} = 2 \cdot {}_cM_u / h_0$$

Where,  $h_0$  = clear height of column

### (2) Shear strength of RC jacketed column

Following equation is used to calculate shear strength of column retrofitted by RC jacketing. This equation was derived from experimental studies.

$$Q_{su} = \left\{ \frac{0.053 \cdot p_{t2}^{0.23} \cdot (F_{cavg} + 18)}{M / (Q \cdot d_2) + 0.12} + 0.85 \sqrt{p_w \cdot \sigma_{wy} + p_{w2} \cdot \sigma_{wy2}} + 0.1 \frac{N}{b_2 \cdot D_2} \right\} \times 0.8 \cdot b_2 \cdot D_2 \quad \text{The J.Guidelines (3.3.4-3)}$$

$\frac{M}{Q \cdot d_2}$  shall be 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.

$\sigma_{wy}$  = Yield strength of shear reinforcement in the existing column (N/mm<sup>2</sup>).

$\sigma_{wy2}$  = Yield strength of shear reinforcement in the jacketing column (N/mm<sup>2</sup>).

$d_2$  = Effective depth of the retrofitted column (mm).

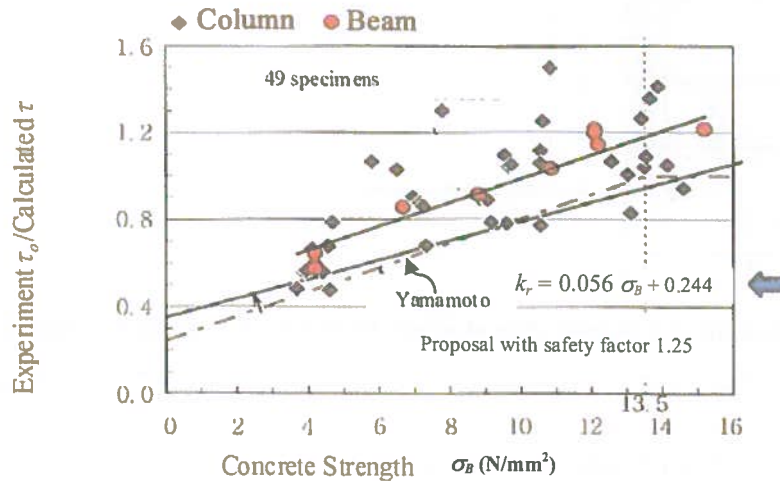
$M/Q$  = It shall be obtained by detailed calculation referring to the section 3.2.2 (2) of the standard.

### (3) Reduction factor in case of low strength concrete, for information only

This reduction factor is used for the evaluation of low strength concrete, and is shown for information only. In case that concrete strength  $F_c$  (or  $\sigma_B$  in Figure 3.4.1) is lower than 13.5N/mm<sup>2</sup> and above 9.0N/mm<sup>2</sup>, following reduction factor (by Prof. Yamamoto’s proposal) shall be multiplied for the evaluation of shear strength.

$$K_r = 0.056 \cdot \sigma_B + 0.244 \quad (3.4.1)$$

Where,  $\sigma_B$  = concrete strength (N/mm<sup>2</sup>)



(Source): Japan Concrete Institute, "Low strength Concrete", by Prof. K. Minami, Concrete Journal, vol. 48, No.7, Jul. 2010

**Figure 3.4.2 Reduction Factor of Shear Strength in case of Low Strength Concrete**

### 3.4.2 Example Calculation of Column Jacketing

Original section:

$$b \times D = 300\text{mm} \times 300\text{mm},$$

Compressive strength of original concrete  $F_c = 13.5\text{N/mm}^2$ ,

Main 4- $\Phi 20\text{mm}$  tie 10mm @ 250mm

Axial Force,  $N = 730\text{kN}$

$h_o$  (clear height) = 2,500mm

Yield strength of main rebar,  $\sigma_y = 275\text{ MPa}$

After jacketing:

$$b_2 \times D_2 = 500\text{mm} \times 500\text{mm},$$

Concrete  $F_c = 25\text{N/mm}^2$ ,

Re-bar,  $f_t = 400\text{N/mm}^2$ ,

Main 8- $\Phi 16\text{mm}$ ,

Tie 10mm@125mm

$F_{cavg}$  = Weighted average of concrete strength of old and new concrete

$$= \frac{13.5 \times 300 \times 300 + 25 \times (500 \times 500 - 300 \times 300)}{500 \times 500} = 20.86\text{ N/mm}^2$$

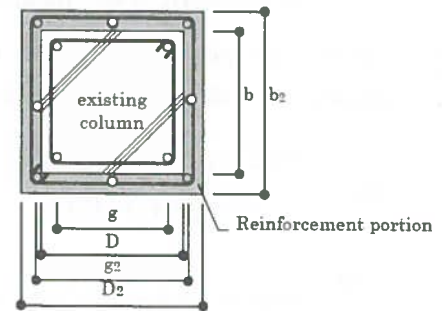
Axial force ratio,  $\sigma = N/b D F_{cavg} = 730 \times 10^3 / (500 \times 500 \times 20.86) = 0.14$

$$\text{Flexural strength, } M_u = a_t \cdot \sigma_y \cdot g + a_{t2} \cdot \sigma_{y2} \cdot g_2 + 0.5 \cdot N \cdot D_2 \cdot \left( 1 - \frac{N}{b_2 \cdot D_2 \cdot F_{cavg}} \right)$$

$$= 2 \times 314 \times 275 \times 188 + 3 \times 201 \times 400 \times 384 + 0.5 \times 730 \times 1000 \times 500 \times \left( 1 - \frac{730 \times 1000}{500 \times 500 \times 20.86} \right)$$

$$= 32.4 \times 10^6 + 92.6 \times 10^6 + 157.0 \times 10^6$$

$$= 282 \times 10^6 \text{ N}\cdot\text{mm}$$



Ultimate Shear Strength for flexural moment,  $Q_{mu} = 2 \cdot M_u / h_o$

$$= 2 \times 282 \times 10^6 / 2,500 = 225.6 \text{ kN}$$

Shear strength,

$$Q_{su} = \left\{ \frac{0.053 \cdot p_{t2}^{0.23} \cdot (F_{cavg} + 18)}{M / (Q \cdot d_2) + 0.12} + 0.85 \sqrt{p_{w1} \cdot \sigma_{wy} + p_{w2} \cdot \sigma_{wy2}} + 0.1 \frac{N}{b_2 \cdot D_2} \right\} \times 0.8 \cdot b_2 \cdot D_2$$

$$\frac{M}{Q \cdot d_2} = \frac{h_o}{d_2} = \frac{2500}{500} = 5 > 3 \quad \text{So,} \quad \frac{M}{Q \cdot d_2} = 3$$

$$p_{t2} = \frac{3 \times 201 \times 100}{500 \times 500} = 0.24\% \text{ (Previous re-bar ignored)}$$

$$p_{w1} = \frac{2 \times 30.6}{500 \times 225} = 0.00054 \quad \text{and} \quad p_{w2} = \frac{2 \times 78.5}{500 \times 125} = 0.00251$$

$$\begin{aligned} Q_{su} &= \left\{ 0.053 \times 0.24^{0.23} \times \frac{20.86 + 18}{3 + 0.12} + 0.85 \sqrt{(0.00054 \times 275 + 0.00209 \times 400)} + \frac{0.1 \times 730 \times 1000}{500 \times 500} \right\} \times 0.8 \times 500 \times 500 \\ &= (0.475 + 0.843 + 0.292) \times 0.8 \times 500 \times 500 \\ &= 322 \text{ kN} > 225.6 \text{ kN} \quad \therefore Q = Q_{mu} = 225.6 \text{ kN} \end{aligned}$$

Ductility index, plastic drift angle is calculated by following equation, refer to supplement A4

Upper limit,  $cR_{max}$

$$cR_{max} = \min \{ cR_{max}(n), cR_{max}(s), cR_{max}(\tau), cR_{max}(b), cR_{max}(h) \} \quad \text{The J. Standard (A.1.2-5)}$$

$$\eta = \frac{N_s}{b \cdot D \cdot F_{cavg}} = \frac{730 \times 10^3}{(500 \times 500 \times 20.86)} = 0.14 < 0.25$$

So,  $\eta_L = 0.25$  and  $\eta_H = 0.5$  and  $cR_{max}(n) = 1/30 (= 0.033)$

$$\tau = \frac{Q_{mu}}{b \cdot D} = \frac{225.6 \times 1000}{500 \times 500} = 0.902 \text{ N/mm}^2$$

$$\frac{\tau}{F_c} = \frac{0.902}{20.86} = 0.433 \quad \text{so,} \quad cR_{max}(s) = 1/30$$

$$p_t = \frac{3 \times 201 \times 100}{500 \times 500} = 0.24\% < 1.0\% \quad \text{so,} \quad cR_{max}(\tau) = 1/30$$

$$s/d_b = 125 \text{ mm} / 16 \text{ mm} = 7.8 < 8 \quad \text{so,} \quad cR_{max}(b) = 1/30$$

$$h_o/D = 2500 \text{ mm} / 500 \text{ mm} = 5 > 2.0 \quad \text{so,} \quad cR_{max}(h) = 1/30$$

$$\therefore cR_{max} = 1/30 \text{ and } cR_{my} = 1/150$$

$$\text{Now } cR_{mp} = 10 \times \left( \frac{cQ_{su}}{cQ_{mu}} - q \right) \times cR_{my} = 10 \times \left( \frac{322}{225.6} - 1.1 \right) \times \frac{1}{150} = 0.0218$$

$$\text{Then } cR_{mu} = cR_{my} + cR_{mp} = 0.00667 + 0.00218 = 0.0285 = \frac{1}{35}$$

Ductility index,  $F$ , refer to supplement A4,

The J. Standard (16)

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

$$= \frac{\sqrt{2 \times \frac{150}{35} - 1}}{0.75 \times \left(1 + 0.05 \times \frac{150}{35}\right)} = \frac{2.75}{0.91} = 3.02$$

Note) Evaluation of column only. Influence of brick standing wall and beam column joint is not considered.

### 3.4.3 Evaluation of Column Strength

Following cases will be incorporated for retrofit design to evaluate flexural strength of existing column, in case of 2<sup>nd</sup> level screening, supposing column collapse.

#### 1) Plain main-bar

It is suggested to reduce 20% of flexural strength of column tentatively, in case of plain main-bar and low strength concrete ( $F_c < 13.5\text{N/mm}^2$ ), of which bond stress is low. Refer to Supplement 2.

#### 2) Short anchor length of beam main-bar at external column

If short anchor length of beam main-bar at an external column is found to be short, it is suggested to reduce 25% and max. 50% (for small depth column such as 250mm) of flexural strength of column by 2<sup>nd</sup> level screening.

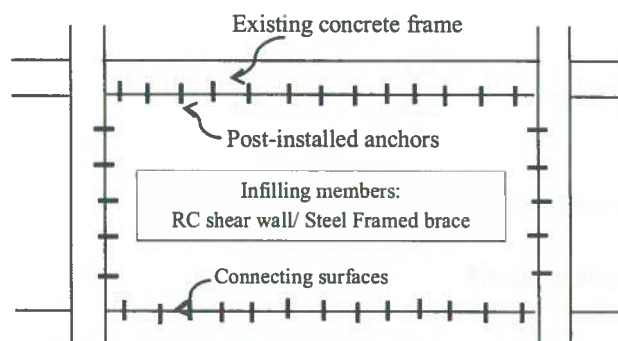
## 3.5 POST INSTALLED ANCHOR

### 3.5.1 General

Post-installed anchors are connecting materials between existing RC frames and newly installed strengthening members so that smooth transfer of the force can be made. Anchors are set and fixed after the drilling of holes on existing frames. There are two types of post installed anchor, expansion anchor as a metal type, and bonded anchor as an adhesive type.

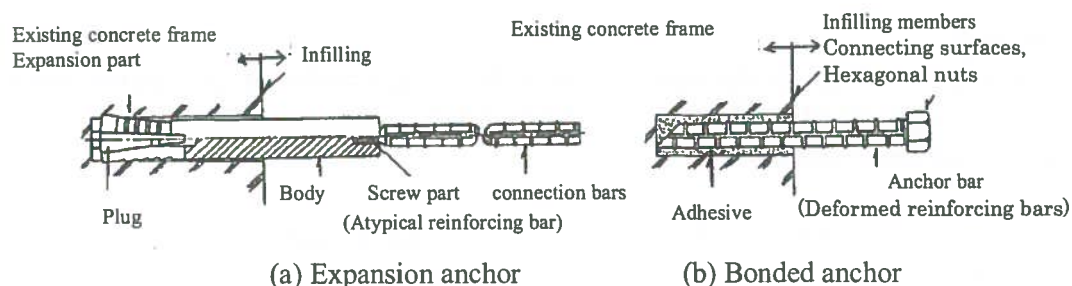
In case of design of post installed anchor, suitable anchor shall be selected with respect to strength, stiffness, ductility and workability.

(1) Post-installed anchors are devised to smoothly transfer force between surfaces connecting existing concrete frames and newly established infilling members for strengthening. For example, as shown in the example of use given in Figure 3.5.1, the anchor body or one end of the anchor bar is established by embedding in the existing concrete frame, and the other end is connected to the infilling strengthening members.



**Figure 3.5.1 Example of Use of Post-Installed Anchors***Source: Figure 3.9.1-1 of the Guideline*

There are two types of anchor as shown in Figure 3.5.2. Materials shall follow related standard/code.

**Figure 3.5.2 Detailed Example of Use of Post-Installed Anchors, and Names of Parts***Source: Figure 3.9.1-2 of the J. Guideline*

**(2) Terminology,** refer to the Section 3.9.1 of the Guidelines.

<b>Post-installed anchor construction method</b>	: A general term for a construction method of perforating existing concrete in order to infill shear walls or wing walls, etc., embedding anchor bars or anchor bodies.
<b>Bonded anchor</b>	: Post-installed anchors with anchor bars fixed into place and adhesive capsules, etc. installed in perforated parts of existing concrete.
<b>Expansion anchor</b>	: Post-installed anchors fixed by means of resistance mechanism between concrete and anchor body cast in parts where existing concrete has been perforated.
<b>Connecting surfaces</b>	: Position of connection with infilling walls newly established on existing concrete or strengthening materials of steel strengthening frame, etc.
<b>Fixation</b>	: Establishment of anchor in existing concrete.
<b>Material strength</b>	: Indicates basic performance of anchor, referring to steel member strength of body and connection bars in expansion anchors, and to steel member strength and adhesive performance in bonded anchors.
<b>Resistance mechanism strength</b>	: Anchor resistance strength when tensile force or shear force has acted in the anchor.
<b>Design strength</b>	: The smaller strength out of material strength and resistance mechanism strength calculated on the basis of material strength.
<b>Cone-shaped failure</b>	: Cone-shaped failure of concrete in concrete failure mode when tensile force has acted in the anchor.
<b>Steel member failure</b>	: Failure of steel members such as the anchor body, connection bars or anchor bars, etc. in the steel member failure mode when tensile force or shear force has acted in the anchor.
<b>Bond failure</b>	: Failure of parts bonded with anchor bars and concrete in the failure mode of bonded parts when tensile force has acted in the adhesive anchor.
<b>Bearing pressure failure</b>	: Concrete failure mode when shear force has acted in the anchor, and is concrete failure by bearing pressure of anchor.
<b>Pitch</b>	: Standard interval between anchors in parallel arrangement.
<b>Gauge</b>	: Interval between lines of anchors.
<b>Edge distance</b>	: Dimensions from core of anchor to concrete end in perpendicular direction of acting stress.
<b>End distance</b>	: Dimensions from core of anchor to concrete end in direction of acting stress.
<b>Effective embedding depth</b>	: Effective fixed length in concrete perforation parts of anchor.
<b>Embedding depth</b>	: Depth of embedding anchor into perforation parts, and length from welding base metal surface to end of expansion part of anchor body or end of anchor bar.

### 3.5.2 Design of Post-Installed Anchor

Shear resistance type or tensile resistance type is used as post-installed anchors. Strength or anchor material and strength of anchored concrete are calculated to decide design capacity of post-installed anchors for shear or tensile force. Then smaller value is the capacity of post-installed anchor against shear or tensile force.

#### (1) Shear capacity $Q_a$

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  $4d_a \leq l_e < 7d_a$

The J. Guidelines

$$Q_a = \min [Q_{a1}, Q_{a2}] \quad (3.9.4-1)$$

$$Q_{a1} = 0.7 m\sigma_y \cdot s a_e \quad (3.9.4-2)$$

$$Q_{a2} = 0.3 \sqrt{E_c \cdot \sigma_B} \cdot s a_e \quad (3.9.4-3)$$

But  $\tau (= Q_a/s a_e)$  shall not be greater than  $245 \text{ N/mm}^2$ .

(b) Expansion anchor in case of  $l_e \geq 7d_a$

The J. Guidelines

$$Q_a = \min [Q_{a1}, Q_{a2}] \quad (3.9.4-4)$$

$$Q_{a1} = 0.7 m\sigma_y \cdot s a_e \quad (3.9.4-5)$$

$$Q_{a2} = 0.4 \sqrt{E_c \cdot \sigma_B} \cdot s a_e \quad (3.9.4-6)$$

But  $\tau (= Q_a/s a_e)$  shall not be greater than  $249 \text{ N/mm}^2$ .

(c) Bonded anchor in case of  $l_e \geq 7d_a$

The J. Guidelines

$$Q_a = \min [Q_{a1}, Q_{a2}] \quad (3.9.4-7)$$

$$Q_{a1} = 0.7 \sigma_y \cdot s a_e \quad (3.9.4-8)$$

$$Q_{a2} = 0.4 \sqrt{E_c \cdot \sigma_B} \cdot s a_e \quad (3.9.4-9)$$

But  $\tau (= Q_a/s a_e)$  shall not be greater than  $294 \text{ N/mm}^2$ .

Where,

$Q_a$  = 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

$\sigma_B$  = Compressive strength of existing concrete. In general, the strength shall be obtained by compression test of concrete cores.

$E_c$  = Young's modulus of existing concrete ( $\text{N/mm}^2$ ). The test value can be used when measured during compression test.

$m\sigma_y$  = yield strength of expansion (mechanical) anchor ( $\text{N/mm}^2$ )

$\sigma_y$  = yield strength of re-bar for bonded (chemical) anchor ( $\text{N/mm}^2$ )

$s a_e$  = Cross section area of expansion anchor at concrete interface, or cross section area of bonded anchorage bar ( $\text{mm}^2$ )

$\tau$  = Shear stress at strength of an anchor (N)

$\tau_a$  = Bond stress at strength of bonded anchor against pull-out force (N/mm<sup>2</sup>)

$\tau_o$  = Basic bond stress at strength of bonded anchor (N/mm<sup>2</sup>)

$l_e$  = Effective embedment length of an anchor (mm)

$d_a$  = Diameter of anchor; nominal diameter of anchorage bar for bonded anchor or diameter of sleeve of expansive anchor (mm)

$F_c$  = Specified compressive strength of existing concrete (N/mm<sup>2</sup>)

## (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 of expansion anchor shall be the smaller value of  $T_{a1}$  and  $T_{a2}$ , which are determined by steel strength and cone failure of concrete, respectively. In case of bonded anchor, cohesive strength  $T_{a3}$  shall also be estimated and the smallest value shall be used.

### (a) Expansion anchor

The J. Guidelines

$$T_a = \min. (T_{a1}, T_{a2}) \quad (3.9.4-10)$$

$$T_{a1} = \min. (m\sigma_y \cdot s a_e, \sigma_y \cdot a_o) \quad (3.9.4-11)$$

$$T_{a2} = 0.23 \sqrt{\sigma_B \cdot A_C} \quad (3.9.4.12)$$

### (b) Bonded anchor

The J. Guidelines

$$T_a = \min. (T_{a1}, T_{a2}, T_{a3}) \quad (3.9.4-13)$$

$$T_{a1} = \sigma_y \cdot a_o \quad (3.9.4-14)$$

$$T_{a2} = 0.23 \sqrt{\sigma_B \cdot A_C} \quad (3.9.4-15)$$

$$T_{a3} = \tau_a \cdot \pi \cdot d_a \cdot l_e \quad (3.9.4-16)$$

$$\tau_a = 10 \sqrt{\frac{\sigma_B}{21}} \quad (3.9.4-17)$$

Where:

$T_a$  = 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 core failure (N)

$T_{a3}$  = Tensile capacity of an anchor determined by bond failure (N)

$l_e$  = Effective embedment length of an anchor (mm)

$d_a$  = Diameter of anchor; nominal diameter of anchorage bar for bonded anchor or diameter of sleeve of expansive anchor (mm)

$a_o$  = Effective cross section area of threaded steel bar, or nominal cross section area of anchorage bar (mm<sup>2</sup>)

$s a_e$  = Cross section area of expansion anchor at concrete interface, or cross section area of bonded anchorage bar (mm<sup>2</sup>)

$\sigma_B$  = Compressive strength of existing concrete (N/mm<sup>2</sup>)

$E_c$  = Yong's modulus of existing concrete (N/mm<sup>2</sup>)

$m\sigma_y$  = yield strength of expansion anchor (N/mm<sup>2</sup>)

$\sigma_y$  = Specified yield strength of steel bar (N/mm<sup>2</sup>)

$A_c$  = Effective projected area of anchor at the surface with 45 degree of cone failure (mm<sup>2</sup>).

**(3) Structural detail requirement**

- (a) Where tensile force acts, bonded anchor with effective length not less than  $10d_a$  shall be used.
- (b) Diameter of anchor  $d_a$  shall be in the range of 12mm to 22mm.
- (c) Interval shall be not be less than  $7.5d_a$  and not exceed 300mm.
- (d) Ladder type re-bars or spiral re-bar shall be provided around anchors to prevent split failure of concrete.

Reinforcement ratio  $p_s$  of ladder type reinforcement or others in the injected mortar shall not be less than 0.4%. The value of  $p_s$  is calculated by the following equation.

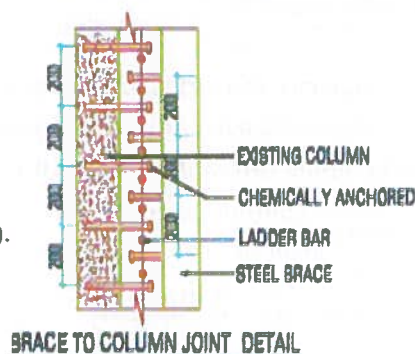
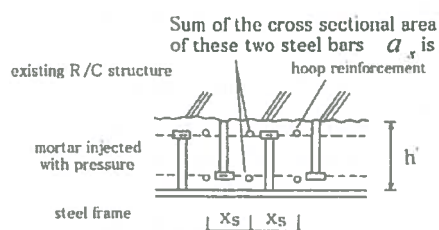
$$p_s = a_s / (h' \cdot X_s)$$

where:

$X_s$  = Interval of reinforcement (mm).

$a_s$  = Cross sectional area of one set of reinforcement ( $\text{mm}^2$ ).

$h'$  = Height of injected mortar (mm).



**Figure 3.5.3 An Example of Ladder Type Re-Bar**

**(4) Calculation example**

Example calculation of shear capacity of bonded (chemical) anchor, in case of  $14\text{N/mm}^2$  and  $18\text{N/mm}^2$  concrete strength is shown respectively in Table 3.5.1.

**Table 3.5.1 Shear Capacity of Bonded Anchor**

	$\sigma = 14\text{N/mm}^2$ $E = 2.1 \times 10^4 \times \sqrt{\left(\frac{14}{20}\right)} = 17,580\text{N/mm}^2$					$\sigma = 18\text{N/mm}^2$ $E = 2.1 \times 10^4 \times \sqrt{\left(\frac{18}{20}\right)} = 19,929\text{N/mm}^2$				
Size & interval of anchor	$Q_{a1} = 0.7\sigma_y \cdot a$ (N)	$Q_{a2} = 0.4 \times \sqrt{(E \cdot \sigma)} \cdot a$ (N)	$\min(Q_{a1}, Q_{a2})$ (kN)	Number of anchor	Total Q (kN)	$Q_{a1} = 0.7\sigma_y \cdot a$ (N)	$Q_{a2} = 0.4 \times \sqrt{(E \cdot \sigma)} \cdot a$ (N)	$\min(Q_{a1}, Q_{a2})$ (kN)	Number of anchor	Total Q (kN)
$\phi 16\text{mm}@200$	56269	39870	39870	26	1037	56269	47266	47266	26	1229
$\phi 16\text{mm}@150$	56269	39870	39870	35	1395	56269	47266	47266	35	1654
$\phi 20\text{mm}@200$	87900	62290	62290	26	1620	87900	73850	66652	26	1920
$\phi 20\text{mm}@150$	87900	62290	62290	35	2180	87900	73850	66652	35	2585
$\phi 22\text{mm}@200$	106383	75380	75380	26	1960	106383	89362	89362	26	2323
$\phi 22\text{mm}@160$	106383	75380	75380	30	2261	106383	89362	89362	30	2681

Note: 1) Effective embedment length  $l_e = 7d_a$ . (Diameter of an anchor).  $\sigma_y = 400\text{N/mm}^2$ .

2) Beam length is 5,500mm for the calculation of number of anchors.

- 3) Minimum interval of anchor is 120mm for 16mm diameter, 145mm for 19mm, and 165mm for 22mm diameter in case of  $18\text{N/mm}^2$  concrete as shown in Additional explanatory Table 1.2 of Section 3.0 of the J. Guideline.
- 4) In case of low strength concrete ( $9.0\text{N/mm}^2 \leq \sigma < 13.5\text{N/mm}^2$ ), reduction factor  $K_r$  (Equation 3.4.1) shall be applied for the capacity calculation  $Q_{a2}$  of an anchor.

### 3.6 NON-STRUCTURAL COMPONENTS (ELEMENTS)

#### 3.6.1 General

Non-structural components of buildings, such as external brick walls and partition brick walls are required to have the performance of,

- 1) Reasonable strength that can resist against the inertia force.
- 2) Reasonable deformability that can follow against the story deflection of structural frames.

Above requirements are necessary to prevent the drop and fall of brick walls to avoid human casualty.

Non-structural components of buildings, such as external brick walls, windows and partition brick walls will be damaged to some extent during an earthquake, but the drop and fall of brick walls should be prevented. In this point, retrofit design of existing brick walls will be considered. Retrofit of in-filled brick wall is required to consider both out of plane and in-plane movement of the wall.

#### 1) Design seismic load of non-structural components by BNBC

The strength is required against inertia force for out of plane direction of brick wall. Strength of brick walls against inertia force for out of plane direction will depend on thickness/ length of brick wall, supporting condition to structural frames, mortar/ brick strength, etc. There is no established method for retrofit of brick wall. Retrofit using steel angle members connected to a brick wall and supported by structural frame may be used. Other appropriate methods such as providing RC lintel will also be proposed where necessary.

#### (a) BNBC2015, Chapter 2 Loads on Buildings and Structures

##### Section 2.5.15 Seismic design of non-structural components

###### 2.5.15.1 Component Importance Factor

###### 2.5.15.2 Component Factor Transfer

###### 2.5.15.3 Seismic Design Force

$$F_c = \frac{\alpha_c \cdot a_h \cdot W_c \cdot I_c}{R_c} \left( 1 + 2 \times \frac{z}{h} \right) \quad (6.2.57, \text{BNBC2015})$$

Where,

$$0.75 \times a_h \cdot W_c \cdot I_c \leq F_c \leq 1.5 \times a_h \cdot W_c \cdot I_c$$

$\alpha_c$  = component amplification factor which varies from 1.0 to 2.5 (Table 6.2.22 or 6.2.23).

$a_h$  = expected horizontal peak ground acceleration (in g) for design =  $0.67 \times Z \cdot S$

In Dhaka,  $0.67 \times 0.20 \times 1.35$  (soil type *SD*) = 0.18, 0.153 (soil type *SC*)

$W_c$  = weight of component

$R_c$  = component response reduction factor which varies from 1.0 to 12.0 (Table 6.2.22 or Table 6.2.23).

$z$  = height above the base of the point of attachment of the component, but  $z$  shall not be taken less than 0 and the value  $z/h$  need not 1.0.

$h$  = roof height of structure above the base

For example, brick walls of ordinary building in Dhaka (soil *SD*), at top floor, seismic design force coefficient  $\frac{F_c}{W_c} = 1.0 \times 0.18 \times 1.0 \times \frac{3}{2.5} = 0.22$ . At ground floor,  $\frac{F_c}{W_c} = 0.07$ , and intermediate is interpolation. It seems that there is no clear requirement for the deformability of non-structural components.

### (b) BNBC 93 Chapter 2 Loads

Section 2.5.8 Seismic Lateral Forces on Components and Equipment Supported by Structures

2.5.8.1 Lateral forces on Structural and non-structural Components, and Equipment

$$F' = Z \cdot I' \cdot C' \cdot W' \quad (2.5.10, \text{BNBC 93})$$

Where:

$F'$  = Total lateral seismic forces

$Z$  = Seismic zone coefficient as given in Table 6.2.22 Dhaka = 0.15

$I'$  = Seismic Importance coefficient for components as given in table 6.2.23

( $I'$ : Essential facilities,  $I'=1.5$ , III&VI: Special and Standard occupancy structures,  $I'=1.0$ )

$C'$  = Horizontal force Coefficient as specified in Sec 2.5.8.2

(For rigid type, Table 6.2.26, II Non-structural components, 1 & 2, Exterior and interior ornamentation and appendages,  $C'=2.0$ )

$W'$  = Weight of an element, component or piece of equipment

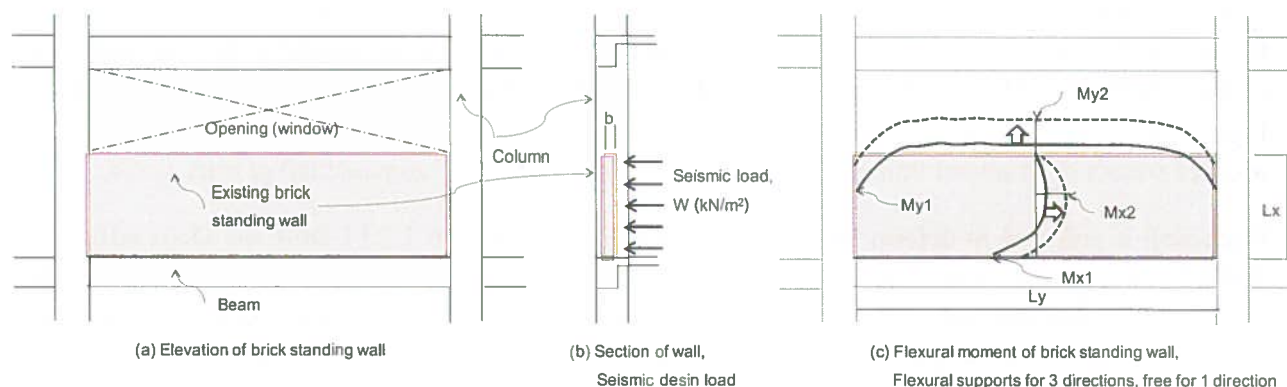
For example, external walls in Dhaka, lateral seismic coefficient  $\frac{F}{W} = Z \cdot I' \cdot C' = 0.15 \times 1.0 \times 2.0 = 0.30$ .

Load factor will be considered for the design of non-structural components.

### 2) In-filled brick wall

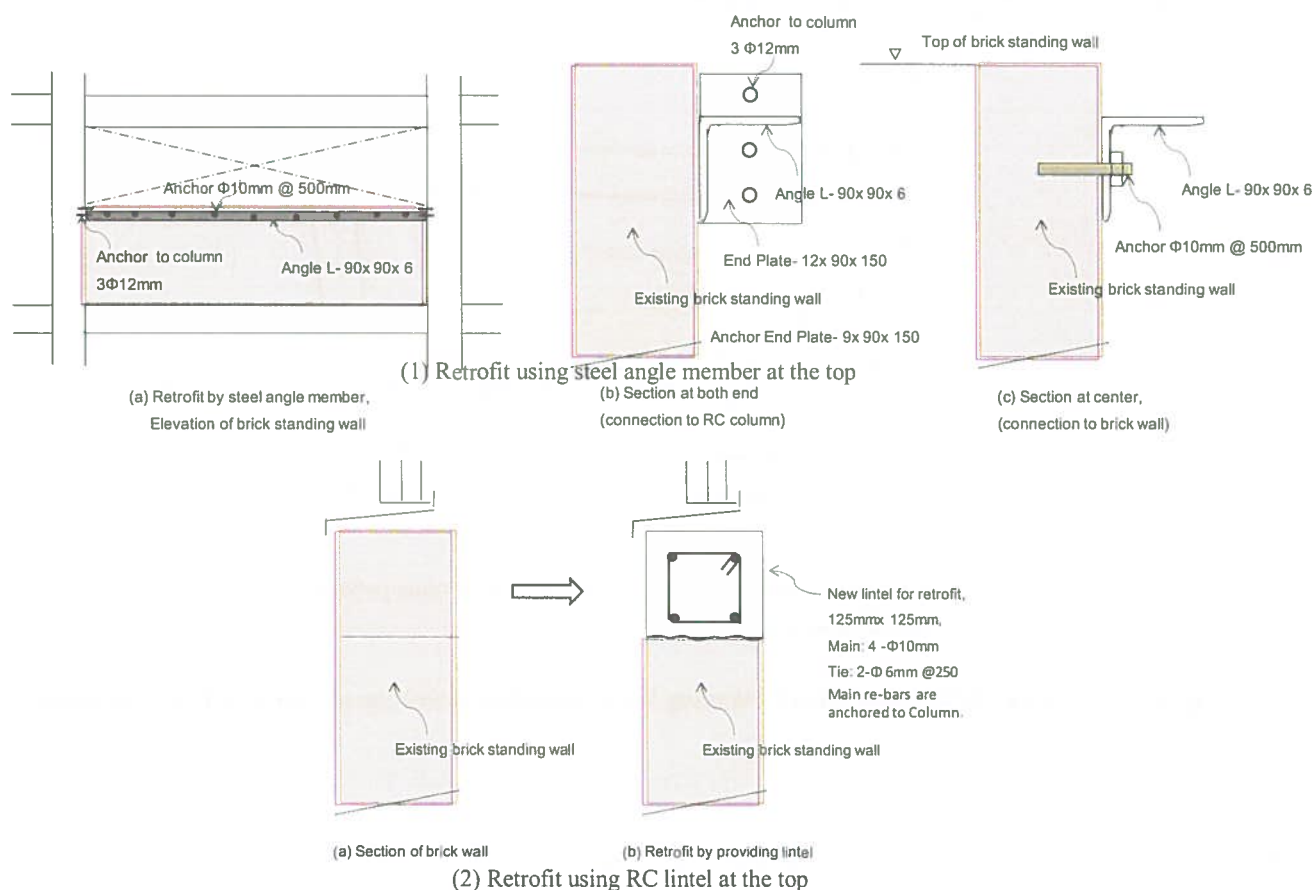
#### (a) Out of plane direction of in-filled brick wall

Reasonable strength of brick wall is required against seismic load of out of plane direction. An example is shown in (a) of Figure 3.6.1. In case of brick wall without any reinforcement, brick wall is required to resist by itself. Supposed flexural moment of brick wall is shown in (c) of the Figure. If the mortar tensile or bond strength is high,  $M_{x1}$  and  $M_{y1}$  will be the flexural moment of out of plane direction. Since the mortar tensile or bond strength at supporting points is low, flexural moment at center portion of wall will increase as shown flexural moment  $M_{x2}$  and  $M_{y2}$ . If these flexural moments exceed the flexural strength of the wall, the collapse of brick walls for out of plane direction will occur. This  $M_{x1}$  and  $M_{y1}$  can be calculated from the equivalent design chart of floor slab against vertical load, will be used for the evaluation purpose instead of  $M_{x2}$  and  $M_{y2}$ . Brick wall with 125mm thickness wall will be damaged first compared with 250mm thickness.



**Figure 3.6.1 In-filled Brick Wall, Seismic Load and Flexural Moment**

An example of retrofit is to provide a steel angle member as shown in (1) of Figure 3.6.2. This angle member is connected to brick wall, and also connected to the column through post installed anchors at both ends. Another method is to provide new RC lintel at the top of brick wall as shown in (2) of the Figure. Main re-bar will be connected by lap joint at both ends to post installed anchor (re-bar) to the column.



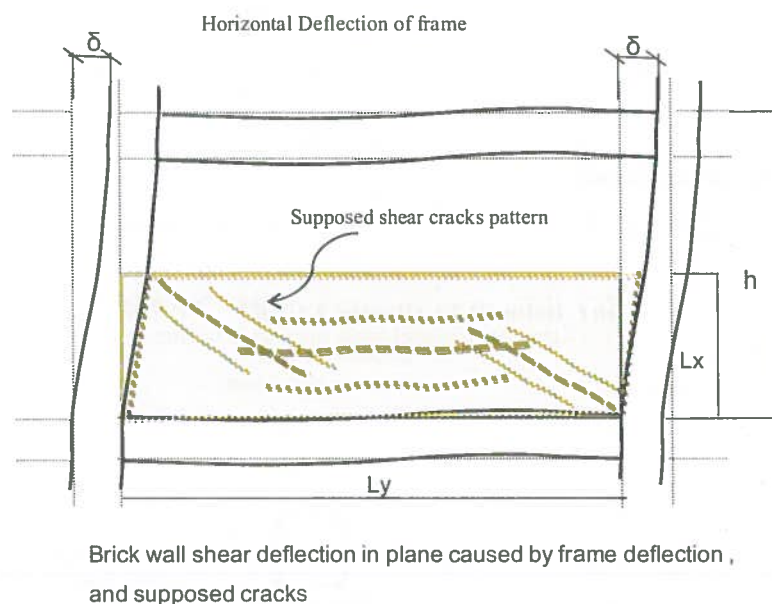
**Figure 3.6.2 An Example of Retrofit for Brick Standing Wall**

**(b) Deflection of brick wall in the plane direction**

The deformability is required against in plane direction of brick wall to meet the deflection of structural frames. Brick walls are generally brittle, but enough data/information is not available. Schematic picture of shear deflection of brick standing wall caused by deflection of frame in the plane direction is shown in Figure 3.6.3. One proposed method of retrofit is to reduce the story deflection (angle) by the strength oriented retrofit of structural frames. This will reduce the possibility of drop and fall of brick walls.

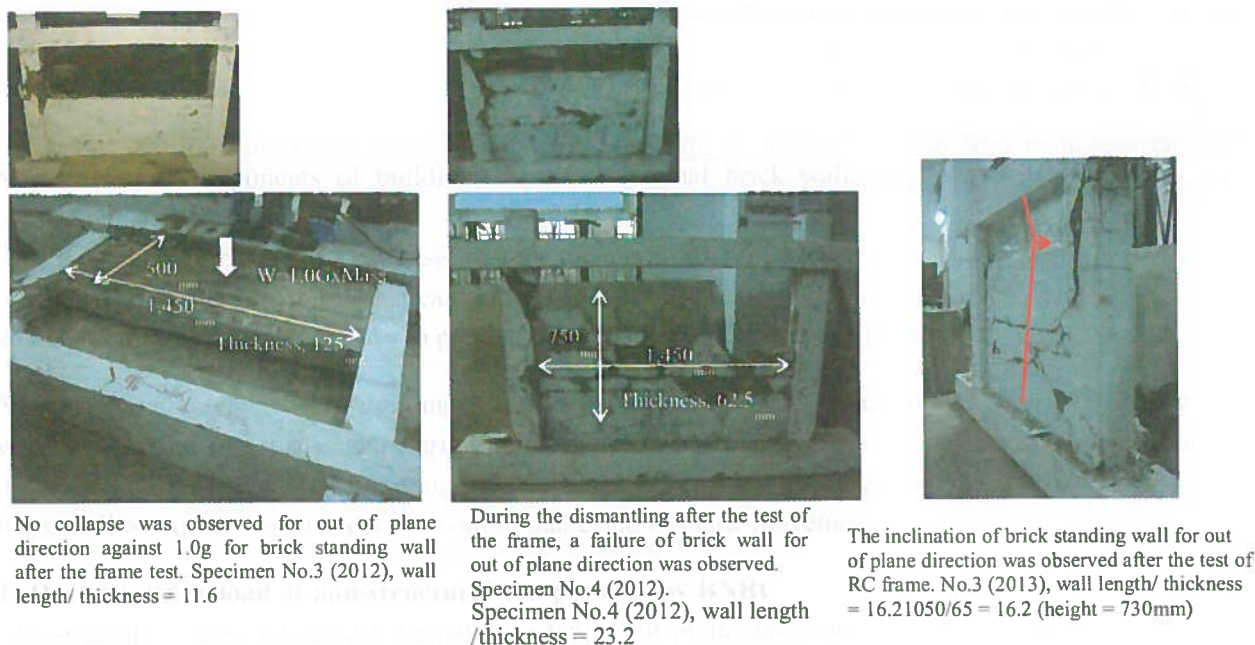
Brick wall is enforced to deform for in plane direction. According to 2.5.14 Drift and Deformation of BNBC2015, the design storey drift ( $\Delta$ ) shall not exceed the allowable drift ( $\Delta_a$ ), which is shown in the Table 6.2.21 Allowable Storey Drift Limit ( $\Delta_a$ ). In case of typical RC frame, allowable storey drift limit is  $0.025 \cdot h_{sx}$  (the storey height below level x), which is 1/40 of storey height. Infilled walls of new construction will be required to have certain performance of deformability in the in plane direction. In case of brick in-filled walls without any reinforcement, brick walls cannot deform up to this level and will be damaged heavily. If the brick walls are damaged in plane, the strength against out of plane direction will be reduced. This will cause the failure and drop of brick walls to the ground.

To prevent this type of failure of brick walls, it is recommended to prevent this type of failure of brick walls, to increase stiffness and strength of RC frames of the existing buildings to reduce horizontal deflection by providing RC walls and or steel framed braces.



**Figure 3.6.3 Shear Deflection of Brick Standing Wall Caused by Deflection of Frame in Plane Direction**

Following Figure 3.6.4 is the specimen after the experiment by CNCRP in 2012 and 2013. It is noted that the strength for out of plane direction will be reduced after the experience of deflection of in plane direction.



**Figure 3.6.4 Failure of Brick Walls after the Loading Test of RC Frames, CNCRP 2012 and 2013**

### 3) Support of M/E equipment

Retrofit of M/E equipment is required to consider both support of equipment and a support of piping.

#### (a) Support of equipment

Horizontal design load of equipment will be calculated by BNBC2015, Section 2.5.15 Seismic design of Non-structural components. Equipment is required to prevent sliding and overturning against this horizontal design load. Design forces for the design of anchor bolts (holding down bolts), is shown in Figure 3.6.5 based on, “Guidelines for the Design and Construction of Building Services 2005 written in Japanese”, by Building Center of Japan.

Tensile force of anchor bolt;  $R_b = R_H \cdot h_G - (W - F_V) \times \frac{L_G}{L \cdot n_t}$  (The J. Guidelines for B/S (10.7)

Shear stress of anchor bolt;  $\tau = \frac{F_H}{n \cdot A}$  or  $Q = \frac{F_H}{n}$  (10.8)

Where;

$G$  : Center of gravity of mass of equipment

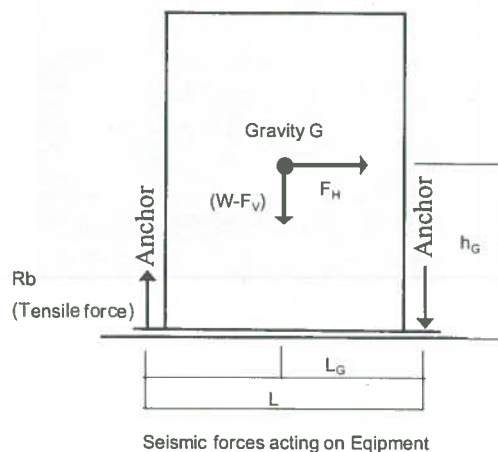
(A1.1-2 of J. Standard)  $W$  : Weight of equipment (kN)

$R_b$  : Tensile force of one number of anchor bolt

$n$  : Total number of anchor bolts

$n_t$  : Total number of anchor bolts at tensile side

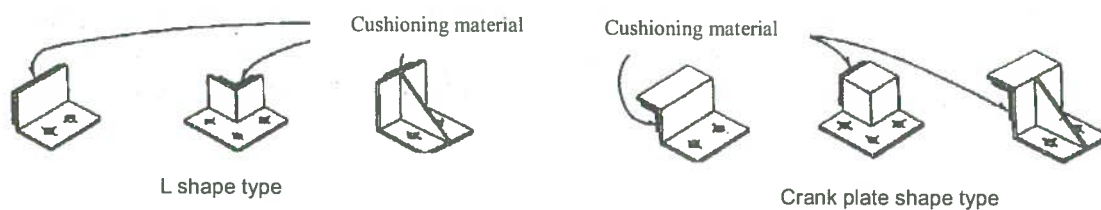
Member design of post installed anchor as anchor bolts (holding down bolts) will be done based on Chapter 3.5 Post installed anchor.



**Figure 3.6.5 Forces Acting on Equipment**

Source: Building Center of Japan, "Guidelines for the Design and Construction of Building Services 2005", in Japanese

In case that anchor bolts are not suitable for retrofit, stoppers as shown in Figure 3.6.6 will be used instead.



(1) Stopper preventing horizontal sliding

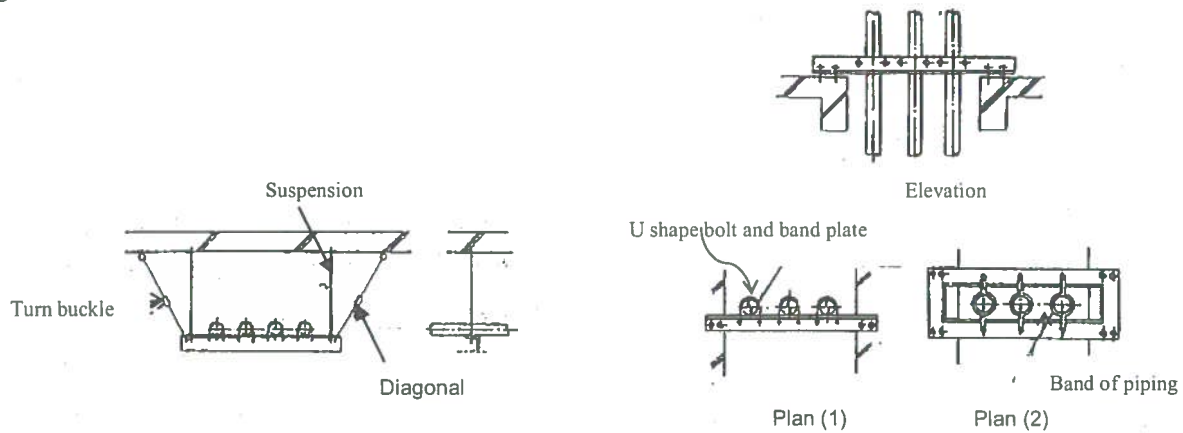
(2) Stopper preventing sliding and overturning

**Figure 3.6.6 Examples of Stopper**

Source: Building Center of Japan, "Guidelines for the Design and Construction of Building Services 2005", in Japanese

**(b) Piping**

It is required to control stresses and deflection of pipes and support against seismic load shown in BNBC2015, Section 2.5.15 Seismic design of Non-structural components. Vertical piping and horizontal piping are also required to provide suitable supporting to reduce excessive stress and deflection and can follow storey deflection of frames. An example of support for horizontal and vertical pipes is shown in Figure 3.6.7.



- (1) An example of support for horizontal piping      (2) An example of support for vertical piping

**Figure 3.6.7 Seismic Support for Piping**

Source: Building Center of Japan, "Guidelines for the Design and Construction of Building Services 2005", in Japanese



Figure 3.10 Typical Reinforcement Details for a Beam-Column Joint

## CHAPTER 4. EXAMPLES OF RETROFIT DESIGN OF BUILDINGS

### 4.1 CASE 1: RETROFIT DESIGN OF A 5 STOREY OFFICES BUILDING IN DHAKA

This retrofit design of an office building as exercise was done by CNCRP. The summary of the retrofit design is shown below. The building is a 5 storey office building. The building was not designed properly against earthquake load. There are no grade beams. Seismic performance was low as per result of seismic evaluation.

**Characteristics of the building:** Some of the important characteristics of the building are given below:

- 1) Typical office building with frame structure constructed in 1985.
- 2) No earthquake resisting design was performed.
- 3) No grade beams exist.
- 4) Low strength concrete was used.

#### (1) Outline of building

Usage	Office Building
Number of storey	Five
Building height	15.25 m
Structural type	R.C. Framed Structure
Foundation type	Shallow Foundation
Building area	377.38 m <sup>2</sup>
Total floor area	1886.9 m <sup>2</sup>
Year of design	1985
Year of construction	1985

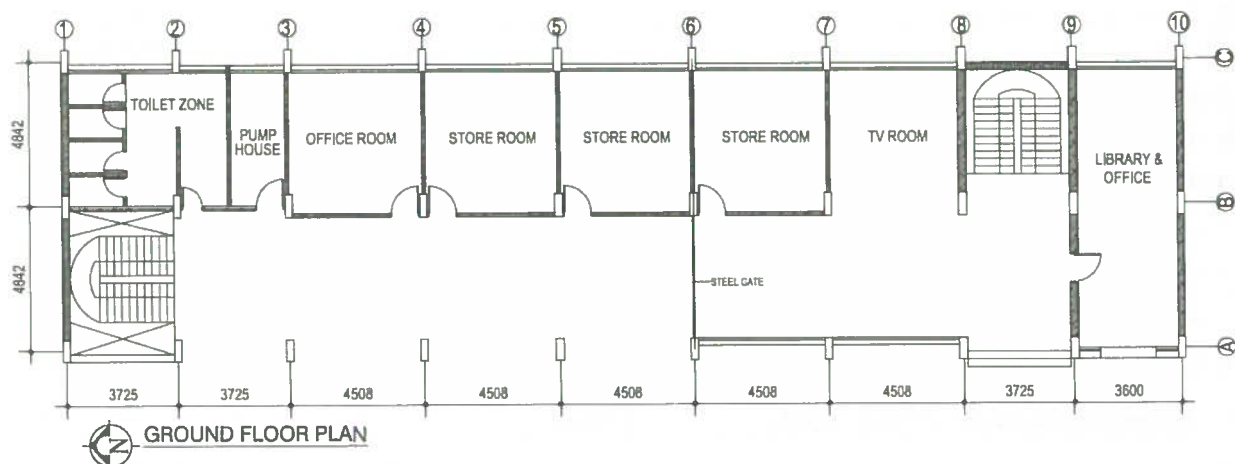


Figure 4.1.1 Typical Plan of Existing Building

**(2) Site survey:** It was conducted to get information of in-situ material strength (by core cutting and testing), foundation type (by excavation), location of brick wall and other important information regarding the structure.

## Summary of findings

Concrete Strength	9.2 N/mm <sup>2</sup> (Mpa) (Design strength $f'_c = 13.7$ Mpa)
Re-bar Yield Stress	275 N/mm <sup>2</sup> (Mpa)
Foundation capacity	1.00 $T_{sf}$ (As per structural drawing)
Brick wall	There is no load bearing wall. There are 125mm/250mm infill brick wall in frame.
Low Strength Concrete (<13.5N/mm <sup>2</sup> ) or not	Low Strength Concrete

**(3) Existing design drawings:** If architectural and structural drawings are available, the assessment becomes relatively easy. Otherwise, as-built drawing needs to be prepared. When drawings are available they need to be verified before start of the work.

Existence of structural design drawings/architectural design drawings	Structural drawings were available. However, no architectural drawings were available. So as built architectural drawings were prepared.
Countermeasures in case of non availability of structural design drawings.	Structural drawings were available in this case.

**(4) Application:** Strength of concrete influence the behavior of structure. When the strength is low it needs special attention.

Items	Findings
Buildings with minimum strength of concrete of 9.0N/mm <sup>2</sup> are eligible for retrofit.	Concrete strength slightly over the minimum requirement.
Code/Standard followed	Technical Manual for Seismic Evaluation and Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001 (Japanese Standard)
Superposition if any (such as impact of brick wall, etc)	Not Considered.

**(5) Performance of existing building:** This table will indicate the characteristics of the buildings which will influence the building performance during earthquake. For example, if the buildings do not have any grade beam the free height will be more which will allow more drift and hence influence the seismic behavior of the building.

**(A) Characteristics of buildings**

Existence of grade beam	No grade beam
Beam at superstructure	Beam at superstructure is present.
Unit weight	Roof = 6.3 kN/m <sup>2</sup> , Typical slab = 10.8 kN/m <sup>2</sup>
Total building weight	18692kN
Vertical irregularity	None
Eccentricity in plan by stiffness/ weight	None

**(B) Result of seismic evaluation:**

Seismic evaluation is done for each major direction and for each floor of a building (shown in Table 4.1.1). The proposed  $I_{so}$  is 0.3 for buildings located at Dhaka based on soil type SC of BNBC 2015. Cumulative strength index  $C_T S_D$  ensures the safety against the total collapse of the structure, which is selected as 0.15 for buildings in Dhaka. The first table below indicates that at lower level  $I_s$  is lower than 0.3 up to 4<sup>th</sup> floor for X-direction and 3<sup>rd</sup> floor for Y direction. Also  $C_T S_D$  is below required level at 1<sup>st</sup> and 2<sup>nd</sup> storey for X-direction. It is needed to take a retrofit plan that will ensure  $I_{so} > 0.3$  and  $C_T S_D > 0.15$  in each level and each direction.

Proposed seismic demand index of structure,  $I_{so}$

Storey	X direction		Y direction	
	$I_{so}$	$C_T S_D$	$I_{so}$	$C_T S_D$
Storey (1 To 5)	0.3	0.15	0.3	0.15

**Table 4.1.1 Result of Seismic Evaluation**

Storey	X direction		Y direction	
	$I_s$	$C_T S_D$	$I_s$	$C_T S_D$
5	0.303	0.433	1.61	1.073
4	0.096	0.137	0.596	0.497
3	0.108	0.154	0.463	0.386
2	0.092	0.032	0.294	0.294
1	0.050	0.072	0.178	0.178

**(6) Outline of retrofitting****(A) Retrofit plan**

Conditions to consider retrofit plan	Minimum disturbance of the structure and function was planned. So, insertion of shear wall was considered. As there was no grade beam in the original structure, addition of new grade beam is pre-requisite for inserting shear wall.
Impact to building function, building operation, obstruction to lighting, ventilation etc.	Placement of shear wall was such that it would only replace the brick wall (Y direction Shear wall). In case of X direction insertion of shear wall can eliminate ventilation and lighting problem.
Concept of layout for retrofit should be such that it eliminates irregularity of building.	The building has no irregularity problem so this was not considered.

(B) Retrofit design: The building behavior mainly depends on strength and ductility. A retrofit design can improve strength or ductility or both. Sometimes building performance is affected due to the presence of irregularity. By improving irregularity the structural performance  $I_s$  can be improved.

Component Type	Description
Strength oriented design or ductility oriented design	Strength oriented design
Improvement of structural irregularity/seismic load transfer.	Seismic load transfer will be provided by adding Shear wall.

**(7) Calculation of Demand:**

Before retrofitting scheme total demand of shear deficiency need to be calculated. A sample calculation of demand at ground floor X-direction for the building is as under:

$$I_s = E_o \cdot S_D \cdot T = \frac{n+1}{n+i} C \cdot F \cdot S_D \cdot T, \text{ Where } C = \frac{\Sigma Q}{W}$$

Considering no change in the system ( $F \cdot S_D \cdot T$ )

Required shear capacity,  $\Sigma Q = (I_{so} - I_s) \times W$

From Table 4.1.1 after evaluation it was found that at GF level seismic index at X-direction is  $I_s = 0.05$ .

Total building weight is 1869 kN.

So, shear requirement in X-direction at GF =  $(0.3 - 0.05) \times 1869 = 4673$  kN.

Alternative methods to find the best solution for the retrofitting work is explored here.

### (8) Calculation of alternate method:

#### A) Column Jacketing: Strength Calculation of Jacketed Column:

Original column:

Size =  $250 \times 750$

Main rebar = 16- 20d and Tie = 10d @ 225

$F_c = 9.2$  Mpa

$\sigma_y = 275$  Mpa

Jacketed part:

Jacketed column size =  $450 \times 950$

Additional main rebar = 12-d16

Hoop spacing = d10@150

$F_c = 25$  Mpa

$\sigma_{y2} = 400$  Mpa

Free height of column at ground floor = 3000

Axial load on column at GF,  $N = 1115$  kN

Flexural strength of column at GF

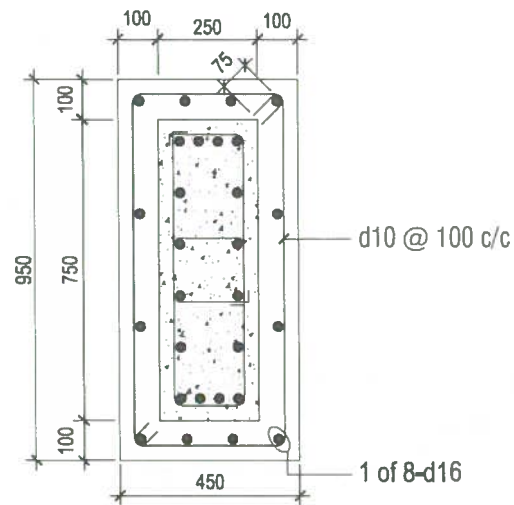


Figure 4.1.2 Plan of Jacketed Column

$$M_u = a_t \cdot \sigma_y \cdot g + a_{t2} \cdot \sigma_{y2} \cdot g_2 + 0.5ND_2 \left( 1 - \frac{N}{b_2 \cdot D_2 \cdot F_{cavg}} \right)$$

Where,

$a_t$  = Cross sectional area of tensile reinforcing bars of existing column.

=  $6 \times 314 \text{ mm}^2$ .

$a_{t2}$  = Cross sectional area of tensile reinforcing bars of jacketing part.

=  $4 \times 201 \text{ mm}^2$ .

$\sigma_{y2}$  = Yield strength of tensile reinforcing bars of existing column.

=  $275 \text{ N/mm}^2$ .

$g$  = Distance between tensile and compressive longitudinal reinforcement of existing column.

=  $250 - 2 \times 50$

=  $150 \text{ mm}$ .

$b_2$  = Width of column after jacketing.

=  $950 \text{ mm}$ .

$D_2$  = Depth of column after jacketing.

=  $450 \text{ mm}$ .

$$F_{cavg} = \frac{25 \times (950 \times 450 - 750 \times 250) + 9.2 \times 750 \times 250}{950 \times 450} = 18.1 \text{ N/mm}^2.$$

$$\begin{aligned} \text{So, } M_u &= 6 \times 314 \times 275 \times 150 + 4 \times 201 \times 400 \times 350 + 0.5 \times 1115 \times 1000 \times 450 \left( 1 - \frac{1115 \times 1000}{950 \times 450 \times 18.1} \right) \\ &= 379.1 \times 10^6 \text{ N-mm.} \end{aligned}$$

So, shear force at flexural strength,

$$\therefore Q_{mu} = \frac{2M_u}{h_0} = \frac{2 \times 379.1 \times 10^6}{3000} = 252.7 \text{ kN}$$

Ultimate shear strength

$$Q_{su} = \left\{ \frac{0.053 \times P_{t2}^{0.23} \times (F_{cavg} + 18)}{\left( \frac{M}{Q \cdot d_2} \right) + 0.12} + 0.85 \sqrt{p_w \cdot \sigma_{wy} + p_{w2} \cdot \sigma_{wy2}} + 0.1 \frac{N}{b_2 \cdot D_2} \right\} \times 0.8 \times b_2 \times D_2$$

Where,

$P_{t2}$  = Tensile reinforcement ratio in % with respect to column size after jacketing

$$= \frac{4 \times 201}{450 \times 950} \times 100 = 0.18\% \quad [\text{Previous rebar ignored}]$$

$P_w$  = Shear reinforcement ratio of existing column

$$= \frac{4 \times 78.5}{750 \times 225} = 0.00186$$

$P_{w2}$  = Shear reinforcement ratio after column jacketing

$$= \frac{2 \times 78.5}{950 \times 150} = 0.0011$$

Check;  $P_w + P_{w2} = 0.00186 + 0.0011 = 0.0030 < 0.012$

$$\sigma_{wy} = 275 \text{ MPa}$$

$$\sigma_{wy2} = 400 \text{ MPa}$$

$$M/Q = \frac{h_0}{2}$$

$$\therefore \frac{M}{Q \cdot d} = \frac{3000/2}{450} = 3.33 > 3$$

So,  $M/Q \cdot d = 3$  [Note that  $M/Q \cdot d$  shall not be more than 3]

$$\begin{aligned} Q_{su} &= \left\{ \frac{0.053 \times 0.18^{0.23} (18.1 + 18)}{3 + 0.12} + 0.85 \sqrt{0.00186 \times 275 + 0.0011 \times 400} + 0.1 \times \frac{1115 \times 1000}{450 \times 950} \right\} \times 0.8 \times 450 \times 950 \\ &= \frac{(0.413 + 0.829 + 0.260) \times 0.8 \times 450 \times 950}{1000} \\ &= 513.7 \text{ kN} > Q_{mu} \end{aligned}$$

$\therefore$  Shear strength of column = 252.7 kN.

Initial capacity of column at ground floor in X-direction is 50kN. After column jacketing at ground floor level X-direction one column will take shear of 252.7kN. Total shear requirement at ground floor X-direction is 4673kN. Approximate no. of column need to be jacketed is  $4673 / (252.7 \cdot 50) = 23$  Nos of column, considering ductility index,  $F = 1.0$ .

### B) Wing wall:

#### X-Direction: Ground Floor

#### Strength Calculation of Wing Wall:

Original column:

Size =  $250 \times 750$

Tie =  $10d @ 225$

$F_c = 9.2 \text{ Mpa}$

$\sigma_y = 275 \text{ Mpa}$

Wing wall each part size =  $250 \times 375$

Additional main rebar = 6 - d16

Hoop spacing =  $d10 @ 150 \text{ c/c}$

$F_{cl} = 25 \text{ Mpa}$

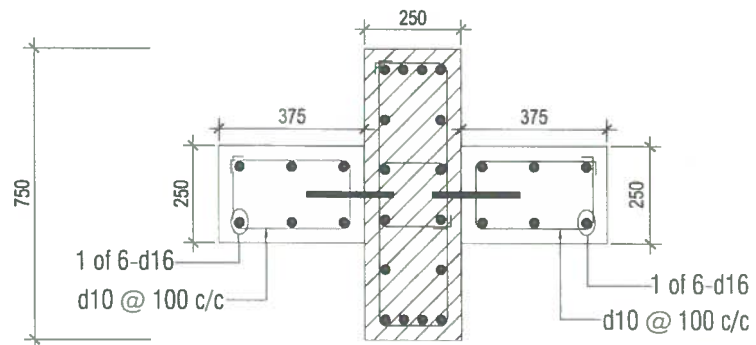


Figure 4.1.3 Plan of Column Wing Wall

Load carrying capacity of the column with wing wall cast-in-situ shall be smaller value of shear force at flexural strength  $Q_{mu}$  and shear strength  $Q_{su}$  indicate 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. For details of wing wall Sec-3.2 of J. guidelines is referred.

$$Q_{mu} = \phi \cdot M_u / h'$$

$$M_u = (0.9 + \beta) \cdot a_t \cdot \sigma_y \cdot D + 0.5N \cdot D \left\{ 1 + 2\beta - \frac{N}{\alpha_e \cdot b \cdot D \cdot F_{cl}} \left( \frac{a_t \cdot \sigma_y}{N} + 1 \right)^2 \right\}$$

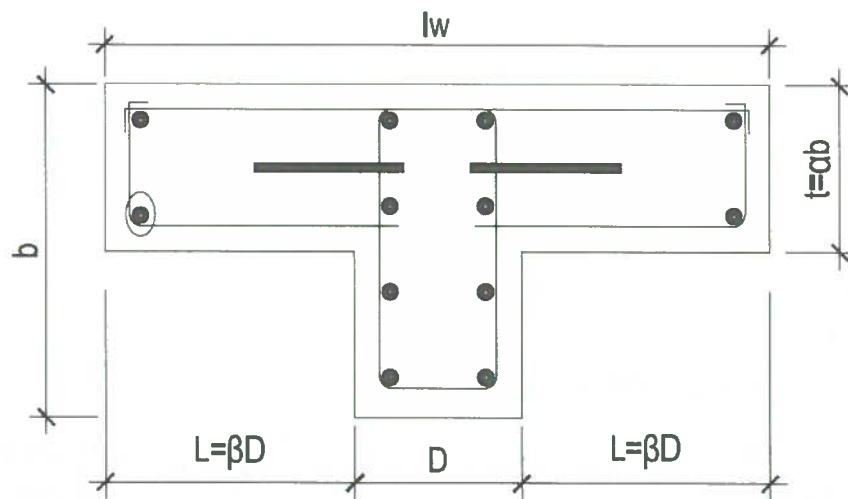


Figure 4.1.4 Notation for Wing Wall-1

Where,

$\alpha_e = (1 + 2\alpha\beta) / (1 + 2\beta)$ ,  $\alpha$  and  $\beta$  is referred to figure.

$\phi$  = Reduction factor (= 0.8) [Note: 20% reduction shows lower limit of experimental result]

$a_t$  = Gross sectional area of longitudinal reinforcing bars of column intension side ( $\text{mm}^2$ ).

=  $6 \times 314 \text{ mm}^2$ .

$\sigma_y$  = Yield strength of longitudinal reinforcing bars of column (N/mm<sup>2</sup>).

$$= 275 \text{ N/mm}^2.$$

$N$  = Axial force of column (N)

$$= 1115 \text{ kN}.$$

$F_{cl}$  = Specified design strength of concrete for wing wall (N/mm<sup>2</sup>).

$$= 25 \text{ N/mm}^2.$$

Where,

$\phi$  = Reduction factor (= 0.8)

$F_c$  = Specified design strength of concrete for existing structure.

$$= 9.2 \text{ N/mm}^2.$$

$M/Q = h'$

$d_e$  = Distance between the center of the tensile reinforcing bars and the extreme fiber of wing wall in comparison side (mm).

$$= (375 + 250 - 50)$$

$$= 575 \text{ mm}.$$

$p_w \cdot \sigma_{wy}$  = Product of hoop ratio and its yield strength in the existing columns (N/mm<sup>2</sup>).

$$= \frac{4 \times 78.5}{750 \times 225} \times 275 = 0.511$$

$$\text{For } 90^\circ \text{ hook, shear reinforcement ratio} = 0.5 p_w = 0.5 \times 0.511 = 0.256$$

$p_{sh} \cdot \sigma_{sy}$  = Product of lateral reinforcement ratio of installed wing wall and its yield strength (N/mm<sup>2</sup>).

$$= \frac{2 \times 78.5}{250 \times 100} \times 400 = 2.512$$

$t$  = Wall thickness of installed wing wall (mm) = 250 mm.

$ab = t$  i.e.  $\alpha(750) = 250$  So,  $\alpha = 0.33$

$$\beta = L/D = 375/250 = 1.5$$

$$\text{Now, } \alpha_e = \frac{1 + 2 \times 0.33 \times 1.5}{1 + 2 \times 1.5} = 0.5$$

$$\text{Therefore, } b_e = \alpha_e \cdot b = 0.5 \times 750 = 375$$

$$j_e = 7 d_e / 8$$

$$= 7 \times 575 / 8 = 503 \text{ mm}.$$

$$d_e = 250 + 375 - 50 = 575.$$

$$p_{te} = 100 a_t / (b_e \cdot d_e)$$

$$= 100 \times (6 \times 314) / (373.125) \times (575)$$

$$= 0.878$$

$$p_{we} \cdot \sigma_{wy} = \rho_w \cdot \sigma_{wy} (b/b_e) + \rho_{sh} \cdot \sigma_y (t/b_e)$$

$$= 0.256 \times \frac{750}{375} + 2.512 \times \frac{750}{375} = 2.187$$

$$\sigma_{oe} = N / b_e \cdot j_e$$

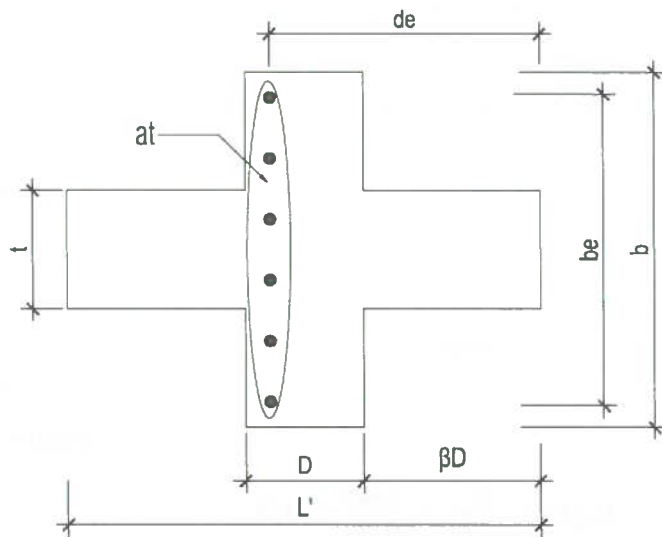


Figure 4.1.5 Notation for Wing Wall -2

$$= \frac{1115 \times 1000}{373.1 \times 503.1}$$

$$= 5.94 \text{ N/mm}^2.$$

$$M/(Q \cdot d_e) = \frac{h_{cwo}}{L'}$$

Where,

$h_{cwo}$  = The inflection height.

$L'$  = Total depth including wing walls.

$$L' = 2(375) + 250 = 1000.$$

The inflection height  $h_{cwo}$  shall be assumed based on the result of elastic or inelastic analysis. In case of non conducting elastic or inelastic analysis, the inflection height can be calculated as follows.

$$h_{cwo} = h_{co} + (h_{wo} - h_{co}) \frac{L_w}{L}$$

$$= h_o / 2 + (h_w / 2 - h_o / 2) \times \frac{L_w}{L}$$

$$= \frac{3000}{2} + \left( \frac{15250}{2} - \frac{3000}{2} \right) \times \frac{750}{4650}$$

$$= 2488.$$

Where

$L$  = Typical column span length = 4650mm

$L_w$  = Total length of wing wall without column  
= 750mm

$h_{wo}$  = Inflection height as a wall with column  
=  $h_w / 2$

$h_{co}$  = Inflection height as a column  
=  $h_o / 2$

$h_o$  = Clear height of column  
= 3000mm

$h_w$  = Total height of wing wall  
= 15250mm

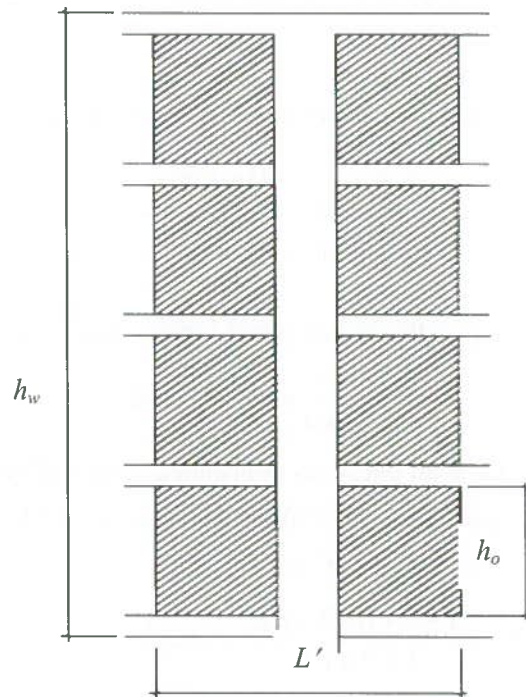


Figure 4.1.6 Total Height of Column with Wing Wall

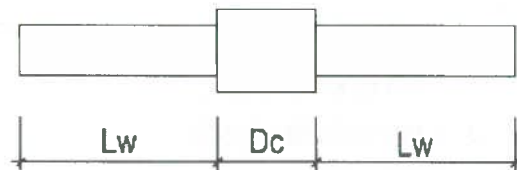


Figure 4.1.7 Total Depth of Column with Wing Wall

$$\therefore M/Q \cdot d_e = \frac{h_{cwo}}{L'} = \frac{2488}{1000} = 2.488 > 2$$

So,  $M/Q \cdot d_e = 2$

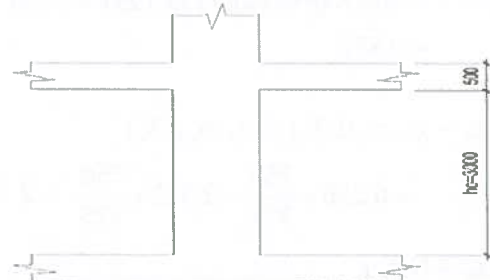


Figure 4.1.8 Elevation of Column

$$\begin{aligned}
\therefore M_u &= (0.9 + 1.5) \times 6 \times 314 \times 275 \times 250 + 0.5 \times 1115 \times 1000 \times 250 \cdot \left\{ 1 + 2 \times 1.5 - \frac{1115 \times 1000}{0.5 \times 750 \times 250 \times 25} \left( \frac{6 \times 314 \times 275}{1115.24 \times 1000} + 1 \right)^2 \right\} \\
&= 310.86 \times 10^6 + 139.40 \times 10^6 \{1 + 3 - 0.476 \times (0.465 + 1)^2\} \\
&= 726.05 \times 10^6 \text{ N-mm.}
\end{aligned}$$

So, shear force at flexural strength,

$$\therefore Q_{mu} = \phi M_u / h_{cwo} = \frac{0.8 \times 726.05 \times 10^6}{2488} = 233.5 \text{ kN.}$$

Ultimate shear strength

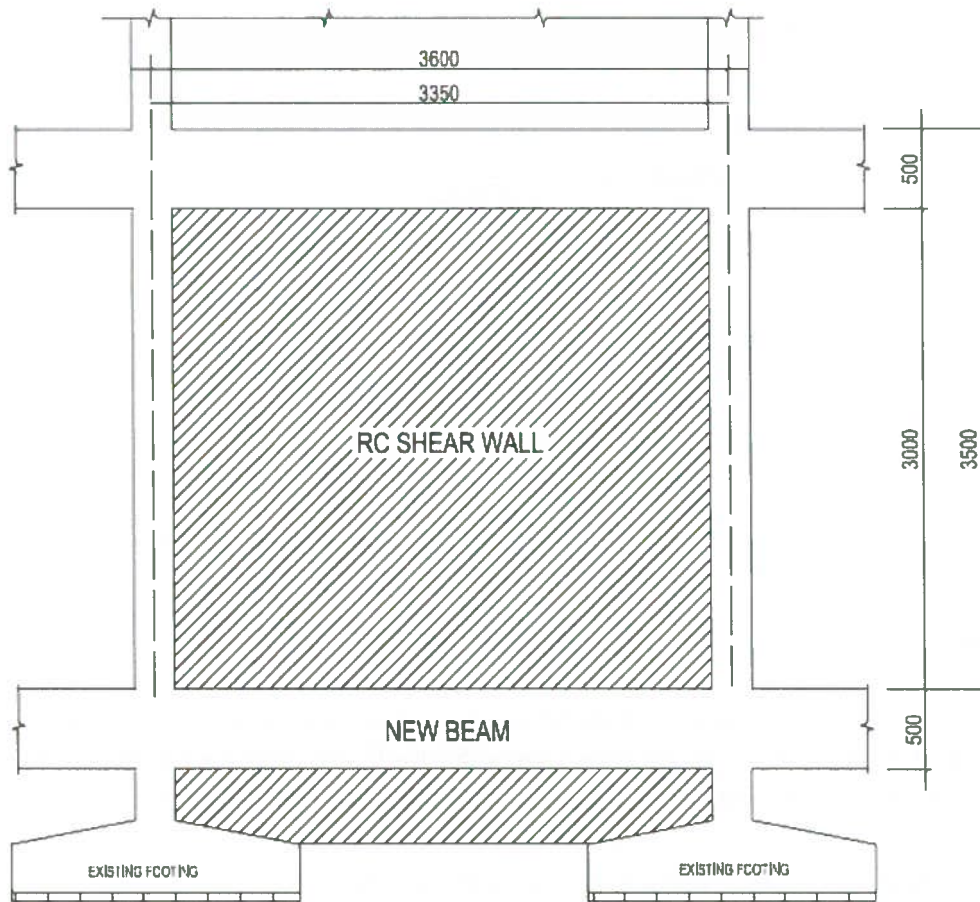
$$\begin{aligned}
Q_{su} &= \phi \left\{ \frac{0.053 \times P_{te}^{0.23} \cdot (F_c + 18)}{M / (Q \cdot de) + 0.12} + 0.85 \sqrt{p_{we} \cdot \sigma_{wy}} + 0.1 \sigma_{oe} \right\} \cdot b_e \cdot j_e \\
&= 0.8 \left\{ \frac{0.053 \times 0.878^{0.23} (9.2 + 18)}{2.0 + 0.12} + 0.85 \sqrt{2.187} + 0.1 \times 5.94 \right\} \times (375 \times 503.1) \\
&= 378.6 \text{ kN} > Q_{mu}
\end{aligned}$$

After adding wing wall at ground floor X-direction one column with wing wall will take 233.5 kN of shear. Total shear requirement at ground floor level at X-direction is 4673 kN which need to be satisfied by  $4673 / (233.5 - 50) = 26$  Nos of wing wall, considering ductility index,  $F = 1.0$ .

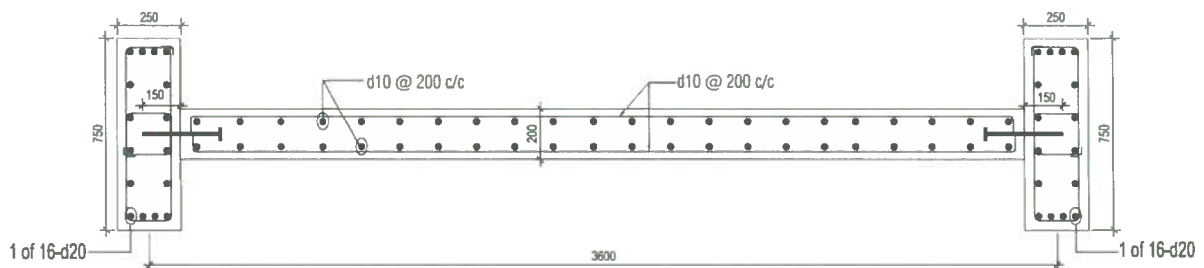
Calculation of anchor bolt will be similar to that of shear wall insertion.

**C) Shear wall:**

**Shear Wall Calculation: (X-Direction)**



**Figure 4.1.9 Detail of RC Shear Wall**



**Figure 4.1.10 Section of RC Shear Wall**

Shear strength of walls which are connected with existing boundary frames by using connectors such as post installed anchors. It shall be calculated by using the following equations in consideration of the load carrying mechanism at the connectors, wall panels and columns.

$${}_wQ_{su} = \min \{ {}_wQ'_{su} + 2 \cdot \alpha \cdot Q_c, Q_j + {}_pQ_c + \alpha \cdot Q_c \}$$

Where:

${}_wQ_{su}$  = Shear strength of shear walls.

${}_wQ'_{su}$  = Shear strength of infilled shear panel (only for the panel part in clear height and width)

$Q_j$  = Sum of the shear strength of connector underneath the beam.

${}_pQ_c$  = 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.

$\alpha$  = Reduction factor in consideration of the deflection condition to allow for load bearing contribution of columns. ( $\alpha=1$  in case of shear failure of columns)

$${}_wQ'_{su} = \max ({}_p{}_w{}_w\sigma_y, F_{cw}/20 + 0.5{}_p{}_w{}_w\sigma_y) \cdot t_w \cdot l'$$

Where:

$\rho_w$  = Wall reinforcement ratio

$$= \frac{2 \times 78.5}{200 \times 200} = 0.003925$$

${}_w\sigma_y$  = Yield strength of the wall reinforcing bar = 400 N/mm<sup>2</sup>.

$F_{cw}$  = Concrete strength of the installed wall panels (N/mm<sup>2</sup>) = 25 N/mm<sup>2</sup>.

$t_w$  = Wall thickness = 200mm.

$l'$  = Clear span of installed wall panel (mm).

$$= 3600 - 250 = 3350\text{mm.}$$

$$\begin{aligned} \therefore {}_wQ'_{su} &= \max \left\{ 0.003925 \times 400, \frac{25}{20} + 0.5 \times 0.003925 \times 400 \right\} 200 \times 3350 \\ &= \max \{ 1.57, 2.035 \} (200 \times 3350) \\ &= 1363.5 \text{ kN.} \end{aligned}$$

Direct shear strength of column.

$${}_pQ_c = k_{min} \cdot \tau_o \cdot b_e \cdot D$$

Where:

$$\begin{aligned} k_{min} &= 0.34 / (0.52 + a/D) \\ &= 0.34 / (0.52 + 1/3) = 0.4 \end{aligned}$$

$$\text{When } a = D/3$$

$$\begin{aligned} \sigma_o &= N / b_e \cdot D \\ &= \frac{1115 \times 1000}{750 \times 250} = 5.94. \end{aligned}$$

$$\begin{aligned} \sigma &= \rho_g \cdot \sigma_y + \sigma_o \\ &= \frac{16 \times 314}{750 \times 250} \times 275 + 5.94 \\ &= 13.30 > 0.66 F_c \end{aligned}$$

$$\text{So, } \tau_o = 0.66 F_c = 0.66 \times 9.2$$

$$= 6.072 \text{ N/mm}^2.$$

$$_p Q_c = 0.4 \times 6.072 \times 750 \times 250$$

$$= 455.4 \text{ kN.}$$

$$\text{Number of } \phi 20 \text{ mm bolt, } n = \frac{3475}{200} \cong 17,$$

$$\text{Capacity of each anchor } Q_a = 0.4 \sqrt{E_c \cdot \sigma_B} a_t$$

$$Q_a = 0.4 \sqrt{14350 \times 9.2} \times 314 / 1000 = 45.6 \text{ kN}$$

$Q_j$  = Sum of the shear strength of connectors underneath the beam.

$$A_s \cdot n = Q_a \cdot n = 45.6 \times 17 = 775.2 \text{ kN}$$

Ultimate flexural strength of existing column-

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

$$N_{max} = (750 \times 250 \times 9.2 + 16 \times 314 \times 275) / 1000 = 3106.6 \text{ kN}$$

$$M_u = \{0.8 \times 6 \times 314 \times 275 \times 250 + 0.12 \times 750 \times 250^2 \times 9.2\} \left\{ \frac{3106.6 - 1115}{3106.6 - 690} \right\}$$

$$= 128.0 \times 10^6 \text{ N-mm.}$$

$$Q_{mu} = \frac{2 \times 128.0 \times 10^6}{3000} = 85.3 \text{ kN for column}$$

$$Q_{su} \text{ (for column)} = \left\{ \frac{0.053 \times 1^{0.23} (18 + 9.2)}{3 + 0.12} + 0.85 \sqrt{0.00186 \times 275} + 0.1 \times 5.96 \right\} \times 750 \times 0.8 \times 250$$

$$= (0.462 + 0.608 + 0.596) 750 \times 0.8 \times 250 = 250 \text{ kN}$$

So Shear strength of column,  $Q_c = 85.3 \text{ kN}$

Now for wall  $Q_{su} = \min \{ {}_w Q_{su}' + 2 \alpha Q_c, Q_j + {}_p Q_c + \alpha Q_c \}$

$$= \min \{ 1363.5 + 2 \times 0.7 \times 85.3, 775.2 + 455.4 + 0.7 \times 85.3 \}$$

$$= \min (1482.92, 1290.31)$$

$$= 1290.31 \text{ kN.}$$

Ultimate flexural strength of wall

$${}_w M_u = a_t \sigma_{sy} \cdot l_w + 0.5 \Sigma (a_{wy} \cdot \sigma_{wy}) \cdot l_w + 0.5 N / l_w$$

Where

$a_t$  = Cross sectional area of flexural reinforcing bars.

$$a_t = 16 \times 314 = 5024 \text{ mm}^2.$$

$\Sigma a_{wy}$  = Vertical reinforcing bars in walls

$$= \frac{2 \times 3350 \times 78.5}{200} = 2630 \text{ mm}^2.$$

$\sigma_{sy}$  = Yield strength of the vertical reinforcing bars of vertical column.

$$= 275 \text{ N/mm}^2.$$

$\sigma_{wy}$  = Yield strength of the vertical reinforcing bars in the walls

$$= 400 \text{ N/mm}^2.$$

$l_y$  = Distance between the centre of the boundary columns of the wall.

$$= 3600 \text{ mm.}$$

$$\begin{aligned}
 {}_wM_u &= 5024 \times 275 \times 3600 + 0.5 (2630 \times 400) \times 3600 + 0.5 \times 1115 \times 1000 \times 3600 \\
 &= 4974 \times 10^6 + 1893 \times 10^6 + 2007 \times 10^6 \\
 &= 8874 \times 10^6 \text{ N-mm.}
 \end{aligned}$$

$$h_{wo} = \frac{15250}{2} = 7625 \text{ mm}$$

$${}_wQ_{mu} = \frac{{}_wM_u}{h_{wo}} = \frac{8874 \times 10^6}{7625 \times 1000} = 1163.8 \text{ kN.}$$

After adding shear wall at ground floor X-direction one shear wall will take 1163.8 kN of shear. Total shear requirement at ground floor level is 4673.05 kN, which need to be satisfied by  $(4673.05 / 1163.8 - 2 \times 50) = 4.4$  nos. of shear wall, considering  $F=1.0$ .

#### D) Carbon fiber wrapping:

##### Carbon Fiber Wrapping:

For details of carbon fibre wrapping sec-3.3.6 of J. Guidelines is referred.

Shear strength of column wrapped with carbon fiber shall be calculated by following equation:

$Q_{su}$  = Ultimate shear strength of column

$$= \left\{ \frac{0.053 \cdot p_t^{0.23} (F_{cl} + 18)}{M / (Q \cdot d) + 0.12} + 0.85 \sqrt{p_w \cdot \sigma_{wy} + p_{wf} \cdot \sigma_{fd} + 0.1 \sigma_o} \right\} \cdot b \cdot j$$

Condition:

- (i)  $M / (Q \cdot d)$  shall be in the range of 1.0 to 3.0
- (ii)  $p_w \cdot \sigma_{wy} + p_{wf} \cdot \sigma_{fd}$  shall not be more than  $9.8 \text{ N/mm}^2$ .

$p_t$  = Tensile reinforcement ratio

$$= \frac{6 \times 314 \times 100}{250 \times 750} = 1.00 \%$$

$$F_{cl} = 9.2 \text{ N/mm}^2$$

$$M / Q \cdot d = \frac{h_o / 2}{d} = \frac{3000 / 2}{250} = 6$$

$$\therefore M / Q \cdot d = 3$$

$p_w$  = Shear reinforcement ratio of existing column

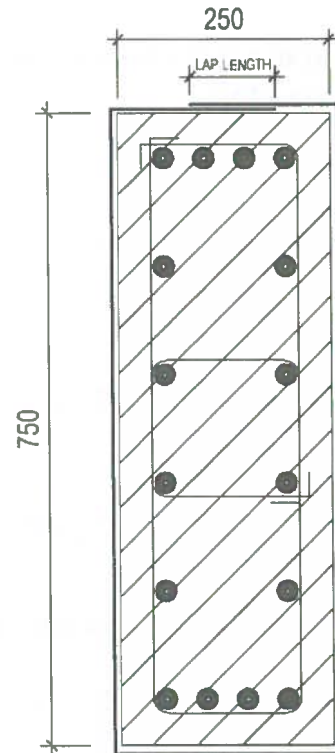
$$= \frac{4 \times 78.5}{225 \times 750} = 0.00186$$

$p_{wf}$  = Shear reinforcement ratio of carbon fiber sheet.

$$= \frac{2 \times 1.2}{750} = 0.0032 \text{ (Thickness of fiber = 1.2mm)}$$

$\sigma_{fd}$  = Tensile strength of carbon fiber sheet for shear design.

$$= \min \left\{ E_{fd} \cdot \epsilon_{fd}, \frac{2}{3} \sigma_f \right\}$$



**Figure 4.1.11 Plan of Carbon Fiber Wrapping**

When

$E_{fd}$  = Specified young's modulus of carbon fiber

$$= 2.30 \times 10^5 \text{ N/mm}^2.$$

$\epsilon_{fd}$  = Effective strain of carbon fiber sheet at shear failure (A value of 0.7% can be used).

$\sigma_f$  = Specified tensile strength of carbon fiber sheet.

$$= 3400 \text{ N/mm}^2.$$

$$\therefore \sigma_{fd} = \min \{2.30 \times 10^5 \times 0.007, \frac{2}{3} \times 3400\}$$

$$= \min \{1610, 2267\}$$

$$= 1610 \text{ N/mm}^2.$$

Now,  $p_w \cdot \sigma_{wy} + p_{wf} \cdot \sigma_{fd}$

$$= 0.00186 \times 275 + 0.0032 \times 1610$$

$$= 5.66 \text{ N/mm}^2 < 9.8 \text{ N/mm}^2.$$

$\sigma_o$  = Axial compressive stress

$$= \frac{N}{bD} = \frac{1115 \times 1000}{250 \times 750}$$

$$= 5.94 \text{ N/mm}^2 < 7.8.$$

$$\therefore Q_{su} = \left\{ \frac{0.053 \times (1.00)^{0.23} (9.2 + 18)}{3 + 0.12} + 0.85 \sqrt{5.66} + (0.1)(5.94) \right\} 750 \times 0.8 \times 250$$

$$= (0.46 + 2.38 + 0.59) \times (750 \times 0.8 \times 250)$$

$$= 514.5 \text{ kN}.$$

Flexural strength:

$$N = 1115 \text{ kN}.$$

$$0.4 bDF_c = 0.4 \times 250 \times 750 \times 9.2$$

$$= 690 \text{ kN} < N.$$

$$N_{max} = bDF_c + ag\sigma_y$$

$$= (250 \times 750 \times 9.2) + (314 \times 16 \times 275)$$

$$= 3106 \text{ kN} > N.$$

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

$$= \left\{ 0.8 \times 6 \times 314 \times 275 \times 250 + 0.12 \times 750 \times 250^2 \times 9.2 \right\} \times \left\{ \frac{3106 - 1115}{3106 - 690} \right\}$$

$$= 155.37 \times 10^6 \times 0.82$$

$$= 128 \times 10^6 \text{ N-mm}.$$

$$Q_{mu} = \frac{2 \times 128 \times 10^6}{3000}$$

$$= 85.3 \text{ kN}.$$

Due to insertion of all around grade beam, thereby reducing clear height, shear strength due to flexural moment,  $Q_{mu}$  have been increased from 50kN to 85.3 kN. For carbon fibre wrapping shear strength have been increase from 250kN to 514.5kN which has no impact on overall increment strength of the structure.

### E) Steel frame bracing:

#### Steel Bracing:

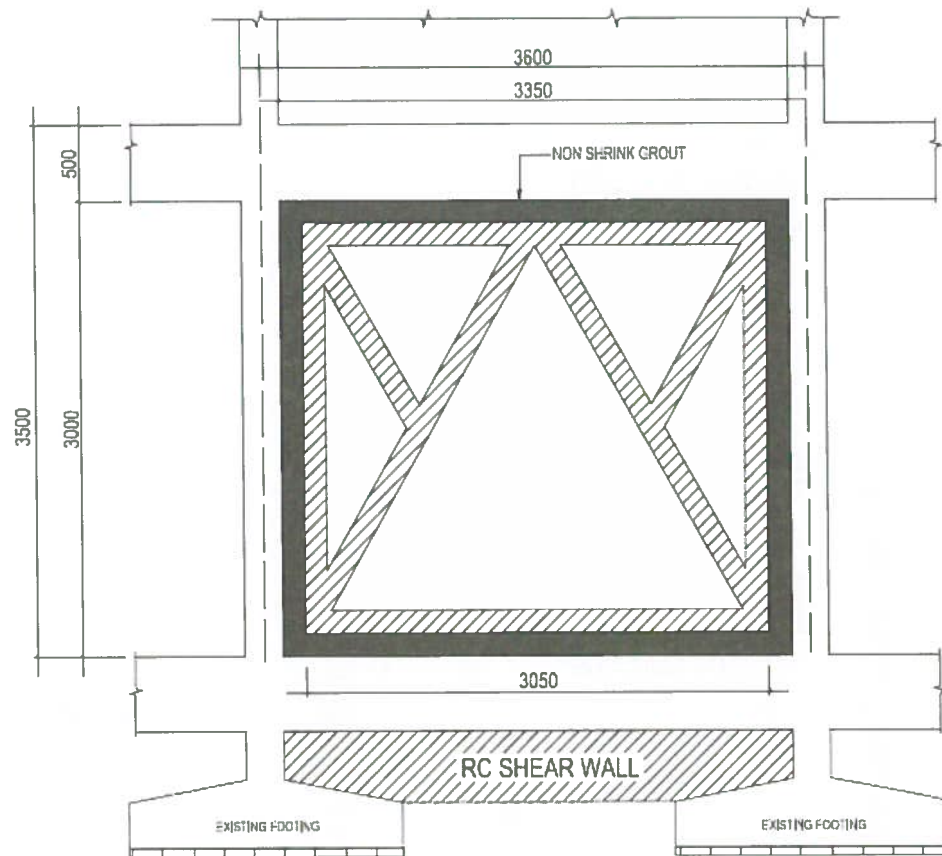


Figure 4.1.12 Detail of Steel Frame Shear Wall

Span  $L = 3600$ mm.

Storey height = 3500mm.

Height  $h_o = 3000$  mm.

Clear gap = 150mm (Between RC face & steel)

Length of steel frame =  $3600 - 250 - 150 \times 2 = 3050$

Height of steel frame =  $3000 - 150 \times 2 = 2700$

Existing concrete =  $9.2 \text{ N/mm}^2$ .

Rebar = 275 Mpa ( $\text{N/mm}^2$ ).

Steel  $\sigma_y = 345 \text{ Mpa}$

$E_c = 1.96 \times 10^4 \text{ Mpa}$

$E_s = 2.06 \times 10^5 \text{ Mpa}$ .

Non shrink grout  $\sigma_M = 30 \text{ Mpa}$ .

Tensile strength of steel  $\sigma_{max} = 400 \text{ Mpa}$ .

Tensile strength bonded anchor = 400 Mpa.

Steel Brace = C- 260  $\times$  130  $\times$  12

Steel Frame = C- 260 × 130 × 12

Ultimate strength:

Cross sectional area of steel

$$A = 260 \times 12 + (130 - 12) \times 12 \times 2$$

$$= 5952 \text{ mm}^2.$$

Radius of gyration:

$$i_x = \sqrt{\frac{BH^3 - bh^3}{12(BH - bh)}}$$

$$= \sqrt{\frac{130 \times 260^3 - 118 \times 236^3}{12(130 \times 260 - 118 \times 236)}}$$

$$= 101.36 \text{ mm}.$$

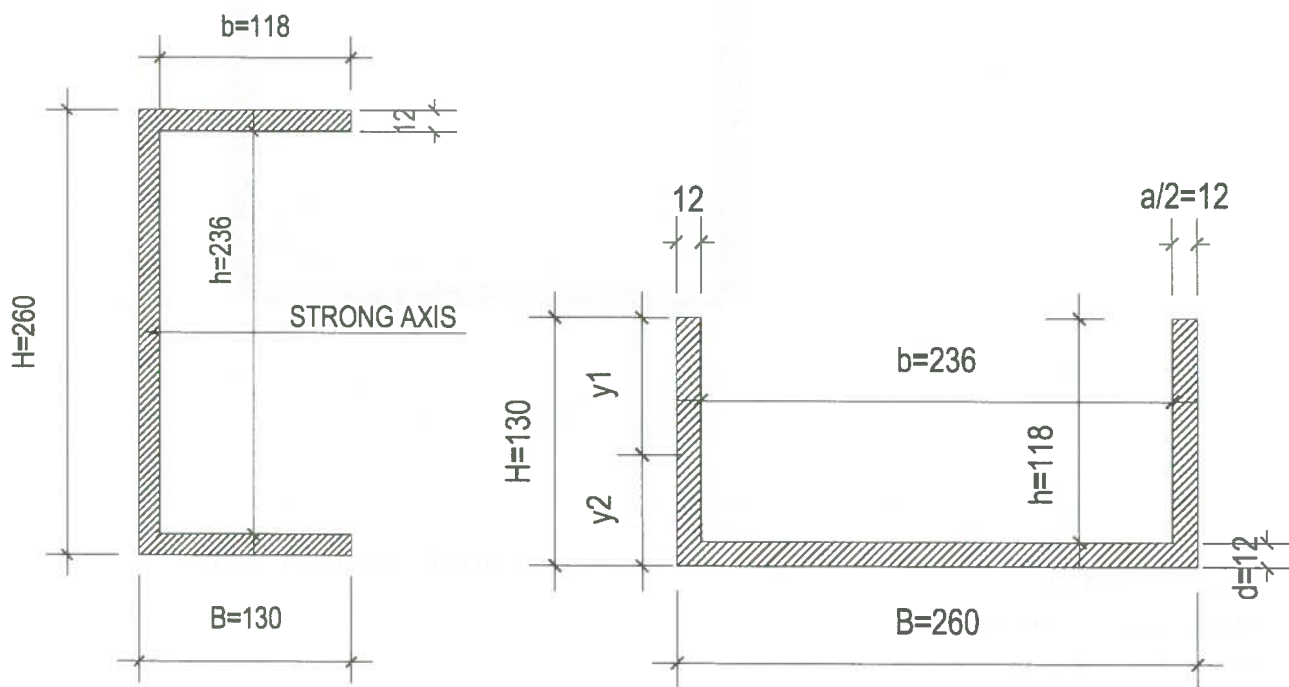


Figure 4.1.13 Brace Section

$$i_y = \sqrt{\frac{By_2^3 + ay_1^3}{3(BH - bh)}}$$

$$= \sqrt{\frac{260 \times 236^3 + 24(92.78)^3}{3(260 \times 130 - 236 \times 118)}}$$

$$= 42.70.$$

$$y_2 = \frac{1}{2} \frac{aH^2 + bd^2}{aH + bd}$$

$$= \frac{1}{2} \frac{(24 \times 130^2) + (260 \times 12^2)}{24 \times 130 + 260 \times 12}$$

$$= 37.21 \text{ mm.}$$

$$y_1 = 92.78 \text{ mm.}$$

Buckling length,

$$lk_x = \sqrt{\left(\frac{3050}{2}\right)^2 + (2700)^2}$$

$$= 3100 \text{ mm.}$$

$$lk_y = lk_x / 2 = 1550 \text{ mm.}$$

Slenderness ratio,

$$\lambda_x = lk_x / i_x = 3100 / 101.36 = 30.58 < 58$$

$$\lambda_y = lk_y / i_y = 1550 / 42.70 = 36.30 < 58$$

$\Lambda$  = Limit aspect ratio

$$= \sqrt{\frac{\Pi^2 \cdot E}{0.6F}} = \sqrt{\frac{\Pi^2 \times 2.06 \times 10^5}{0.6 \times 345}}$$

$$= 99.05 > 36.30.$$

Critical compressive stress:

$$\sigma_{cr} = [1 - 0.4(\lambda / \Lambda)^2] \cdot F_y$$

$$= [1 - 0.4(36.30/99.05)^2] \times 345$$

$$= 326.5 \text{ N/mm}^2.$$

Compressive & tensile strength of brace

$$N_c = \sigma_{cr} \cdot A = 326.5 \times 5952 / 1000 = 1943.3 \text{ kN.}$$

$$N_t = F \cdot A \times 0.8 = 345 \times 5952 \times 0.8 / 1000 = 1642.8 \text{ kN.}$$

Ultimate strength of brace

$$\cos \theta = \frac{1525}{3100} = 0.49$$

$$sQ_u = (N_c + N_t) \cos \theta = (1943.3 + 1642.8) \times 0.49 = 1757.2 \text{ kN.}$$

Average shear stress of mortar =  $\tau_m$

$$= sQ_u / A_M$$

$$= \frac{1757.2 \times 10^3}{(3050 \times 260)} = 2.22 \text{ N/mm}^2.$$

## Strength of brace frame

$${}_sQ_{su1} = {}_sQ_u + 2 \times Q_u = 1757.2 + 2 \times 50 = 1857.2 \text{ kN.}$$

After adding steel frame at ground floor X-direction one steel frame will take 1857.2 kN shear wall. Total shear required at ground floor level X-direction is 4673.05 kN, which need to be satisfied by  $\{4673.05 / 1857.2 - (50 \times 2)\} = 2.66 \cong 3$  Nos of steel frame.

## Summary of retrofitting method:

Type	Requirement of shear at GF X-direction, 1st level kN	Original shear capacity of one column 50kN, GF X-direction	Shear gain by each method	Total requirement
Column jacketing	4673.05	50 kN	252.7	$\frac{4673.05}{(252.7 - 50)} = 23$
Wing wall		50 kN	233.5	$\frac{4673.05}{(233.5 - 50)} = 26$
Shear wall		$50 \times 2 = 100 \text{ kN}$	1163.8	$\frac{4673.05}{(1163.8 - 2 \times 50)} \cong 4.4$
Carbon fiber wrapping		50 kN	85.3	Impractical solution
Steel bracing		$50 \times 2 = 100 \text{ kN}$	1857.2	$\frac{4673.05}{(1857.2 - 2 \times 50)} \cong 2.66$

## Remark:

Among the above methods column jacketing, wing wall needs more location of column to be disturbed to retrofit. Carbon fiber seems to be not a practical solution in this case. Shear wall and steel frame bracing are the two likely methods of retrofit. Based on economy and easy construction, Shear wall option has been chosen. Due to sake of symmetry 4nos shear wall instead of 3nos at GF 1st level X-direction. Similarly for each level in each major directions need to be calculated.

## (9) Outline of selected retrofit work

Component Type	Description
Material (concrete, re-bar, anchor, structural steel)	Concrete $f'_c = 25 \text{ N/mm}^2$ Re-bar $f_y = 400 \text{ N/mm}^2$ Anchor $f_y = 400 \text{ N/mm}^2$
Layout of retrofit elements	X-direction 4Nos SW (Storey-1) 2Nos SW (Storey 2 to 4) Y-direction 3Nos SW (Only Storey-1)
Connection of existing and new member through post-installed anchor	New infill Shear wall and existing frame should be connected by post installed anchor. Details of anchor are: Length = 450mm Diameter = D19 Spacing = 150mm c/c

## (10) Result after seismic retrofit

Table 4.1.2 Result after retrofit

Storey	X direction		Y direction	
	$I_s$	$C_T S_D$	$I_s$	$C_T S_D$
5	0.433	0.433	1.61	1.073
4	0.62	0.57	0.596	0.497
3	0.41	0.38	0.463	0.386
2	0.32	0.30	0.301	0.294
1	0.36	0.36	0.45	0.37

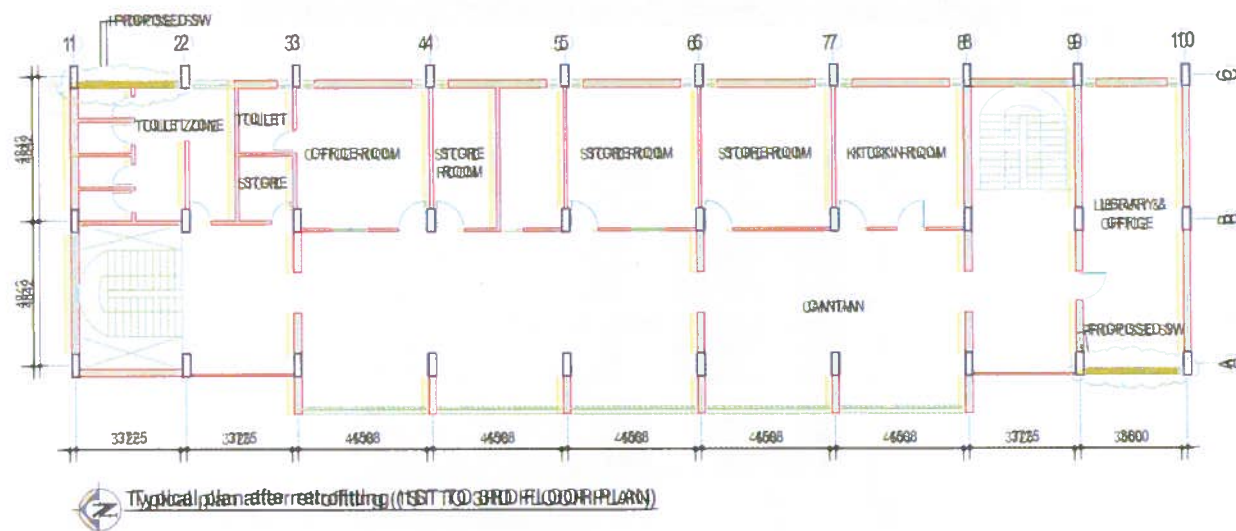
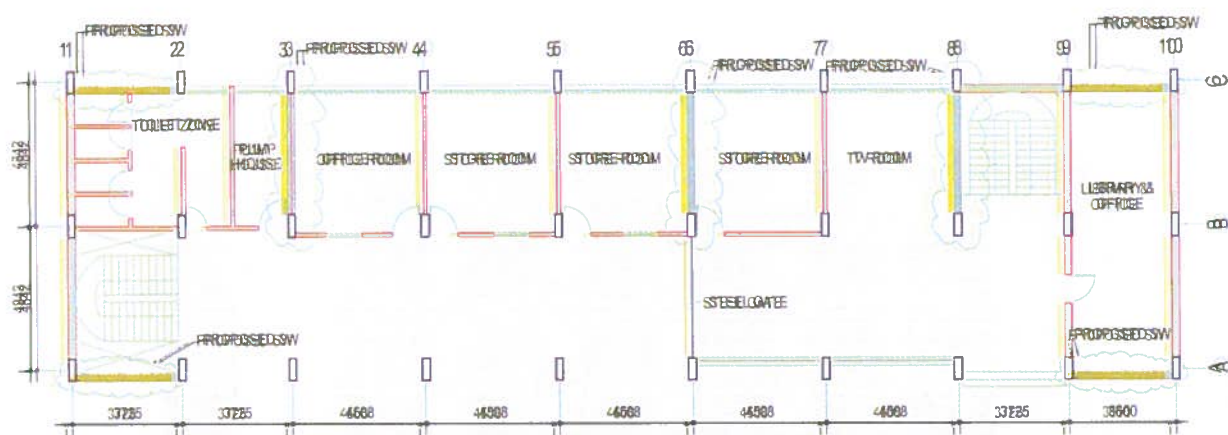


Figure 4.1.14 Typical Framing Plan after Retrofitting

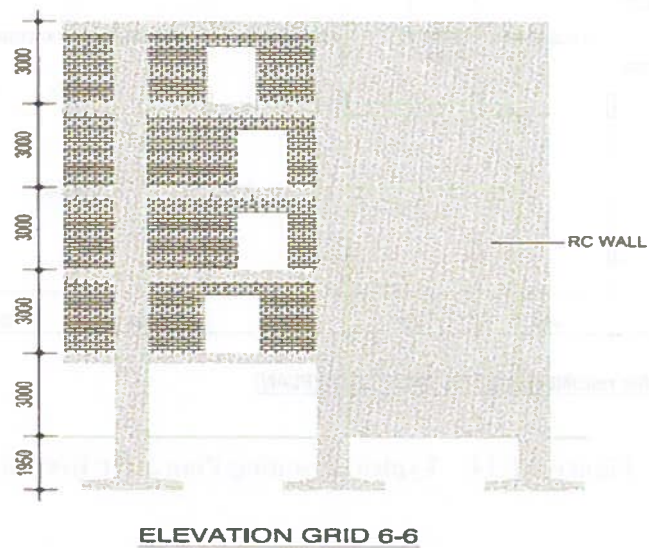
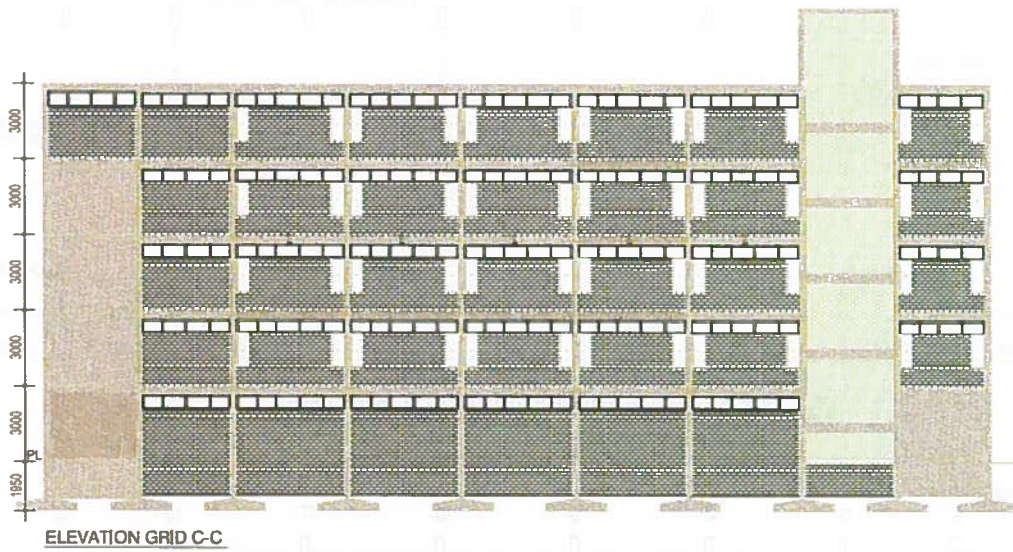
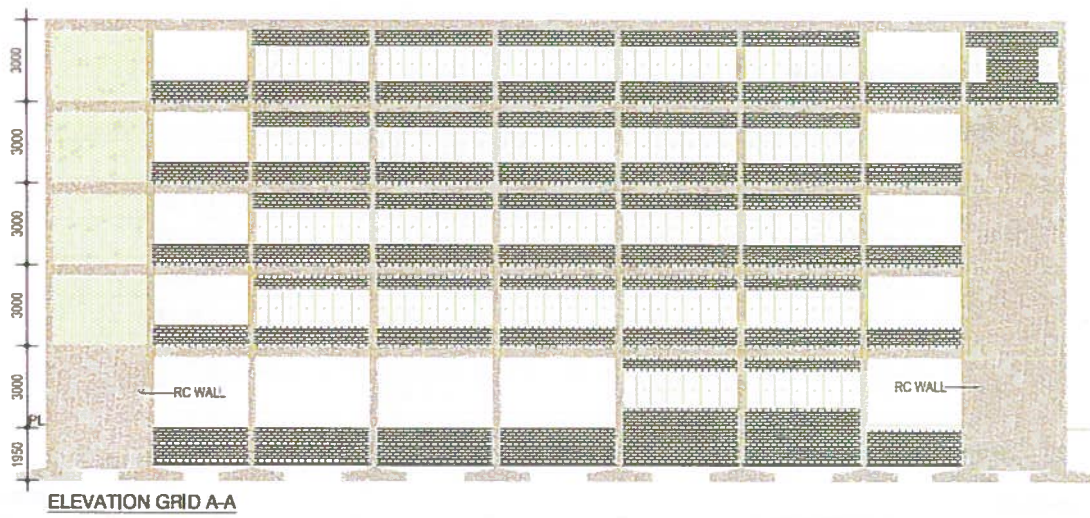


Figure 4.1.15 Typical Elevation after Retrofitting

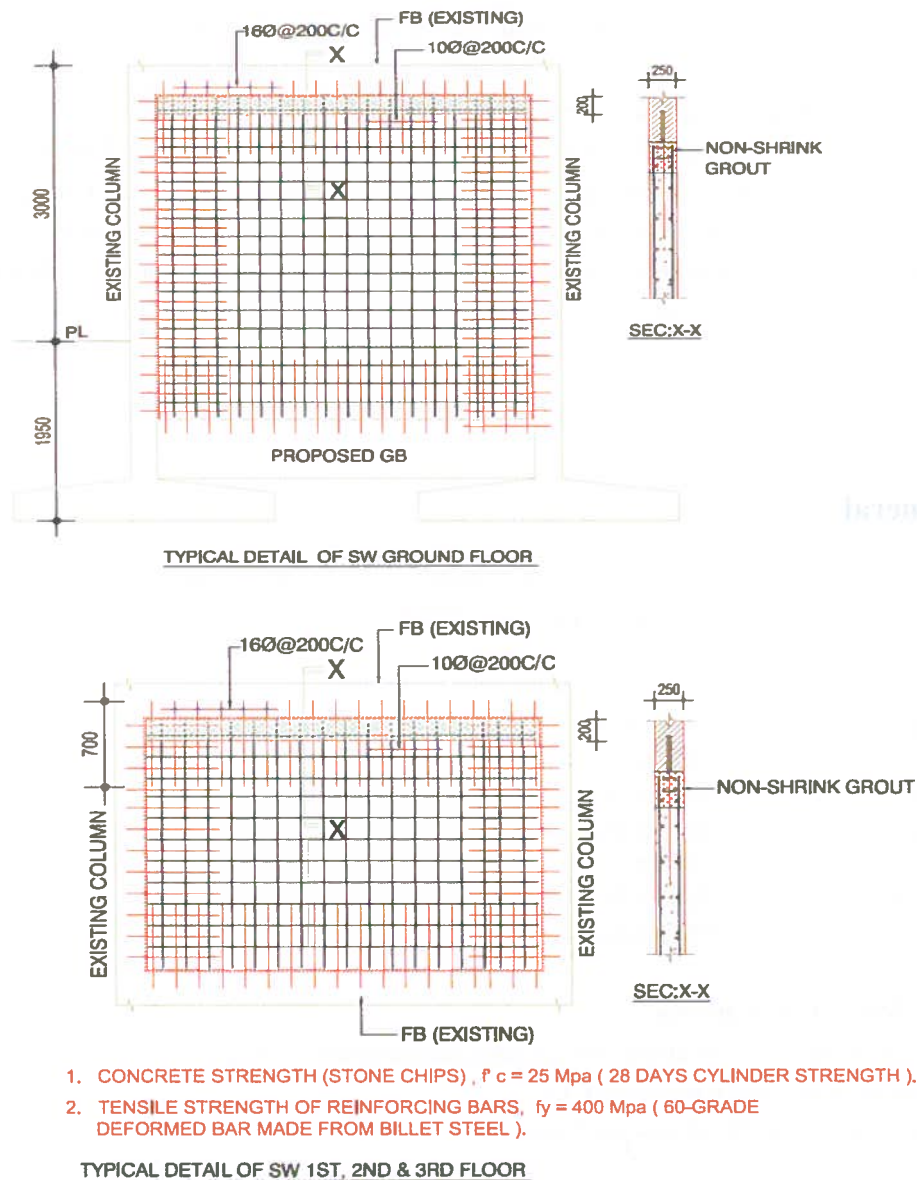


Figure 4.1.16 Typical Detail of RC Wall

(11) Others, if any such as comment by the Engineer

In this structure strength based design was considered due to lack of ductility of the structure. Insertion of Shear wall was found to be better solution as it will only change the infill brick wall to shear wall.

Check against vertical load:

In this example, unit weight of typical floor was  $10.8 \text{ kN/m}^2$  and roof was  $6.3 \text{ kN/m}^2$ . A typical column in the middle grid C1 was considered for checking against vertical load. The tributary area considered  $22.5 \text{ m}^2$ . The selected column size  $250\text{mm} \times 750\text{mm}$  and rebar was 16Nos 20mm diameters rebar. Total un-factored load in this column is 187.61 Kip and factored capacity (as per BNBC 1993) of the column considering  $f'_c = 1,335 \text{ psi}$  (9.2 Mpa),  $f_y = 40,000 \text{ psi}$  (275 Mpa) is 364 Kip which results axial force ratio ( $P \text{ un-factored} / b \cdot D \cdot f'_c$ ) of 0.47. It satisfied the vertical load carrying requirements of BNBC (1993).

## 4.2 CASE 2: RETROFIT DESIGN OF 4 STOREY GARMENT FACTORY BUILDING IN DHAKA AREA

Retrofit design as an example was done and the summary of the retrofit design is shown below. Characteristics of the building, 1) There is a double height space at one side of the building which cause torsion, 2) Structural height at ground floor is big compared with other floors and become a relatively weak story. To improve the irregularity is one of the targets of the retrofit. The level of foundation footing is deep. Factory under operation is one of the conditions for the plan of retrofit design. Steel framed brace is planned at perimeter of the building considering the windows and opening for exhaust fan. Steel framed brace is also planned at storage, which will minimize the impact to the operation of the factory. RC wall is provided beneath the steel framed brace below the ground level up to foundation level to transfer the seismic load.

It is noted that some data of actual buildings have been modified for the introduction of retrofit design, since this is a private building.

### 4.2.1 General

**(1) Outline of building** (following information is reference only for the example)

Name	: A Garment factory in Dhaka area
Usage	: Garment Factory
Number of storey	: 4
Building height	: 15,292mm
Storey height	: 3,658mm, (structural height 4,878mm at level 1)
Structural type	: Reinforced Concrete Frame Structure
Foundation type	: Independent footing
Building area	: 1,811m <sup>2</sup>
Total floor area	: 65,000sft (6,038.7m <sup>2</sup> )
Year of design	: 2002 (approved)

**(2) Characteristics of the building,**

- 1) 4 storey garment factory with double height area, constructed in 2002.
- 2) Grade beams exist but much lower than GL.
- 3) Foundation is located at GL minus 3m level.

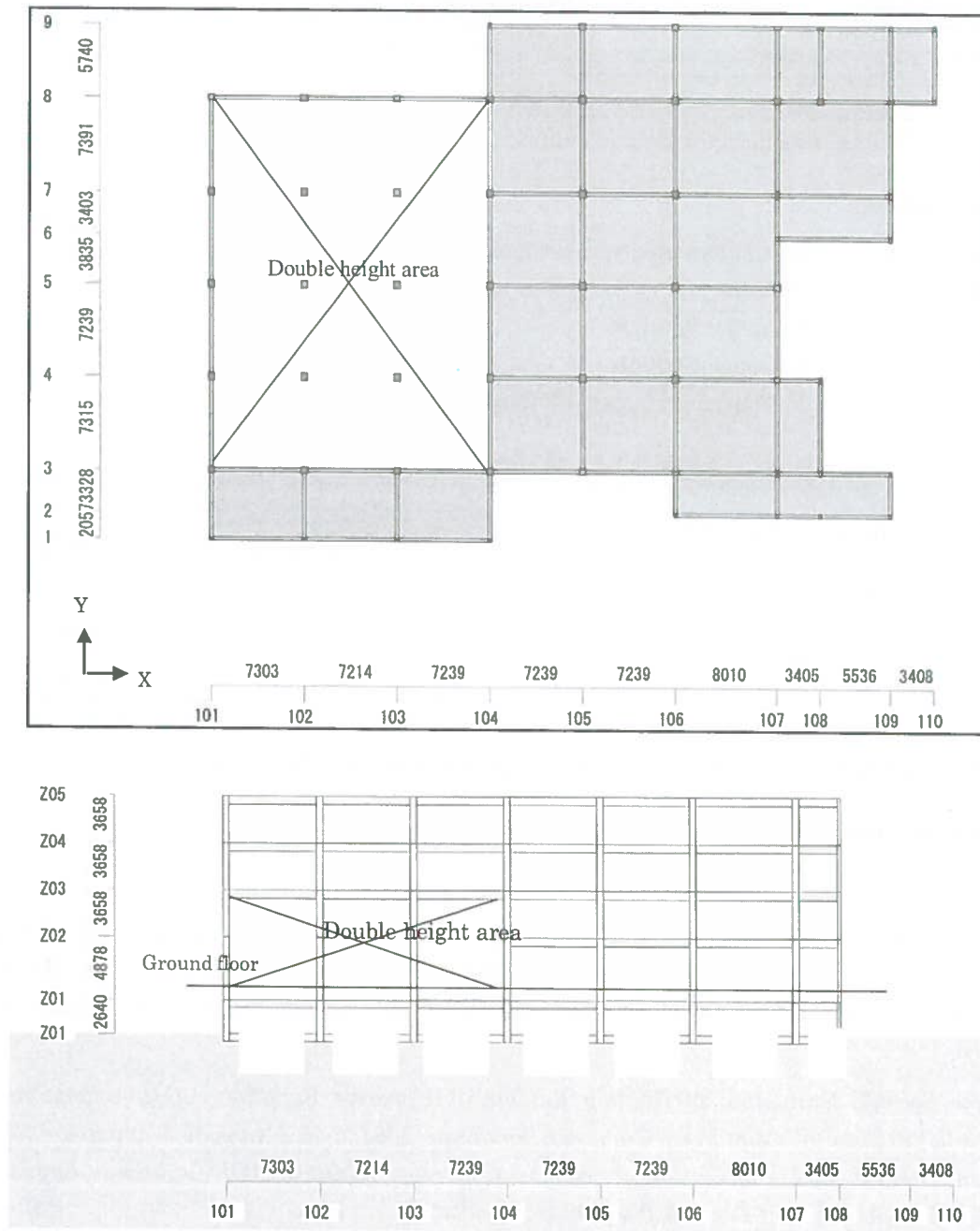


Figure 4.2.1 Framing Plan of 1<sup>st</sup> Floor and Typical Elevation of a Sample Building

### (3) As- built drawings

Preparation of as-built drawings of structural design drawings and architectural design drawings was done including related material tests by a detail building survey.

## 4.2.2 Structural Assessment

### (1) Materials

Concrete strength by core sampling test (diameter; 50mm),

“Average strength – standard deviation/2” was used for the evaluation.

$\sigma = 12.7 - 4.0/2 = 10.7 \text{ N/mm}^2$  is used for concrete strength.

Re-bar, diameter and yield strength

Diameter (mm)	Yield strength (N/mm <sup>2</sup> )
25mm	437N/mm <sup>2</sup> (average of two)
12mm	441N/mm <sup>2</sup>

\*  $\sigma_y = 400\text{kN/mm}^2$  was used for the evaluation.

## (2) Weight of Building

Building weight for the evaluation of seismic load is shown, and load factor has not been considered.

Floor slab thickness : 175mm,

Typical beam size : 600mm × 350mm,

Typical column size : 508mm × 660mm

Brick wall thickness : 250mm for external, 125mm for internal.

**Table 4.2.1 Weight of Building**

Number of Storey	$W_i$ (kN)	Total $W$ (kN)	Floor area (m <sup>2</sup> )	W i/ Area (kN/m <sup>2</sup> )	
				Dead load (kN/m <sup>2</sup> )	Live load (kN/m <sup>2</sup> )
4 (RF)	14,965	14,965	1,666	7.98 (average)	1.0 (RF, BNBC)
3	17,913	32,878	1,666	8.75 (average)	2.0 (Garment)
2	18,023	50,903	1,666	8.82 (average)	2.0 (Garment)
1	13,588	64,491	1,042	11.04 (average)	2.0 (Garment)

Note: Dead load (kN/mm<sup>2</sup>) is calculated from total weight of each floor/ floor area

## (3) Evaluation Method and Criteria of Judgment

### (a) Method

General requirement of BNBC 93 is followed excluding seismic retrofit design, which is not covered by BNBC. Manuals for Seismic evaluation and Retrofit design of existing RC buildings, 2015 by CNCRP is followed. Items not covered by CNCRP manuals, the following Japanese standard and guidelines has been followed:

*“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” written in English and published by Japanese Building Disaster Prevention Association.*

### (b) Criteria of judgment

Seismic index of structure,  $I_s$  is calculated at each direction and each floor, and compared with Seismic demand index,  $I_{so}$ .

$I_s \geq I_{so}$ , then it is acceptable.

2<sup>nd</sup> level screening method is applied.

**Proposed seismic demand index of structure,  $I_{so}$ ,**Based on **BNBC 2015**  $I_{so} = 0.8 \cdot 2/3 \cdot Z \cdot I \cdot C_s$ 

Where:

 $Z$  : Seismic zone coefficient, as defined in Section 2.5.4.2 of BNBC 2015. $I$  : Structure importance factor, as defined in Section 2.5.5.1 of BNBC 2015 $C_s$  : Normalized acceleration response spectrum, which is a function of structure (building) period and soil type (site class) as defined by Equations 6.2.35a to 6.2.35d of BNBC 2015

$$T = C_t \cdot (h_n)^m = 0.466 \cdot (15.2)^{0.9} = 0.54 \text{sec.} < 0.6 \text{sec. Soil type SC, } C_s = 2.875$$

$$I_{so} = 0.8 \cdot 2/3 \cdot 0.2 \cdot 1.0 \cdot 2.875 = 0.304 > 0.262 \text{ by the following BNBC93, then } I_{so} = 0.30 \text{ is selected.}$$

Based on **BNBC 93**  $I_{so} = 0.8 \cdot Z \cdot C \cdot I$ 

Where:

 $Z$  : Seismic zone coefficient given in Table 6.2.22 of BNBC 93 $I$  : Structure importance coefficient given in Table 6.2.23 $C$  : Numerical coefficient given by the relation,  $C = 1.25S/T^{2/3}$ ,  $S$ : Site coefficient for soil characteristics,  $T$ : Fundamental period of vibration in seconds, the value of  $C$  need not exceed 2.75.

In case of RC frame structure,

$$T = C_t \cdot (h_n)^{3/4} = 0.073 \cdot (15.2)^{3/4} = 0.562 \text{sec.}$$

$$\text{Soil type } S_2 = 1.2, C = 1.25 \cdot S / (T^{2/3}) = 1.25 \times 1.2 / 0.668 = 2.18 < 2.75.$$

$$\text{In Zone 2 (Dhaka), } I_{so} = 0.8 \times 0.15 \times 2.18 \times 1.0 = 0.8 \times 0.327 = 0.262 \quad (Z = 0.15, I = 1.0, C = 2.75)$$

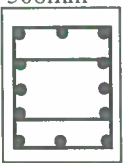
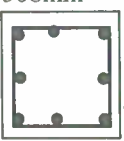
**(c) Application**

In case of low strength concrete (concrete strength  $< 13.5 \text{N/mm}^2$ ), reduction factor for shear strength of column will be used. Concrete smaller than  $9.0 \text{N/mm}^2$  is out of scope in principle, and will be evaluated carefully.

**(4) Assessment Result****(a) Evaluation of column**

Typical column section at ground level is shown in Table 4.2.2.

**Table 4.2.2 Flexural Strength ( $M_u$ )/Shear Strength ( $Q_u$ ), and Ductility Index,  $F$** 

	Internal column (grid 5-103)	External column (grid 5-105)
Section size, number of main bar, tie and length	<p>610mm</p>  <p>Tie: 4-10mm @ 200 (<math>P_w = 0.257\%</math>)</p> <p>Number of man bar: 12-25mm <math>L_o = 4,010</math> mm (clear height)</p>	<p>508mm</p>  <p>Tie: 2-10mm @ 200 (<math>P_w = 0.155\%</math>)</p> <p>Number of man bar: 8-25mm <math>L_o = 4,010</math> mm (clear height)</p>
Flexural strength, Shear capacity (kN), and strength coefficient, $C$	<p><math>M_u = 521.3 \text{ kN}\cdot\text{m}</math>  <math>Q_{mu} = 2 \cdot M_u / 4.01 \text{ m} = 260 \text{ kN}</math>  <math>Q_{su} = 408.7 \cdot K_r = 408.7 \cdot 0.854 = 410.5 \text{ kN}</math>  <math>Q_{su} / Q_{mu} = 1.58</math>  This column is "Flexural column"  <math>C = Q_{mu} / N(\text{axial force}) = 260,884 = 0.138</math></p>	<p><math>M_u = 395.6 \text{ kN}\cdot\text{m}</math>  <math>Q_{mu} = 2 \cdot M_u / 4.01 \text{ m} = 197.1 \text{ kN}</math>  <math>Q_{su} = 322.5 \cdot K_r = 322.5 \cdot 0.854 = 275.4 \text{ kN}</math>  <math>Q_{su} / Q_{mu} = 1.40</math>  This column is "Flexural column"  <math>C = Q_{mu} / N = 197.1 / 1,201 = 0.164</math></p>
Ductility index, $F$	<p>Plastic drift angle,  <math>cR_{mp} = 10 \cdot (cQ_{su} / cQ_{mu} - q) \cdot cR_{my}</math>  <math>= 10 \cdot (1.58 - 1.1) \cdot 1/150 = 4.8/150 = 0.032</math>  Drift angle at the ultimate flexural strength,  <math>cR_{mu} = cR_{my} + cR_{mp} = 0.0067 + 0.032 = 0.0387 = 1/25.8 &gt; 1/30</math>,</p> $F = \frac{\sqrt{\left(\frac{2R_{mu}}{R_y} - 1\right)}}{\left\{0.75 \times \left(1 + 0.05 \times \frac{R_{mu}}{R_y}\right)\right\}}$ $F = \frac{\sqrt{\left(\frac{2 \times 0.0387}{0.0067} - 1\right)}}{\left\{0.75 \times \left(1 + 0.05 \times \frac{0.0387}{0.0067}\right)\right\}} = 3.36 > 3.2$ <p>Check upper limit of <math>F</math> by followings,  <b>1) Axial force ratio,</b>  <math>N/(bDF_c) = \eta =</math>  <math>1,884,000 / 508 \times 610 \times 10.7 = 0.574 &gt; 0.4, &lt; 0.6</math>,  then <math>F = 1.27</math> is used instead of 3.2 (note)  <b>2) Shear stress of column,</b>  <math>c\tau_u / F_c = Q_u / b \times d \times F_c = 0.077 &lt; 0.2</math>  <b>3) Tensile reinforcement ratio,</b>  <math>P_t = 5 \times 490 / b \times d = 0.79\% &lt; 1.0\%</math>  <b>4) Spacing of hoop,</b>  <math>s/db = 200/25 = 8 \leq 8</math>  <b>5) Clear height,</b>  <math>H_o/D = 4010/508 = 7.9 &gt; 2.0</math>  The lowest value, <math>F = 1.27</math> is used. (*refer to the Chapter 1.1 and Supplement A3)</p>	<p><math>N/bDF_c = \eta =</math>  <math>1,201,000 / 508 \times 508 \times 10.7 = 0.435 &lt; 0.6</math>.  <math>F = 1.27</math> is used. (*refer to the technical manual)</p>

Note: \* Axial force ratio  $N/(b \times D \times F_c)$  in the range of 0.4 to 0.6, and ductility index of 1.27 is proposed.

**(b) Column and Brick standing wall**

Short column (shear failure) caused by standing brick wall has not been evaluated quantitatively. Ductility index,  $F$  of 1.27 is selected for columns as shown Chapter 1.11.

**(c) Beam column joint**

This seismic evaluation is 2<sup>nd</sup> level screening method, which consider column collapse mechanism. Beam column joint is relatively weak in case of low strength concrete ( $F_c < 13.5 \text{ N/mm}^2$ ), and Ductility index,  $F$  of 1.50 is used as shown in Chapter 1.11 and Supplement 7. As a result,  $F = 1.27$  is selected.

**(d) Irregularity Index  $S_D$** **Table 4.2.3 Irregularity Index  $S_D$** 

( $q_{2e} = 0.95$ ,  $q_{2f} = 0.975$ ), refer to the Table 6 of “the J. Standard”)

Number of Storey	Vertical Irregularity	Horizontal Irregularity	Irregularity index, $S_{D2}$	
			X-direction	Y-direction
4	1.0	1.0	0.95	0.95
3	1.0	1.0	0.95	0.95
2	1.0	0.9	0.95	0.86
1	0.8	1.0	0.76	0.76

Above summary is calculated as follows.

Item		$G_i$ (grade)	$q_{2i}$	Note
a	Regularity	1.0	1.0	
b	Aspect ratio, plan	1.0	1.0	
c	Narrow part	1.0	1.0	
d	Expansion joint	--	1.0	
e	Well-style area	0.8	0.95	$e > 0.3$
f	Eccentric-well	0.9	1.0	$f_1 = r/y = 0.37$ , $f_2 = r/x = 0.24$
i, j		1.0	1.0	Based on detail calculation by item n
l	Eccentricity	Floor L2, X, 1.0 Y, 0.9	1.0 0.9	$R_e = 0.12 < 0.15$ , for safety side $G_l = 0.9$ is used. $R_e$ : eccentricity ratio, studied separately.
n	Stiffness/mass ratio	Floor L1, X, 0.8 Floor L1, Y, 0.8	0.8 0.8	For reference only, $R_s = 0.42 < 0.6$ , $F_s = 2.0 - R_s/0.6 = 1.30$ , $G'_n = 1/1.32 = 0.77$ $R_s = 0.44 < 0.6$ , $F_s = 2.0 - R_s/0.6 = 1.27$ , $G'_n = 1/1.27 = 0.79$ $R_s$ : stiffness ratio, $F_s$ : shape factor

Item a to item j are applied for all stories. Item l and item n are applied for each storey and direction. Eccentricity (item l) and vertical stiffness distribution (item n) shown at Note are evaluated by separate calculation for reference only.

Typical level and each direction,  $S_{D2} = q_{2a} \times \dots \times q_{2n} = 1.0 \times 0.95 \times 1.0 \times 1.0 \times 1.0 = 0.95$ ,

Level 2, Y direction,  $S_{D2} = q_{2a} \times \dots \times q_{2n} = 1.0 \times 0.95 \times 1.0 \times 0.9 \times 1.0 = 0.86$

L1, X and Y direction,  $S_{D2} = q_{2a} \times \dots \times q_{2n} = 1.0 \times 0.95 \times 1.0 \times 1.0 \times 0.8 = 0.76$

(e) Time index,  $T$ 

Time Index,  $T$  is estimated as 0.998 from following Table, and  $T = 1.0$  is used for further calculation.

**Table 4.2.4 Time Index,  $T$ , Refer to Table 8 of the J. Standard.**

Item		Structural cracking and deflection						Deterioration and aging					
		a		b		c		a		b		c	
Portion	Degree	a		b		c		a		b		c	
		1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>
I Slab including sub-beam													
	Range												
	I 1/3 or more of total floor	0.017		0.005		0.001		0.017		0.005		0.001	
	II 1/3~1/9	0.006		0.002		0		0.006		0.002		0	
	III 1/9 or less	0.002		0.001		0		0.002		0.001		0	
II Beam													
	I 1/3 or more of total number of members for each direction	0.05		0.015		0.004		0.050		0.015		0.004	
	II 1/3~1/9	0.017		0.005		0.001		0.017		0.005		0.001	
	III 1/9 or less	0.006		0.002		0		0.006		0.002		0	
III Wall & Column													
	I 1/3 or more of total number of members	0.15		0.046		0.011		0.150		0.046		0.011	
	II 1/3~1/9	0.050		0.015		0.004		0.050		0.015		0.004	
	III 1/9 or less	0.017		0.005		0.001		0.017		0.005		0.001	
Mark-down													
	Subtotal	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>	1 <sup>st</sup> 2 <sup>nd</sup> 3 <sup>rd</sup> 4 <sup>th</sup> 5 <sup>th</sup> 6 <sup>th</sup>
Total	Ground total	0.001	0.001	0.001	0.001			0.001	0.001	0.001	0.001	0.001	0.001
Time index ( $T$ )		$= \frac{[(\text{Time index of } 1^{\text{st}}) + (\text{Time index of } 2^{\text{nd}}) + \dots + (\text{Time index of } N^{\text{th}})] / (\text{Exclusion for number of story } N)}{(0.998 + 0.998 + 0.998 + 0.998 + 0.998) / 5}$											
		$= \frac{0.998 + 0.998 + 0.998 + 0.998 + 0.998}{5} = 0.998$											

**(f) Result of seismic evaluation**

$I_s$  value at level 1 and level 2 are lower than  $I_{so}$  ( $= 0.30$ ), and retrofit is required.

**Table 4.2.5 Result of Seismic Evaluation**

Storey	(n+1) / (n+i)	X- direction				Y- direction			
		$C$	$F$	$E_o$	$I_s$	$C$	$F$	$E_o$	$I_s$
4	0.63	0.634	1.50	0.0594	0.564	0.598	1.50	0.561	0.533
3	0.71	0.331	1.50	0.355	0.337	0.324	1.50	0.347	0.330
2	0.83	0.187	1.27	0.198	0.188	0.169	1.27	0.179	0.154
1	1.0	0.123	1.27	0.156	0.119	0.109	1.27	0.163	0.124

Note:  $E_o$ : Basic seismic index of structure,  $I_s = E_s \cdot S_D \cdot T$

Calculated  $F$  is higher than 1.5 at level 3 and level 4, but  $F=1.5$  was estimated considering “low strength concrete”.

**Table 4.2.6 Judgment**

Storey	$I_{so}$	X-direction		Y-direction	
		$I_s$	Judgment	$I_s$	Judgment
4	0.30	0.564	Satisfactory	0.533	Satisfactory
3		0.337	Satisfactory	0.330	Satisfactory
2		0.188	Not satisfactory	0.154	Not satisfactory
1		0.119	Not satisfactory	0.124	Not satisfactory

**(g) Checking of column against vertical load**

Checking of column axial strength against vertical load by BNBC ( $P_u = 0.85 \phi [0.85 f'_c \times A_c + f_y A_{st}]$ ) was done for a typical column at grid 5-D. The result shows that the column axial force capacity is satisfactory against vertical load or weight by dead and live load.

**4.2.3. Retrofit Design****(1) General****(a) Summary of seismic assessment**

1. Building irregularity is big.

1.1 Seismic assessment, horizontal stiffness is low at ground floor level for both directions. The level of grade beam is lower than ground floor, and the large structural storey height causes lower horizontal stiffness.

1.2 Torsional behavior occurs for transverse direction because of double height area at one side.

2. Ground floor and 1<sup>st</sup> floor are required to retrofit, to improve the irregularity and the strength oriented retrofit is planned.

**(b) Requirements**

1. The factory is under operation. Impacts to the operation by retrofit work shall be minimized.

**(c) Concept of retrofit**

1. Retrofit elements are provided at perimeter area and outside of existing brick wall. Impact to production activities shall be minimized.
2. Steel framed brace is provided and not RC wall, since there are windows and openings for ventilation at perimeter walls.
3. In-filled RC wall is provided under the steel framed brace up to the existing foundation footing to transfer the strength of steel framed brace.
4. As a result, the improvement of the irregularity is planned. Horizontal strength is increased.

**(d) Retrofit design**

1. Proposed Seismic demand index,  $I_{so} = 0.30$  based on BNBC2015, is used as 2<sup>nd</sup> level seismic screening method.

Natural period after retrofit will reduce because of stiffness increase, but 0.288 is the upper limit and same value is used for the evaluation after retrofit.

2. Ductility index,  $F = 1.27$  ( $= 1/150$ ), used for columns at ground storey and 1<sup>st</sup> storey.

Ductility index  $F$  of steel framed brace is can be expected in the range of 1.5 to 2.0 generally. There is an experimental study of steel framed brace using low strength concrete RC frame, and ductility index  $F$  up to 1.5 is suggested. In case of the retrofit design, ductility index of steel braced frame 1.27 ( $= 1/150$ ) was proposed to meet the condition of existing columns. This will reduce the damage of brick walls, non-structural elements. Ductility index of 1.5 will be used for 2<sup>nd</sup> and 3<sup>rd</sup> storey, and no retrofit will be required.

3. Indirect connection method is used.

When steel framed brace is designed at perimeter, additional RC beam and column are casted to provide indirect connection, and the eccentricity of steel framed brace is minimized.

4. Design of steel framed brace

Channel section by the combination of angle members will be used. Connection at a factory is based on fillet welding only and is not butt welding. Connection at a site is high strength bolting connection.

5. Post installed anchor

Chemical anchors are used and longer embedment is considered due to low strength concrete.

6. Design of RC wall under the steel framed brace

Design shear force of RC infill-wall will be more than horizontal strength of steel framed brace.

**(2) Materials for Retrofit****Table 4.2.7 Materials for Retrofit**

Material (concrete, re-bar, anchor, structural steel)	Concrete: $F_c = 25\text{N/mm}^2$ Re bar: $400\text{N/mm}^2$ (main bar), $400\text{N/mm}^2$ (tie) Structural steel: $\sigma_y = 345\text{N/mm}^2$ High strength bolt: $\sigma_y = 800\text{N/mm}^2$ Non-shrink grout mortar: $300\text{N/mm}^2$
Layout of retrofit elements	Well balanced layout for two directions
Connection of existing and new member through post-installed anchor	Indirect connection method between steel framed brace and existing RC frame will be applied.

**(3) Required Numbers of Steel Framed Brace**

$$E = \frac{n+1}{n+i} \times C \times F$$

$$I_s = E \cdot S_D \cdot T = \frac{n+1}{n+i} \times C \cdot F \cdot S_D \cdot T$$

After retrofit,  $C \cdot F = \frac{n+i}{n+1} \times \frac{I_s}{S_D \cdot T}$ , where  $C = \frac{Q}{\sum W}$

Then,  $\text{req } Q = \frac{n+i}{n+1} \times \frac{I_{so} \times \sum W}{(F \cdot S_D \cdot T)} - \frac{n+i}{n+1} \times \text{original } C \text{ (at } F=1.27) \times \sum W$

In case of ground floor,  $\text{req } Q = \frac{I_{so} \times \sum W}{(F \cdot S_D \cdot T)} - \text{original } C \text{ (at } F=1.27) \times \sum W$

**Table 4.2.8 Required numbers of Steel Framed Brace**

	$\Sigma W$ , Weight (kN), load factor is not used.	$\frac{n+i}{n+1}$	Design shear coefficient, $\frac{n+i}{n+1} \times \frac{0.30}{1.27 \times 0.95 \times 1.0}$ $I_{so} = 0.30, F = 1.27, S_D$ $= 0.95$ (after retrofit)	Design shear strength $Q$ , after retrofit, $\frac{n+i}{n+1} \times \frac{0.30}{1.27 \times 0.95 \times 1.0} \times \Sigma W_i$ (1)	Original strength, $C$ (at $F$ ) $\times \Sigma W_i$ , (2)		Required additional strength, $Q$ (1)- (2) (kN)	
					$Q$ (kN)	$C$	$Q$	
4	14,965 + 100	1.6	0.337 ( $F=1.5$ )	5,077 in case $F=1.5$	x	9,481	0.629	---
					y	8,955	0.594	---
3	32,878 + 300	1.4	0.295 ( $F=1.5$ )	9,788 in case $F=1.5$	x	10,892	0.328	---
					y	10,646	0.321	---
2	50,903 + 900	1.2	0.298 ( $F=1.27$ )	15,437	x	9,512	0.184	5,925
					y	8,606	0.166	6,831
1	64,491 + 1,900	1.0	0.249 ( $F=1.27$ )	16,531	x	7,909	0.119	8,622
					y	7,013	0.106	9,518

Note 1. Weight increase by additional staircase and steel framed brace with additional concrete was evaluated.

Level 4: 100kN, level 3: 200kN, level 2: 600kN, level 1: 1,000kN, total: 1,900kN

	Required additional strength, $Q$ (1)-(2) (kN)	Required no. of steel framed brace a) 2L-130×130×12 $Q_b = 2,600\text{kN}$ (standard span)	Required no. of steel framed brace b) 2L-130×130×12 $Q_b = 2,200\text{kN}$ (smaller span)	Required no. of steel framed brace c) 2L-100×100×12 $Q_b = 1,900\text{kN}$ (standard span)	Required no. of steel framed brace d) 2L-100×100×12 $Q_b = 1,650\text{kN}$ (smaller span)
4	---	----	----	----	----
3	---	----	----	----	----
2	X, 6,133	---	---	3.23 → 4	3.72 → 4
	Y, 7042	---	---	3.71 → 4	4.27
1	X, 8,822	3.39 → 4			
	Y, 9,718	3.74 → 4	4.42		

\*Steel,  $\sigma_y = 345\text{kN/mm}^2$  (ASTM A572)

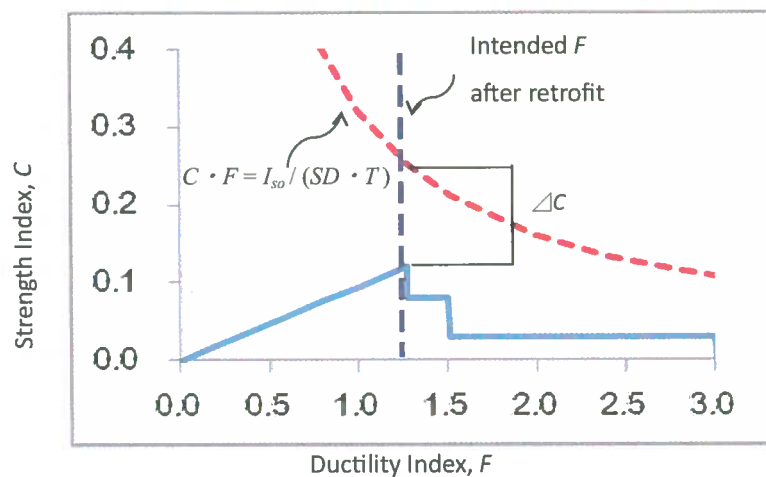
Following combination of steel framed brace is proposed. Well balanced layout of steel brace is planned to improve the irregularity. Irregularity index of each floor is 0.95.

X-direction: 4 Nos of type a) at level 1 and 4 Nos of type c) at level 2.

Y-direction: 2 Nos of type a) and 2 Nos type b) at level 1 and 2 Nos of type c) and 2 Nos type d) at level 2.

**Table 4.2.9 Additional Strength by Steel Framed Brace**

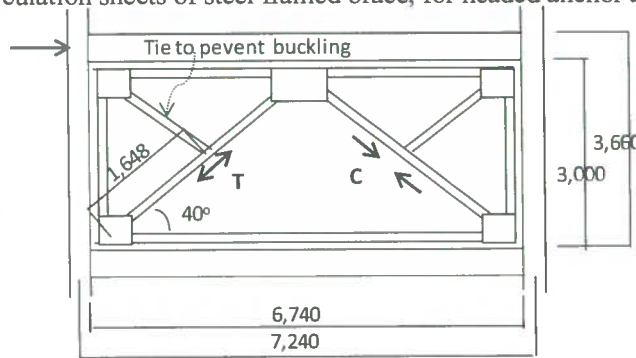
	Building weight (kN)	Additional strength, $Q$ (kN)		Additional strength coefficient, $C$	
		X-direction	Y-direction	X-direction	Y-direction
4	15,065	---	---	---	---
3	33,178	---	---	---	---
2	51,803	7,600 > 5,925	7,100 > 6,831	0.147	0.137
1	66,391	10,400 > 8,622	9,600 > 9,518	0.157	0.144



**Figure 4.2.2 Strength Index and Ductility Index Relation**

**(4) Design of Steel Framed Brace (SFB)**

Refer to Chapter 3 for the calculation sheets of steel framed brace, for headed anchor and post-installed anchor.



**Figure 4.2.3 Steel Framed Brace**

Case 1: **2 × L (channel shape) -130 × 130 × 12**, standard column span 7,240mm. Storey height 3,660mm

$$A = 29.76x^2 = 5950\text{mm}^2, w = 23.4x^2 = 46.8\text{kg/m}$$

$$l_x \text{ (buckling length)} = 4400\text{mm}, l_y = 2,200\text{mm (typical at ground floor level)}$$

$$\text{Radius of gyration, } i_x = \sqrt{(BH^3 - bh^3)/12 \cdot (BH - bh)} = \sqrt{(73384/714.24)} = 101.2\text{mm}, i_y = 39.6\text{mm}$$

$$\lambda_1 \text{ (slenderness ratio)} = 4400/101.2 = 43.4 \text{ (out of plane)}, \lambda_2 = 2200/39.6 = 55.5 \text{ (in-plane)}, \text{ then } \lambda = 55.5$$

$$f_c \text{ (limit compressive stress)} = (1 - 0.4 \cdot (\lambda/\lambda)^2) \cdot F = (1 - 0.4 \cdot (55.5/98.9)^2) \cdot 345 = 0.874 \times 345 = 301\text{N/mm}^2$$

$$C \text{ (limit compressive force)} = 301 \times 5950 = 1794\text{kN}$$

$$T \text{ (limit tensile force)} = 345 \times 5950 \times 0.8 \text{ (reduction by bolt hole)} = 1642\text{kN}$$

$$\text{Total } Q = (C + T) \times \cos 40.0\text{degree} = (1794 + 1642) \times 0.76 = \mathbf{2,611\text{kN}}$$

High strength bolt ( $\sigma_y = 800\text{N/mm}^2$ ), allowable force of M25 diameter = 177.5kN [This value provided by manufacturer.]

$$\text{No of bolt} = 1,642/177.5 = 9.5, \text{ then } 10 \text{ nos.}$$

**Headed stud**, ( $f_y = 400\text{N/mm}^2$ ), 2  $\phi$  16mm,

$$\text{Shear capacity } Q_{d_s} = 0.64 \cdot \sigma_{max} \cdot a_s = 0.64 \cdot 400 \cdot 201 = 51.4\text{kN}$$

$$\text{Required No.} = 2,611/51.4 = 50.8, \text{ then } 52 \text{ nos, } 2\phi 16\text{mm @ } 200$$

**Post installed anchor at upper beam**, ( $f_y = 400\text{N/mm}^2$ ),  $\phi$  20mm,

$$\text{Shear capacity by steel strength, } Q_{a1} = 0.7 \cdot \sigma_y \cdot s_{a_e} = 0.7 \cdot 400 \cdot 314 = 87.9 \text{ kN}$$

$$\text{Shear capacity by bearing strength of concrete } Q_{a2}. \text{ Young's modulus, } E_c = 2.1 \times 10^4 \cdot \sqrt{(\sigma_B/20)}$$

$$Q_{a2} = 0.4 \cdot \sqrt{E_c \cdot \sigma_B} \cdot s_{a_e} = 0.4 \cdot \sqrt{(23478 \cdot 25)} \cdot 314 = 96.2 \text{ kN (in case } \sigma_B = 25\text{N/mm}^2, \text{ new concrete)} > Q_{a1}$$

$$= 0.4 \cdot \sqrt{(14850 \cdot 10.7)} \cdot 314 = 50.0\text{kN (in case } \sigma_B = 10.7\text{N/mm}^2, \text{ existing concrete)} < Q_{a1}$$

$$\text{Required No.} = 2,611/87.9 = 29.7, \text{ then } 30 \text{ nos. } \phi 20\text{mm @ } 200\text{mm against new concrete}$$

$$= 2,611/50.0 = 52.2, \text{ then } 53 \text{ nos. } 2\phi 20\text{mm @ } 200\text{mm against existing concrete}$$

Case 2: **2 × L (channel shape) - 130 × 130 × 12**, in case of short span at ground floor level, 5,740mm  
 $\cos 50\text{degree} = 0.643$

$$\text{Total } Q = (C + T) \times \cos 50\text{degree} = (1794 + 1642) \times 0.643 = \mathbf{2,209 \text{ kN}}$$

**(5) Design of RC Wall Below Ground Floor Level**

RC in-filled wall below the ground floor, under the steel framed brace. Refer to Appendix 7 for the calculation sheets of steel framed brace, for headed anchor and post-installed anchor. Design shear force of RC in-filled wall is bigger than strength of steel brace.

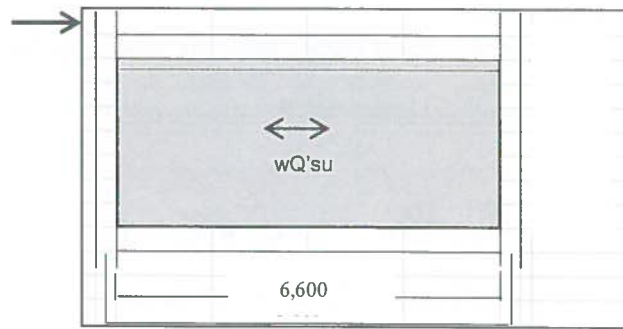


Figure 4.2.4 RC In-filled Wall

Calculation of shear wall, refer to Chapter 3.2.3. Shear strength of in-filled shear wall panel is shown here.

$$wQ_{su} = \max. (P_{w,w}\sigma_y, F_{cw}/20 + 0.5 \cdot P_{w,w}\sigma_y) \cdot \tau_w \cdot L$$

Re-bar: D10@150 double layer,  $P_w$  (shear reinforcement ratio) =  $78.5 \cdot 2 / (150 \times 200) = 0.00523$ ,

$$w\sigma_y = 400 \text{ N/mm}^2, F_{cw} = 25 \text{ N/mm}^2, \tau_w = 200 \text{ mm}, L = 6,600 \text{ mm (clear length)}$$

$$= \max (0.00523 \times 400, 25/20 + 0.5 \times 0.00523 \times 400) \times 200 \times 6,600$$

$$= \max (2.092, 2.296) \times 200 \times 6,600 = 3,032,000 \text{ N,}$$

$$wQ_{su} = 3.032 \text{ kN} > 2,600 \text{ kN (Strength of steel brace)} \times 1.1 = 2,860 \text{ kN (design strength of in-filled wall)} \text{ O.K.}$$

Note 1.1 \*strength of steel brace of ground floor was used as design strength of in-filled shear wall panel.

## (6) Result of Seismic Retrofit Design

### (a) Seismic index of structure $I_s$ after retrofit

Seismic index of structure,  $I_s$  at level 1 and level 2 are more than  $I_{so}$  ( $= 0.30$ ) and are satisfactory.

Table 4.2.10 Result after Retrofit

Storey	$\frac{n+1}{n+i}$	X-direction				Y-direction			
		C (Column +SFB)	F	$E_o$	$I_s$	C (Column +SFB)	F	$E_o$	$I_s$
4	0.63	0.629	1.50	0.594	0.565	0.574	1.50	0.561	0.533
3	0.71	0.328	1.50	0.349	0.332	0.321	1.50	0.342	0.325
2	0.83	$0.184 + 0.147 = 0.331$	1.27	0.349	0.331	$0.166 + 0.137 = 0.303$	1.27	0.319	0.303
1	1.0	$0.119 + 0.157 = 0.275$	1.27	0.350	0.332	$0.106 + 0.145 = 0.251$	1.27	0.318	0.302

Note: 1) Irregularity Index,  $S_{D2}$ , is 0.95 after retrofit. 2) Time Index,  $T = 1.0$  3)  $I_{so} = 0.30$

$E_o$ : Basic seismic index of structure,  $I_s = E_o \cdot S_D \cdot T$

2) Present C was modified based on the building weight after retrofit.

### (b) Requirement for Retrofit Construction

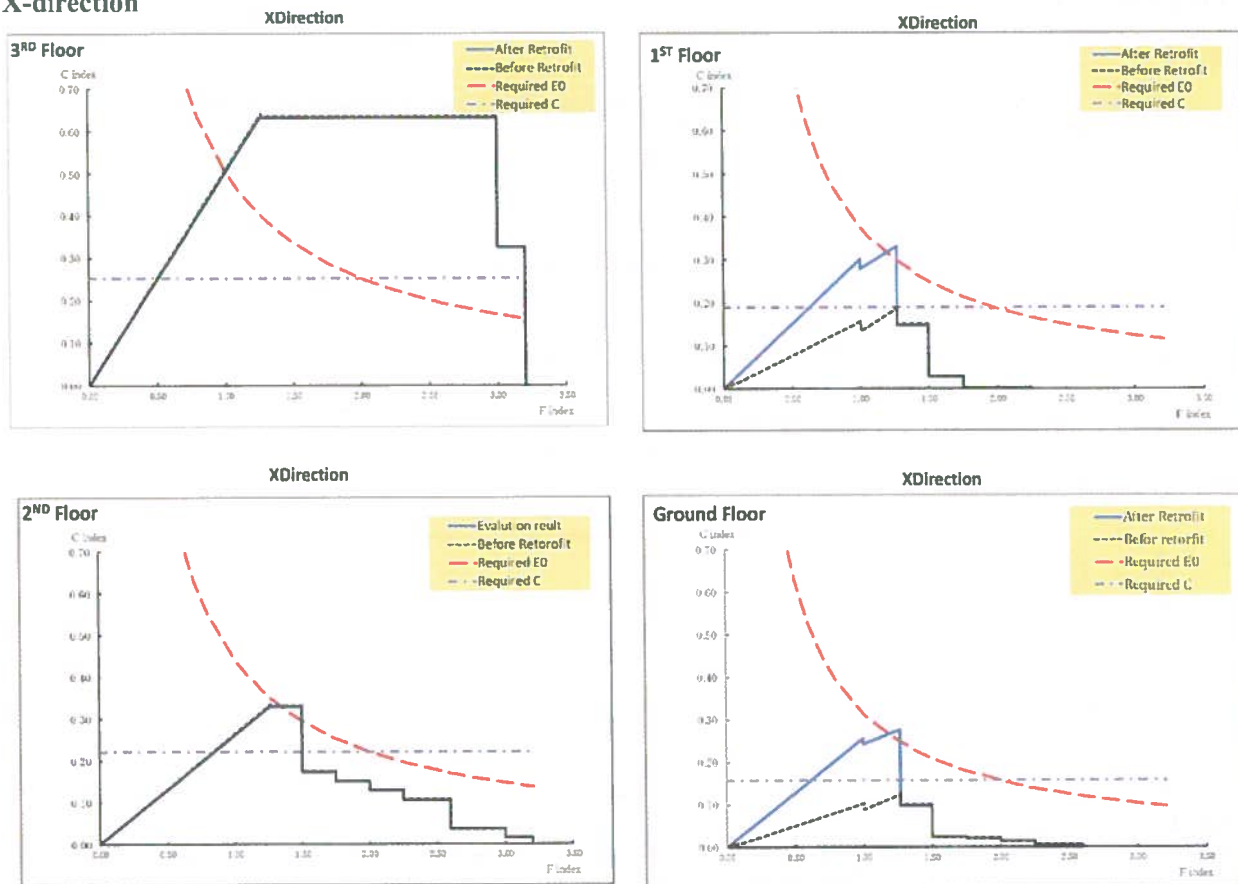
- 1) Erection method of steel framed brace will be studied at the beginning of the construction.
- 2) Excavation and shuttering work for RC wall work at underground will be studied so as not to affect the existing slab on grade.
- 3) Coordination work with existing pipes and cables for the production will be done before the construction.

### (c) C-F relation after retrofit,

C (strength index) F (ductility index) relation at each floor before and after retrofit is shown in Figure 4.8. Right upper side of the hyperbolic curve shown is the target area of seismic performance. This hyperbolic curve shows target  $E_o$  or  $I_{so}$  expressed by,

$$C \cdot F = \frac{n+i}{n+1} \times \frac{0.30}{0.95 \times 1.0}$$

## X-direction



## Y-direction

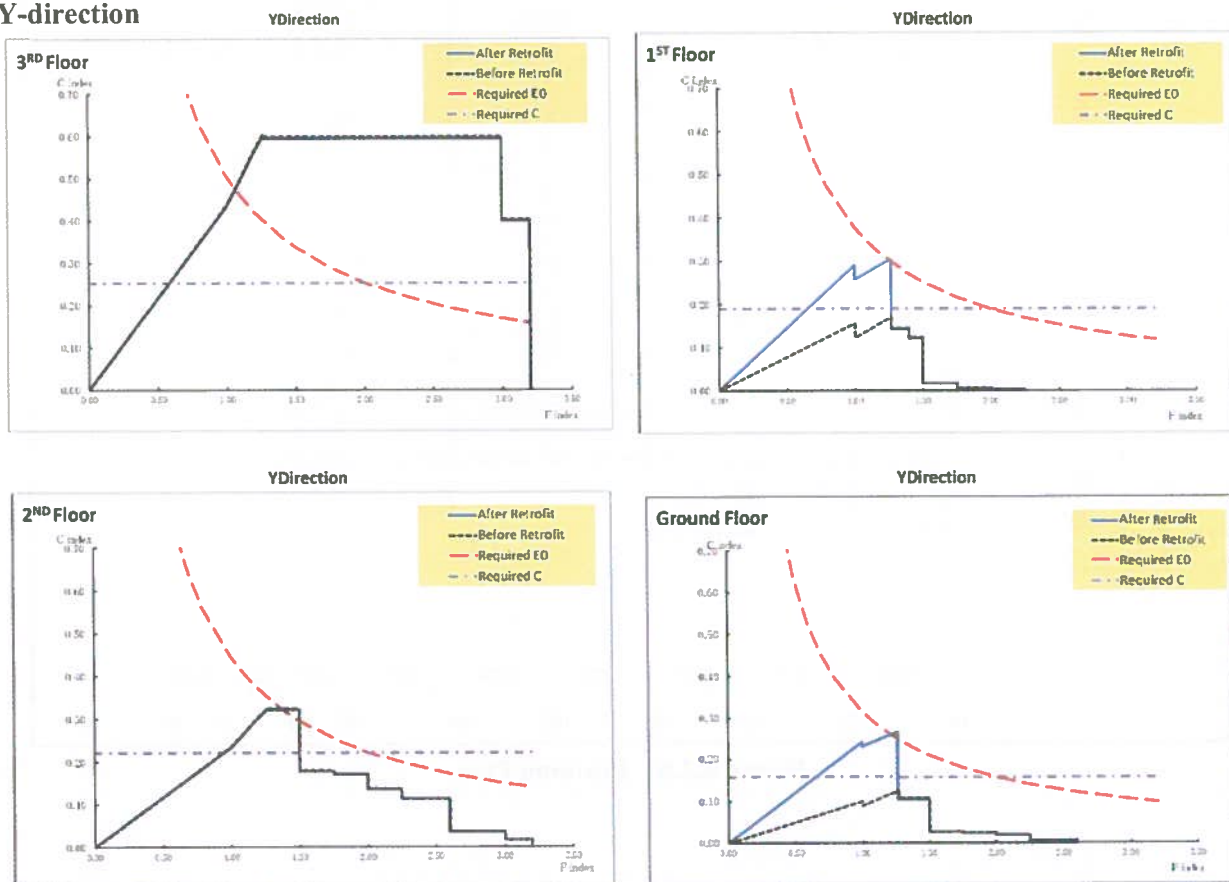
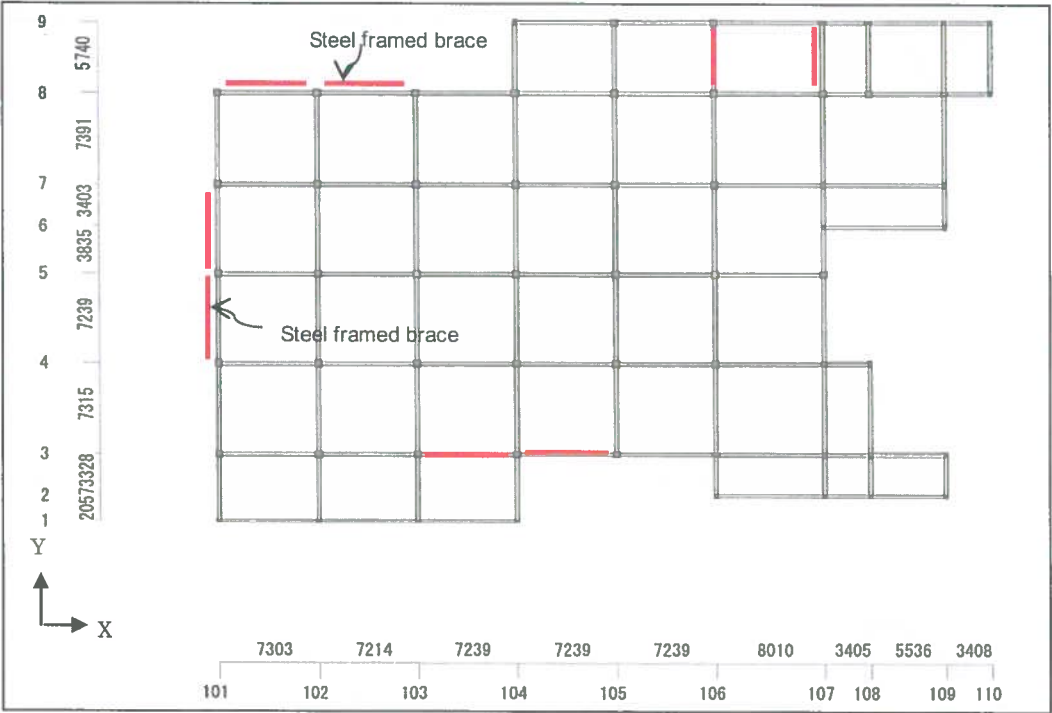


Figure 4.2.5 C-F Relation Curve after Retrofit

(7) Retrofit Plan

Level 1



Level 2

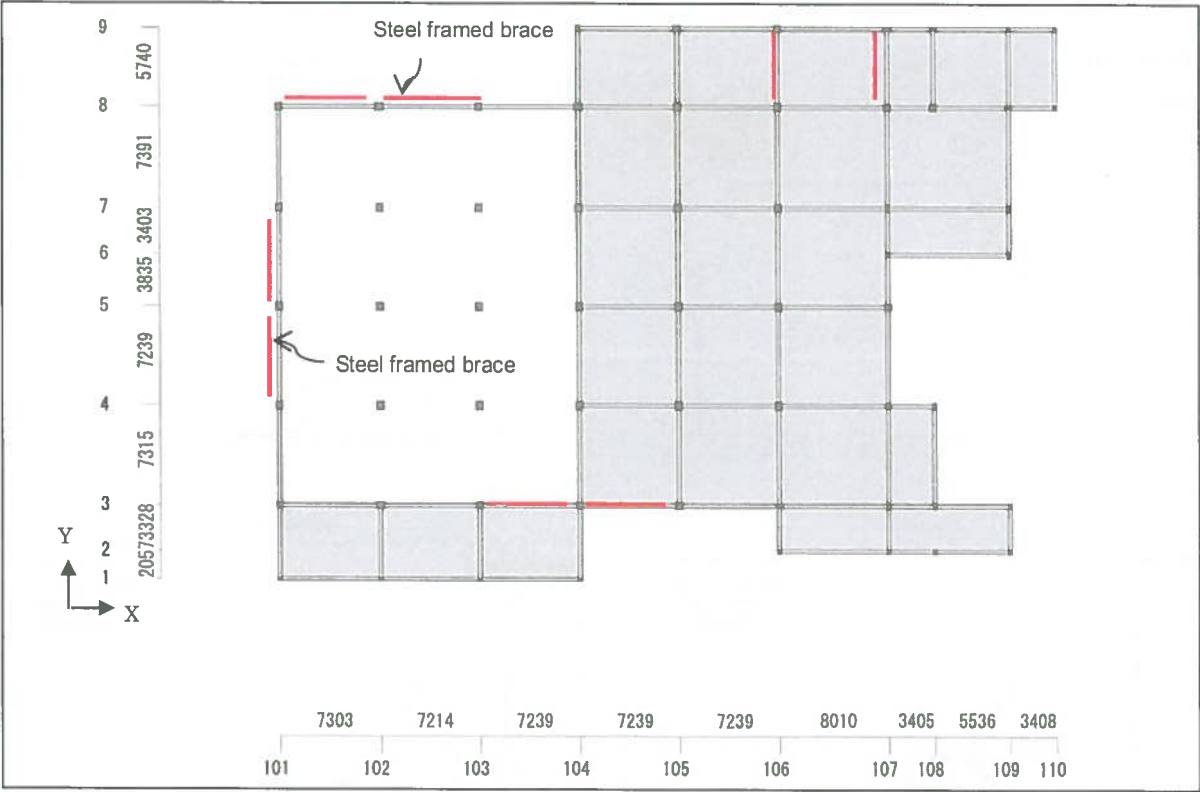
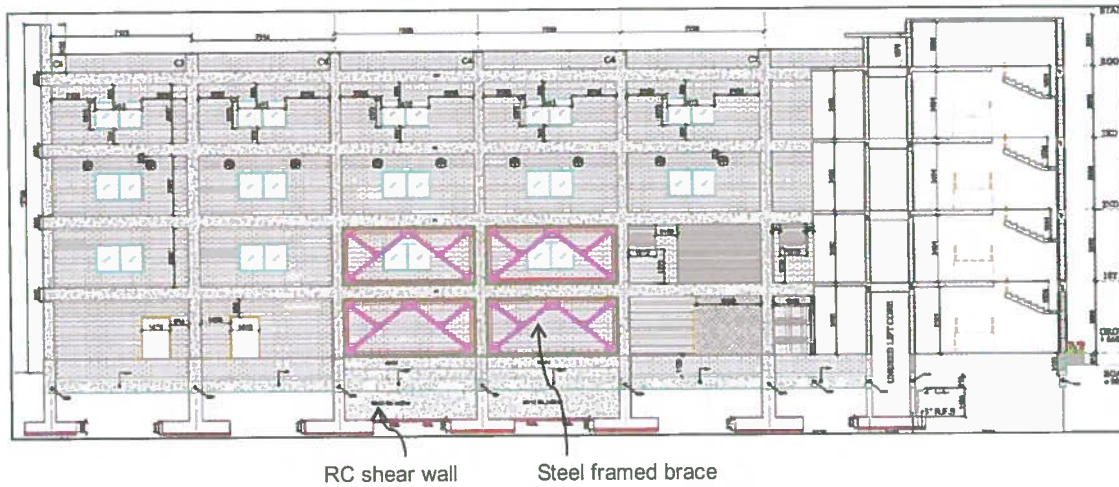


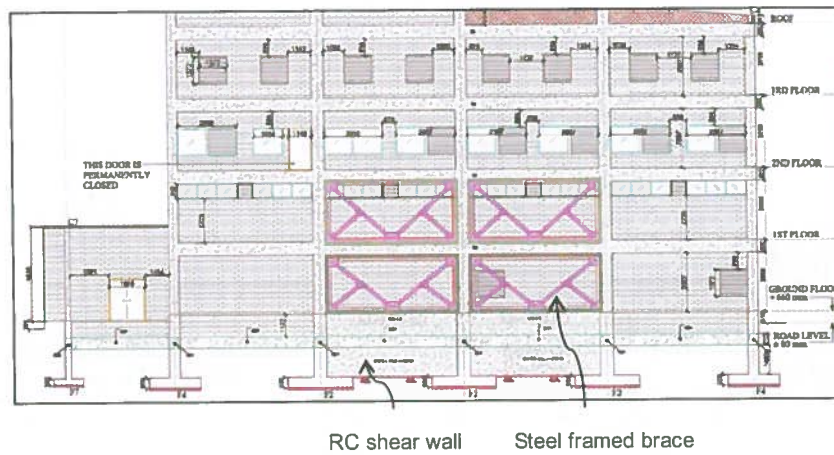
Figure 4.2.6 Framing Plan

## (8) Retrofit Elevation

### Grid 3



### Grid 101



### Grid 106

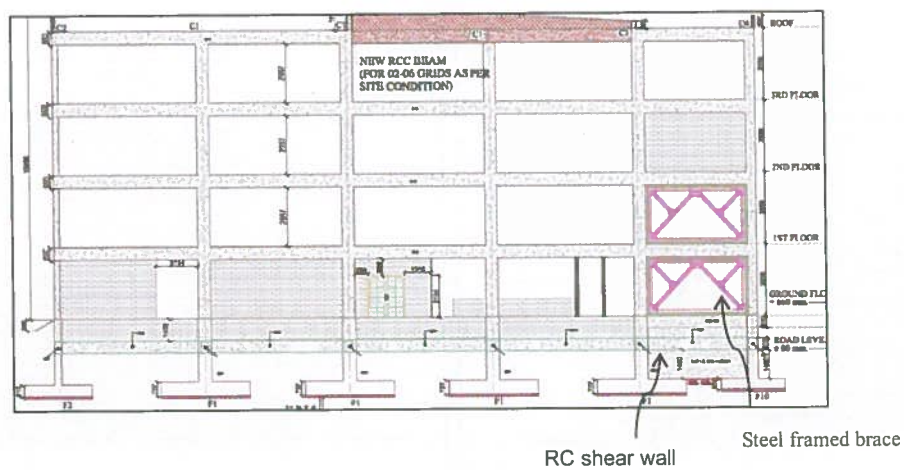
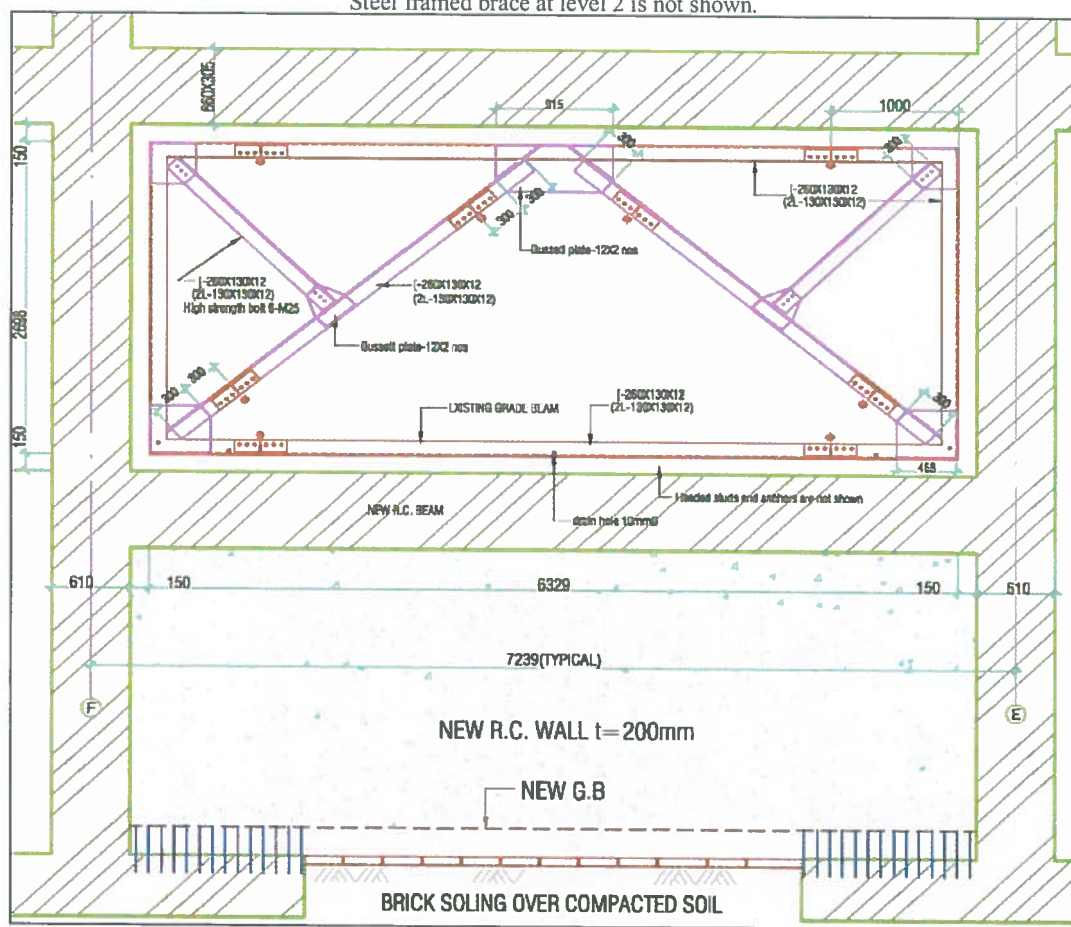


Figure 4.2.7 Framing Elevation

(9) Detail Framing Elevation and RC Wall Detail

Steel framed brace at level 2 is not shown.



RC wall type 1

RC wall type 2

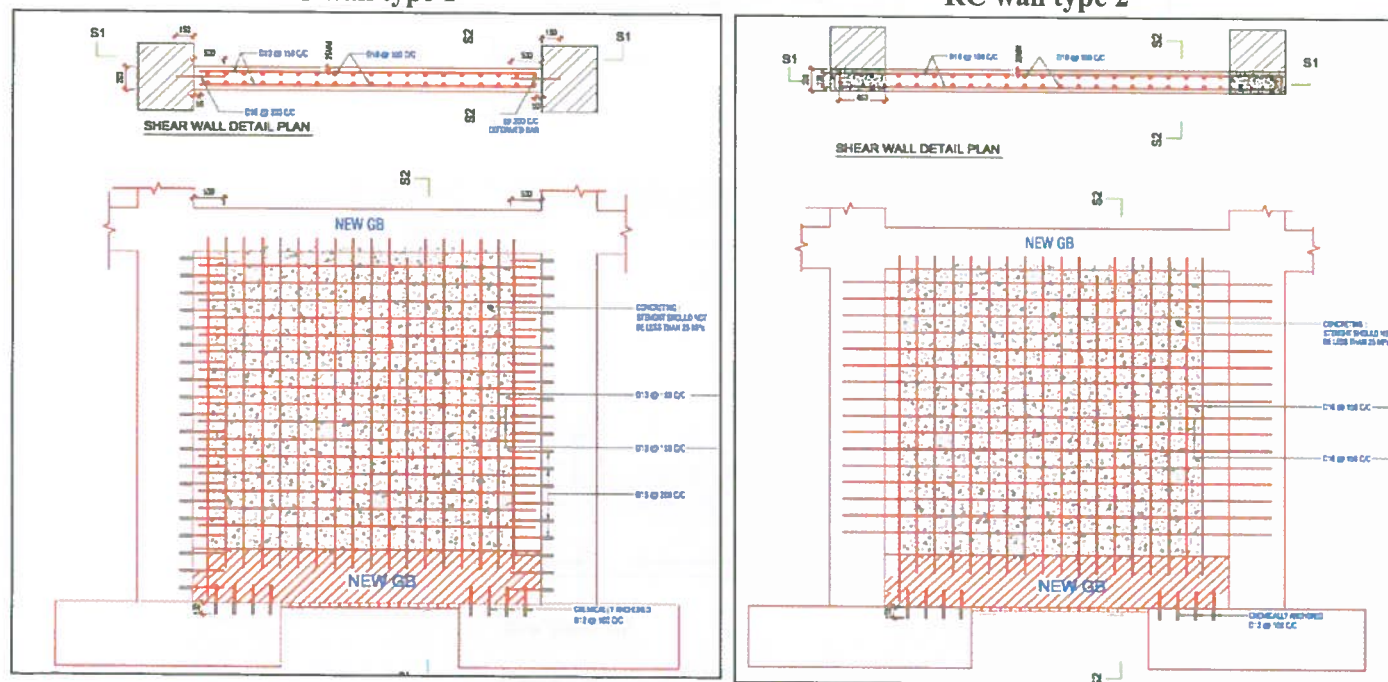


Figure 4.2.8 Structural Details (1/2)

# (10) Structural Detail

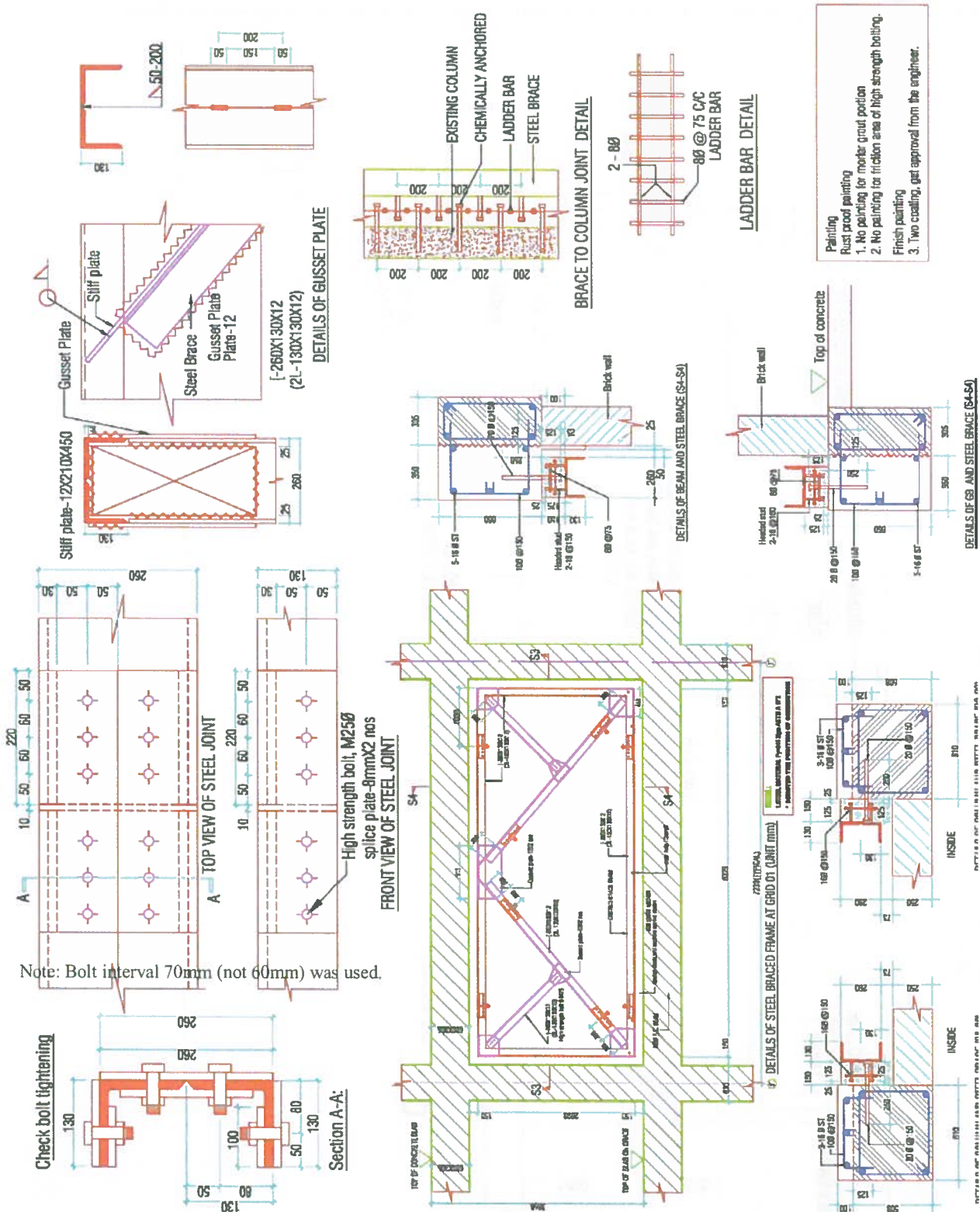


Figure 4.2.9 Structural Details (2/2)

**(11) Proposed Construction Sequence for RC Shear Wall**

Excavation sequence related to RC shear wall below ground floor level is proposed as follows.

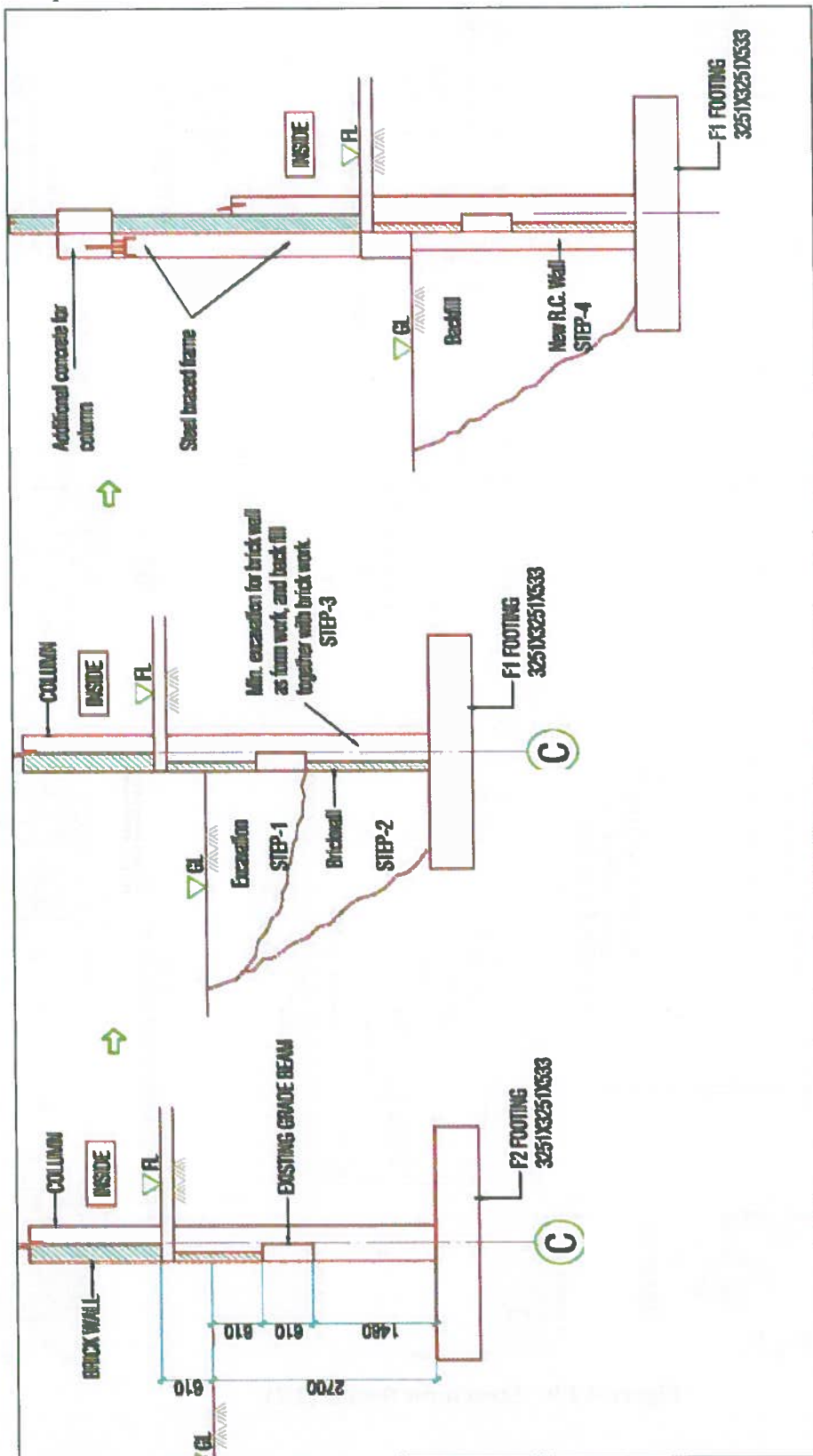


Figure 4.2.10 Proposed Excavation Work Sequence for RC Shear Wall

### 4.3 CASE 3: A SAMPLE RETROFIT DESIGN OF MIXED TYPE STRUCTURE IN DHAKA

#### 4.3.1. General

A sample retrofit design of two storey's mixed type structure of RC frame and brick masonry is introduced. 2nd level screening procedure by Japanese standard was applied. Since this manual doesn't cover brick masonry and the existing concrete is of extremely low strength the retrofit design calculation is for reference only. Engineers are requested to investigate different aspects carefully and take decision judiciously. During seismic evaluation and retrofit design of this building suggested modification for Bangladesh building (sec-1.11) was not considered.

#### 4.3.2. Seismic Evaluation

##### (1) Basic information about the building

- i. Usage of the building – Fire station
- ii. Occupancy type – IV
- iii. Year of construction – 1963
- iv. Two storied building with each floor approximately 272sqm  
Usage of ground floor – Office + Car parking  
Usage of 1<sup>st</sup> floor – Residence  
Stair portion continues up to roof.
- v. Structure type – Mixed type structure with brick masonry at both side and RC frame at middle part.
- vi. Ground floor height = 3100mm (Parking to beam bottom) + 400mm beam  
First floor height = 3000mm.
- vii. Top of grade beam = 800mm below parking level.

##### (2) Information of existing structural material

###### a) Brick-wall

According to the in-situ test of cement mortar joint of brick wall done at the site, shear strength was found to be  $0.188\text{N/mm}^2$  including the overburden pressure. This is very low compared with existing data (Average of 32nos of data;  $0.75\text{N/mm}^2$ , and Standard deviation;  $0.35\text{N/mm}^2$ ). Axial (vertical) stress of typical brick wall is estimated from  $0.16\text{N/mm}^2$  to  $0.32\text{N/mm}^2$  at ground floor. In this evaluation shear strength  $\tau = 0.2\text{N/mm}^2$  is taken by engineering judgment and was applied accordingly. Young's modulus will be approximately  $E = 0.60 \times 10^4 \text{N/mm}^2$  which is approximately 0.3 times that of ordinary concrete.

###### b) Concrete

Concrete strength of existing column,  $F_c = 7.8\text{N/mm}^2$  (Average – standard deviation/2) is calculated from 3 no's drilled concrete core test results. Due to extremely low strength concrete, reduction factor  $K_r$  is used for the evaluation of shear strength of column.

###### c) Reinforcement

Reinforcing bar strength,  $\sigma_y = 275\text{N/mm}^2$  is assumed considering construction period.

##### (3) Building weight

Unfactored unit weight and floor area is given below. Unit load of specific floor level is calculated considering 50% load from upper floor and 50% load from lower floor. Live load is considered as  $1.45 \text{kN/m}^2$ .

	Brick Masonry Part		RC frame part	
	Unit weight	Floor area	Unit weight	Floor area
Staircase at roof	$3.59 \text{ kN/m}^2$	$17.8\text{m}^2$		
Roof	$10.27\text{kN/m}^2$	$102.8\text{m}^2$	$6.87 \text{ kN/m}^2$	$168.8\text{m}^2$
1 <sup>st</sup> floor	$12.47 \text{ kN/m}^2$	$102.8\text{m}^2$	$7.21 \text{ kN/m}^2$	$168.8\text{m}^2$

Building weight of roof level,  $W = 64 + 1,056 + 1,160 = 2280\text{kN}$

Building weight of 1<sup>st</sup> floor level,  $W = 1,284 + 1,218 = 2,502\text{kN}$

Total weight;  $\Sigma W = 4,782\text{kN}$

#### (4) Proposed Seismic Demand Index $I_{SO}$

Using building and subsoil characteristic according to BNBC 2015 proposed seismic demand index,  $I_{SO}$  is calculated as follows and this value have been applied for assessment and retrofitting design by 2<sup>nd</sup> level screening method as per Japanese Standard.

$$I_{SO} = 0.80 \times \frac{2}{3} \times Z \times I \times C_S$$

Where:

$Z$  = Zone co-efficient = 0.2

$I$  = Importance factor = 1.5

$C_S$  = Normalized response spectrum

$$= S \left\{ 1 + \frac{T}{T_B} (2.5\eta - 1) \right\}$$

Where:

$T$  = Building period

$$= C_i (h_n)^m$$

For masonry structure

$$C_i = 0.0488 \text{ and } m = 0.75$$

$h_n$  = Building height = 6.5m

$$\therefore T = 0.0488 (6.5)^{0.75} = 0.2 \text{ sec}$$

$\eta$  = Damping correction factor = 1 for 5% viscous damping

For soil type SC

$$S = 1.15 \text{ and } T_B = 0.2$$

$$\text{Now, } C_S = 1.15 \left\{ 1 + \frac{0.2}{0.2} (2.5 \times 1 - 1) \right\}$$

$$= 2.875$$

$$\text{Therefore, } I_{SO} = 0.80 \times \frac{2}{3} \times 0.2 \times 1.5 \times 2.875$$

$$= 0.46$$

Note, for soil type SD;  $I_{SO} = 0.54$

and in case of BNBC 93,  $I_{SO} = 0.41$  (for stiff soil)

Floor plan and wall length (mm): Brick wall thickness, 250mm and 125mm.

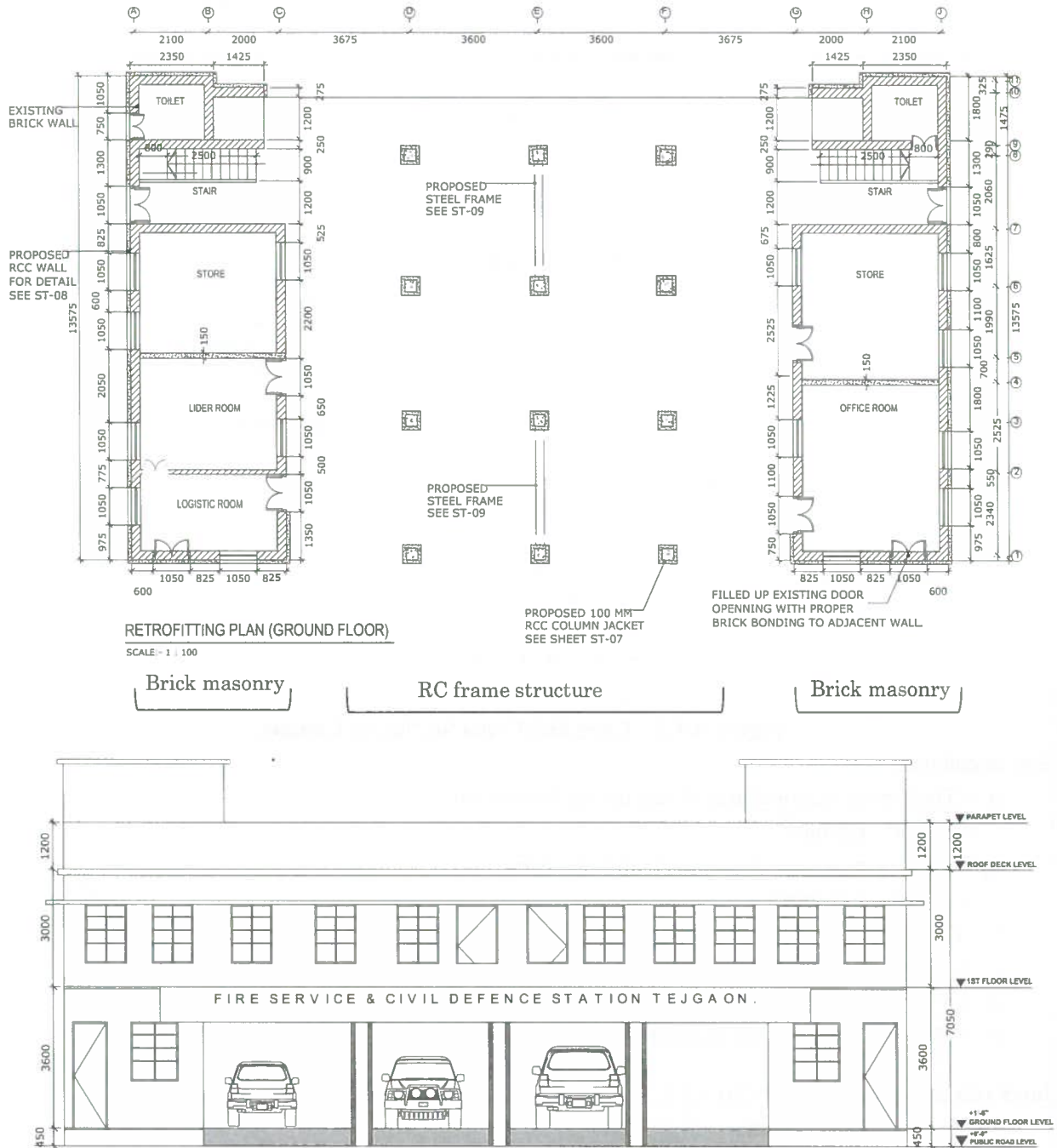


Figure 4.3.1 Floor Plan and Elevation

### (5) Seismic Index of Structure, $I_s$

2<sup>nd</sup> level screening procedure by Japanese Standard

#### (a) Strength Calculation of Column

Unit weight for roof =  $6.87 \text{ kN/m}^2$

Unit weight for 1<sup>st</sup> floor =  $7.21 \text{ kN/m}^2$

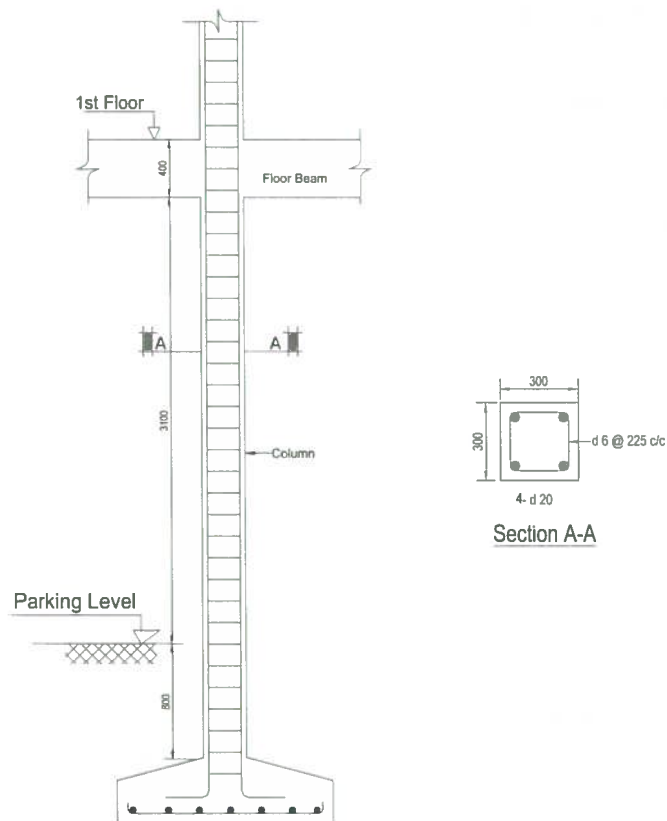


Figure 4.3.2 Long and Cross Section of Column

For calculation-

$a_t$  = Total cross sectional area of tensile reinforcing bars

$$= 2 \times 314 = 628 \text{ mm}^2$$

$a_g$  = Total cross sectional area of reinforcing bars.

$$= 4 \times 314 = 1256 \text{ mm}^2$$

$b$  = Column width = 300mm

$D$  = Column depth = 300mm

$\sigma_y$  = Yield strength of tensile reinforcing bars =  $275 \text{ N/mm}^2$

$F_c$  = Compressive strength of concrete =  $7.8 \text{ N/mm}^2$

Inner column tributary area =  $3.6 \times 3.5$

$$= 12.6 \text{ sqm}$$

Axial force on column,

$$N = (6.87 + 7.21) \times 12.6$$

$$= 177.4 \text{ kN.}$$

Axial strength of column,

$$N_{max} = b \cdot D \cdot F_c + a_g \cdot \sigma_y$$

$$= 300 \times 300 \times 7.8 + 4 \times 314 \times 275$$

$$= 1047.4 \text{ kN}$$

Check,  $0.4 \cdot b \cdot D \cdot F_c = 0.4 \times 300 \times 300 \times 7.8$

$$= 280.8 \text{ kN} > N (177.4 \text{ kN})$$

∴ Flexural strength of column will be calculated by the following equation:

$$\begin{aligned}
 M_u &= 0.8a_t \cdot \sigma_y \cdot D + 0.5N \cdot D \left( 1 - \frac{N}{b \cdot D \cdot F_c} \right) \\
 &= 0.8 \times 2 \times 314 \times 275 \times 300 + 0.5 \times 177.8 \times 1000 \times 300 \left( 1 - \frac{177.4 \times 1000}{300 \times 300 \times 7.8} \right) \\
 &= 41.4 \times 10^6 + 19.9 \times 10^6 \\
 &= 61.3 \times 10^6 \text{ N-mm}
 \end{aligned}$$

Shear force at flexural strength:

$$\therefore Q_{mu} = \frac{2M_u}{h_o} = \frac{2 \times 61.3 \times 10^6}{800 + 3100} = 31.4 \text{ kN}$$

Ultimate shear strength:

$$Q_{su} = K_r \left\{ \frac{0.053 p_t^{0.23} (18 + F_c)}{M/(Q \cdot d) + 0.12} + 0.85 \sqrt{p_w \times_s \sigma_{wy}} + 0.1 \sigma_o \right\} \cdot b \cdot j$$

Where reduction factor for low strength concrete:

$$\begin{aligned}
 K_r &= 0.244 + 0.056 F_c \\
 &= 0.244 + 0.056 \times 7.8 \\
 &= 0.68
 \end{aligned}$$

$$\text{Tensile reinforcement ratio in \%} = p_t = \frac{2 \times 314 \times 100}{300 \times 300} = 0.697\%$$

$$\text{Shear reinforcement ratio} = p_w = \frac{2 \times 28.3}{300 \times 225} = 0.00084$$

$$\text{Axial stress in column} = \sigma_o = \frac{N}{b \cdot D} = \frac{177.4 \times 1000}{300 \times 300} = 197 \text{ N/mm}^2$$

$$\text{Shear span} = M/Q = \frac{h_o}{2} = \frac{3900}{2} = 1950 \text{ mm}$$

$$\frac{M}{Qd} = \frac{h_o/2}{d} = \frac{1950}{300} = 6.5 > 3$$

If  $M/(Q \cdot d)$  is greater than 3, then its value shall be taken as 3

$$\therefore M/(Q \cdot d) = 3$$

$$J = 0.8D = 240 \text{ mm}$$

$$\begin{aligned}
 \therefore Q_{su} &= 0.68 \left\{ \frac{0.053 \times 0.697^{0.23} (18 + 7.8)}{3 + 0.12} + 0.85 \sqrt{0.00084 \times 275} + 0.1 \times 1.97 \right\} \times 300 \times 240 \\
 &= 0.68 \{ 0.403 + 0.408 + 0.197 \} \times 300 \times 240 \\
 &= 49.3 \text{ kN} > 31.4 \text{ kN} (Q_{mu})
 \end{aligned}$$

∴ Shear strength of column = 31.4 kN

Total shear strength of columns at ground floor =  $12 \times 31.4 = 377 \text{ kN}$

If shear stress of long column, i.e.  $\tau = 0.7$  is considered.

Then Shear strength of 12 columns will be

$$= 0.7 \times 300 \times 300 \times 12/1000$$

$$= 756 \text{ kN} > 377 \text{ kN}$$

So, total shear strength of column = 377 kN

Clear height of column at 1<sup>st</sup> floor,  $h_o = 2600 \text{ mm}$

If same  $M_u$  for column at 1<sup>st</sup> floor is considered

$$\text{then } Q_{mu} = \frac{2 \times 61.3 \times 1000}{2600} = 47.2 \text{ kN} < 49.3 \text{ kN}$$

So, total shear strength of column at 1<sup>st</sup> floor =  $47.2 \times 12 = 566.4 \text{ kN}$

### (b) Strength Calculation of Brick Wall

#### Considerations:

- i) Brick walls with 250mm thickness at window level are estimated for strength calculation.
- ii) Brick walls located out side of projected line of RC frame portion are excluded.
- iii) Brick walls length less than 400mm are excluded due to flexural behavior.

Calculation of  $I_s$  at ground floor:

i) At Ground floor,

X-direction total section area of brick wall is  $4,925,000 \text{ mm}^2$ .

So, shear strength of brick wall =  $4,925,000 \times 0.2/1000$   
 $= 985 \text{ kN}$ .

Y-direction total section area of brick wall is  $7,755,000 \text{ mm}^2$ .

So, shear strength of brick wall =  $7,755,000 \times 0.2/1000$   
 $= 1,551 \text{ kN}$ .

i) At 1<sup>st</sup> floor,

X-direction total section area of brick wall is  $6,439,750 \text{ mm}^2$ .

So, shear strength of brick wall =  $6,439,750 \times 0.2/1000$   
 $= 1,288 \text{ kN}$ .

Y-direction total section area of brick wall is  $8,550,000 \text{ mm}^2$ .

So, shear strength of brick wall =  $8,550,000 \times 0.2/1000$   
 $= 1,710 \text{ kN}$ .

### (c) Seismic index of structure, $I_s$

Time index,  $T = 0.8$  (For age > 30 years)

Irregularity index  $S = 0.9$  (For soft ground floor)

Considering allowable deflection angle of brick masonry is  $1/500$ .

So, ductility index;  $F = 0.8$  and effective strength factor of column,  $\alpha = 0.5$  [Ref: Table 3, J. Standard]

(i) Calculation of  $I_s$  at ground floor:

i) In X-direction (long direction)

Building weight at middle of the ground floor = 4782 kN

$\therefore$  Strength index,

$$C = \frac{Q}{\sum W} = \frac{\text{Strength of brick wall in X - direction} + \text{Strength of all column}}{\text{Building Weight}} = \frac{985 + 0.5 \times 377}{4782} = 0.245$$

$$\begin{aligned}
 \therefore I_s &= \frac{n+1}{n+i} \times C \times F \times T \times S \\
 &= 1 \times 0.245 \times 0.8 \times 0.8 \times 0.9 \\
 &= 0.14 < 0.46
 \end{aligned}$$

ii) In Y direction (Transverse direction)

$$\begin{aligned}
 \therefore \text{Strength index, } C &= \frac{1551 + 0.5 \times 377}{4782} \\
 &= 0.364
 \end{aligned}$$

$$\begin{aligned}
 \therefore I_s &= \frac{n+1}{n+i} \times C \times F \times T \times S \\
 &= 1 \times 0.364 \times 0.8 \times 0.8 \times 0.9 \\
 &= 0.21 < 0.46
 \end{aligned}$$

(ii) Calculation of  $I_s$  at 1<sup>st</sup> floor

i) In X-direction (long direction)

Building weight at 1<sup>st</sup> floor level = 2280 kN

$$\begin{aligned}
 \text{So, strength index, } C &= \frac{Q}{\sum W} \\
 &= \frac{1288 + 0.5 \times 566.4}{2280} \\
 &= 0.689
 \end{aligned}$$

$$\begin{aligned}
 \text{Basic seismic index, } E_o &= \frac{n+1}{n+i} \times C \times F \\
 &= \frac{2+1}{2+2} \times 0.689 \times 0.8 \\
 &= 0.413
 \end{aligned}$$

$$\begin{aligned}
 \therefore I_s &= E_o \times S \times T \\
 &= 0.413 \times 0.9 \times 0.8 \\
 &= 0.30 < 0.46
 \end{aligned}$$

ii) In Y-direction (Short direction)

$$\begin{aligned}
 \text{Strength index, } C &= \frac{1710 + 0.5 \times 566.4}{2280} \\
 &= 0.874
 \end{aligned}$$

$$\begin{aligned}
 \text{Basic seismic index, } E_o &= \frac{n+1}{n+i} \times C \times F \\
 &= \frac{2+1}{2+2} \times 0.874 \times 0.8 \\
 &= 0.524
 \end{aligned}$$

$$\begin{aligned}
 \therefore I_s &= E_o \times S \times T \\
 &= 0.524 \times 0.9 \times 0.8 \\
 &= 0.38 < 0.46
 \end{aligned}$$

**Table 4.3.1 Result of Seismic Evaluation**

Storey	X direction				Y direction			
	$C$	$E$	$I_s$	$I_{so}$	$C$	$E$	$I_s$	$I_{so}$
2	0.69	0.41	0.30	< 0.45	0.87	0.52	0.38	< 0.45
1	0.25	0.20	0.14	< 0.45	0.36	0.29	0.21	< 0.45

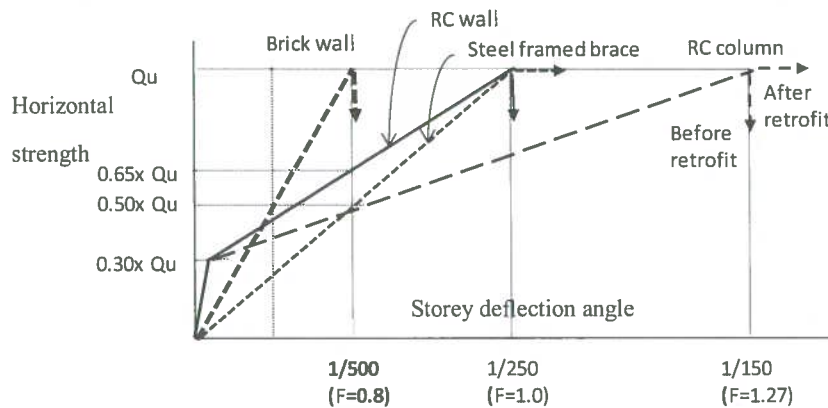
Proposed  $I_{so} = 0.46$  (2<sup>nd</sup> level screening, BNBC 2015, soil type SC) (Soil type SD,  $I_{so} = 0.54$ )

Seismic index of structure  $I_s$  of each floor and each direction is lower than proposed Seismic demand index of structure  $I_{so}$ , and seismic retrofit is required.

### 4.3.3 Seismic Retrofit Design

#### (1) Basic Idea

- 1) To prevent shear failure of brick masonry walls, which may cause collapse against vertical load.
- 2) To increase horizontal strength and to unite the brick masonry area, floor slab and RC frame area, “concrete jacketing” of perimeter walls are provided.
- 3) Steel framed brace is introduced at Y direction to increase horizontal strength and to reduce shear stress of floor slab at 1<sup>st</sup> floor.
- 4) Strength oriented retrofitting is used to reduce horizontal deflection.
- 5) Load-deflection characteristics of each member are as follows,

**Figure 4.3.3 Load-deflection Characteristics**

Reference: Japan Building Disaster Prevention Association, “Examples of Seismic Retrofit for Existing RC buildings 2009 (in Japanese)” Curve of steel framed brace is added.

- 6) Strength contribution factor  $\alpha$ ,  
Strength index  $C$  is evaluated incorporating the Strength contribution factor  $\alpha$  of RC shear wall, steel framed brace and RC column.

**RC shear wall;  $\alpha = 0.65$ , Steel framed brace;  $\alpha = 0.50$ , RC column;  $\alpha = 0.50$**

Ductility index of brick masonry,  $F$  (ductility index) = **0.8** (storey deflection angle, 1/500).

**(2) Basic Design****(a) Brick masonry portion**

- (i) To increase shear strength, “concrete jacketing with thickness of 100mm with single layer of reinforcing bars” at outside of perimeter walls is provided. RC floor slab is connected to RC jacketed wall at perimeter by chemical anchors. Vertical load is supported by existing brick masonry walls. Overturning moment due to seismic load will be resisted by RC jacketed walls.
- (ii) To increase the shear strength of X direction (long direction) at ground floor, RC wall with thickness 150mm with double layer reinforcement is provided at grid 4 of both sides. This wall is connected to 1st floor slab by chemical anchors.
- (iii) X-direction; to increase the strength door opening at Grid 1 is closed and RC wall is provided. To increase the unity of frames, Y direction wall with 750mm~1,350mm length is provided at grid C and grid G near Grid 1.

**(b) RC frame portion**

- (i) Existing RC column is jacketed at ground floor.
- (ii) Steel framed brace is added at grid E (Y-direction) to increase horizontal strength and to improve the irregularity related problem of Y-direction (transverse direction). RC wall is provided beneath steel framed brace.

**(c) Non-structural components (elements)**

- (i) Internal walls are investigated against possible horizontal acceleration for out of-plane movement. Horizontal acceleration of  $0.63G \{= 0.38(n+i)/(n+1) \times 1.25\}$  and more will be applied for walls of 125mm thickness at 1<sup>st</sup> floor and will be retrofitted by providing steel members such as angle and channel where required.

*Note: RC post (125mmx125mm) will be provided at the edge of 125mm wall as a retrofit.*

- (ii) Non-structural wall at perimeter of RC frame portion (grid 1 and grid 9) will be made by RC wall.
- (iii) Parapet wall at roof is brick wall and is jacketed by RC wall.
- (iv) Protrusion (protrude objects) such as staircase at roof will be jacketed by RC wall.
- (v) Water tank at roof will be investigated and retrofitted where necessary.
- (vi) Existing standing wall at veranda of 1<sup>st</sup> floor is RC wall.

**(d) Materials**

Existing portion: concrete,  $F_c = 7.8\text{N/mm}^2$  (extremely low strength concrete), re-bar,  $\sigma_y = 275\text{N/mm}^2$ , brick wall, shear strength of mortar by test at the site,  $\tau = 0.2\text{N/mm}^2$

Retrofit portion: Concrete,  $F_c = 25\text{N/mm}^2$ , re-bar,  $\sigma_y = 400\text{N/mm}^2$ , Structural steel,  $\sigma_y = 425\text{N/mm}^2$

Related material specification of BNBC shall be applied.

**(3) Retrofit Design of Structural Members**

Additional weight by retrofit, Additional concrete wall, 412kN ( $412 \times 0.5 = 206\text{kN}$  for 1<sup>st</sup> storey)

Column jacketing,	203kN	
Steel framed bracing,	29kN	Total 645kN

**(a) Strength Calculation of Jacketed Column:**

Original column:

Size = 300 × 300

Main rebar = 4-d20

Tie = 6d @ 225

 $F_c = 7.8 \text{ Mpa}$  $\sigma_y = 275 \text{ Mpa}$ 

Jacketed part:

Jacketed column size = 500 × 500

Additional main rebar = 8-d16

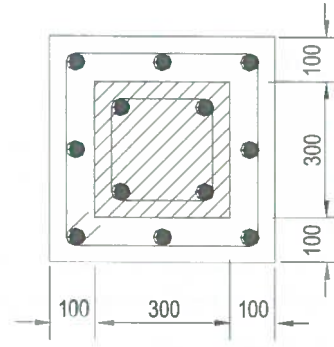
Hoop spacing = d10@150

 $F_c = 25 \text{ Mpa}$  $\sigma_{y2} = 400 \text{ Mpa}$ 

Free height of column at ground floor = 3900

Axial load on column at GF,  $N = 177.4 \text{ kN}$ 

Flexural strength of column at GF

**Figure 4.3.4 Section of Jacketed Column**

$$M_u = a_t \cdot \sigma_y \cdot g + a_{t2} \cdot \sigma_{y2} \cdot g_2 + 0.5 \cdot N \cdot D_2 \left( 1 - \frac{N}{b_2 \cdot D_2 \cdot F_{cl}} \right) \quad (\text{J. Guidelines 3.3.4-2})$$

Where,

$a_t$  = Cross sectional area of tensile reinforcing bars of existing column.  
 $= 2 \times 314 \text{ mm}^2$ .

$a_{t2}$  = Cross sectional area of tensile reinforcing bars of jacketing part.  
 $= 3 \times 201 \text{ mm}^2$ .

$\sigma_y$  = Yield strength of tensile reinforcing bars of existing column.  
 $= 275 \text{ N/mm}^2$ .

$\sigma_{y2}$  = Yield strength of tensile reinforcing bars of jacketing part.  
 $= 400 \text{ N/mm}^2$ .

$g$  = Distance between tensile and compressive longitudinal reinforcement of existing column.  
 $= 188 \text{ mm}$ .

$g_2$  =  $g$  for jacketing part of column.  
 $= 384 \text{ mm}$ .

$b_2$  = Width of column after jacketing.  
 $= 500 \text{ mm}$ .

$D_2$  = Depth of column after jacketing.  
 $= 500 \text{ mm}$ .

$F_{cl}$  = Compressive strength of concrete for jacketing part.  
 $= 25 \text{ N/mm}^2$ .

$$\begin{aligned} \text{So, } M_u &= 2 \times 314 \times 275 \times 188 + 3 \times 201 \times 400 \times 384 + 0.5 \times 177.4 \times 1000 \times 500 \left( 1 - \frac{177.4 \times 1000}{500 \times 500 \times 7.8} \right) \\ &= 32.47 \times 10^6 + 92.62 \times 10^6 + 40.32 \times 10^6 \\ &= 165.41 \times 10^6 \text{ N}\cdot\text{mm} \end{aligned}$$

*Note: In this calculation, concrete strength of existing column was used. Average strength of original and new concrete will also be acceptable to use.*

So, shear force at flexural strength,

$$\therefore Q_{mu} = \frac{2M_u}{h_o} = \frac{2 \times 165.41 \times 10^6}{3900} = 84.8 \text{ kN}$$

Ultimate shear strength of column retrofitted by RC jacketing

$$Q_{su} = \left\{ \frac{0.053 \cdot P_{t2}^{0.23} \cdot (F_{c1} + 18)}{M/(Q \cdot d_2) + 0.12} + 0.85 \sqrt{p_w \cdot \sigma_{wy} + p_{w2} \cdot \sigma_{wy2}} + 0.1 \frac{N}{b_2 \cdot D_2} \right\} \times 0.8 \cdot b_2 \cdot D_2$$

$M/Q \cdot d_2$  shall be in the range of 1.0 to 3.0 (J. Guidelines 3.3.4-3)

Where,

$P_{t2}$  = Tensile reinforcement ratio in % with respect to column size after jacketing

$$= \frac{3 \times 201}{500 \times 500} \times 100 = 0.24\% \quad [\text{Previous rebar ignored}]$$

$P_w$  = Shear reinforcement ratio of existing column

$$= \frac{2 \times 28.3}{500 \times 225} = 0.00050$$

$p_{w2}$  = Shear reinforcement ratio after column jacketing,  $p_w + p_{w2}$  shall be 0.012 if it is more than 0.012

$$= \frac{2 \times 78.5}{500 \times 150} = 0.00209$$

Check;  $P_w + P_{w2} = 0.00050 + 0.00209 = 0.00259 < 0.012$

$\sigma_{wy} = 275 \text{ MPa}$  (Yield strength of shear reinforcement in the existing column)

$\sigma_{wy2} = 400 \text{ MPa}$  (Yield strength of shear reinforcement in the jacketing column)

$$M/Q = \frac{h_o}{2}$$

$$\therefore \frac{M}{Q \cdot d} = \frac{3900/2}{500} = 3.9 > 3$$

So,  $M/Q \cdot d = 3$  [Note that  $M/Q \cdot d$  shall not be more than 3]

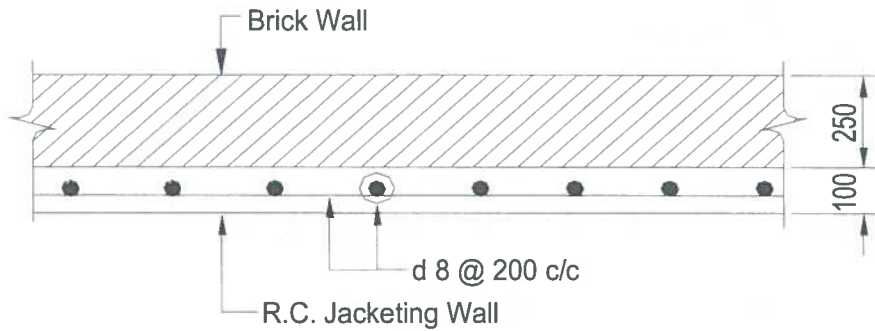
$$Q_{su} = \left\{ \frac{0.053 \times 0.24^{0.23} (7.8 + 18)}{3 + 0.12} + 0.85 \sqrt{0.00050 \times 275 + 0.00209 \times 400} + 0.1 \times \frac{177.4 \times 1000}{500 \times 500} \right\} \times 0.8 \times 500 \times 500$$

$$= \{0.315 + 0.839 + 0.071\} \times 0.8 \times 500 \times 500$$

$$= 245 \text{ kN} > Q_{mu}$$

$\therefore$  Shear strength of column = 84.8 kN.

Note: There will be some limitation of flexural strength after jacketing of column considering the flexural strength of beams. In this case, stress level is low and no reduction was done.

**(b) Strength Calculation of RC Jacketing over Existing Masonry Wall.****Figure 4.3.5 Section of Wall Jacketing**

Thickness of jacketing wall,  $t_w = 100\text{mm}$ .

Length of jacketing wall at grid-1,  $l = 2 \times (2500 + 850) = 6700$  [Entrance opening is filled by RC jacketing wall over new brick wall]

at Grid-11,  $l = 2 \times (2450 + 1500) = 7900$

$F_{cw}$  = Compressive strength of concrete of RC jacketing wall.  
 $= 25\text{N/mm}^2$ .

$\sigma_{wy}$  = Yield strength of reinforcement of jacketing wall.  
 $= 400\text{N/mm}^2$ .

Reinforcement =  $\phi 8 @ 200\text{mm c/c}$  each way shear stress is calculated as follows,

$$\tau = \max \left\{ p_w \cdot \sigma_y, \frac{F_{cw}}{20} + 0.5 p_w \cdot \sigma_y \right\} \quad (\text{Ref. 3.1.5-4 J. Guidelines})$$

Where,

$p_w$  = Wall reinforcement ratio

$$= \frac{50.24}{100 \times 200} = 0.00251$$

$$\therefore \tau = \max \left\{ 0.00251 \times 400, \frac{25}{20} + 0.5 \times 0.00251 \times 400 \right\}$$

$$= \max \{ 1.00, 1.75 \}$$

$$= 1.75\text{N/mm}^2.$$

In this case, lower value  $1.5\text{N/mm}^2$  (capacity of wall as indicated in 1<sup>st</sup> level screening method) is used for the strength evaluation considering design of post installed anchor.

In X-direction,

$$\therefore \text{Shear strength of grid-1, } Q_{wi} = 1.5 \times 100 \times 6700 = 1005\text{kN}$$

$$\text{Shear strength of grid-11, } Q_w = 1.5 \times 100 \times 7900 = 1185\text{kN}$$

In Y-direction,

Shear strength of wall jacketing at grid- A, C, G and J

$$= 1.5 \times t \times l$$

$$= 1.5 \times 100 \times (7550 + 5750 + 4220 + 8200)$$

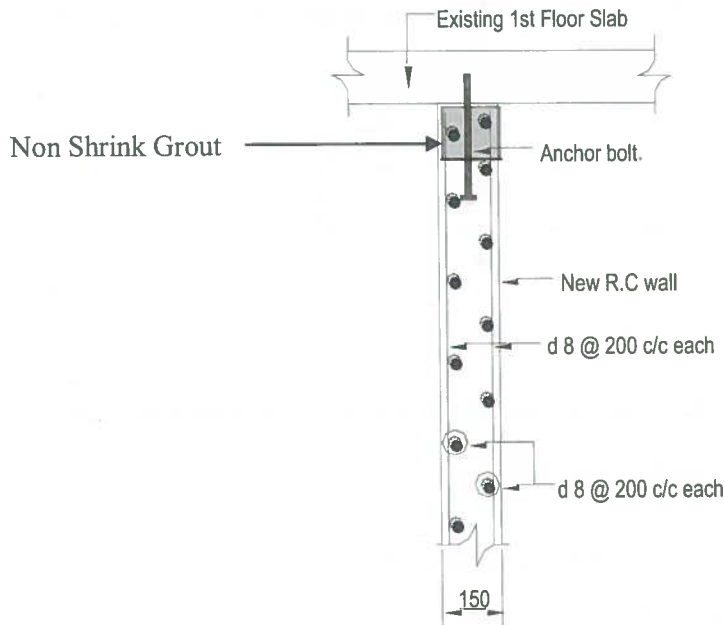
$$= 3858\text{kN}$$

**(c) Strength Calculation of RC Wall at Grid-4**

Length of RC wall = 3750mm, Thickness of wall = 150mm.

The RC wall will be provided to meet the shear demand at ground floor only. This new RC wall has a continuous footing at existing foundation level and will be anchored to 1<sup>st</sup> floor slab by post installed anchor. Since it is a very low height RC wall, so it will fail in shear mode instead of flexure mode.

Shear stress,  $\tau = \max \{P_{w \cdot w} \sigma_y, F_{cw}/20 + 0.5 P_{w \cdot w} \sigma_y\}$



**Figure 4.3.6 Section of RC Wall at Grid-4**

Where,

$P_w$  = Wall reinforcement ratio

$$= \frac{2 \times 50.24}{150 \times 200} = 0.00335$$

$F_{cw}$  = Compressive strength of concrete of RC wall

$$= 25 \text{ N/mm}^2.$$

$\sigma_{wy}$  = Yield strength of reinforcement of RC wall

$$= 400 \text{ N/mm}^2.$$

$$\text{So, } \tau = \max \left\{ 0.00335 \times 400, \frac{25}{20} + 0.5 \times 0.00335 \times 400 \right\}$$

$$= \max \{1.34, 1.92\} = 1.92 \text{ N/mm}^2.$$

In this case, lower value  $1.5 \text{ N/mm}^2$  is used for the strength evaluation of considering design of post installed anchor.

So, shear strength of RC wall =  $\tau \cdot l \cdot t$

$$= 1.5 \times 3750 \times 150 \times 2 \text{ nos.}$$

$$= 1687 \text{ kN.}$$

(d) Calculation for post installed anchor for RC jacketing over existing masonry wall at Grid-1 and newly inserted RC wall at Grid-4.

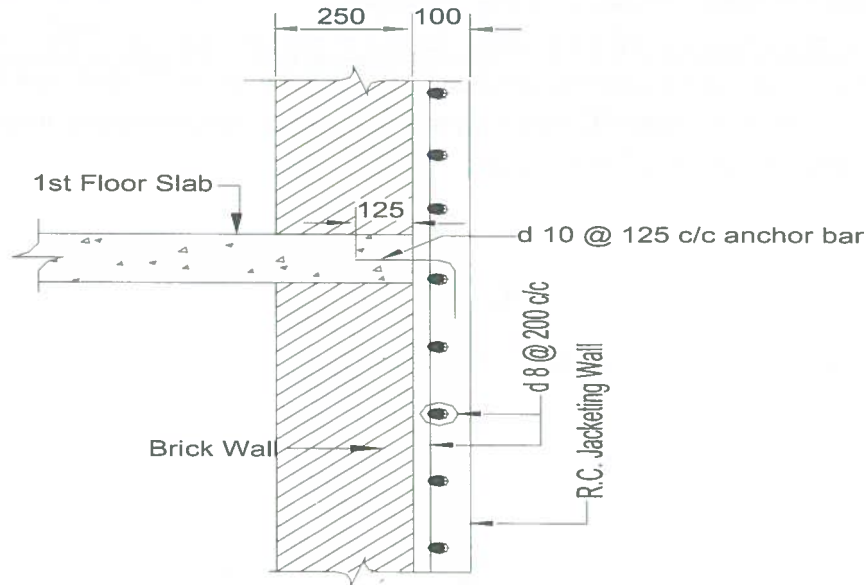


Figure 4.3.7 Section Showing Anchoring of Jacketed Wall with Slab

Shear capacity of bonded anchor is calculated as follows:

If anchor length  $l_e \geq 7d_a$

$$Q_a = \min [Q_{a1}, Q_{a2}] \quad (\text{J. Guidelines 3.9.4-7})$$

$$= \min [0.7\sigma_y \cdot s a_e, 0.4\sqrt{E_c \cdot \sigma_B} \cdot s a_e]$$

$$Q_{a1} = 0.7 \sigma_y \cdot s a_e \quad (\text{J. Guidelines 3.9.4-8}),$$

$$Q_{a2} = 0.4\sqrt{E_c \cdot \sigma_B} \cdot s a_e \quad (\text{J. Guidelines 3.9.4-9})$$

But  $\tau (= Q_a / s a_e)$  shall not be greater than  $294 \text{ N/mm}^2$ . 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.

Where,

$\sigma_y$  = Yield strength of bolt  
=  $400 \text{ N/mm}^2$ .

$s a_e$  = Cross sectional area of anchor.  
=  $78.5 \text{ mm}^2$  [For 10mm bar]

$E_c$  = Young  $I_s$  modulus of existing concrete.  
=  $13126 \text{ N/mm}^2$ .

$\sigma_B$  = Compressive strength of existing concrete  
=  $7.8 \text{ N/mm}^2$ .

$$\text{So, } Q_a = \min [0.7 \times 400 \times 78.5, 0.4\sqrt{13126 \times 7.8} \times 78.5]$$

$$= \min [21980, 10047] \text{ N}$$

$$= 10 \text{ kN. [Note that, } Q_a / s a_e \text{ shall be less than } 294 \text{ N/mm}^2]$$

Total shear strength of RC jacketing wall at grid-1

$$= 1005 \text{ kN [From see 3(b)].}$$

Assuming half of this shear force will be transferred from level-2 floor slab and another half is transferred from level-3 (roof slab) number require bolt =  $1005/10 \cong 101$

Total length of wall at grid-1 = 6700mm

$$\text{So, bolt spacing} = \frac{6700 \times 2}{101} = 133\text{mm}$$

Provide  $\phi$  10mm bolt @ 125mm.

Embedment length =  $11 \times$  bar diameter will be applied for low strength concrete,

For wall at Grid-4

Shear capacity of each bolt

$$\begin{aligned} Q_a &= 0.4 \sqrt{E_c \cdot \sigma_B} \cdot s \cdot a_e \\ &= 0.4 \sqrt{13126 \times 7.8} \times 113 \quad [\text{For } 12\text{mm } \phi \text{ bolt}] \\ &= 14.5\text{kN}. \end{aligned}$$

Strength contribution factor 0.65 is used, since ductility index 0.8 is considered. If ductility index not less than 1 is used, then full strength (1687kN) have to be considered for the anchor design.

$$\text{So, required number of bolt} = \frac{1687 \times 0.65}{14.5} = 76$$

$$\text{Bolt spacing} = \frac{7500}{76} = 99\text{mm, say } 100\text{mm}.$$

#### (e) Strength of Existing Brick Wall

Shear capacity of brick wall =  $0.2\text{N/mm}^2$ .

Cross sectional area of brick wall in X-direction =  $4,925,000\text{mm}^2$ .

So, shear strength of brick wall in X-direction

$$\begin{aligned} Q_b &= 0.2 \times t \times l \\ &= 0.2 \times 4,925,000 \\ &= 985 \text{ kN} \end{aligned}$$

Cross sectional area of brick wall in Y-direction =  $7,755,000\text{mm}^2$

So, shear strength of brick wall in Y-direction

$$\begin{aligned} &= 0.2 \times 7,755,000 \\ &= 1,551 \text{ kN}. \end{aligned}$$

#### (f) Strength Calculation of Steel Framed Brace at Grid-1

Length of steel framed brace = 3010mm

Height of steel framed brace = 2650mm

Length of diagonal member = 3834mm

Section of brace =  $\square 225 \times 125 \times 10$

Section property:

Area,  $A = 4550\text{mm}^2$ .

Moment of Inertia  $I_x = 3860 \times 10^4\text{mm}^4$

$$I_y = 616 \times 10^4\text{mm}^4$$

Radius of gyration,  $r_x = 92.1 \text{ mm}$

$$r_y = 36.8 \text{ mm}.$$

$$\text{Slenderness ratio, } \tau_x = \frac{l_z}{r_x} = \frac{3834}{92.1} = 41.6$$



$$\tau_y = \frac{l_y}{r_y} = \frac{3834/2}{36.8} = 52.1$$

$$\text{Limit slenderness ratio, } \Lambda = \sqrt{\left(\Lambda^2 \cdot E\right) / (0.6F)} \\ = 99$$

$$f_{cr} = \text{Limit compressive strength} \\ = \{1 - 0.4(\lambda / \Lambda)^2\} \cdot F \\ = \{1 - 0.4(52.1/99)^2\} \times 345 \\ = 306 \text{ N/mm}^2.$$

For tension side 20% reduction for bolt hole is assumed. Only main diagonal member will be considered for strength calculation. One braced frame will take load by compression and another will participate by tension. Inclination of this diagonal member is  $41.4^\circ$  with base.

$$\text{So, strength} = A_{s1} f_{cr} \cos\theta + A_{s2} f_y \cos\theta \\ = (4550 \times 306 + 4550 \times 0.8 \times 345) \times \cos 41.4 \\ = 1986 \text{ kN.}$$

#### (g) Calculation for Bolt Connection

The diagonal member will be connected by bolt with steel frame. If ordinary bolt (SS 400 equivalent) with M24 is used then strength of each bolt,  $Q_{bolt} = 120\text{kN}$ .

$$\text{So, total no. of bolt required} = \frac{A_s f_y}{Q_{bolt}} \\ = \frac{4550 \times 345}{120 \times 1000} = 13.1, \text{ say 14nos on each side.}$$

$$\text{If 20\% area is reduce for bolt hole then no. of bolt required} = \frac{0.8 \times 4550 \times 345}{120 \times 1000} = 10.5, \text{ say 12nos.}$$

If high strength bolt, F10T equivalent is used then  $Q_{bolt} = 199\text{kN}$ .

$$\text{and no. of bolt required} = \frac{4550 \times 345}{199 \times 1000} = 7.9, \text{ say 8nos.}$$

#### (h) Calculation for Headed Stud

Assuming double layer headed stud  $\phi 12\text{mm @}200\text{mm}$

$$\text{Total no. of headed stud at top of the frame} = \frac{3010}{200} \times 2 = 30\text{nos.}$$

Shear strength of each stud,  $q_{ds} = 0.64 \sigma_{max} a_s$

$$\sigma_{max} = \text{Tensile strength of stud} \\ = 400 \text{ N/mm}^2.$$

$$a_s = \text{Cross sectional area of stud} \\ = 113\text{mm}^2.$$

$$\therefore q_{ds} = 0.64 \times 400 \times 113 \\ = 28.9\text{kN.}$$

Total capacity of headed stud =  $30 \times 28.9 = 867\text{kN}$ .

For ductility index 0.8, effective strength factor of steel framed bracing is 0.5.

So, strength provided by each bracing

$$= 0.5 \times \frac{1986}{2}$$

$$= 496.5 \text{ kN} < 867 \text{ kN},$$

If ductility index of not less than 1.0 is used, then strength of 993kN shall be used for the stud design.

#### (i) Calculation for Post Installed Anchor

Assuming  $\phi 16\text{mm}$  @ 200 post installed anchor,

$$\begin{aligned} \text{Capacity of each anchor, } &= 0.4 \sqrt{E_c \cdot \sigma_c} \cdot a_e \\ &= 0.4 \sqrt{13126 \times 7.8} \cdot 201 \\ &= 25.7 \text{ kN.} \end{aligned}$$

$$\text{Total no. of bolt} = \frac{3010}{200} - 1 = 14 \text{ nos.}$$

$$\text{Capacity} = 14 \times 25.7 = 359.8 \text{ kN} < 496.5 \text{ kN.}$$

So, increasing the number of bolt @ 100mm spacing capacity =  $29 \times 25.7 = 745.3 \text{ kN} > 496.5 \text{ kN}$

Use  $\phi 16\text{mm}$ @100mm or  $\phi 20\text{mm}$ @150mm at upper beam only and other portion  $\phi 16\text{mm}$ @200mm. Embedment length will be  $11 \times$  bar diameter for low strength concrete.

Note: Strength contribution factor 0.5 (at ductility index 0.8) is used. If ductility index of not less than 1.0 is used. Then strength of 993kN shall be used for the stud design.

#### (j) Calculation for Ladder Type Rebar

According to the standard, minimum cross section area required for ladder rebar = 4%

If  $\phi 12\text{mm}$  @ 200 ladder is used and thickness of non-shrink grout is 125nos

$$\text{then ratio of rebar} = \frac{113 \times 100}{125 \times 200} = 0.45\% > 0.4\%$$

#### (4) Seismic Index of Structure $I_{so}$ and Judgment

For ductility index 0.8, strength contribution of different member is as follows-

Brick wall = 1.0

Jacketing wall = 0.65

RC shear wall = 0.65

Jacketed column = 0.5

Steel from bracing = 0.5

Total shear strength in X-direction at ground floor

$$\begin{aligned} Q &= Q_b + \alpha_{jw} \cdot Q_{jw} + \alpha_{sw} \cdot Q_{sw} + \alpha_c \cdot Q_c \\ &= 985 + 0.65 \times (1005 + 1185) + 0.65 \times 1687 + 0.5 \times 12 \times 84.8 \\ &= 4013.8 \text{ kN.} \end{aligned}$$

Total weight at ground floor = Existing weight + Additional weight for retrofitting element

$$= 4782 + 645$$

$$= 5427 \text{ kN.}$$

$$\therefore \text{Strength index, } C = \frac{Q}{W}$$

$$= \frac{4013.8}{5427} = 0.74$$

So, seismic index of structure in X-direction at ground floor

$$I_s = E_o \times S_D \times T$$

$$= \frac{n+1}{n+i} \times C \times F \times S_D \times T$$

$$= 1 \times 0.74 \times 0.8 \times 1.0 \times 0.95$$

$$= 0.56 > I_{so} (0.45)$$

Total shear strength in Y-direction at ground floor,

$$Q = Q_\sigma + \alpha_{jw} \cdot Q_{jw} + \alpha_{sw} \cdot Q_{sw} + \alpha_c \cdot Q_c + \sigma_{br} \cdot Q_{br}$$

$$= 1551 + 0.65 \times 3858 + 0 + 0.5 \times 12 \times 84.8 + 0.5 \times 1986$$

$$= 5560.5 \text{ kN.}$$

$$\text{Strength index, } C = \frac{Q}{W}$$

$$= \frac{5560.5}{5427} = 1.025$$

So, seismic index of structure in Y-direction at ground floor

$$I_s = E_o \times S_D \times \tau$$

$$= \frac{n+1}{n+i} \times C \times F \times S_D \times \tau$$

$$= 1 \times 1.025 \times 0.8 \times 1.0 \times 0.95$$

$$= 0.78 > I_{so} (0.45)$$

**Table 4.3.2 Result after Retrofit**

Storey	X direction				Y direction			
	<i>C</i>	<i>E</i>	<i>I<sub>s</sub></i>	<i>I<sub>so</sub></i>	<i>C</i>	<i>E</i>	<i>I<sub>s</sub></i>	<i>I<sub>so</sub></i>
2	0.97	0.58	0.55	> 0.45	1.05	0.63	0.60	> 0.45
1	0.75	0.60	0.57	> 0.45	1.03	0.83	0.78	> 0.45

Proposed  $I_{so} = 0.45$  (2<sup>nd</sup> level screening, BNBC 2015, soil type SC) (Soil type SD,  $I_{so} = 0.54$ )

(5) Retrofit Plan

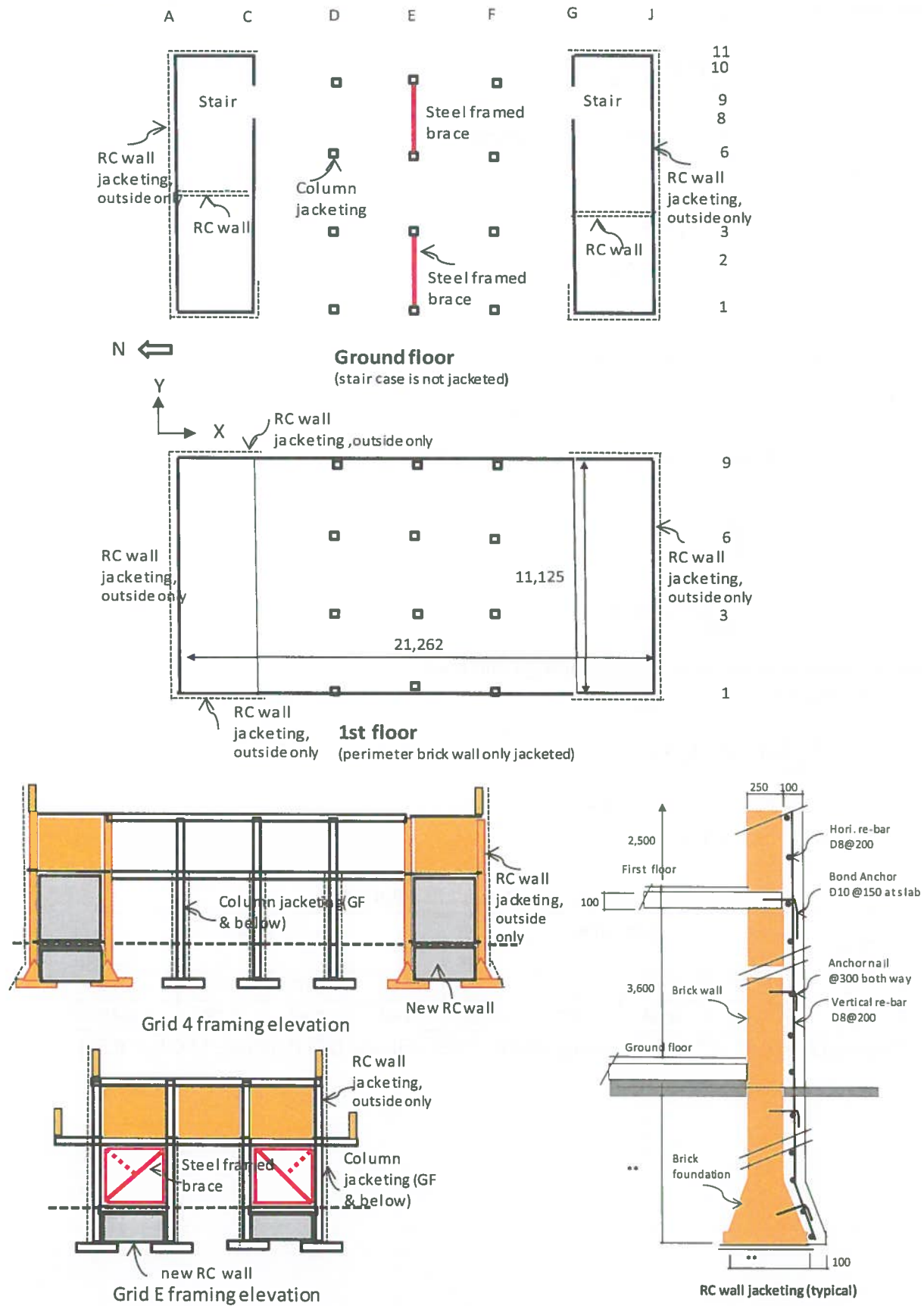
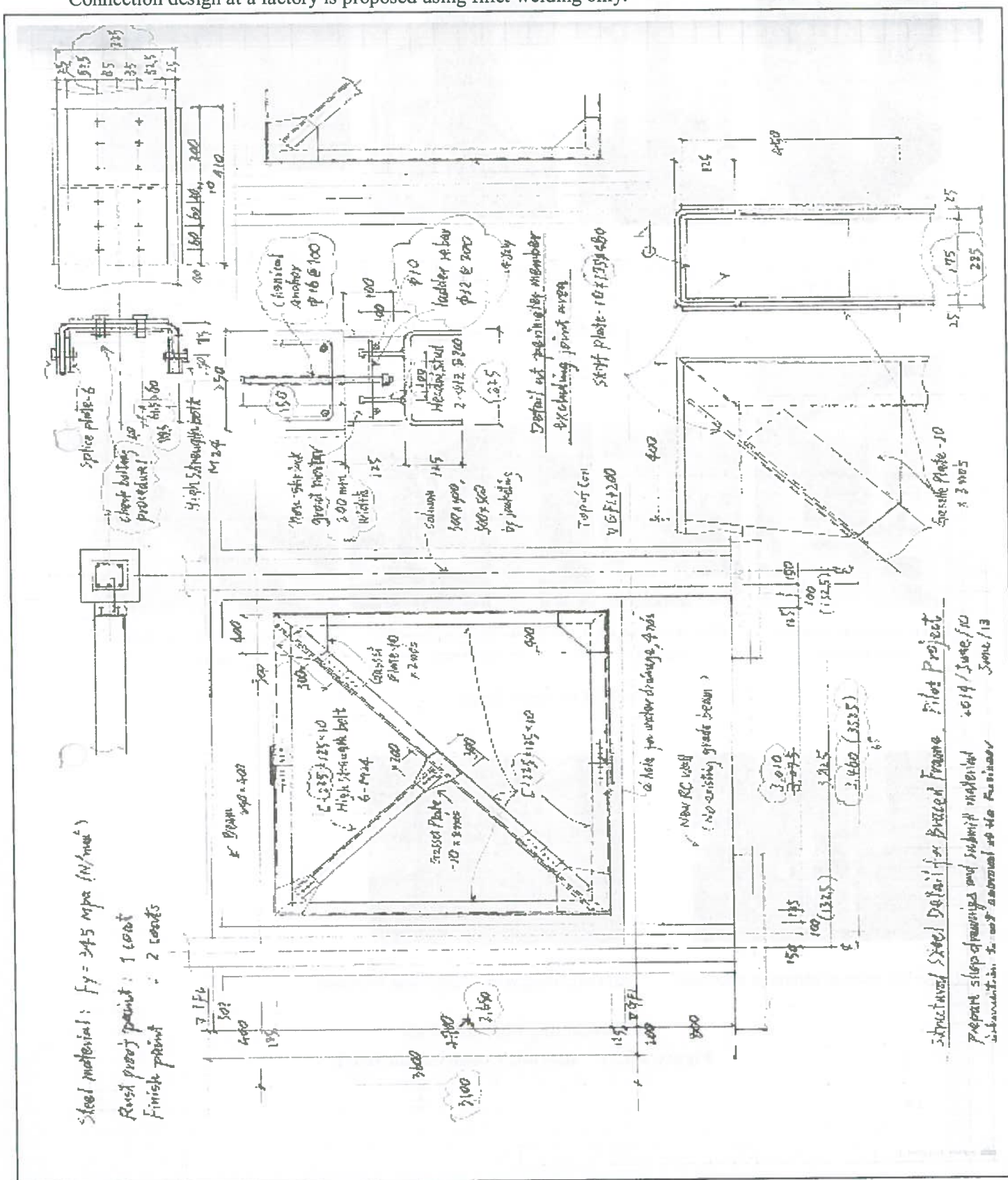


Figure 4.3.9 Proposed Retrofit Plan



**Figure 4.3.10 Detail of Structural Framed Brace**

**(7) Photos during Construction**



(1) Chemical anchor and re-bar work



(2) Formwork and concreting work



(3) Non-shrink grout mortar work at top of column, 1<sup>st</sup> story

**(1) RC Column Jacketing**



(1) Chemical anchor at beam and column



(2) Chain block for erection



(3) Erection work of steel framed brace



(4) Connection of steel framed brace

**(2) Steel Framed Brace**



(1) Re-bar work of perimeter brick wall



(2) Concreting work of perimeter brick wall

**(3) Jacketing of Brick Wall**

**Figure 4.3.11 Retrofit Construction Work**

**(8) Shear strength of brick mortar joint by in-situ shear test, by NSET of CDMP project**

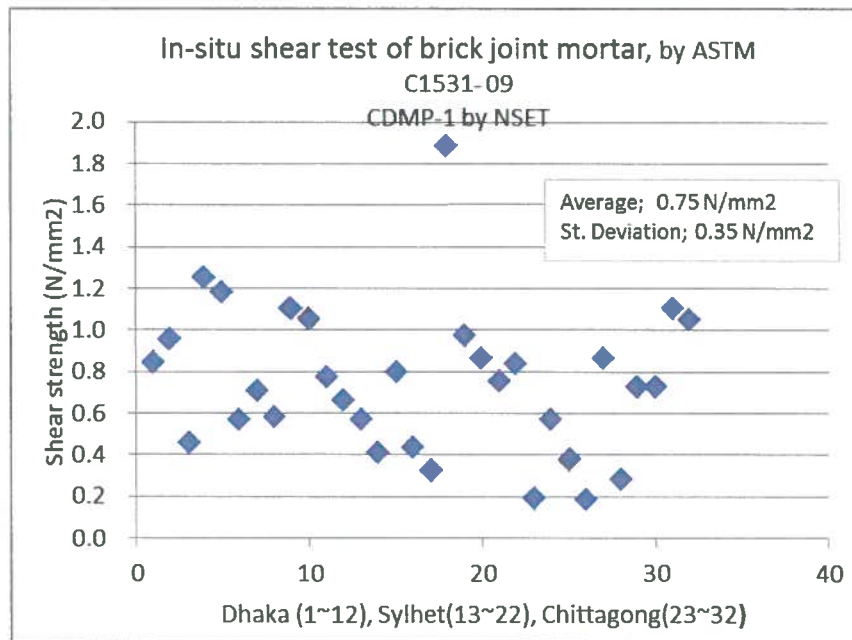
Existing data of shear strength test of brick joint mortar at site is shown as reference information.

Average;  $0.75 \text{ N/mm}^2$

Standard deviation;  $0.35 \text{ N/mm}^2$

Shear strength for planning purpose:  $\tau = 0.4 \text{ N/mm}^2 (= 0.75 - 0.35)$

Factor by vertical stress is excluded.



Result of in-situ test is adjusted by  $\tau - (\text{minus}) \mu \cdot \sigma$  ( $\tau$ ; shear stress by test,  $\mu$ ; coefficient 1.0,  $\sigma$ ; vertical stress  $\text{N/mm}^2$ , by FEMA 356, 7.3.2.6)

**Figure 4.3.12 In-situ Shear Test of Brick Joint Mortar**  
(Courtesy by NSET of CDMP project)



# SUPPLEMENT

# SUPPLEMENT

## SUPPLEMENT A

### SUPPLEMENT A1. PROPOSED SEISMIC DEMAND INDEX OF STRUCTURE, $I_{so}$

#### 1.1 GENERAL

##### (1) Purpose

An investigation of proposed Seismic demand index of structure,  $I_{so}$  based on BNBC 2015 was done. Proposed  $I_{so}$  for soil type SC (hard soil) is 0.30, which is the peak value and is 80% of elastic response shear coefficient 0.38. Proposed  $I_{so}$  for soil type SD (soft soil) is 0.36, which is the peak value and is 80% of elastic response shear coefficient 0.45.

##### (2) Method

Time-history response analysis was done based on supposed restoring force characteristics and artificial earthquake waves. Degrading tri-linear model as restoring force characteristics was used, and parameter is shear strength and storey deflection angle at yield.

The response of shear force coefficient and storey deflection angle was studied. Proposed  $I_{so}$  was investigated using ductile 1 storey frame through case 1 to case 6. Brittle frame of 1 storey frame was studied through case 7 and 8 for comparison purpose. 3 storey brittle frames were used for case 9 and 10.

##### (3) Time-History Response Analysis

**Structure Vibration Model:** RC frame with 1 lumped mass shear type model for case 1 to case 8. RC frame with 3 lumped mass shear type models for case 9 and case 10. Response at peak area of spectrum was assumed providing short period of 1 lumped mass.

**Restoring Force Characteristics:** Degrading tri-linear type model (type 4),  $Q_c = 0.4 Q_y$  is supposed.  $Q_y$ : Yield shears force.  $Q_c$ : Shear force when crack occurs. Initial stiffness is supposed as two times of yield stiffness.

Case 1 to Case 6: Storey deflection angle at yield is supposed as 1/150, yield strength is changed for 3 types for SC and SD respectively. Refer to Figure SA 1.1.

Case 7 to Case 10: Storey deflection angle at yield is supposed as 1/250, Yield strength is changed for SC and SD respectively. Refer to Figure SA 1.2.

**Input earthquake waves:** Artificial wave corresponding to response spectrum of soil type SC (hard) and SD (soft) 3 wave each as shown. Max. Acceleration is not  $\alpha = 0.133g$ . Refer to the attachment.

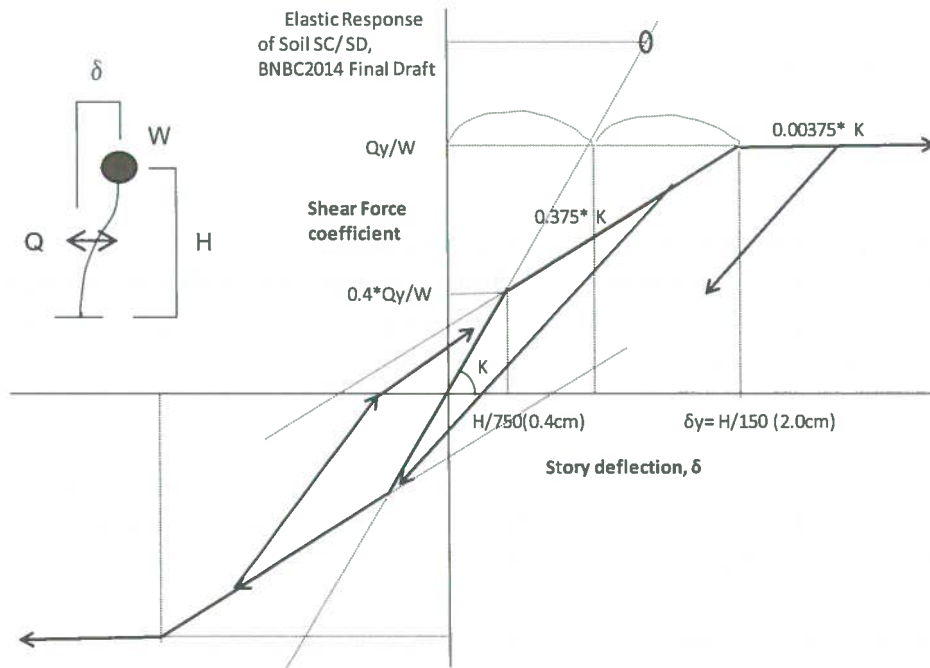
**Damping constant:** Stiffness proportional type 5% was supposed. Building data: Building weight  $W = 5,040kN$ , storey height  $H = 300cm$ .

Case 1 to case 3 for soil type SC and case 4 to case 6 for soil type SD, supposing ductile RC frames.

Case 7 & case 8: 1 lumped mass system, restoring force characteristics is degrading tri-linear type. Yield shear force coefficient is 0.30 for soil type SC, and 0.36 for soil type SD. Storey deflection angle at yield is assumed as 1/250. Ductility index,  $F$  is supposed as 1.0. Target response ductility factor is less than 1.0.

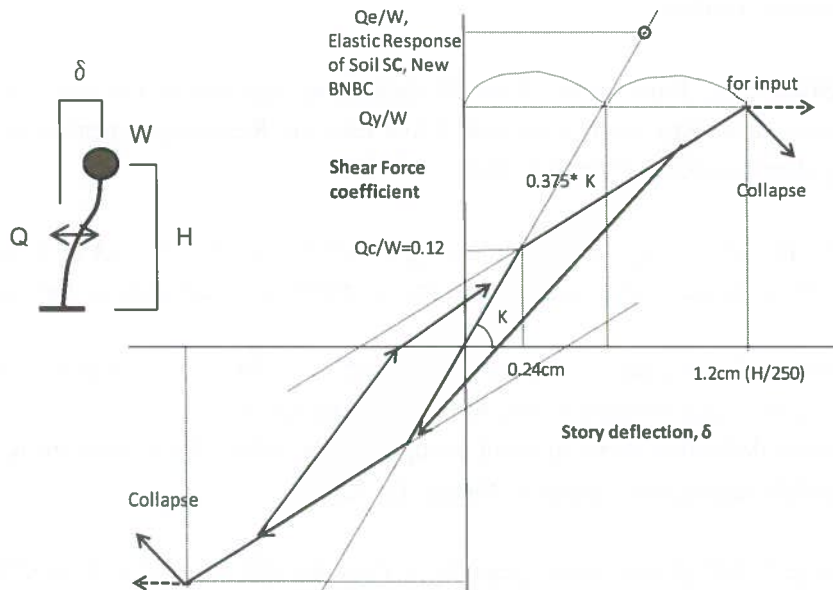
Case 9 and case 10 is 3 lumped mass systems, and other condition is same to those of case 7 and 8.

The result of case 1 to case 8 is introduced first, and result of case 9 and case 10 is shown later.



**Figure SA 1.1 Restoring Force Characteristics (1)**

A model supposing storey deflection angle 1/150 when the frame yield.



**Figure SA 1.2 Restoring Force Characteristics (2),**

A model supposing storey deflection angle 1/250 when the frame yield.

## 1.2 RESULTS OF CASE 1 TO CASE 8

### (1) Elastic response

Stiffness proportional type damping constant 5% was applied. In case of waves of soil type SC, the result of shear force coefficient distribute in the range of plus minus 10% from the peak design value 0.38. In case of waves of soil type SD, the result of shear force coefficient distribute in the range of plus minus 10% from the peak design value 0.45. It is reasonable to use damping constant 5%.

**(2) Response of D-trilinear**

All results excluding case 3 are evaluated as the response within the peak range of response spectrum. Table S1.1 shows the results of case 1~ case 6. Table SA1.2 shows the results of case 7 and case 8.

**Table SA 1.1 Results of Response Analysis (Case 1~ case 6)**

		Soil type SC			Soil type SD		
		Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Assumed restoring force characteristics							
R at yield		$R=1/150$			$R=1/150$		
Ini. Stiffness(kN/cm)		1,189	1,008	862	1,426	1,210	1,038
Yield shear force co. $Q_y/W (=I_{so}/F)$		0.236(=0.30/1.27)	0.200(=0.30/1.5)	0.171(=0.30/1.75)	0.283(=0.36/1.27)	0.240(=0.36/1.5)	0.206(=0.36/1.75)
Nat. period (sec.)		0.410	0.448	0.485	0.377	0.409	0.442
Elastic response shear force co. $Q_e/W$ , 5% damping ( $R=\delta/h$ )	1st	0.350(0.00493)	0.413(0.00687)	0.374(0.00729)	0.449(0.00529)	0.414(0.00574)	0.498(0.00806)
	2nd	0.387(0.00546)	0.390(0.00650)	0.374(0.00729)	0.461(0.00542)	0.462(0.00641)	0.462(0.00747)
	3rd	0.360(0.00509)	0.387(0.00645)	0.379(0.00737)	0.411(0.00483)	0.418(0.00579)	0.454(0.00735)
Target response ductility factor, ( $R$ )		1.00( $R=1/150$ )	1.21( $R=1/124$ )	1.50( $R=1/100$ )	1.00( $R=1/150$ )	1.21( $R=1/124$ )	1.50( $R=1/100$ )
Degrading tri-linear, stiffness proportional damping constant 5%							
Response shears co. $Q/W, (Q/Q_e)$	1st	0.236 (0.674 )	0.201 (0.487)	0.171 (0.457)	0.284(0.633)	0.242 (0.585 )	0.208 (0.493)
	2nd	0.224 (0.579 )	0.200 (0.513 )	0.171 (0.457)	0.283 (0.614)	0.241 (0.522 )	0.207 (0.416)
	3rd	0.220 (0.611 )	0.200 (0.517)	0.171 (0.451 )	0.276 (0.672 )	0.240 (0.574)	0.207 (0.456)
ductility factor, $\mu$ ( $\delta/\delta_y$ )	1st	1.151	1.281	1.191	1.212	1.531	1.636
	2nd	0.932	1.227	1.195	1.021	1.242	1.359
	3rd	0.914	1.042	1.091	0.967	1.082	1.264
Average		1.00 = < 1.00	1.18 < 1.21	1.16 (ref.)	1.07 > 1.0	1.28 > 1.21	1.42 < 1.50

Note: R: storey deflection angle ( $\delta/h$ )

Case 7 & Case 8: 1 lumped mass system, restoring force characteristics is degrading tri-linear type. Yield shear force coefficient is 0.30 for soil type SC, and 0.36 for soil type SD. Storey deflection angle at yield is assumed as 1/250. Ductility index,  $F$  is supposed as 1.0. Target response ductility factor is less than 1.0.

**Note: Time-history response analysis**

Time-history response analysis is a numerical analysis and is a useful tool to get the dynamic response of buildings by earthquakes. Input earthquake waves are expressed by acceleration data at the ground level. Lumped mass type shear model was used at here. Building static behavior is expressed by restoring force characteristics, and is derived from static structural evaluation. In case of RC frame structure, Degrading Tri-linear model is supposed generally. This is expressed by tri-linear model and the stiffness is degraded gradually by repeated loading. Damping constant is also supposed, which affect the response. Output is such as, maximum storey deflection (angle), maximum plastic ratio, maximum storey shear force (coefficient), and others.

**Table SA 1.2 Results of Response Analysis (Case 7 & Case 8)**

	Soil type SC (3 waves)			Soil type SD (3 waves)		
	Case 7			Case 8		
Assumed restoring force characteristics						
$R(=\delta/h)$ at yield	1/250			1/250		
Ini. Stiffness (kN/cm)	2,520			3,023		
Yield shear force co. $Q_y / W (=I_{so} / F)$	0.30 (=0.30/ 1.0 )			0.36 (=0.36/ 1.0 )		
Target response ductility factor, $(R=\delta/h)$	<1.0 (<R=1/250)			<1.0 (<R=1/250)		
Natural period (sec.)	0.283			0.259		
Elastic response shear force co. $Q_e / W, (R)$	1st	2nd	3rd	1st	2nd	3rd
	0.353(0.002353)	0.384(0.00256)	0.398(0.00265)	0.459(0.00255)	0.474(0.00264)	0.494(0.00274)
Degrading tri-linear, stiffness proportional damping 5%						
Response shear co. $Q/W, (Q/Q_e)$	0.246 (0.697 )	0.245 (0.638 )	0.229 (0.575 )	0.325(0.708)	0.272 (0.574)	0.286 (0.579 )
Ductility factor, $\mu$ $(\delta/\delta_y), \mu$	0.760	0.755	0.685	0.871	0.676	0.727
Average	0.733<1.0			0.758 <1.0		

**(3) Strength index and ductility index (C-F) relation, soil type SC**

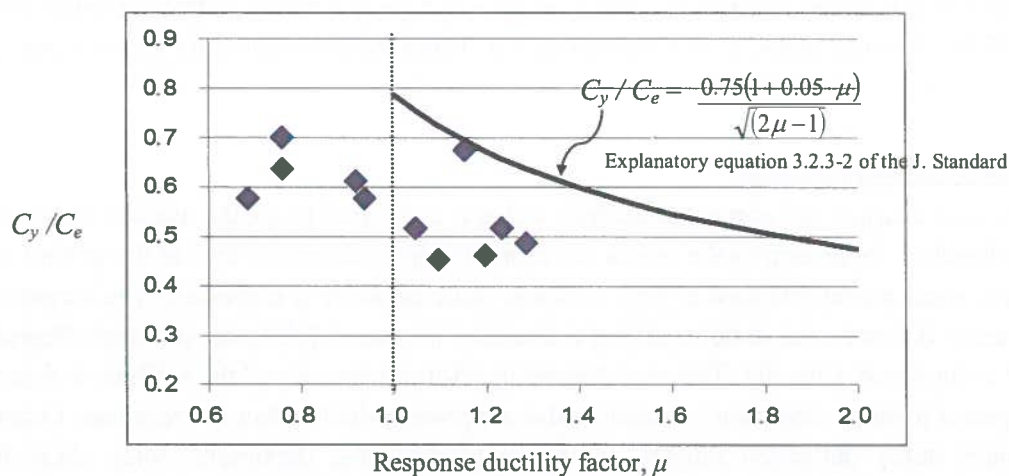
Yield shear force coefficient and response ductility ratio ( $C_y/C_e - \mu$ ) is shown. 3 waves x 4 models, stiffness proportional type damping constant 5%.

$$C_y/C_e = \frac{0.75(1 + 0.05 \cdot \mu)}{\sqrt{(2\mu - 1)}}$$

Commentary eq. 3.2.3-2 of the J. Standard

$C_y/C_e$  = Response (yield) storey shear force coefficient/ elastic response shear force coefficient.

Commentary equation 3.2.3-2 of the J. Standard, which is the original equation (16) of the Standard, is the envelope of the response of soil type SC.

**Figure SA 1.3 Yield Shear Force Coefficient and Response Ductility Ratio ( $C_y/C_e - \mu$ ), Soil Type SC**

### Strength Index and Ductility Index ( $C - F$ ) Relation

Conversion from response ductility factor  $\mu$  to ductility index  $F$  is as follows.

$$F = \frac{\sqrt{(2\mu - 1)}}{0.75(1 + 0.05\mu)} \quad \text{Commentary eq. 3.2.3-3 of the J. Standard}$$

Equation (15) of the Standard is applied in case that  $\mu$  is less than 1.0.

Case 7,  $\mu = 1 (= 1/250)$  and  $\mu$  is less than 1.0,  $F = 0.8 + \frac{0.2(0.004\mu - 0.002)}{0.002}$  was used.

Some results exceed  $C - F = 0.304$  relation. It is slightly in the safe side (overestimated) that proposed value of 0.304 compared with average value of the response.

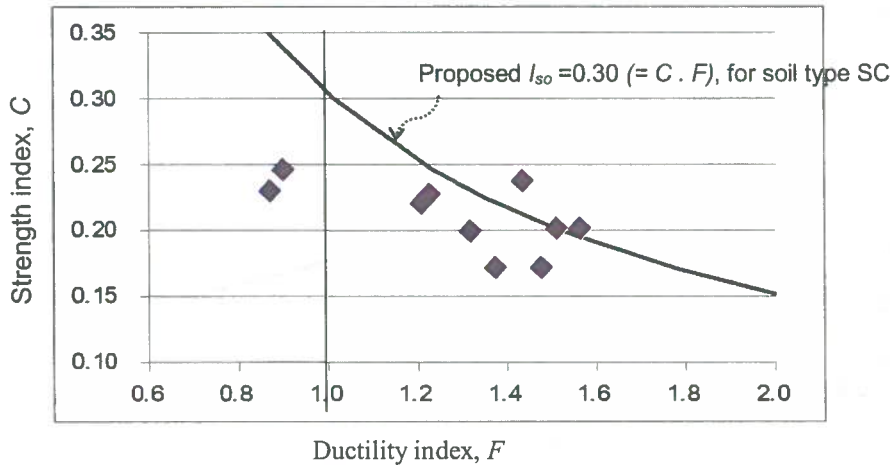


Figure SA 1.4 Strength Index and Ductility Index ( $C-F$ ) Relation, Soil Type SC

### (4) Strength Index and Ductility Index ( $C-F$ ), Soil Type SD

Yield shear force coefficient and response ductility ratio ( $C_y/C_e - \mu$ ). 3 waves x 4 models, stiffness proportional type damping constant 5%.

$$C_y/C_e = \frac{0.75(1 + 0.05 \cdot \mu)}{\sqrt{(2\mu - 1)}} \quad \text{Commentary eq. 3.2.3-2 of the Standard}$$

$C_y/C_e$  = Response (yield) storey shear force coefficient/ elastic response shear force coefficient  
Envelope curve of eq. 3.2.3-2 of the Standard almost covers the response of soil type SD.

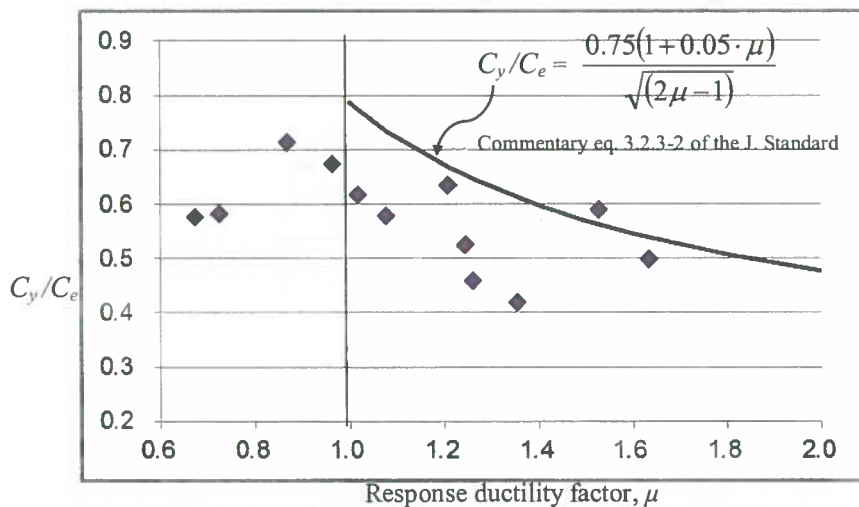


Figure SA 1.5 Yield Shear Force Coefficient and Response Ductility Ratio ( $C_y/C_e - \mu$ ), Soil Type SD

### Strength Index and Ductility Index ( $C-F$ ) Relation

Conversion from response ductility factor  $\mu$  to ductility index  $F$  is as follows.

$$F = \frac{\sqrt{(2\mu - 1)}}{0.75(1 + 0.05\mu)} \quad \text{Commentary eq. 3.2.3-3 of the J. Standard}$$

Equation (15) of the Standard is applied in case that  $\mu$  is less than 1.0.

Case 8,  $\mu = 1 (= 1/250)$  and  $\mu$  is less than 1.0,  $F = 0.8 + \frac{0.2(\mu - 0.002)}{0.002}$  was used.

5 cases out of 12 cases exceed the curve of  $I_{so} = 0.36$ , and proposed  $I_{so} = 0.36$  will be average of response in case of soil type SD.

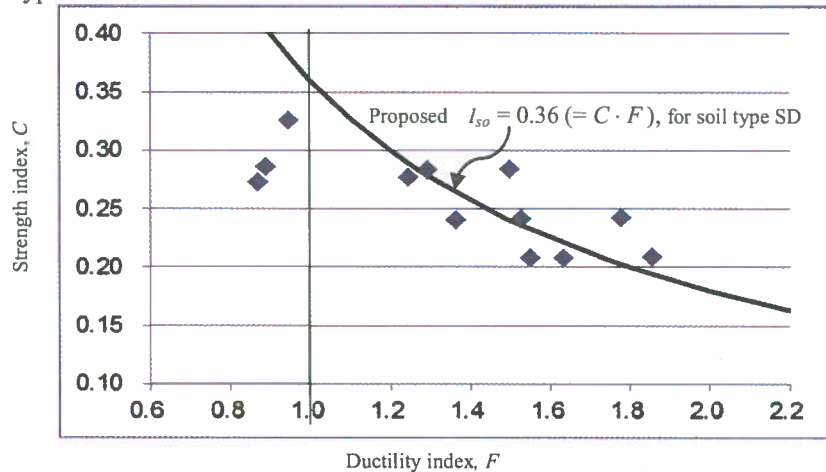


Figure SA 1.6 Strength Index and Ductility Index ( $C-F$ ), Soil Type SD

### (5) Response Shear Force Coefficient and Storey Deflection Angle Relation

Relation of shear force coefficient and storey deflection angle by elastic response and degrading tri-linear are shown as follows. Damping constant of stiffness proportional type 5% is used.

**Soil type SC** (hard soil, suffix e of each case shows elastic response, red color circle (○) shows target allowable response). Response of case 3 seems not the response of the peak of response spectrum.

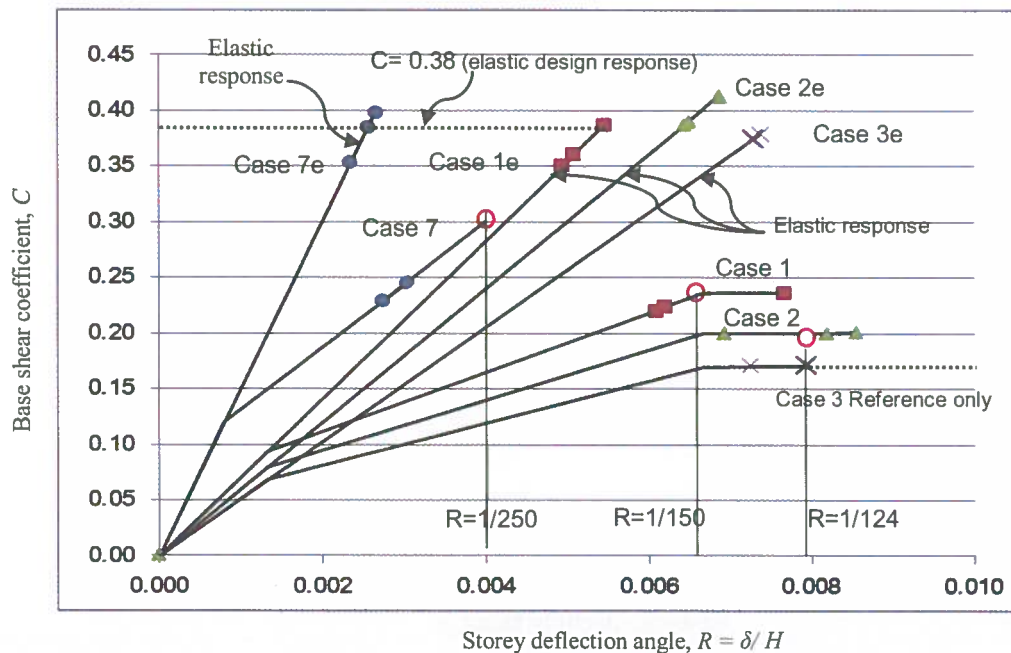
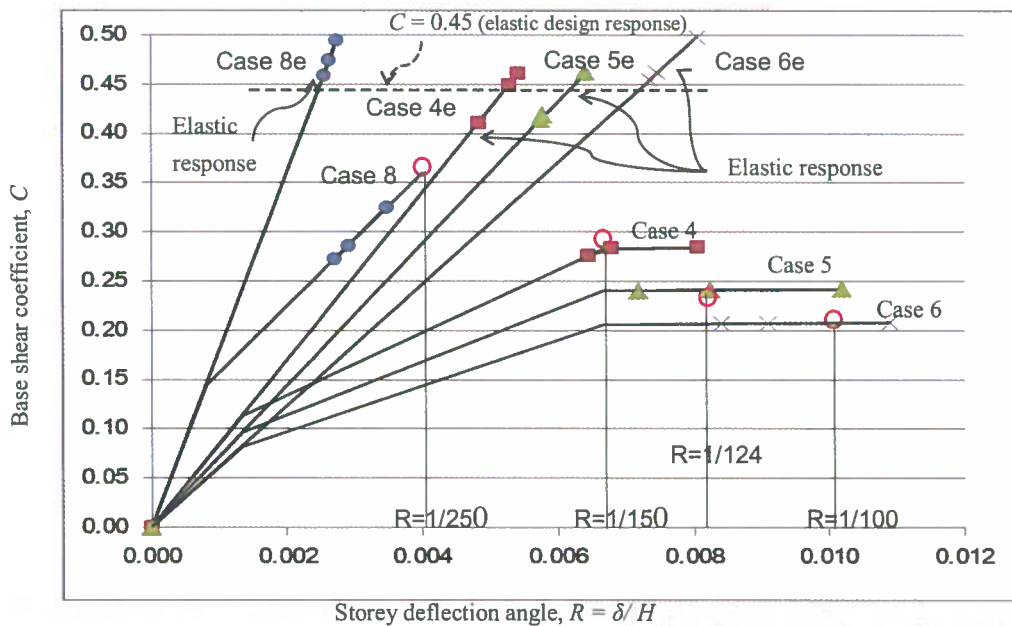


Figure SA 1.7 Response Shear Force Coefficient and Storey Deflection Angle, Soil SC

**Soil type SD** (soft soil, suffix e of each case shows elastic response, red color circle (○) shows target allowable response)



**Figure SA 1.8 Response Shear Force Coefficient and Storey Deflection Angle, Soil SD**

#### (6) Summary

1. Time history response analysis was done using artificial waves, to evaluate the strength index (response shear force coefficient) and storey deflection angle.
2. Proposed  $I_{so} = 0.30$ , 80% of peak value of elastic response shear force coefficient of soil type SC, and proposed  $I_{so} = 0.36$ , 80% of peak value of elastic response shear force coefficient of soil type SD will be reasonable.
3. This reduction has been proposed incorporating the effect of energy absorption (hysteresis) by crack occurrence for Case 7 and Case 8, and the energy absorption (hysteresis) by crack occurrence and yield for Case 1 to Case 6.
4. Damping constant of stiffness proportional type 5% was supposed. If tangential stiffness proportional type 5% is assumed, the response was increased approximately 20% to 40%. This will be an issue to investigate further. If this tangential stiffness type damping is reasonable, the value of elastic response (without any reduction), 0.38 for soil type SC and 0.45 for soil type SD might be used as  $I_{so}$ . This will be an issue in future.

### 1.3 RESULTS OF CASE 9 AND CASE 10

#### (1) Restoring Force Characteristics

Restoring force characteristics of degrading tri-linear type is assumed, incorporating crack occurrence, and no-ductility frame, and with yield shear force  $0.3 (= \frac{Q_y}{W})$ . Response up to yield is supposed, which is degrading bi-linear. Negative slope is supposed after the yield point but zero stiffness was provided for the limitation of the software. Storey deflection angle at yield is assumed 1/200 and 1/250 of storey height 300cm (2 cases). 3 storey lumped mass shear model is used. Tangential stiffness proportional type damping constant is used.

**Table SA 1.3 Structural Input Data of a Sample Building for Time-History Response Analysis**

	Weight(kN)	Total (kN)	Initial Stiffness (kN/cm)	Shear force at Crack Occurrence (kN)	Stiffness Degrading ratio	Yield strength (Q <sub>y</sub> , kN)
3	4030	4030	3133 (3917)	940	0.375	2350
2	5040	9070	4986 (6233)	1496	0.375	3740
1	5040	14110	5640 (7050)	1692	0.375	4234

**Natural period** (Two different stiffness or natural period is considered).

(1/200), T<sub>1</sub> = 0.426sec. T<sub>2</sub> = 0.176sec. T<sub>3</sub> = 0.117sec.

(1/250), T<sub>1</sub> = 0.381sec. T<sub>2</sub> = 0.157sec. T<sub>3</sub> = 0.104sec.

**BNBC 2015, Fundamental building period, T**

$$T = C_t (h_n)^m \quad (6.2.38) \text{ BNBC 2015}$$

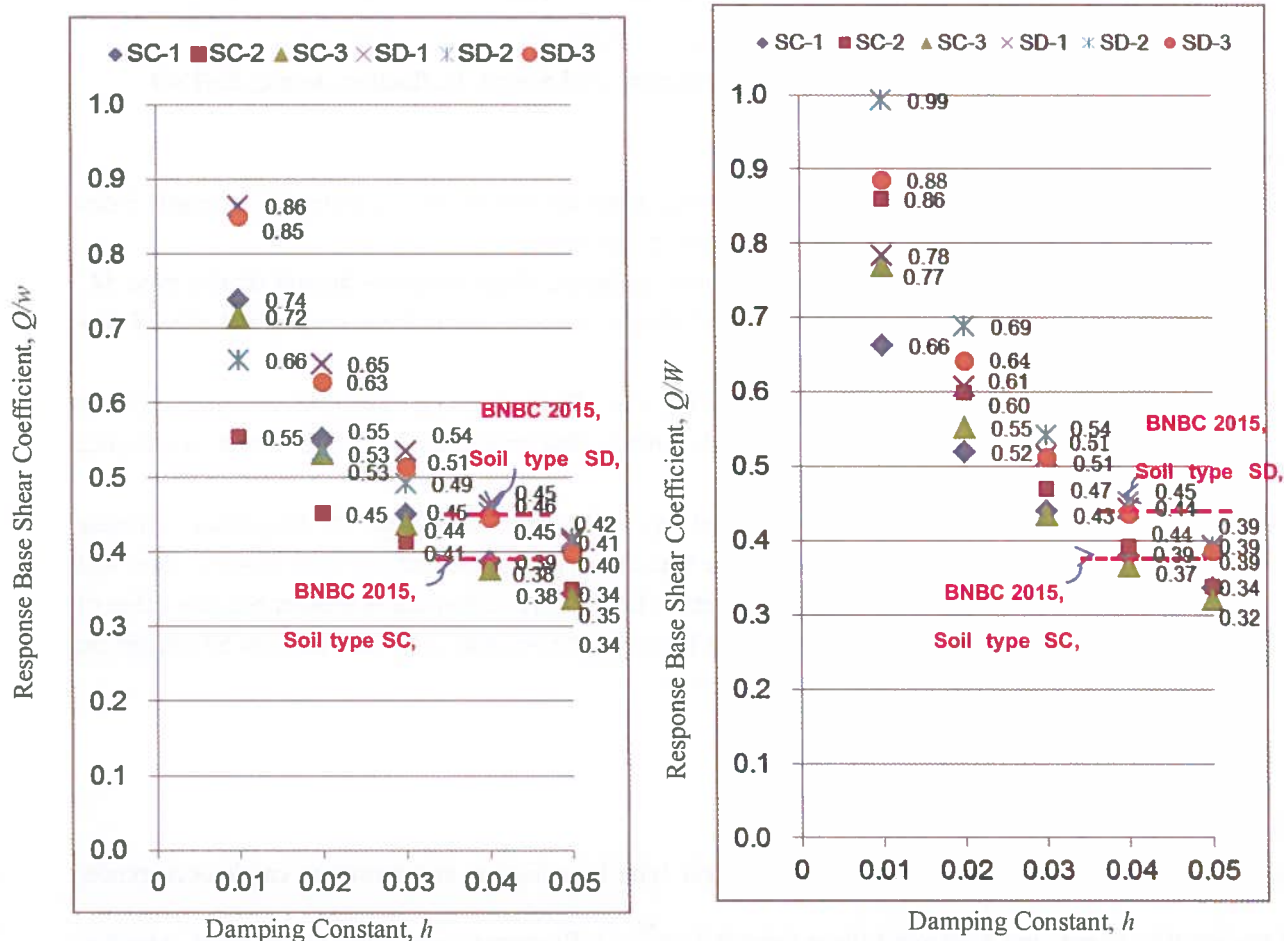
Concrete moment-resisting frame,  $C_t = 0.0466$ ,  $m = 0.9$ , 3 storey ( $h_n = 9\text{m}$  height),  $T = 0.337\text{sec}$ .

**(2) Elastic Response and Damping Constant,**

**Soil type SC and SD of BNBC 2015, example; 3 storey RC building with 2 different stiffness**

T<sub>1</sub> = 0.42sec.

T = 0.38sec.



**Figure SA 1.9 Elastic Response with Different Damping Constant**

Zone 2;  $Z=0.2$

Peak value of acceleration response spectrum, in case of elastic response

Soil type SC;  $S_a = \frac{2}{3} \frac{ZIC_s}{R} = \frac{2}{3} \times 0.2 \times 1 \times \frac{2.875}{1.0} = 0.383$  ( $R=1.0$ , Elastic response, in case of damping constant 5%)

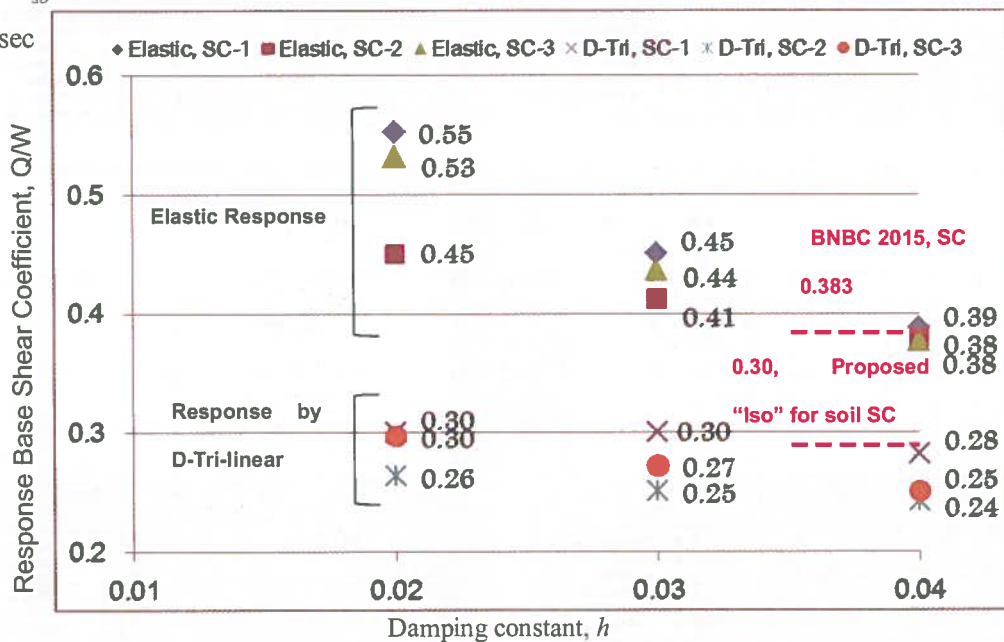
Soil type SD;  $S_a = \frac{2}{3} \frac{ZIC_s}{R} = \frac{2}{3} \times 0.2 \times 1 \times \frac{3.375}{1.0} = 0.45$  ( $R=1.0$ , Elastic response, in case of damping constant 5%)

### (3) Response Base Shear Coefficient by Degrading Tri-Linear

Soil type SC

Comparison between the elastic response and D-tri-linear response is shown in Figure SA 1.10. In case of model  $T_1=0.42$ sec. and  $T_1=0.38$ sec. Response shear force coefficient is lower than 0.30, which is proposed " $I_{so}$ ".

$T_1=0.42$ sec



$T_1=0.38$ sec.

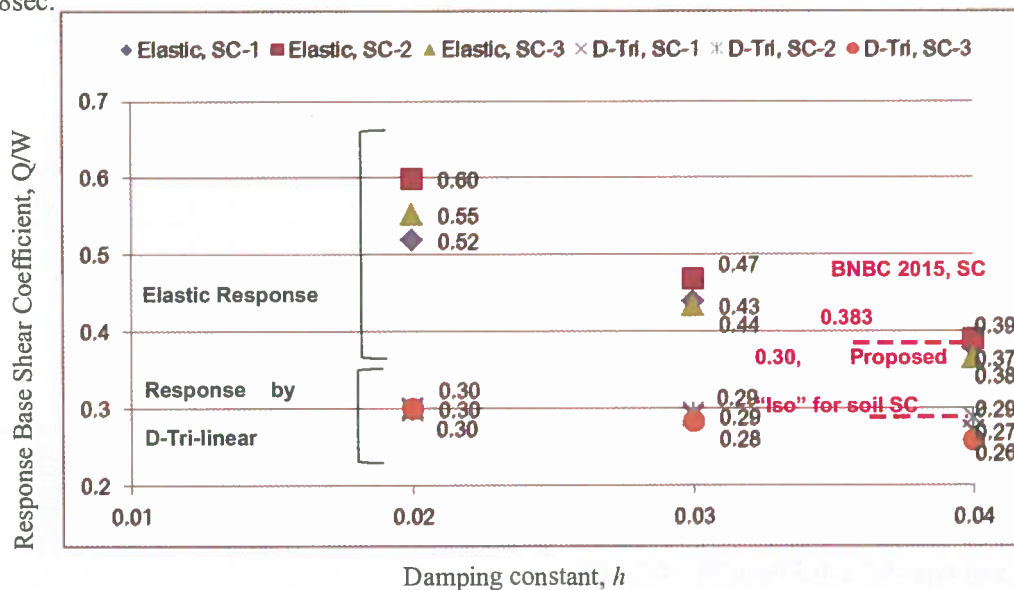


Figure SA 1.10 Comparison between Elastic Response and D-Tri-Linear Response

(4) Result of Response of case 9 and 10, 3 storey,  $\delta/h = 1/200$ ,  $T_1 = 0.426\text{sec}$ .  $T_2 = 0.176\text{sec}$ .  $T_3 = 0.117\text{sec}$ . Damping  $C =$  Tangential stiffness 4%, BNBC 2015 Soil type SC (3 waves) and SD (3 waves)

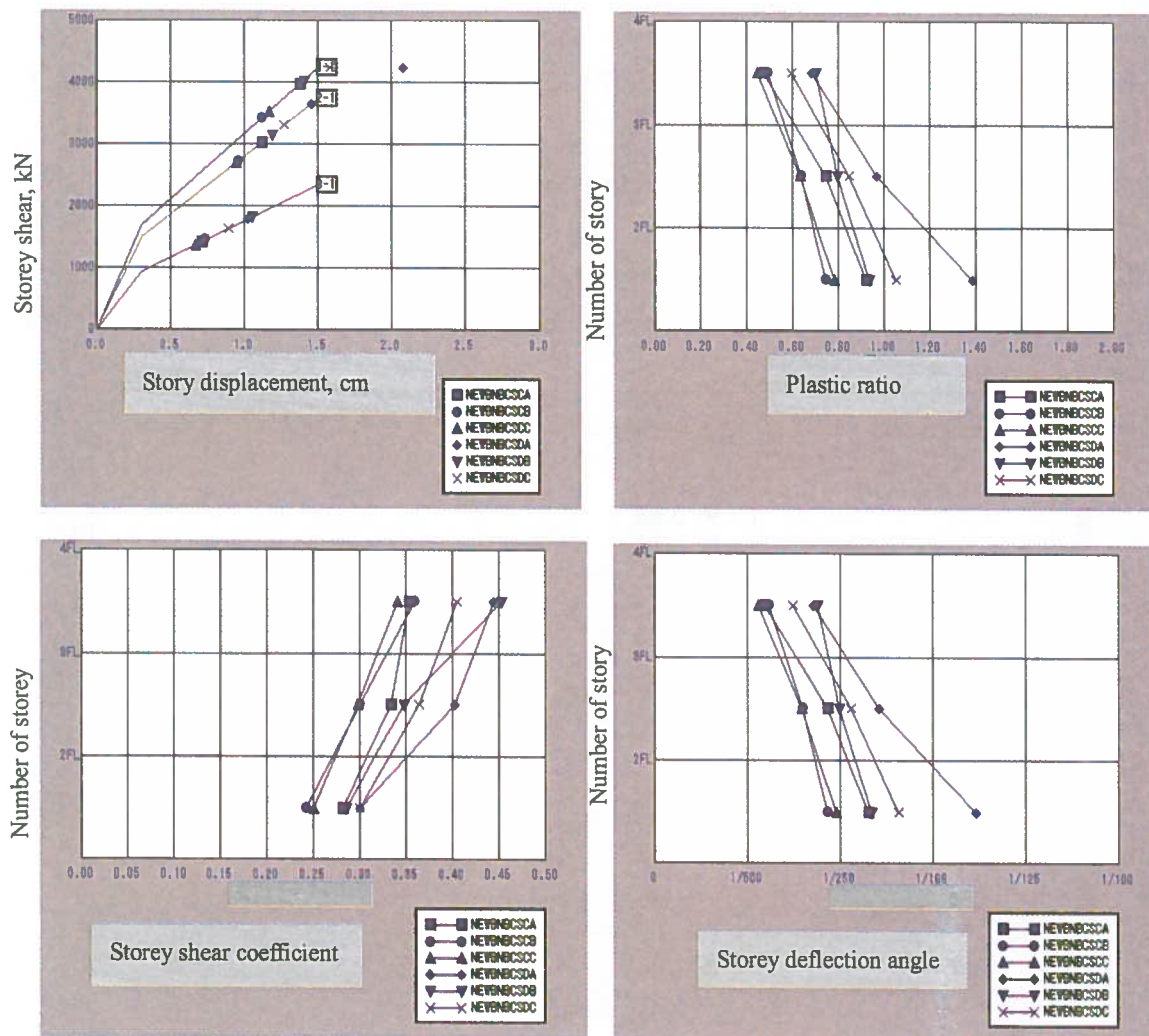


Figure SA 1.11 Response of Each Storey (Wave SC-A Means SA-1.)

#### Response Ductility Factor

Storey	BNBC Type SC 1, 2, 3			BNBC Type SD 1, 2, 3		
3	0.474	0.490	0.445	0.684	0.703	0.594
2	0.746	0.637	0.630	0.969	0.793	0.845
1	0.918	0.744	0.779	>1.387	0.935	>1.051

Average of soil type SC is 0.814, average of soil type SD is more than 1.0.

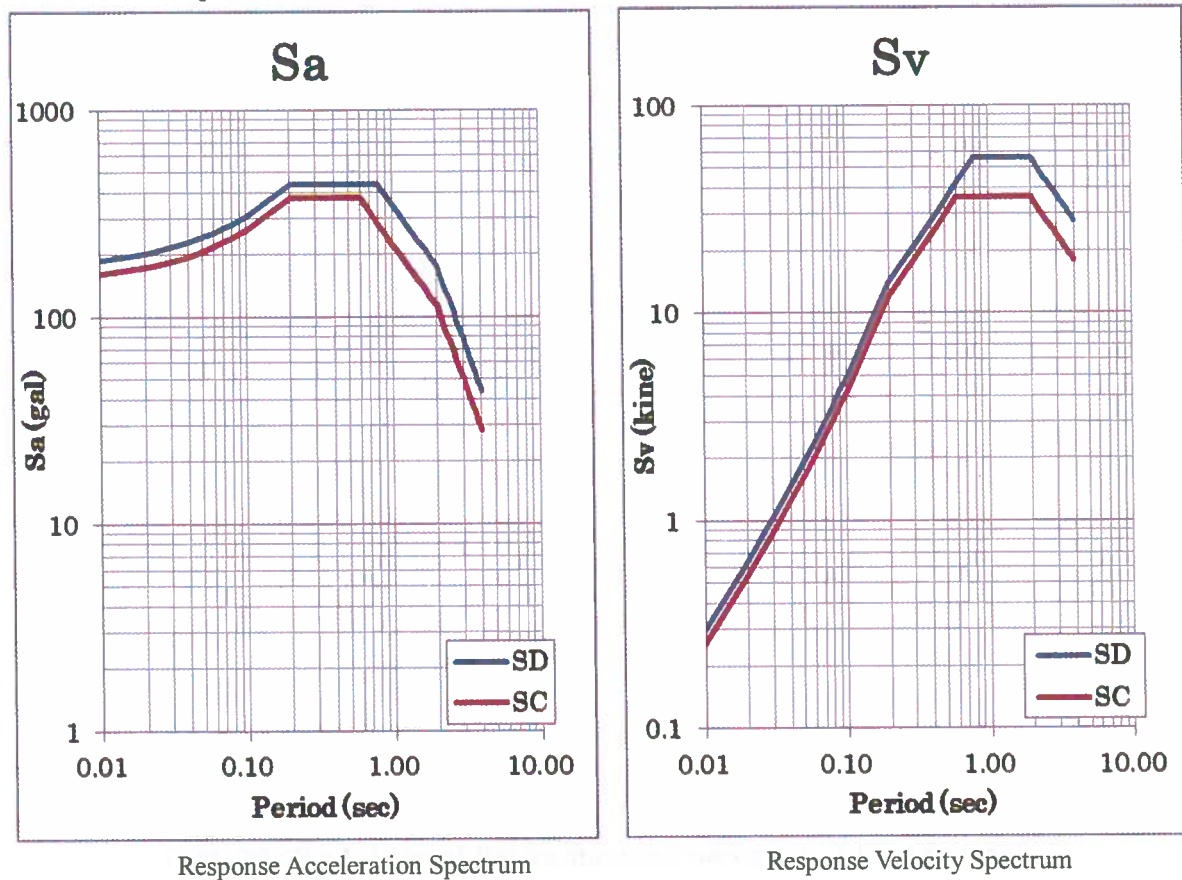
#### Response Shear Force Coefficient

Storey	BNBC Type SC 1, 2, 3			BNBC Type SD 1, 2, 3		
3	0.353	0.360	0.340	0.445	0.340	0.445
2	0.334	0.300	0.298	0.403	0.298	0.403
1	0.281	0.242	0.250	0.300	0.250	0.300

Average of soil type SC is 0.258 (67% of 0.383).

### (5) Example of Artificial Earthquake Waves

Artificial earthquake wave is prepared to produce the response spectrum of BNBC 2015. The Specification of response of soil classification type SC and type SD of BNBC 2015 is shown below.



**Figure SA 1.12 Specification of Response Spectrum by BNBC 2015**

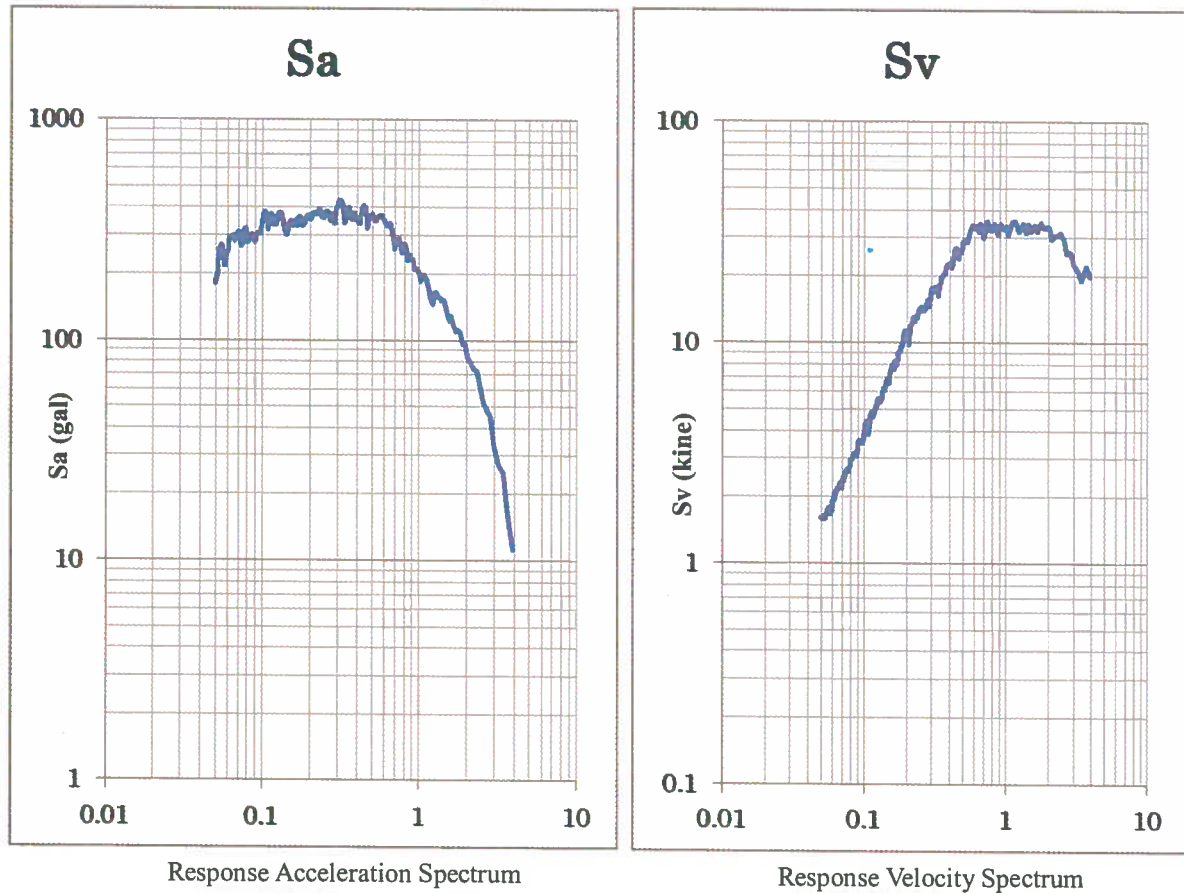
**Table SA 1.4 Peak Ground Acceleration and Velocity of Each Wave**

Name	gal	kine
New_SC_1	179.7	16.50
New_SC_2	215.4	14.14
New_SC_3	187.5	14.85
New_SD_1	218.6	23.01
New_SD_2	256.1	21.46
New_SD_3	210.0	21.53
1993_1	181.4	16.39
1993_2	193.7	16.35
1993_3	186.9	18.27

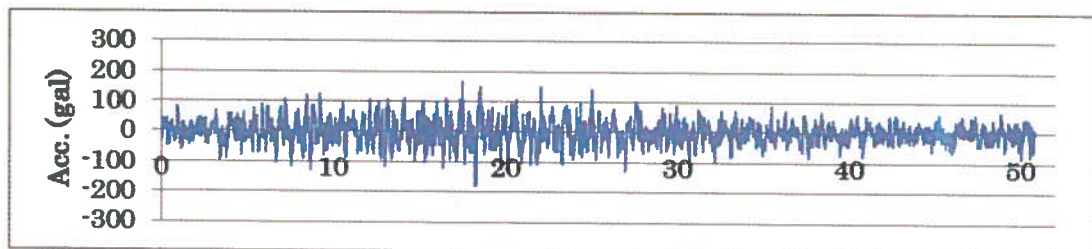
Result is not indicated  
in this supplement.

Note: Response of acceleration was controlled, but acceleration amplification factor was not controlled.

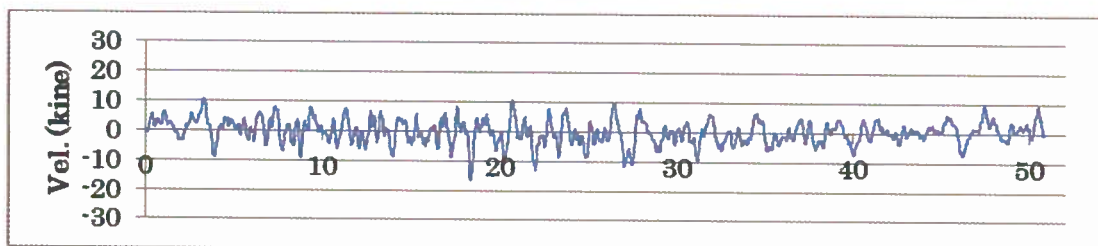
BNBC 2015, Response spectrum of Soil type SC-1



**Figure SA 1.13 Response Spectrum of Soil Type SC by BNBC 2015**



Acceleration at ground level



Velocity at ground level

**Figure SA 1.14 Acceleration and Velocity Waves of Soil Type SC by BNBC 2015**

BNBC93, Specification of Soil type 2, Reference Only

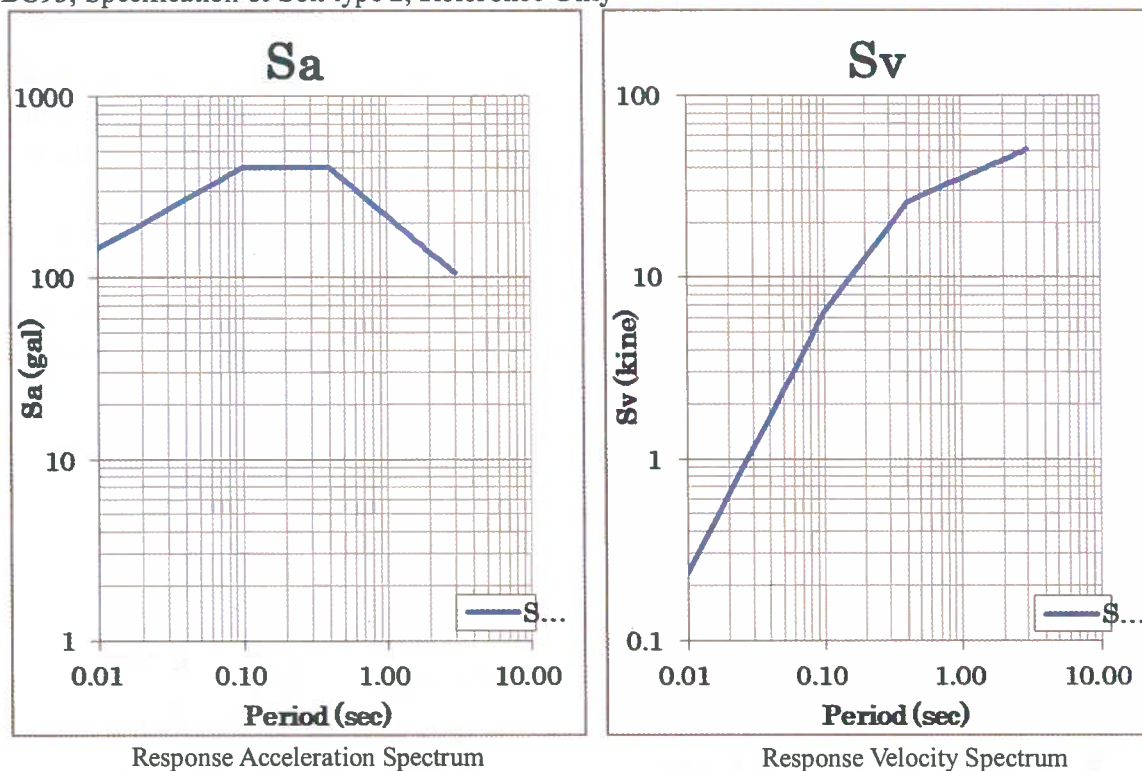


Figure SA 1.15 Specification of Response Spectrum of Soil Type 2 by BNBC 93

BNBC 93, Response spectrum of Soil Type 2-1, Reference Only

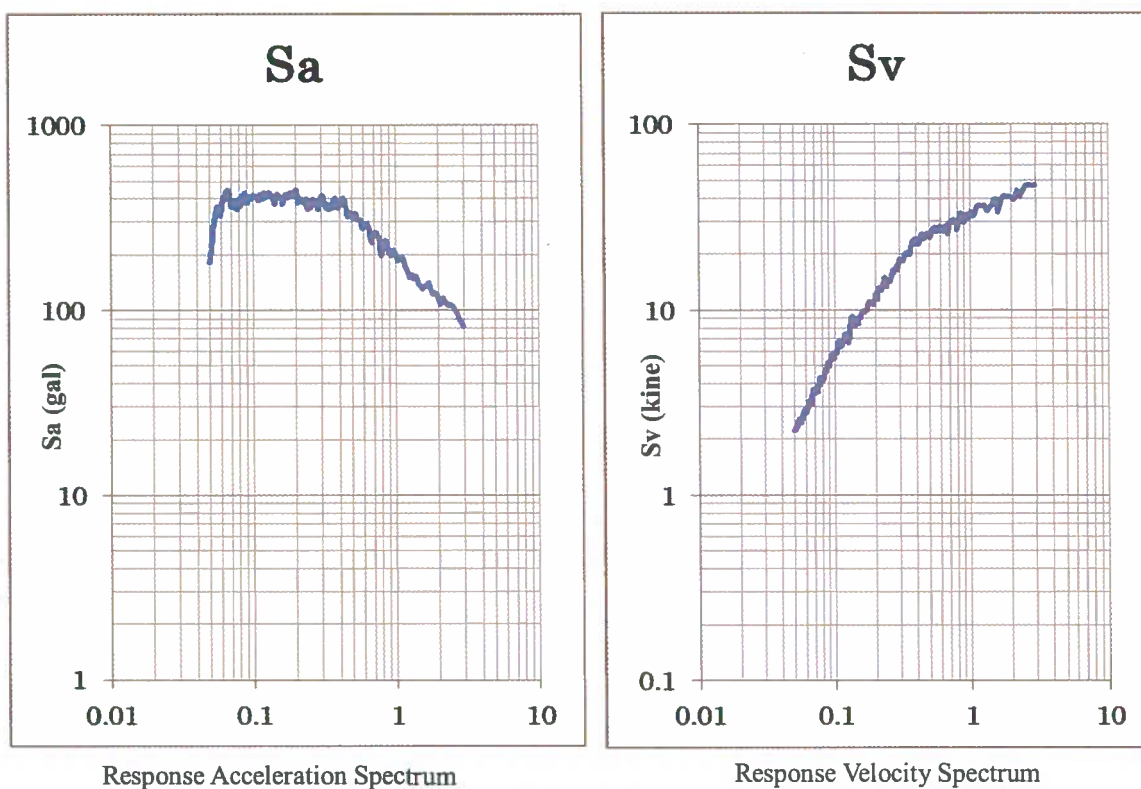


Figure SA1. 16 Response Spectrum of Soil Type 2 by BNBC 93

#### 1.4 COMPARISON OF PROPOSED $I_{so}$ AND ACTUAL $I_s$ OF BUILDINGS BY BNBC93

Proposed Seismic Demand Index  $I_{so}$  is compared with Seismic Index of Structure  $I_s$  of recent buildings in Zone 2 (Dhaka) designed by BNBC 93. Seismic evaluation of a 6 storey residential building and a 6 storey hospital building in Zone 2 and Zone 3 (Sylhet) designed by BNBC 93 was done, and the result is shown in Figure SA1.17.  $C$ - $F$  relation at ground floor of residential building is shown in Figure SA1.18. The values shown are  $x$  and  $y$  direction at ground floor level. The line of  $I_{so}$  equal to 0.30 is shown for reference for mid-rise RC buildings in Zone 2 (Dhaka), and this value of  $I_{so}$  (0.30) is recommended for zone-2 of Bangladesh. On the other hand, the line of  $I_{so}$  equal to 0.55 is shown in the figure for reference in Zone 3 (Sylhet). The accumulation of this kind of data will be recommended for further study.

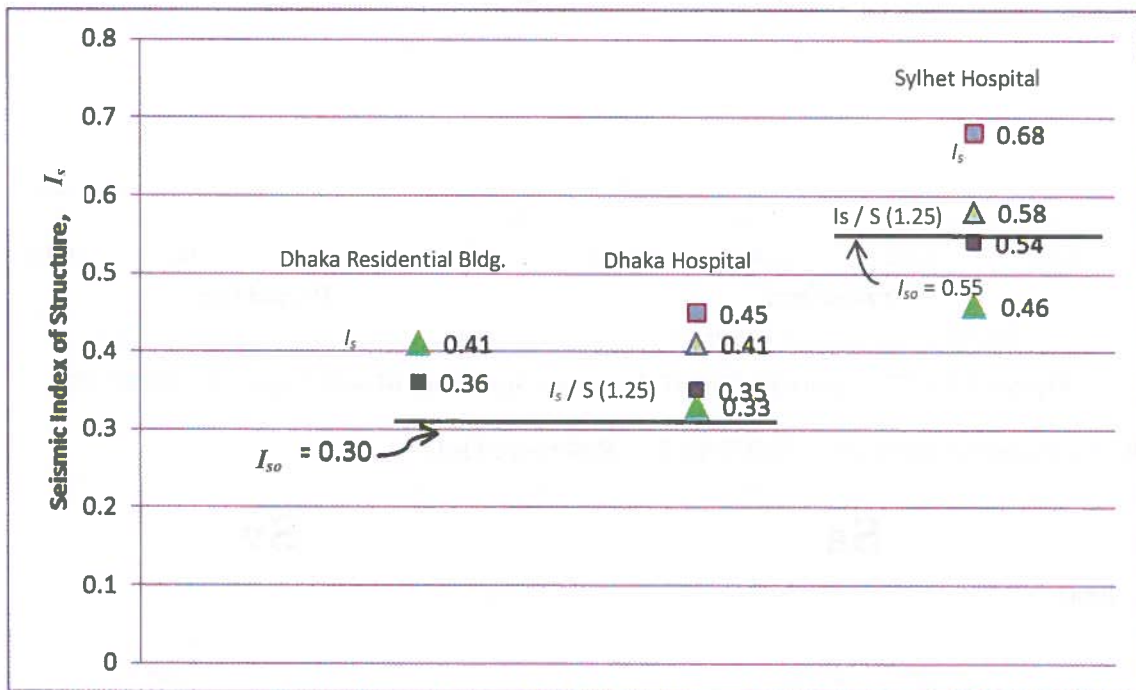


Figure SA 1.17 Seismic Index of Structure of Buildings Designed by BNBC93

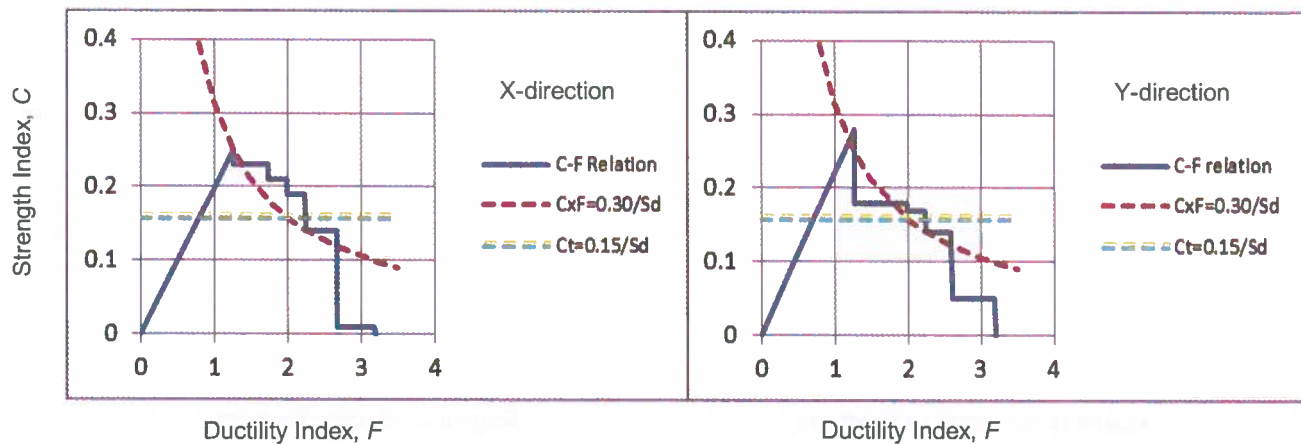


Figure SA 1.18  $C$ - $F$  Relation at Ground Floor, Dhaka Residential Building

Ductility index and shear force at Ground story is shown in Figure SA 1.19 for a residential building designed based on BNBC93.

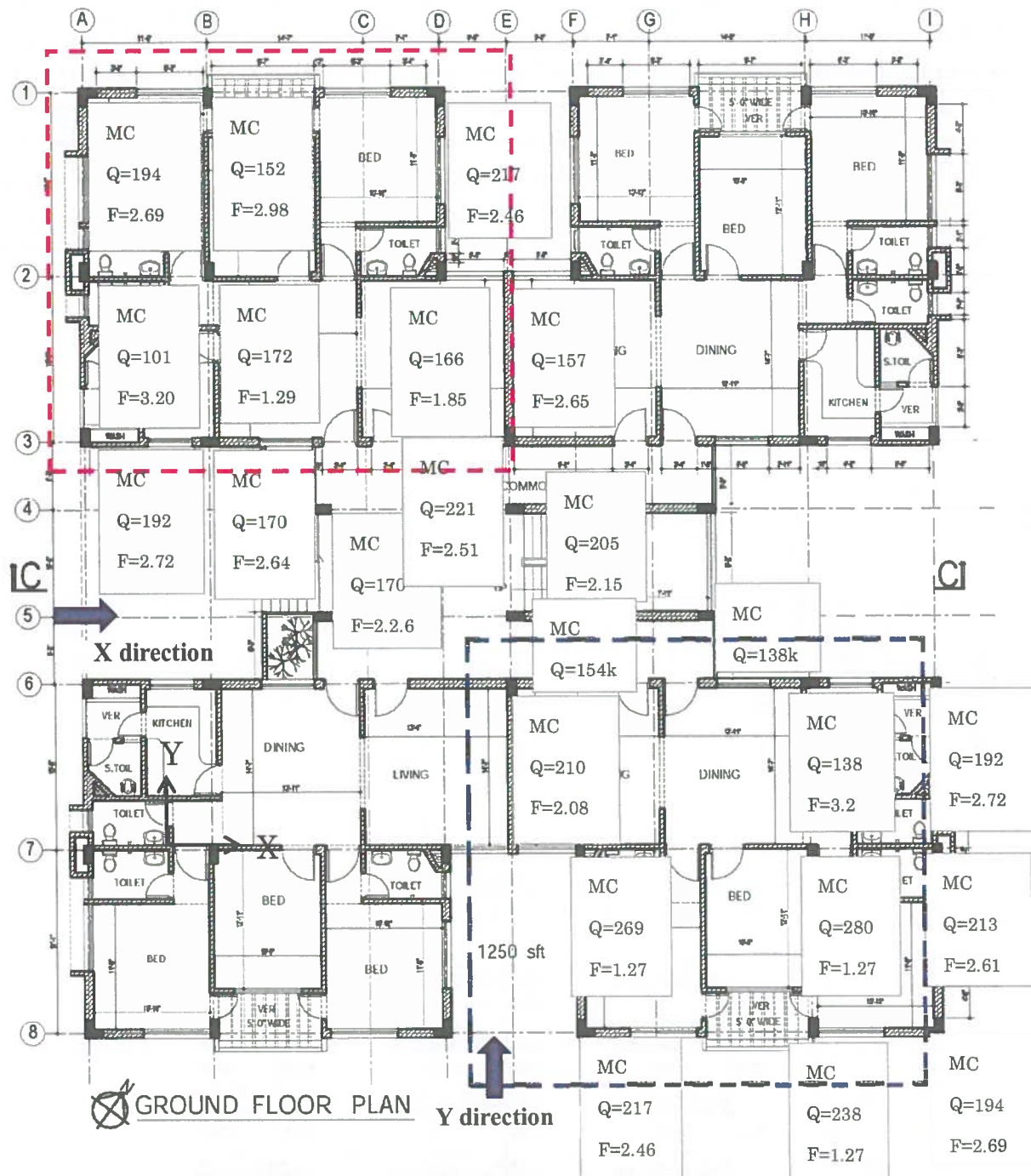


Figure SA 1.19 A Sample of Strength  $Q$  (Kn) and Ductility Index ( $F$ ) of Columns at Ground Floor, Residential Building in Zone 2 (Dhaka), Designed by BNBC93 1/4 of Floor Area is Shown for X and Y Direction Respectively

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amage will be  
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s such as shea

## 1.6 NUMERICAL COEFFICIENT $C$ OF BNBC93 AND BUILDING HEIGHT

Numerical coefficient  $C$  related to building height or natural period of building by BNBC93 was compared with  $R_t$  of Japanese Seismic Design Standard for reference. BNBC 2015 was not used for the purpose of this comparison. For example, natural period by simple method is calculated for 6 storey and 24.3m height building. According to BNBC93, natural period is  $T = 0.80\text{sec.}$ , and  $C = 1.2$  in case of ground type 2. This is 63% of max. value of  $C$  as shown in Figure SA 1.21. In case of Japanese code, natural period is estimated as  $T = 0.49\text{sec.}$  by height and no reduction from the max. value. It is recognized that there is approximately 40% difference by the application of coefficient  $C$ .

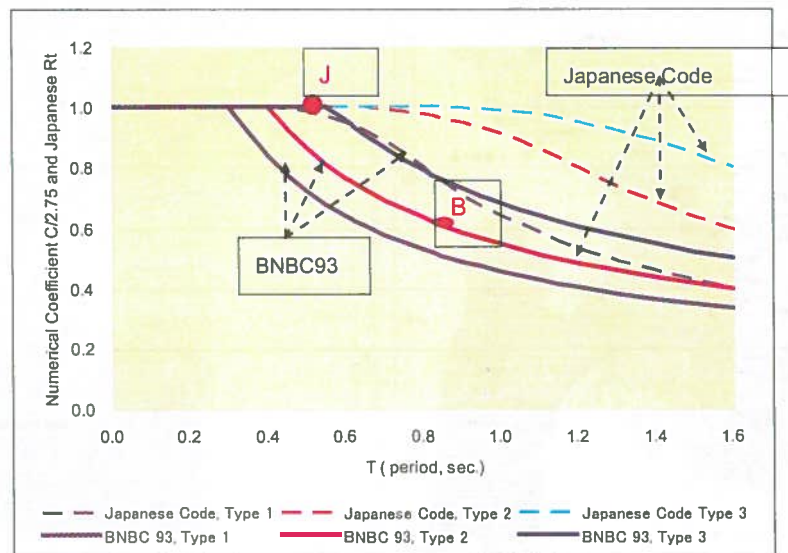


Figure SA 1.21 Comparison of Numerical Coefficient  $C/2.75$  of BNBC93 and Japanese Code  $R_t$

Example: 6 storey new RC building with 24.3m height, located on ground type 2

Natural period by BNBC93,

$$T = C_t (h_n)^{3/4} = 0.80 \text{ sec.}$$

RC moment frame,  $C_t = 0.073$ ,  $h_n$  = building height by meter,

Reference only, by BNBC 2015:  $T = C_t (h_n)^m = 0.82 \text{ sec.}$

In case concrete moment-resisting frames,  $C_t = 0.0466$ ,  $m = 0.9$ ,  $h_n$  = height of building in meters

Numerical coefficient  $C = 1.25 \times S / T^{2/3}$ .  $S = 1.2$  on ground type 2

$C/2.75 = 0.633$ . (Point B of Figure)

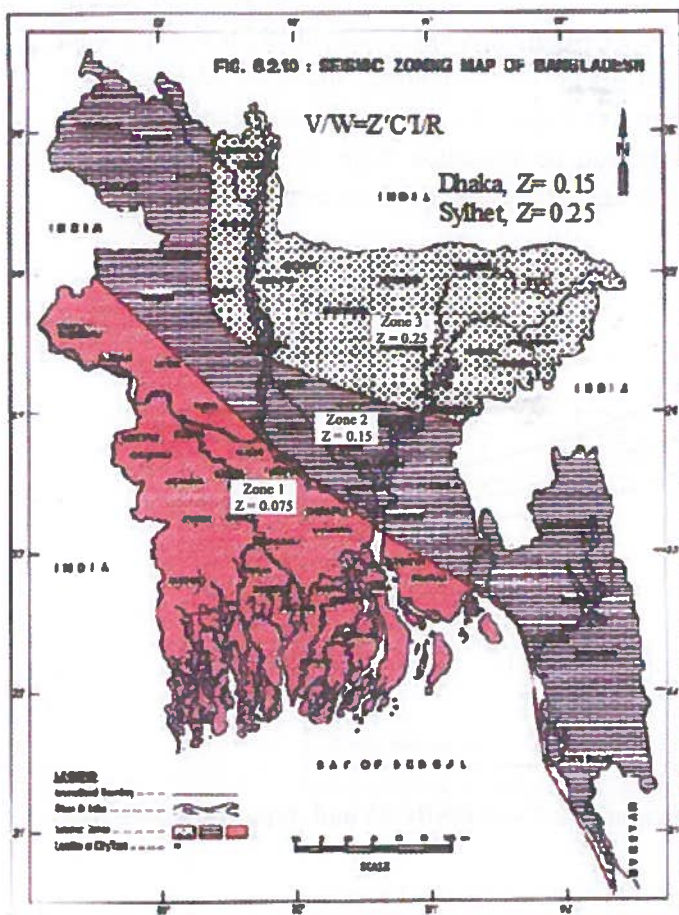
Reference only, Natural period by Japanese Standard: approximate  $T = 0.02x$ ,  $h = 0.02x$ ,  $24.3 = 0.486 \text{ sec.}$

Natural period is 0.59 times of that of BNBC93, because of the stiffness difference.

Vibration characteristics coefficient,  $R_t = 1.0$ . (Point J of Figure), and much higher than that of BNBC93.

## 1.7 SEISMIC ZONING MAP OF BNBC

Seismic Zoning map, BNBC1993



Seismic Zoning map, BNBC 2015



Figure SA 1.22 Seismic Zoning Map of BNBC 93 and BNBC 2015 (Figure 6.2.24)

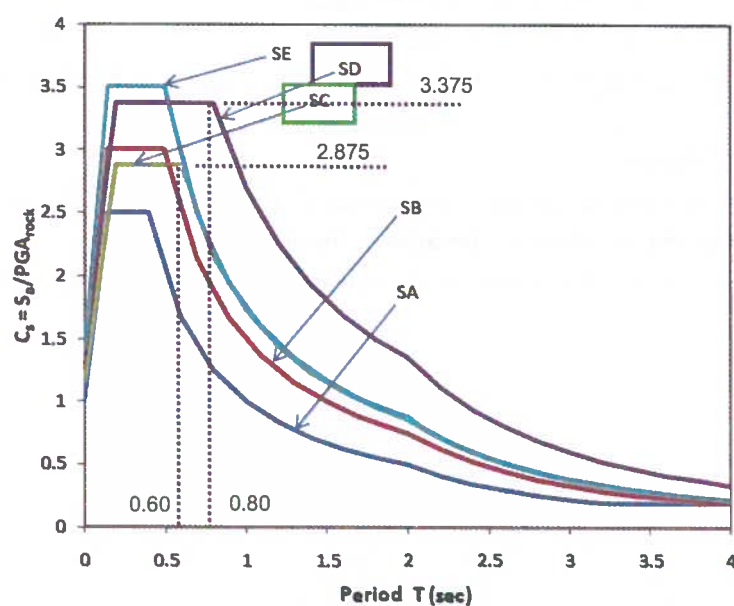


Figure SA 1.23 Design Acceleration Response Spectrum, BNBC 2015

## SUPPLEMENT A2. SUMMARY OF STRUCTURAL EXPERIMENT, 2013

To understand the lateral load-deflection behavior of existing RC building of Bangladesh, a set of structural experiments were conducted by CNCRP in 2012 and 2013. Following is the summary of the experiment in 2013.

### 1) Test Model

Existing mid-rise RC frame, a scaled 1 span 1 story frame model has been selected for the structural experiment. Structural experiment was done in 2013 to supplement the experiments done in 2012. To simulate the characteristics of existing RC buildings of Bangladesh, sample test specimens were prepared using brick aggregate concrete with low strength, 40 grade reinforcing bars with detailing generally practiced during that construction period.

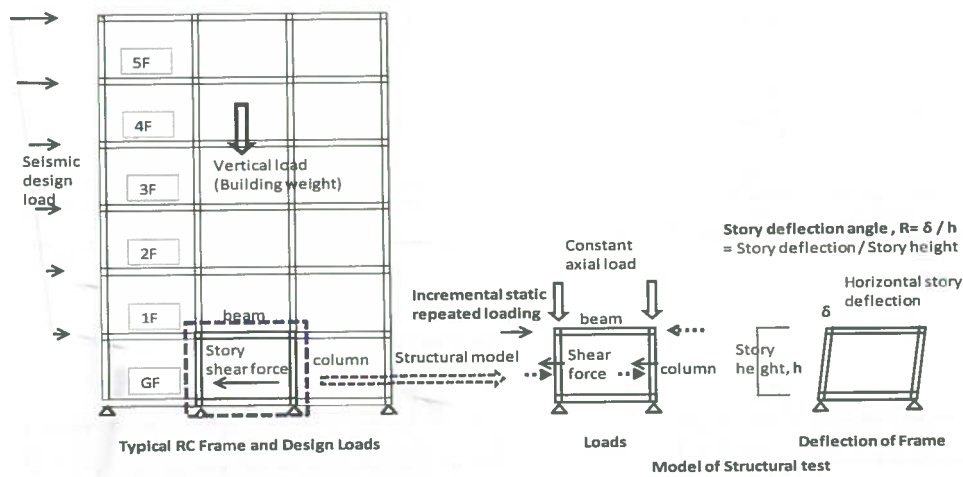


Figure SA 2.1 Test Model

### 2) Testing Apparatus

Load-meter and dial gauge relation has been controlled at the lab. Data from Load-cell and displacement transducer has been recorded by data-logger. Constant vertical load ( $N = 16$  tonf) and repeated static incremental horizontal loading were provided.

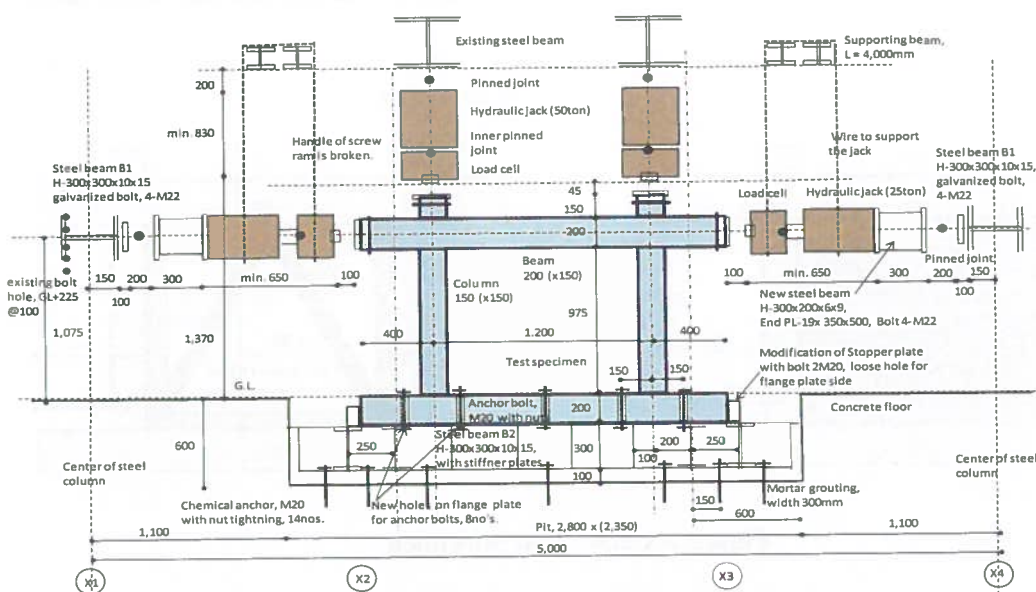


Figure SA 2.2 Testing Apparatus, Unit (mm)

### 3) Specimen

Total 6 specimens including 2 retrofitted specimens were tested in 2013. Specimen No.1 is a standard specimen of a typical frame. Column size is 150mm × 150mm. Beam size is 150mm × 200mm. In specimen No.2 main re-bars of beam has 180 degree hooks. Specimen No.3 has brick standing wall with thickness 65mm, and the height is 3/4 of clear height. Glass window is installed at the opening. Specimen No.4 has brick wall with no opening. Specimen No.5 is a retrofitted specimen by RC wall. Specimen No.6 is a retrofitted specimen by steel framed brace. Because of the capacity limitation of the horizontal hydraulic jacks, relatively low strength retrofit was planned.

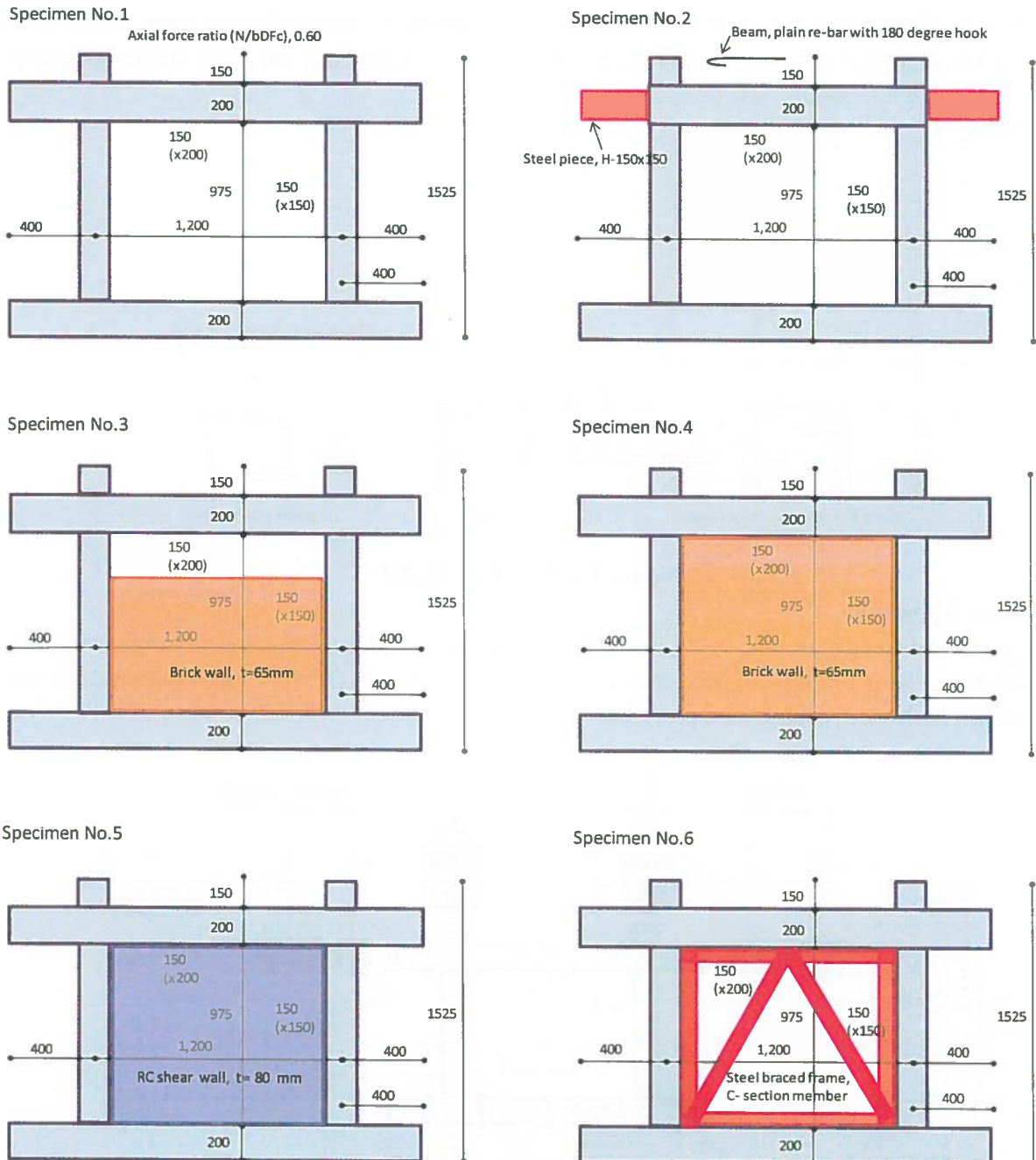


Figure SA 2.3 Test Specimen

Specimens were prepared with low strength concrete of  $10.8\text{N/mm}^2$ . High axial force ratio ( $N/b \cdot D \cdot F_c = 0.68$ ) was simulated. Poor shear reinforcement was provided. Joint of plain bars is lap joint. Column tie was changed to @195 instead of @150, due to the use of re-bar  $\phi 7.45\text{mm}$  instead of beam, from No.1 to No.6. Detail of specimen No.1 is shown below.

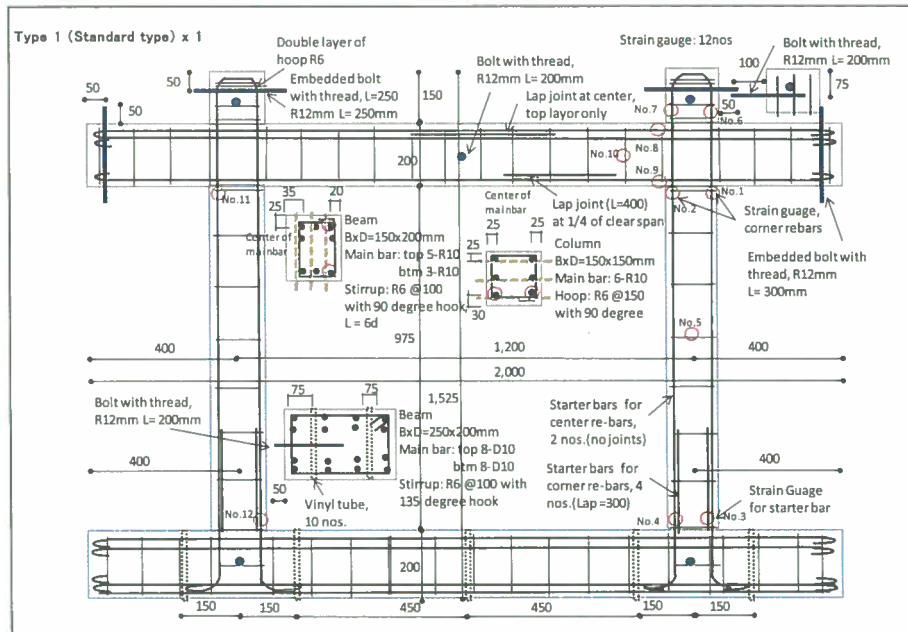


Figure SA 2.4 Test Specimen No.1

#### 4) Horizontal Load and Deflection Curve (Unit: tonnage and mm)

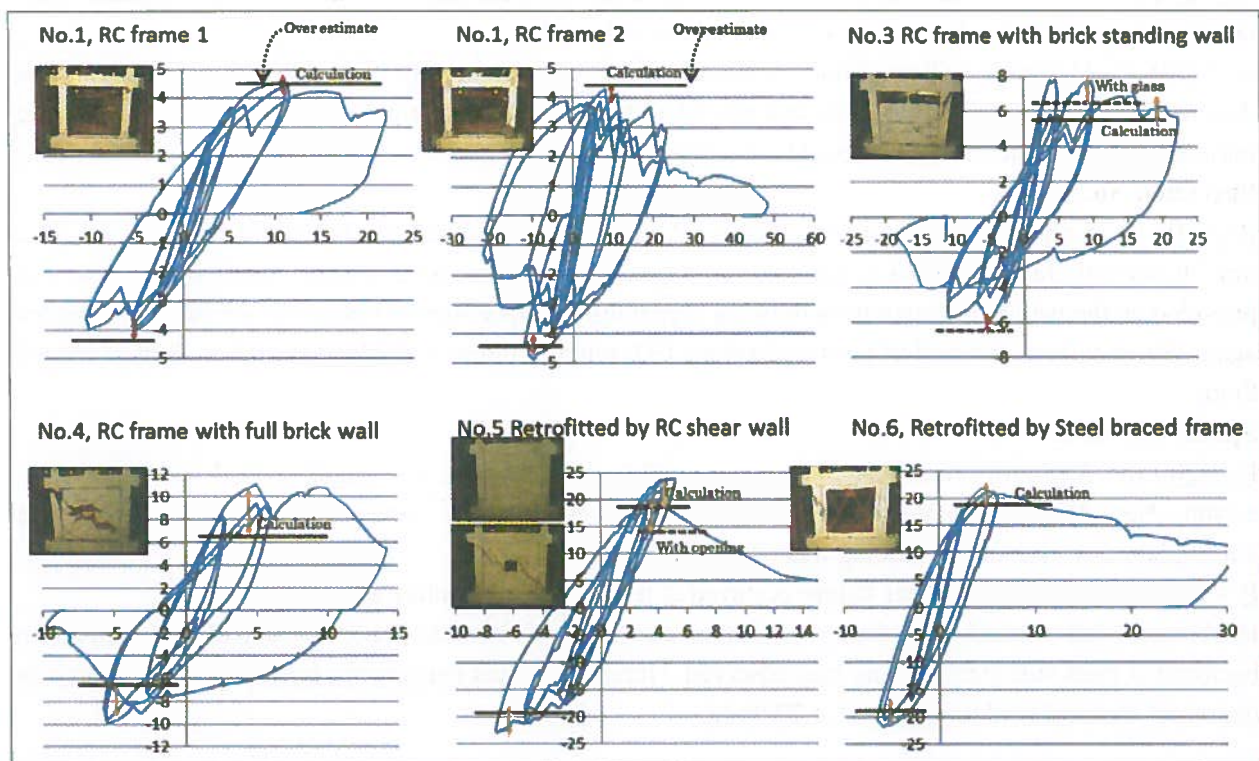


Figure SA 2.5 Horizontal Load and Deflection Curve

**Outline** (displacement shown is the reading value of the dial gauge at upper side of beam)

**Specimen No. 1:**

R<1/100 ( $\delta < 11.75\text{mm}$ ), max. Load was positive 4.5 ton and negative 4.0 ton. Flexural cracks occurred at top and bottom of columns. Vertical cracks also occurred at bottom of columns. Diagonal crack occurred at the top of column. Diagonal crack at left column occurred at R<1/200 ( $\delta < 5.88\text{mm}$ ).

R<1/50 ( $\delta < 23.5\text{mm}$ ), shear failure at the top of right column occurred at positive 3.5 ton at (25.7mm), and the vertical load dropped.

**Specimen No. 2:**

R<1/100 ( $\delta < 11.75\text{mm}$ ), max. load was observed positive 3.9 ton and negative 5.0 ton. Diagonal crack extended at both side of beam column panel zone, and strength dropped from 3.5 ton (at 7.8mm) to 3.0 ton (12.4mm). Flexural cracks occurred at bottom of columns.

R<1/50 ( $\delta < 23.5\text{mm}$ ), Diagonal crack extended at both panel zone. Cover concrete at bottom of column (especially rear side) was detached.

R<1/25 ( $\delta < 47.0\text{mm}$ ), Bottom of column was failed. Cracks of panel zone were extended. Vertical load was reduced at 30mm.

**Specimen No. 3:**

R<1/200 ( $\delta < 5.88\text{mm}$ ), max. Load was positive 7.7 ton and negative was 6.0 ton. Cracks extended through columns and brick walls.

R<1/100 ( $\delta < 11.75\text{mm}$ ), Diagonal and vertical crack extended on right column. Many flexural cracks at left column observed.

R<1/50 ( $\delta < 23.5\text{mm}$ ), Positive load 7.0 ton and dropped to 5.8 ton due to breaking of Glass. Shear failure of right column occurred at (- 17.1mm) of negative loading, and vertical load also dropped.

**Specimen No. 4:**

R<1/200 ( $\delta < 5.88\text{mm}$ ), max. Load at positive was 11.0 ton, and negative was 10.0 ton. Diagonal crack occurred at top of left column. Cracks extended to brick wall.

R<1/100 ( $\delta < 11.75\text{mm}$ ), Shear failure occurred at left column at (10mm) of positive loading. Wide diagonal crack extended to brick wall and extended through bottom of right column. At negative loading, horizontal stiffness decreased and could not support the vertical load.

**Specimen No. 5:**

R<1/200 ( $\delta < 5.88\text{mm}$ ), max. Load was positive 23.3 ton (limit of jack, at 4.3mm) and negative was 21.2 ton. Horizontal crack observed at left column, diagonal crack developed at wall. Small square hole was provided on the wall at the laboratory drawing preparatory of specimen to reduce the strength, and loaded again. Shear failure occurred at positive loading 19.6 ton at 5.0mm. Axial load started to drop at around 8mm.

**Specimen No. 6:**

R<1/200 ( $\delta < 5.88\text{mm}$ ), max. Load was positive 22.0 ton at 5.2mm, and negative load was 22.0 ton at 6.2mm. New diagonal crack occurred at middle of left column at 3<sup>rd</sup> positive cycle. Slight Buckling of out of plain direction of left bracing was observed.

R < 1/100 ( $\delta < 11.75\text{mm}$ ), Shear failure occurred at left column at positive 20 ton at 9.0mm.

Buckling of left vertical steel frame was observed. Yield of left side brace and out of plane direction buckling at right side steel bracing was observed. Horizontal crack beneath the beam at grout mortar was observed. Vertical load drop started at 20.9mm.

### 5) Evaluation of Horizontal Strength

Horizontal strength of each specimen was calculated as follows. Calculated value is shown as horizontal line on Figure SA 2.5. Calculated strength of **specimen No.1** as flexural failure was slightly over estimated, possible reason is the use of plain bar and lap joint of main bar. Calculated strength of **specimen No.2** was also slightly over estimated. Some drop of strength was expected due to the anchor of beam re-bar, but there was no such clear reduction of the strength in this specimen. **Specimen No.3** was evaluated as flexural failure column and shear failure column. Contribution of the strength of window with frame was not negligible, which was estimated as 1.2 ton. **Specimen No.4** was estimated as shear failure column at both side, and excessive strength was the contribution of brick wall. **Specimen No.5** is the shear failure column at both sides with shear failure RC wall. Strength evaluation without opening and with opening seems reasonable. Strength evaluation of **Specimen No.6** is the summation of shear failure strength of both column and strength of brace (yield strength of tension side and buckling strength of compression side). Strength evaluation seems reasonable.

#### Column

Flexural strength is calculated by Equation A1.1-1 of the J. Standard, Shear strength of column is calculated by Equation A1.1-1 of the J. Standard. Estimated collapse mechanism of the frame is shown in Figure SA 2.6.

#### Shear wall for retrofit

Shear strength of concrete panel plus column shear strength, shear strength of post-installed anchor and grout mortar were calculated.

#### Steel framed brace for retrofit

Steel brace of tension and compression (buckling strength) plus column shear strength, shear strength of stud/ post-installed anchor and grout mortar were calculated.

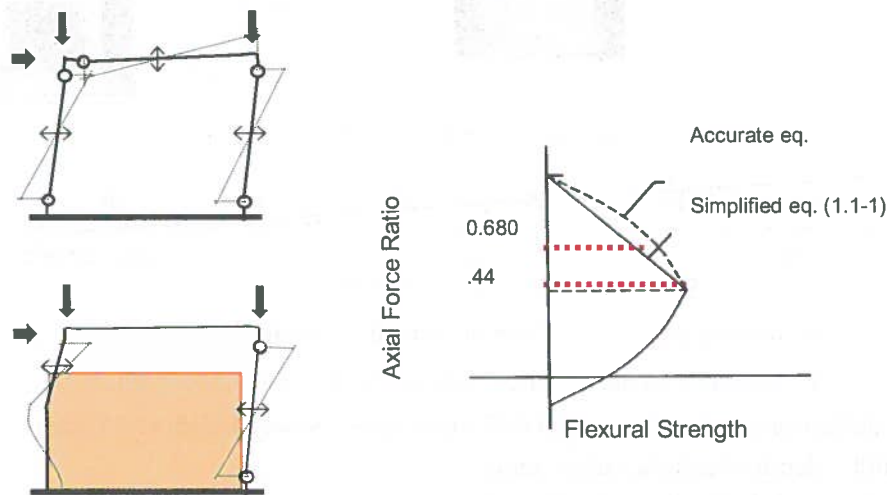


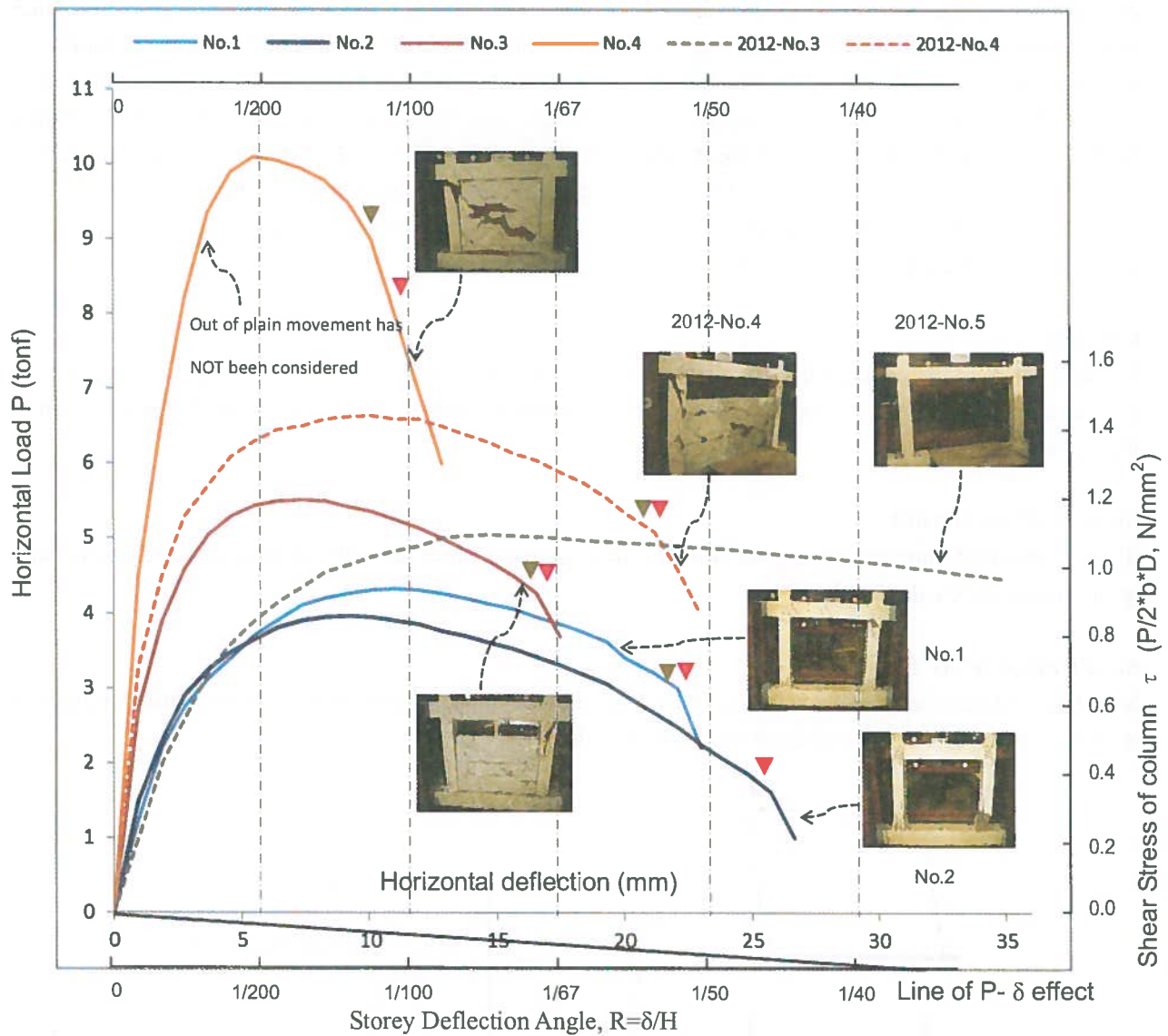
Figure SA 2.6 Estimated Collapse Mechanism of Frame and M-N Interaction Curve

### 6) Simplified Monotonic Horizontal Load- Deflection Curve

Following simplified monotonic load-deflection curves are drawn based on the result of cyclic loading and the engineering judgment.

Axial force ratio: Specimen No.1 ~ No.4,  $N/(b \cdot D \cdot F_c) = 0.68$  ( $F_c = 10.6 \text{ N/mm}^2$ ,  $N = 163 \text{ kN}$ )

Specimen 2012-No.4, 5,  $N/(b \cdot D \cdot F_c) = 0.44$  ( $F_c = 16.5 \text{ N/mm}^2$ ,  $N = 163 \text{ kN}$ )



Note: Marking: ▼ denotes a point of "Drop in vertical strength".

▼ denotes a point of "Shear failure" by the visual observation.

$R$ : Storey deflection angle = Horizontal deflection (mm)/ Storey height (1,175mm)

$b \cdot D$ : Width  $\times$  depth of column (mm  $\times$  mm)

$F_c$ : Concrete compressive strength ( $\text{N/mm}^2$ )

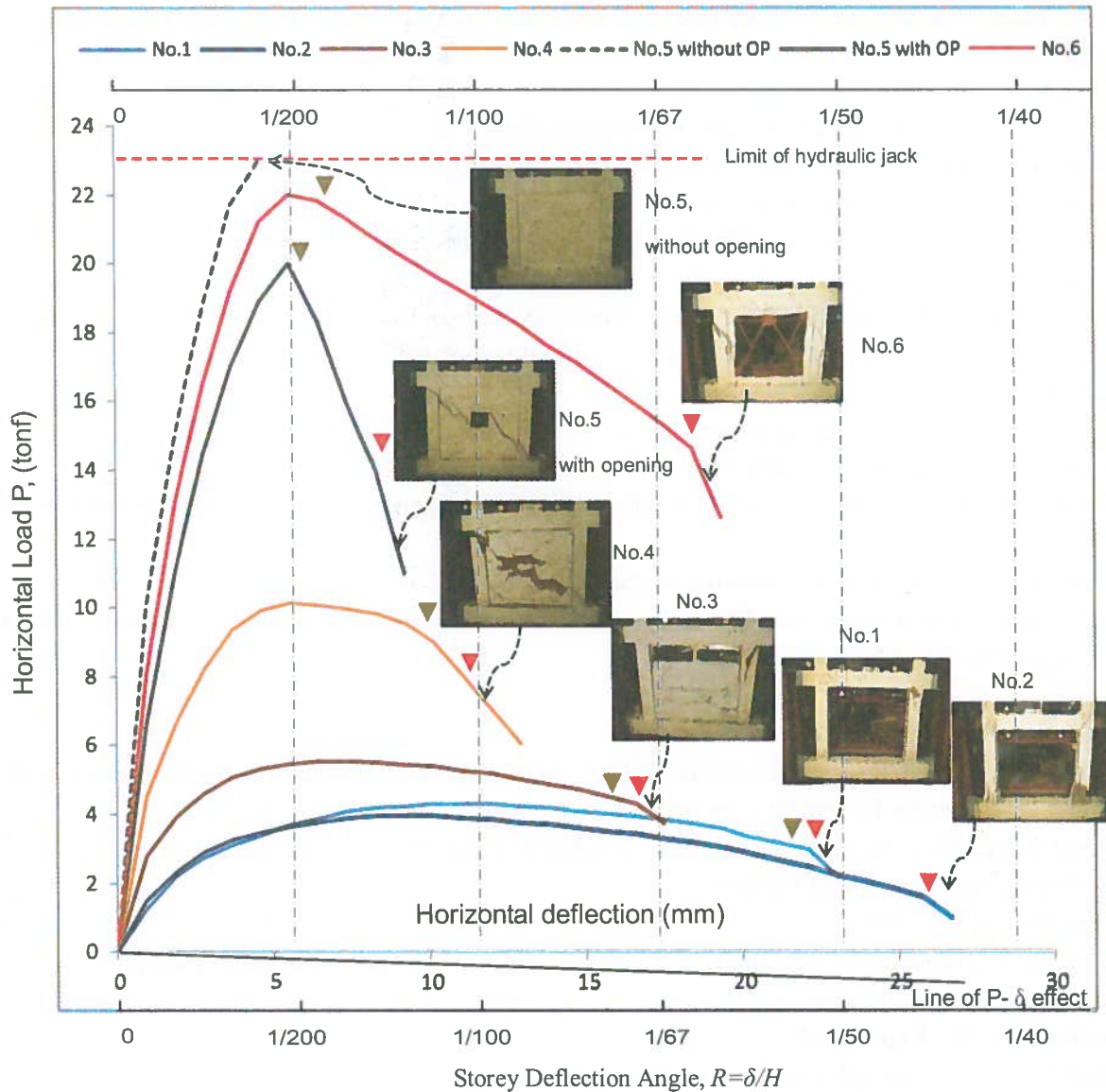
1 tonf = 2, 205 lbf = 9.8 kN

1 Mpa =  $1 \text{ N/mm}^2$

$1 \text{ N/mm}^2 = 145 \text{ psi}$

Figure SA 2.7 Simplified Monotonic Load- Deflection Curve

## Simplified Monotonic Horizontal Load- Deflection Curve including Retrofitted Specimen No.5 and No.6



Note: Marking: ▼ denotes a point of “Drop in vertical strength”.

▼ denotes a point of “Shear failure” by the visual observation.

$R$  : Storey deflection angle = Horizontal deflection (mm)/ Storey height (1,175mm)

$b \cdot D$  : width and depth of column (mm × mm)

$F_c$  : Concrete strength ( $\text{N/mm}^2$ )

Axial force ratio: Specimen No.1 ~ No.6,  $N/b \cdot D \cdot F_c = 0.68$  ( $F_c = 10.6 \text{ N/mm}^2$ ,  $N = 163 \text{ kN}$ )

Retrofit: Specimen No.5,  $F_c$  of wall =  $10.7 \text{ N/mm}^2$

**Figure SA 2.8 Simplified Monotonic Load- Deflection Curve**

### 7) Summary of Structural Experiment, 2013

Basic structural experiment for the seismic evaluation and retrofit design of existing RC buildings in Dhaka were done by CNCRP. Summary of the experiment result is as follows:

**(1) Specimen and loading:**

- RC frame of plain bar and low strength concrete ( $F_c = 10.6 \text{ N/mm}^2$ ) with/without brick-wall.
- High axial force ratio ( $N/b \cdot D \cdot F_c$ ) of column 0.68.
- Repeated horizontal load is provided until the point of drop in vertical load (= drop of floor).

**(2) Results:**

- Horizontal load- story deflection angle relation was shown.
- High axial force ratio causes low ductility of frame in addition to strength reduction.
- Shear failure of column causes the drop in vertical strength.
- Brick-wall affects the behavior of frame, with respect to stiffness, strength and ductility.

**(3) Retrofit:**

- RC shear wall and steel braced frame is effective to increase shear strength of frames, but reduces the deformability with no column jacketing, in case of vulnerable column.

**(4) Stiffness**

- Observed initial stiffness of each specimen is, No.1: 1.3 tonf/mm, No.2: 1.8 tonf/mm, No.3: 6.6 tonf/mm, No.4: 6.6 tonf/mm, No.5: 25 tonf/mm, No.6: 25 tonf/mm
- Initial stiffness of a frame with brick standing wall No.3 was approximately 4 times that of average of No.1 and No.2.  
There was no clear difference of initial stiffness between No.3 and No.4.  
Initial stiffness of a frame with RC wall or steel framed brace was approximately 16 times that of average of No.1 and No.2.

**(5) Strength evaluation:**

- Flexural and shear strength of column, shear strength of RC wall and strength of steel brace including connection for retrofit were evaluated and compared with test results.
- Flexural strength by the calculation should be reduced in case of low strength concrete.

**(6) The Issue to be investigated further:**

- Strength of beam column connection, evaluation of poor quality control at sites.  
Quantitative evaluation of 90 degree hooks of column tie, use of plain main bar and lap joint.
- Ductility evaluation of above item 1.

**(7) Limitation of the Experiment**

**Material** (Def. of low strength concrete, less than  $13.5 \text{ N/mm}^2$ )

- Concrete; low strength concrete,  $10.6 \text{ N/mm}^2$  at 8 weeks. Compared to,  $16.5 \text{ N/mm}^2$  in 2012.

	Yield stress	Original requirement	2012
b. Plain Re-bar $\phi$ 10mm	$\sigma_y = 350 \text{ N/mm}^2$	$\phi$ 6mm	$327 \text{ N/mm}^2$
c. Plain Re-bar $\phi$ 7.4mm	$\sigma_y = 353 \text{ N/mm}^2$		$(560 \text{ N/mm}^2)$
d. Deformed re-bar D10mm	$\sigma_y = 274 \text{ N/mm}^2$		$387 \text{ N/mm}^2$
e. Steel angle plate 4.2mm	$\sigma_y = 363 \text{ N/mm}^2$		$3.0 \text{ mm}, \sigma_y = 250 \text{ N/mm}^2$

Interval of shear reinforcement (column tie) and detail of steel bracing were modified accordingly.

**Loading**

- Inclination of foundation beam under the specimen; Slight inclination of foundation steel beam was observed in plane and out of plane direction, and filler plates were provided where a gap exists under the specimen.
- Limitation of horizontal hydraulic jacks; capacity was 23 ton (230kN) only.

### SUPPLEMENT A3. PROPOSED DUCTILITY INDEX $F$ RELATED TO AXIAL FORCE RATIO

Axial force ratio  $N/b \cdot D \cdot F_c$  is an important factor to evaluate ductility of columns. Japanese standard states that the ductility index  $F$  is 1.0, when axial force ratio exceeds 0.4 and tie interval is more than 100mm. On the other hand, BNBC 93 specifies that allowable axial force of column is approximately 60% of combined strength of concrete and re-bars. It is proposed incorporating the requirement of BNBC. In case of axial force ratio  $N/b \cdot D \cdot F_c$  exceeds 0.4 and up to 0.60, proposed ductility index  $F$  is 1.27 for low strength concrete, from the structural experiment by CNCRP. Ductility index of column will be 1.0 in case axial force ratio exceeds 0.6.

#### 1) Ductility index of a flexural failure column with high axial force ratio by the experiment

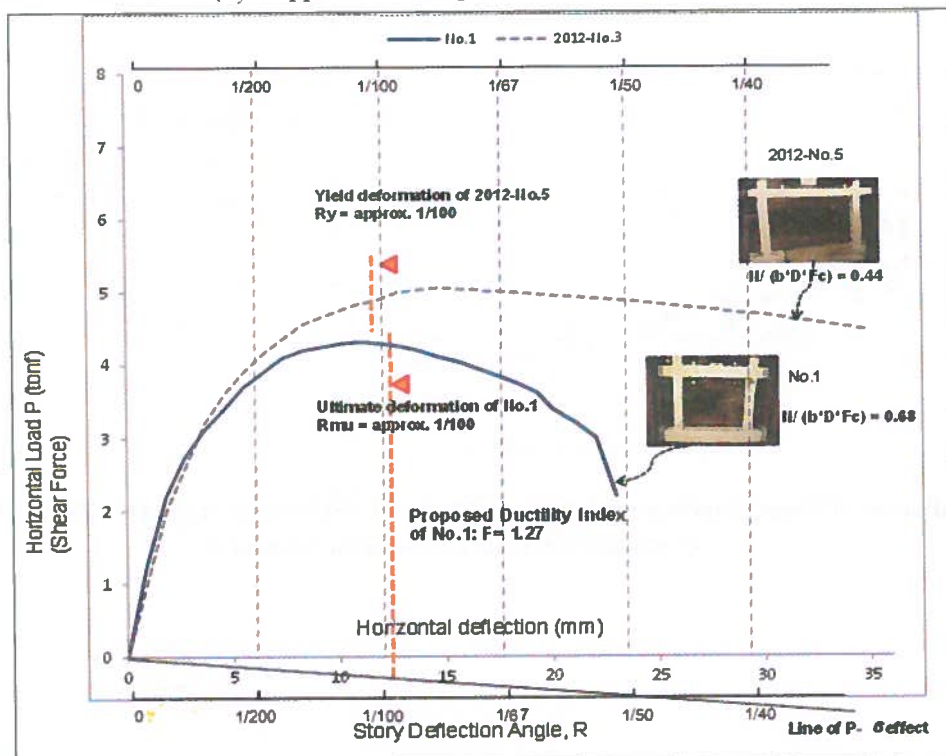
Simplified monotonic load-deflection curve of two frame specimens are shown in Figure SA 3.1. Specimen 1 is low strength concrete and axial force ratio of column is 0.68. Storey deflection angle of this specimen at ultimate capacity  $R_{mu}$  is estimated as approximately 1/100. Specimen 2012-No.5 is ordinary concrete and axial force ratio is 0.44. Storey deflection at yield deformation  $R_y$  is estimated as approximately 1/100. (Note: If the deflection in Japanese Standard is used, it is 1/150.)

**In this condition, Ductility Index is calculated as follows:**

In case  $R_{mu} \geq R_y$

$$F = \frac{\sqrt{2R_{mu}/R_y} - 1}{0.75 \cdot (1 + 0.05R_{mu}/R_y)} \quad F \leq 3.2 \quad \text{The J. Standard (16)}$$

$F = 1.27$  is calculated ( $R_y = \text{approx. } 1/100$ , specimen 2012-No.5,  $R_{mu} = \text{approx. } 1/100$ , specimen No.1)



Note:  $R$ : Story deflection angle = Horizontal deflection (mm)/ Storey height (1,175mm)

**Figure SA 3.1 Simplified Monotonic Load-Deflection Curve of Two Frame Specimens**

## 2) Axial force ratio $N/b \cdot D \cdot F_c > 0.4$ and Ultimate story deflection angle

It is general understanding that ultimate deflection angle is reduced in case that axial force ratio  $N/b \cdot D \cdot F_c$  exceeds 0.4. Experimental study of ultimate deflection related to axial force ratio and shear reinforcement ratio of column by the J. Standard is introduced in Figure SA 3.2 and SA 3.3.

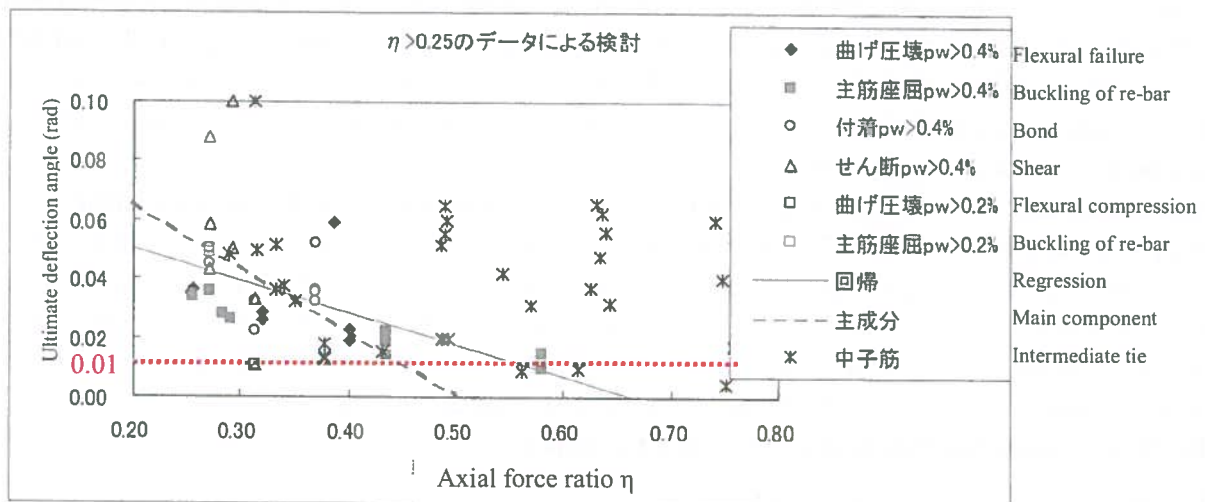


Figure SA 3.2 Influence of Axial Force Ratio (by the Data  $\eta > 0.25$ ) (Commentary Figure 1.2-3 of the Standard)

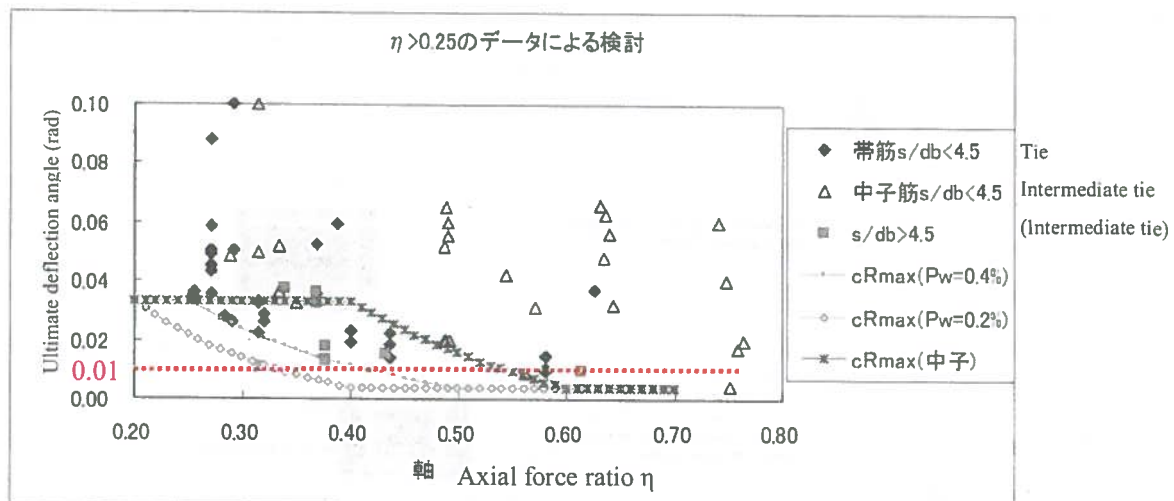


Figure SA 3.3 Influence of Shear Reinforcement Ratio and Interval of Tie under High Axial Force of Column (Commentary Figure 1.2-4 of the Standard)

### 3) Relation of Deflection Limit and Axial Force Ratio

Deflection member angle limit and ductility factor related to axial force ratio is introduced by RC structural design code 2010, Japan

(Source: Architectural Institute of Japan, "Structural Calculation Code of Reinforced Concrete Structure 2010")

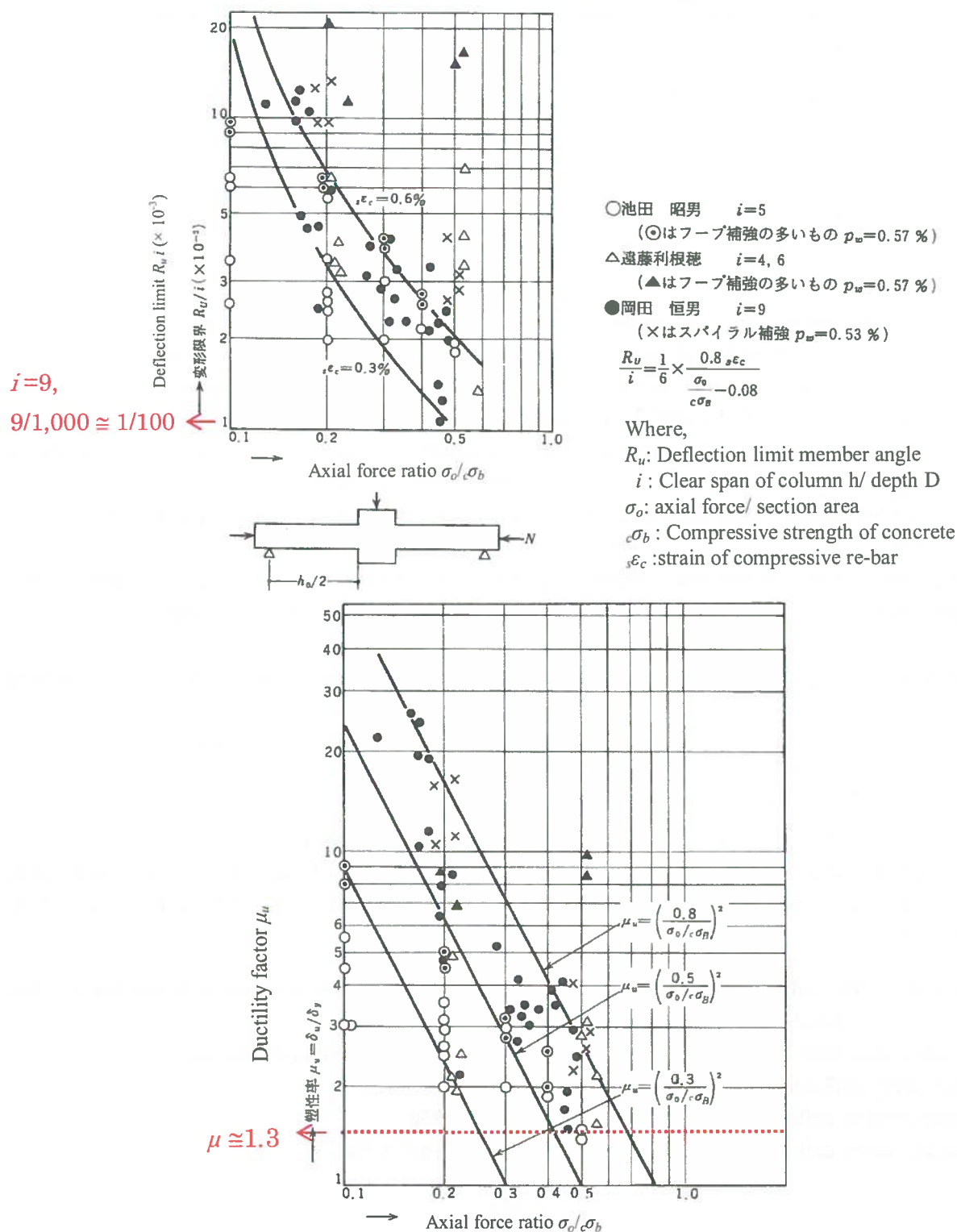


Figure SA 3.4 Ductility Factor and Axial Force Ratio Relation (Commentary Figure 14.8 of above AIJ)

## SUPPLEMENT A4 DUCTILITY INDEX $F$ AND DEFORMATION CAPACITY OF COLUMN

### A. Ductility Index of a Flexural Failure Column

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

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

$$F = 1.0 + 0.27 \frac{R_{mu} - R_{250}}{R_y - R_{250}} \quad \text{The J. Standard (15)}$$

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

$$F = \frac{\sqrt{2R_{mu}/R_y - 1}}{0.75 \cdot (1 + 0.05R_{mu}/R_y)} \leq 3.2 \quad \text{The J. Standard (16)}$$

Where:

$R_y$  = Yield deformation in terms of inter-storey drift angle, which in principle is taken as  $R_y = 1/150$ .

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

$R_{mu}$  = Inter-storey 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).

In the J. Standard, equation (16) is introduced and explained from the estimation of non-linear response analysis. Following explanation is shown in the Japanese version of the Standard.

1. Time-history analysis using degrading tri-linear model of flexural failure type frame as restoring force characteristics.
2. Relationship between maximum plastic response  $\mu$  (= maximum response displacement / yield displacement,  $C_Y$ ) and ductility index (Elastic response displacement,  $C_E$ /yield displacement,  $C_Y$ ) is calculated.
3. Envelope curve against the result of parametric study, following equation, like Newmark's equation, is derived.

$$C_Y / C_E = 0.75(1 + 0.05 \times \mu) / \sqrt{(2 \cdot \mu - 1)} \quad \text{The J. Standard Commentary eq. 3.2.3-2}$$

Since Ductility index  $F = C_E / C_Y$ , the reverse of the equation is,

$$F = \sqrt{(2 \cdot \mu - 1)} / (0.75 \times (1 + 0.05 \times \mu)) \quad \text{The J. Standard Commentary eq. 3.2.3-3}$$

In case that  $R_{mu} / R_y$  is used instead of  $\mu$ , then equation (16) is derived. It is emphasized that  $R_{mu}$  as the inter-storey drift angle at the ultimate deformation capacity in flexural failure of the column is the drift angle at the peak in the load-deflection curve, and the negative slope region after the peak will not be evaluated for  $R_{mu}$ .

The relation of Ductility factor,  $\mu$  and Ductility index,  $F$ , of equation (16) is shown in Figure SA 4.1. The range of  $F \leq 2.0$ , which is general case, is shown for convenience.

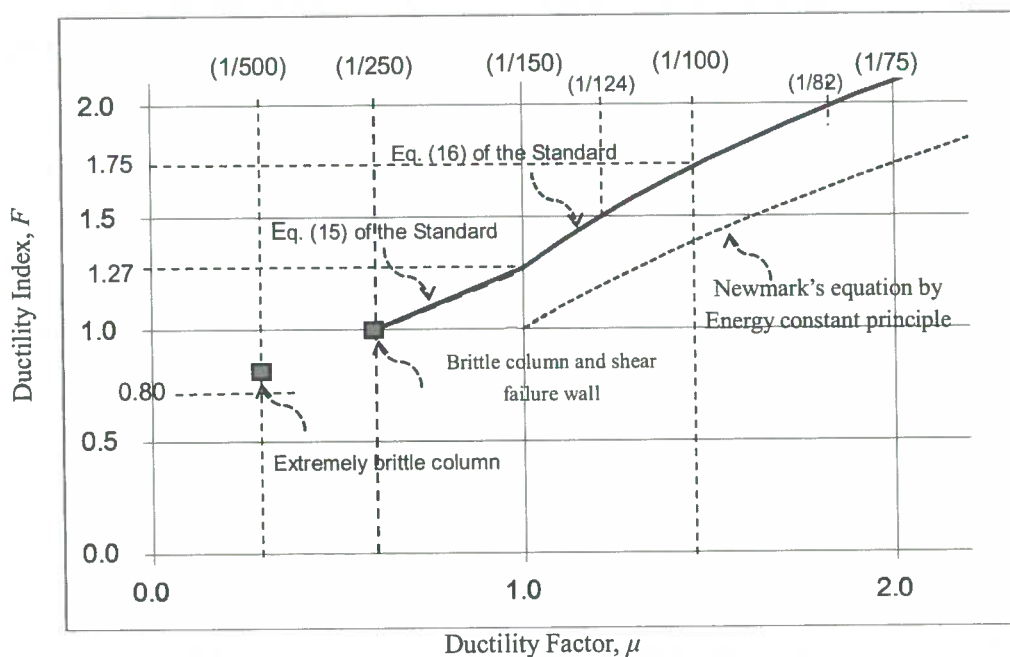
Following supposition on ultimate deflections in the Standard is included in the Figure.

Yield storey deflection angle of standard column =  $1/150$

Ultimate storey deflection angle of shear failure wall =  $1/250$

Ultimate storey deflection angle of extremely brittle column =  $1/500$

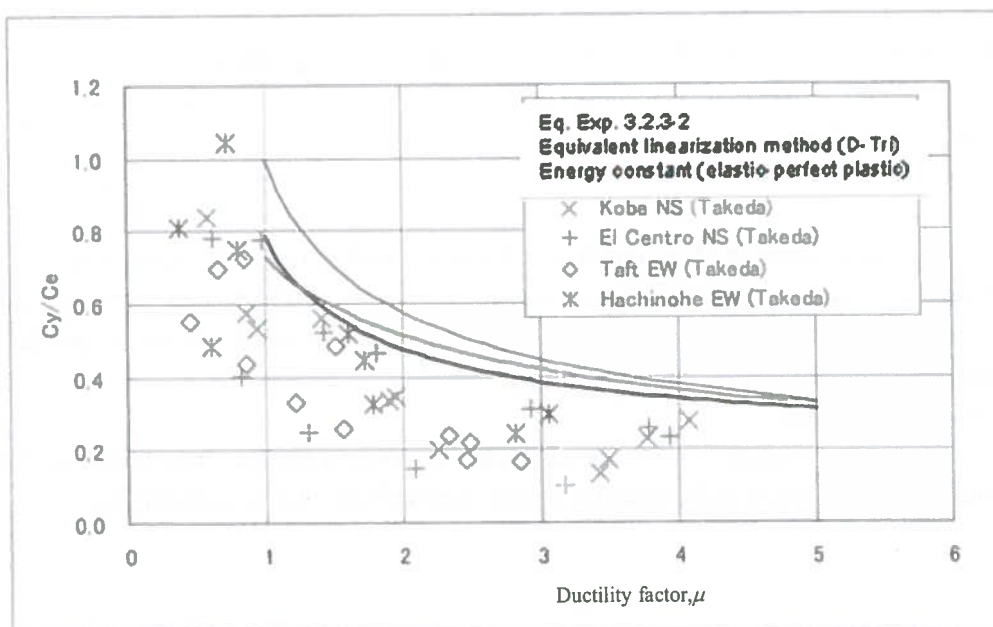
Equation (15), which indicates flexural column of less than  $1/150$  and shear failure column, is also included.



**Figure SA 4.1 Relation of Ductility Factor and Ductility Index**

Source: Commentary figure 3.2.3-7 of the Standard, the range of  $F \leq 2.0$  only is shown.

According to equation (16), ductility index  $F = 1.27$  in case of  $R_{mu} = 1/150$  corresponding to  $\mu = 1.0$ . It is said in the Standard that this value of flexural failure column, 1.27, is reasonable compared with  $F = 1.0$  of RC shear failure wall.



The Standard

**Figure SA 4.2 Comparison of Result of Time-History Analysis and Response Prediction (Commentary Figure 3.3.3-8 of the J. Standard)**

Commentary Figure 3.3.3-8 of the J. Standard shows the Comparison of result of time-history analysis and response prediction. Conditions of the analysis are as follows. Restoring force characteristics is "Takeda model". Damping constant is 2%. Response shear force coefficient  $C_c$  is calculated based on initial stiffness and period  $T_E$ . Input earthquake is standardized as 50kine. The ratio of yield stiffness and elastic stiffness is 0.3, and the ratio of crack strength and yield strength is 0.3. ( $T_E=0.55T_Y$ ;  $T_Y=0.5, 0.75, 1.0\text{sec}$ ). The enveloping curve of the result shows that similar evaluation is given even if different analysis methods are provided.

### B. Ultimate Deformation Capacity of Column

Inter-story drift angle at the ultimate deformation capacity in flexural failure of the column member ( $R_{mu}$ ), is calculated by Eq. (A1.2-1) as follows.

$$R_{mu} = (h_o / H_o) \cdot {}_cR_{mu} \geq R_{250} \quad (\text{A1.2-1}) \quad \text{The J. Standard}$$

In case clear height of column,  $h_o$ , is equal to standard clear height of column,  $H_o$ ,  $h_o/H_o$  is 1.0, and  $R_{mu} =$

$${}_cR_{mu} \cdot {}_cR_{mu} = {}_cR_{my} + {}_cR_{mp} \leq {}_cR_{30} \quad (\text{A1.2-2}) \quad \text{The J. Standard}$$

Where:

$h_o$  = Clear height of column.

$H_o$  = 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 of the Standard.

${}_cR_{mu}$  = 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 of the Standard..

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

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

The plastic drift angle of the column  ${}_cR_{mp}$  is calculated by the following equations.

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

$$q = 1.0 \text{ for } s \leq 100 \text{ mm}$$

$$q = 1.1 \text{ for } s > 100 \text{ mm} \quad (\text{A1.2-4}) \quad \text{The J. Standard}$$

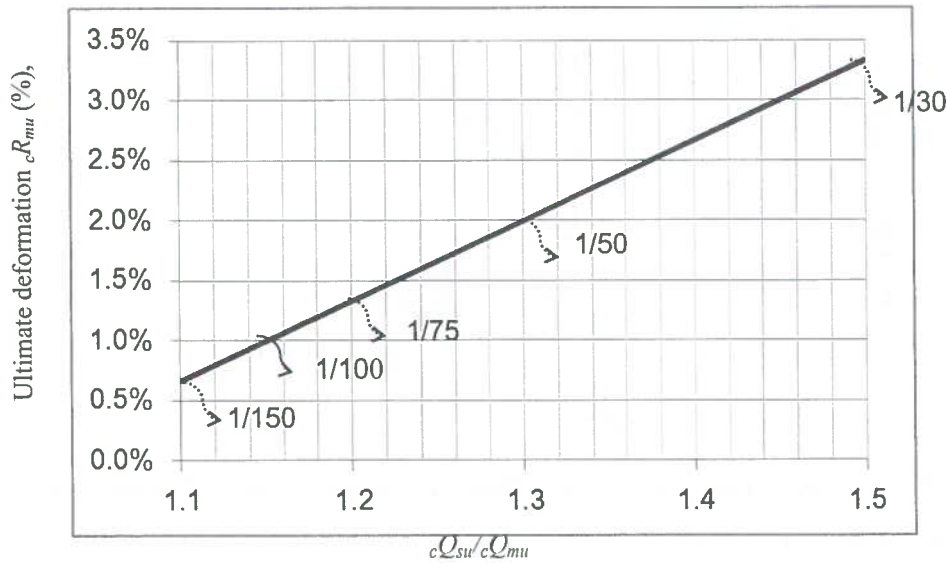
Where:

${}_cQ_{su}$  = 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.

As shown in the above and Figures SA 4.3, the allowance of column against shear failure will decide the plastic drift angle, and drift angle at the ultimate flexural strength of column accordingly.



**Figure SA 4.3** Relation of Ultimate Deformation  $cR_{mu}$  and Allowance against Shear Failure  $cQ_{su}/cQ_{mu}$  in case of  $q=1.1$ ,  $cR_{my}=1/150$  (0.67%)

### C. Upper Limit of the Drift Angle of Flexural Columns

The upper limit of the drift angle of flexural column  $cR_{max}$  is calculated with the following equations, in principle. These are the factor that affects the ductility of columns.

$$cR_{max} = \min\{cR_{max(n)}, cR_{max(s)}, cR_{max(t)}, cR_{max(b)}, cR_{max(h)}\} \quad (\text{A 1.2-5}) \quad \text{The J. Standard}$$

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

$$cR_{max(n)} = cR_{250} \quad \text{for } \eta > \eta_H$$

$$cR_{max(n)} = C R_{30} \cdot \left( \frac{C R_{250}}{C R_{30}} \right)^{n'} \leq C R_{30} \quad \text{for other case} \quad (\text{A 1.2-6}) \quad \text{The J. Standard}$$

Where:

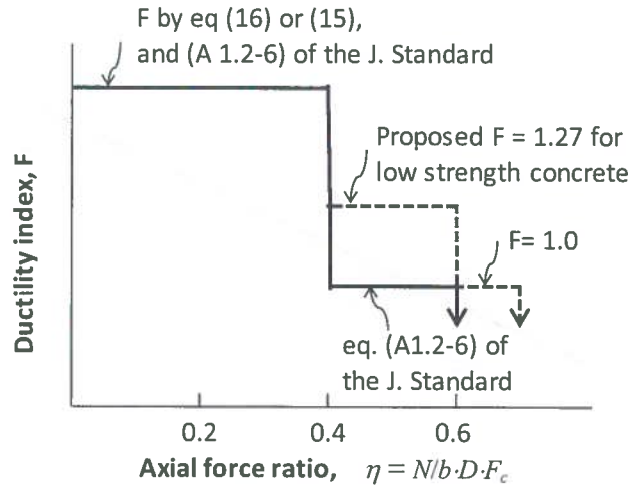
$$n' = (\eta - \eta_L) / (\eta_H - \eta_L)$$

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

$$\eta_L = 0.25 \text{ and } \eta_H = 0.5 \text{ for } s \leq 100 \text{ mm, } s \text{ is interval of column hoop (tie).}$$

$$\eta_L = 0.2 \text{ and } \eta_H = 0.4 \text{ for } s > 100 \text{ mm}$$

It is typical that  $s > 100\text{mm}$  for existing buildings in Bangladesh,  $\eta_H = 0.4$  is used generally.  $\eta_H$  is axial force ratio, and a column of axial force ratio exceeding 0.4, upper limit of drift angle become  $cR_{250}$ , which is  $1/250$ , and ductility index  $F = 1.0$  accordingly. There are some columns designed that exceeds 0.4 of axial force ratio by BNBC 93. In spite of this requirement of the J. Standard incorporating the condition of column design by BNBC 93, it will be allowed that ductility index up to 1.27 for low strength concrete as shown in the Supplement 3 (Proposed Ductility index  $F$  related to axial force ratio) in this manual. Ductility index of a column exceeding 0.6 will be 1.0 ( $1/250$ ). It is noted that ductility index in the range of  $\eta = 0.8$  will be 0.8 or less. Relation of Axial force ratio and upper limit of Ductility index is shown in the Figure SA 4.3.



**Figure SA 4.4 Relation of Axial Force Ratio and Upper Limit of Ductility Index**

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

$$cR_{max(s)} = cR_{250} \quad \text{for } c\tau_u/F_c > 0.2$$

$$cR_{max(s)} = cR_{30} \quad \text{for other case}$$

(A1.2-7) The J. Standard

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

$$cR_{max(t)} = cR_{250} \quad \text{for } p_t > 1.3\%, \text{ refer to Supplement 4 of Evaluation Manual.}$$

$$cR_{max(t)} = cR_{30} \quad \text{for other case}$$

(A 1.2-8) The J. Standard

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

$$cR_{max(b)} = cR_{250} \quad \text{for } s/d_b > 8$$

$$cR_{max(b)} = cR_{30} \quad \text{for other case}$$

(A 1.2-9) The J. Standard

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

$$cR_{max(h)} = cR_{250} \quad \text{for } h_o/D \leq 2.0$$

$$cR_{max(h)} = cR_{30} \quad \text{for other case}$$

(A 1.2-10) The J. Standard

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.

$C_{tu}$  = Shear stress at the column strength.

$$= \min\{cQ_{mu}/(b \cdot j), cQ_{su}/(b \cdot j)\}$$

$cQ_{mu}$  = 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  $0.8 \times D$ .

$p_t$  = 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.

**D. Ductility of Column with Hoop (Tie) of 90 Degree Hook**

Above explanations of A, B, and C are based on columns with hoop (tie) of 135 degree hook. If 90 degree hook is used, some reduction of the ductility will be required. There is no clear data for this condition. Tentative idea is to reduce shear reinforcement ratio to  $0.5P_w$  (reduce to half), in case of 90 degree hook, and evaluate ductility index  $F$  accordingly.

## 2) Axial Force of Column and Deformability

Axial force ratio  $\eta = N/(b \cdot D \cdot F_c)$  is an important factor that affect the deformability of columns, together with the allowance against shear failure. N-M interaction curve and related strain distribution of column section is shown in Figure SA 5.2.

High axial force ratio; Region B shows that tension re-bar will not yield, and low ductility of the section (member) is supposed.

Medium axial force; Region C shows that tension re-bars will yield, and reasonable ductility of the section (member) are supposed.

It is requested to control axial force ratio in the region C which is not larger than 0.4 in case of ordinary shear reinforcement of column. If the concrete is low strength concrete, this axial force ratio will become high, and the ductility will be limited.

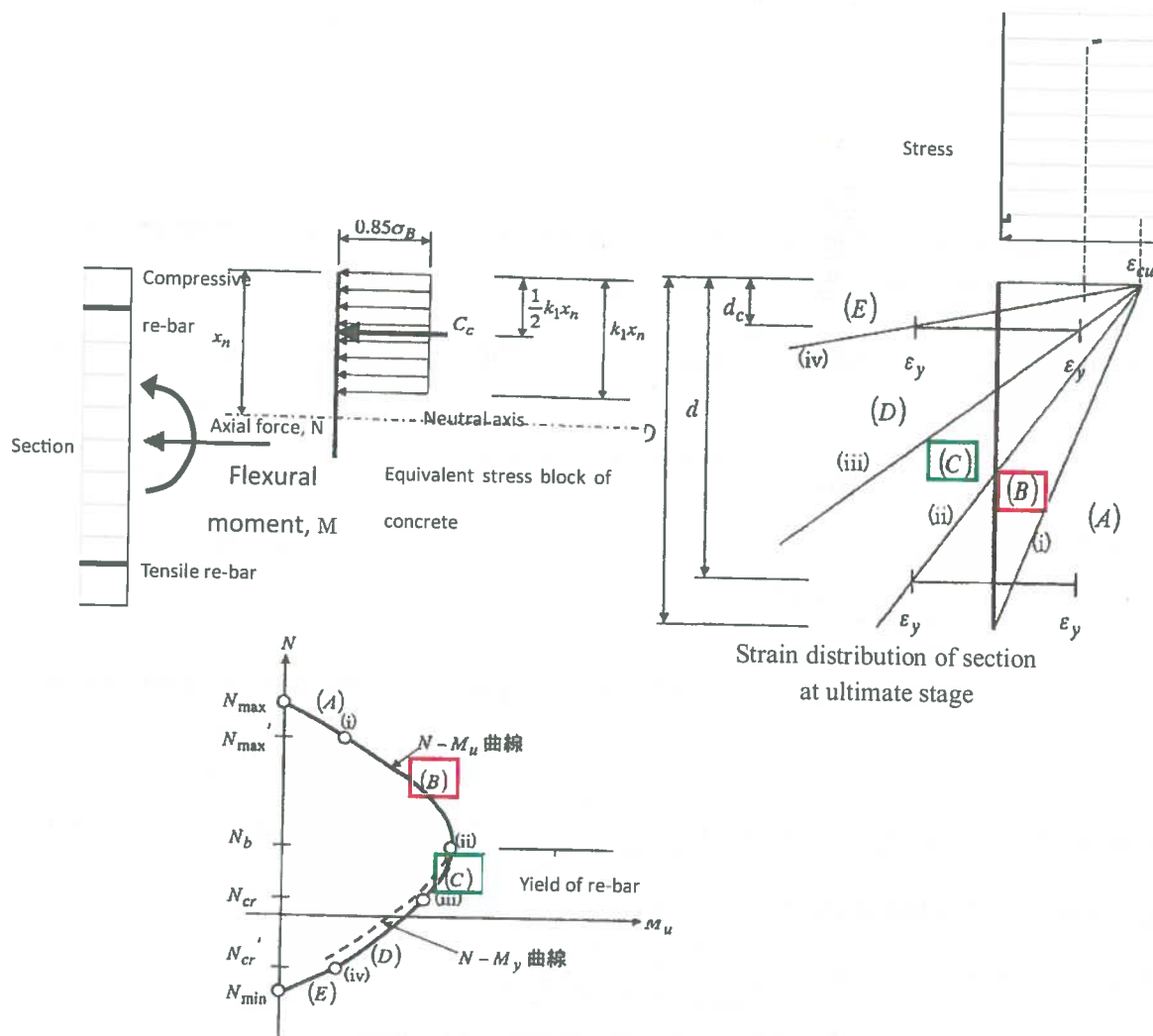


Figure SA 5.2 Axial Force of Column and Deformability

Source: Architectural Institute of Japan, Kanto Branch, "Design of Earthquake Resistant Structures, Chapter 5 Reinforced Concrete Structure, (in Japanese)"

## SUPPLEMENT A6. COMPARISON OF FLEXURAL AND SHEAR STRENGTH OF COLUMN BETWEEN JAPANESE CODE AND BNBC

### 1) Flexural Strength of Typical RC Column

Comparison between Japanese Standard and BNBC93 was made.

#### Example;

Size:  $b \cdot D = 400\text{mm} \times 400\text{mm}$ , Clear span length: 2,400mm,

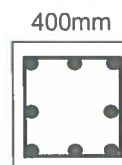
Main bars: 8-22mm  $\phi$  ( $P_t = 0.712\%$ ), Yield strength 400N/mm<sup>2</sup>

Concrete strength: 14N/mm<sup>2</sup>, 9N/mm<sup>2</sup>,

(Tie: 10mm@ 150( $P_w = 0.262\%$ ), 200( $P_w = 0.196\%$ ), 250( $P_w = 0.157\%$ ), Yield strength 280N/mm<sup>2</sup>)

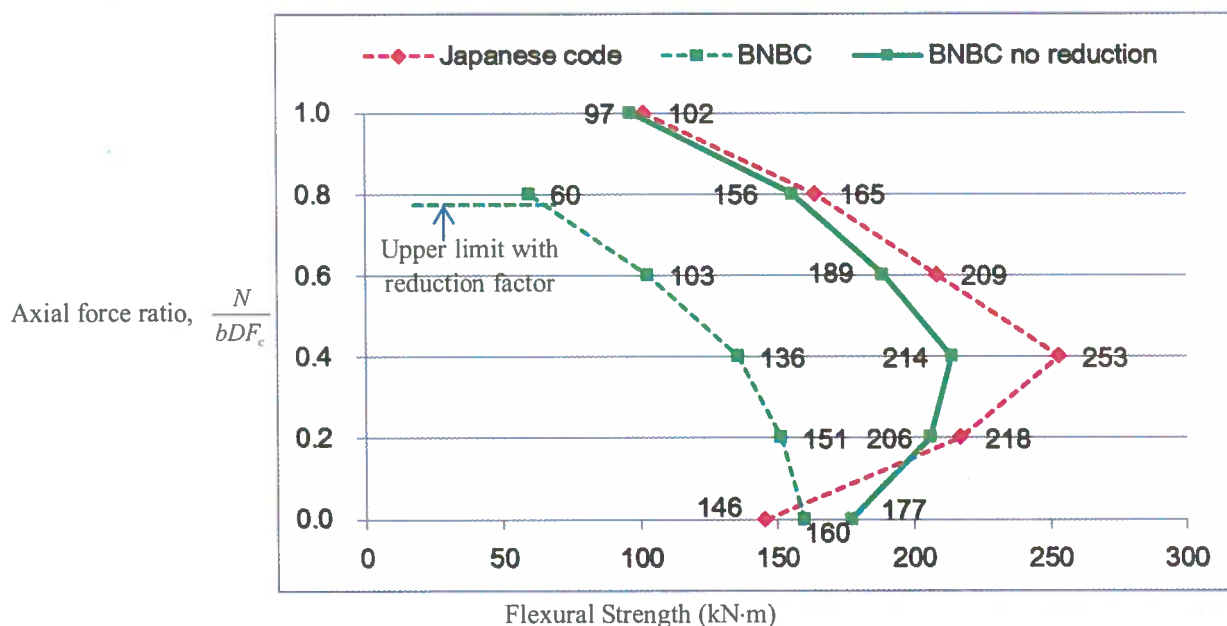
Axial force ratio:  $N/(b \cdot D \cdot F_c) = 0, 0.2, 0.4, 0.6, 0.8, 1.0$

a)  $F_c = 14\text{N/mm}^2$



Main: 8-22mm  $\phi$

Tie: 10mm



b)  $F_c = 9\text{N/mm}^2$

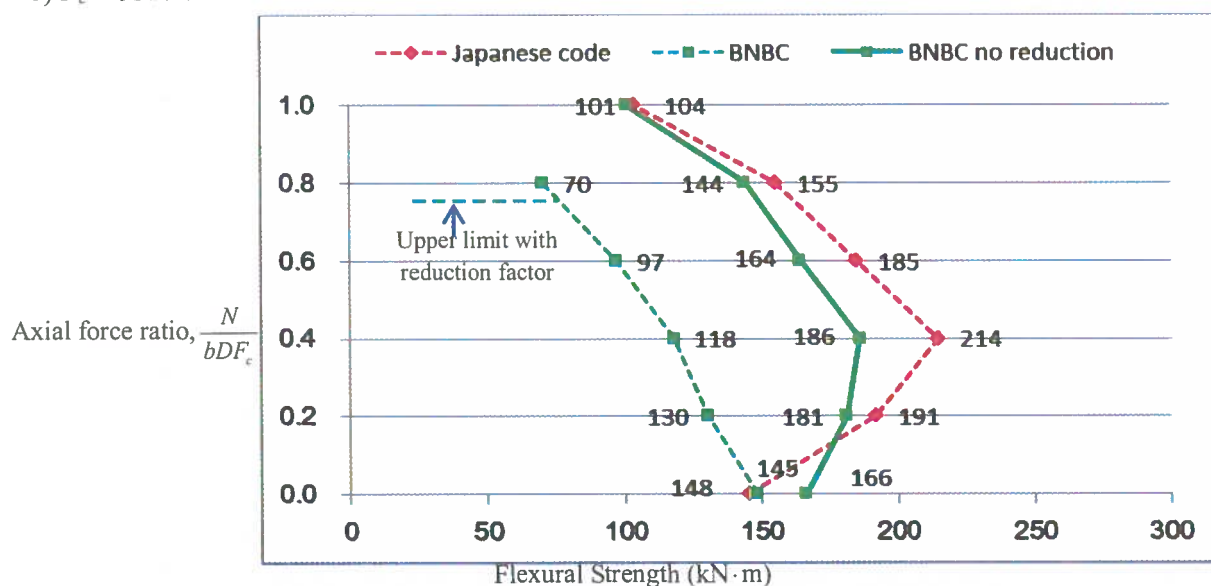


Figure SA 6.1 Comparison of Flexural Strength of Column

### Ultimate Flexural Strength of Column by Japanese Standard

Following equation of the “Standard” was applied.

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

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

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

$$M_u = 0.8a_t \cdot \sigma_y \cdot D + 0.4N \cdot D \quad \text{The J. Standard}$$

(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 ( $\text{mm}^2$ ).

$a_g$  = Total cross sectional area of reinforcing bars ( $\text{mm}^2$ ).

$b$  = Column width (mm).

$D$  = Column depth (mm).

$\sigma_y$  = Yield strength of reinforcing bars ( $\text{N}/\text{mm}^2$ ).

$F_c$  = Compressive strength of concrete ( $\text{N}/\text{mm}^2$ ).

### Flexural Moment Calculation Procedure According to BNBC-93 [Sec-6.3.4, Part-6]

Maximum axial load in column

For members with tie reinforcement conforming to sec 8.1.10.4

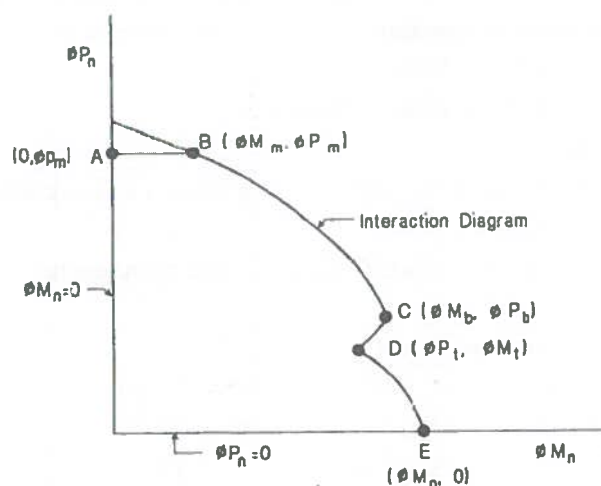
$$\phi P_n(\max) = 0.80\phi [0.85f'_c(A_g - A_{st}) + f_y A_{st}]$$

$\phi = 0.7$  for tied column

$A_g$  = Gross cross sectional area of column

$A_{st}$  = Total area of main reinforcement

For column subject to bending and axial force we usually take the support of Software. Following are the criteria for interaction diagram:



**Figure 6.2 Typical Column Interaction Diagram**

- |         |   |   |              |
|---------|---|---|--------------|
| Point A | : | Point of pure axial compression.  | $= \phi P_m$ |
|         |   | Maximum allowable axial load  | $= 0$        |
|         |   | Associated moment   |              |
| Point B | : | Maximum allowable axial load  | $= \phi P_m$ |
|         |   | Associated moment   | $= \phi M_m$ |
| Point C | : | Point of balanced conditions, i.e. when the compressive strain in concrete reaches 0.003 and the tensile stress in steel reaches $f_y$ simultaneously | $= \phi P_b$ |
|         |   | Axial load  | $= \phi M_b$ |
|         |   | Associated moment   |              |
| Point D | : | Point of transition from a compression member ( $\phi = 0.7$ for tied column, $\phi = 0.75$ for spiral column) to a flexural member ( $\phi = 0.9$ ). |              |
|         |   | Axial load  | $= \phi P_t$ |
|         |   | Associated moment   | $= \phi M_t$ |
| Point E | : | Point of pure flexure   |              |
|         |   | Axial load  | $= 0$        |
|         |   | Associated moment   | $= \phi M_o$ |

## 2) Shear Strength Calculation of Typical RC Column

Comparison of shear strength of RC column between Japanese code and BNBC 93 was made.

**Example;** same section used for the flexural strength.

Size: 400mm × 400mm, Clear span length: 2,400mm,

Main bars: 8-22mm  $\phi$  ( $P_t = 0.712\%$ ), (Yield strength 400N/mm<sup>2</sup>)

Concrete strength: 14N/mm<sup>2</sup>, 9N/mm<sup>2</sup>,

Tie: 10mm@ 150( $P_w = 0.262\%$ ), 200( $P_w = 0.196\%$ ), 250( $P_w = 0.157\%$ ), Yield strength 280N/mm<sup>2</sup>

Axial force:  $N/b \cdot D \cdot F_c = 0, 0.2, 0.4, 0.6, 0.8$

$Q$  (shear force by flexural strength) =  $2 \cdot M_y / h = M_y / 1200$  (half of clear span length)

a)  $F_c = 14\text{N/mm}^2$

N/b · D · F <sub>c</sub>	Shear force by Flexural strength (kN)		Shear strength (kN)					
			Tie @150 P <sub>w</sub> = 0.262%		@200 P <sub>w</sub> = 0.196%		@250 P <sub>w</sub> = 0.157%	
	J. code	BNBC	J. code	BNBC	J. code	BNBC	J. code	BNBC
0.0	121	133 (148)	157	158 (186)	145	137 (161)	136	124 (146)
0.2	181	126 (172)	193	173 (204)	180	152 (179)	172	139 (164)
0.4	211	113 (178)	229	188 (221)	216	167 (196)	208	154 (181)
0.6	174	86 (158)	260	203 (234)	247	182 (214)	238	169 (199)
0.8	137	50 (130)	260	216 (254)	247	195 (229)	238	182 (214)

Note: 1) In case of  $N/b \cdot D \cdot F_c = 0.6$  and  $0.8$ ,  $\sigma_o = 6\text{N/mm}^2$  was used as upper limit for Japanese code.

2) Maximum allowable axial load for this section is  $0.77b \cdot D \cdot F_c$  by BNBC, in case  $\phi$  factor is used.

3) Strength reduction factor 0.85 was used for the shear strength calculation by BNBC. The values shown in parenthesis with red color are without strength reduction factor.

b)  $F_c = 9\text{N/mm}^2$

N/b · D · F <sub>c</sub>	Shear force by Flexural strength (kN)		Shear strength					
			Tie @150 P <sub>w</sub> = 0.262%		@200 P <sub>w</sub> = 0.196%		@250 P <sub>w</sub> = 0.157%	
	J. code	BNBC	J. code	BNBC	J. code	BNBC	J. code	BNBC
0.0	121	123 (138)	110 (147)	144 (169)	100 (134)	123 (145)	94 (126)	110 (129)
0.2	160	108 (151)	127 (170)	151 (178)	118 (158)	130 (153)	111 (149)	118 (139)
0.4	179	98 (155)	144 (194)	159 (187)	135 (180)	138 (162)	128 (172)	125 (147)
0.6	154	81 (137)	162 (216)	167 (196)	152 (203)	146 (172)	146 (195)	133 (156)
0.8	129	58 (120)	162 (216)	175 (206)	152 (203)	154 (181)	146 (195)	141 (166)

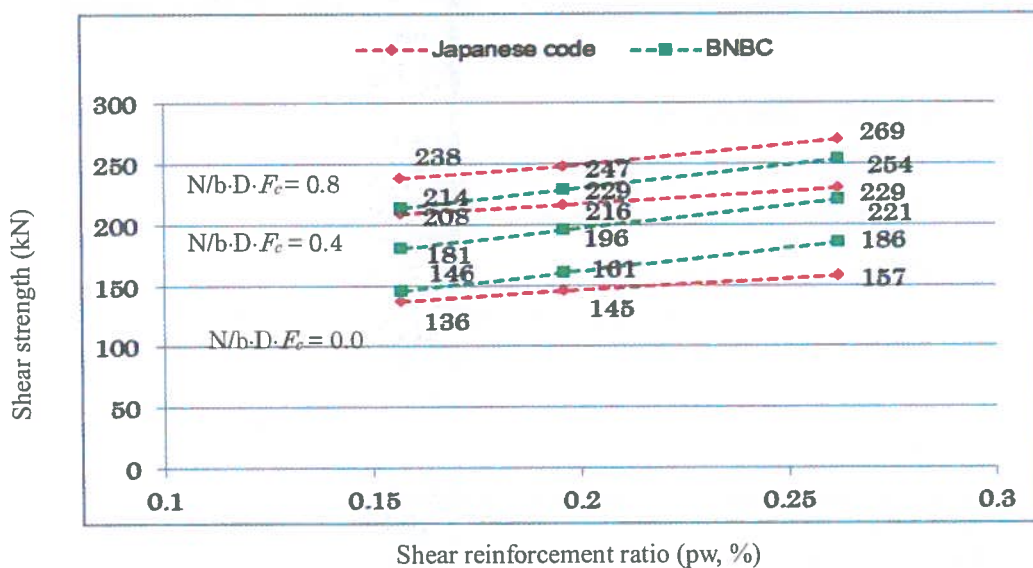
Note: 1) Strength reduction factor 0.85 was used for the strength calculation by BNBC. The values shown in parenthesis with red color are without strength reduction factor.

2) Japanese Standard covers concrete not less than 13.5N/mm<sup>2</sup>. Reduction factor  $K_r$  is applied for low strength concrete in this Manual. The values shown in parenthesis in Japanese code are without reduction factor, and reduced value is used for comparison in case of low strength concrete.

### 3) Comparison of Shear Strength of Typical RC Column

a)  $F_c = 14\text{N/mm}^2$

Shear strength without reduction factor of BNBC 93 is shown. Shear strength at higher axial force ratio especially low shear reinforcement ratio of Japanese code is higher than those of BNBC 93.



b)  $F_c = 9\text{N/mm}^2$

Shear strength without reduction factor of BNBC is shown. Reduction factor for low strength concrete  $K_r$  ( $= 0.056 \cdot \sigma_o + 0.244 = 0.748$ ) is applied for Japanese code, and there is clear difference, and will be safety side. Refer to  $K_r$  of Section 3.4 for information.

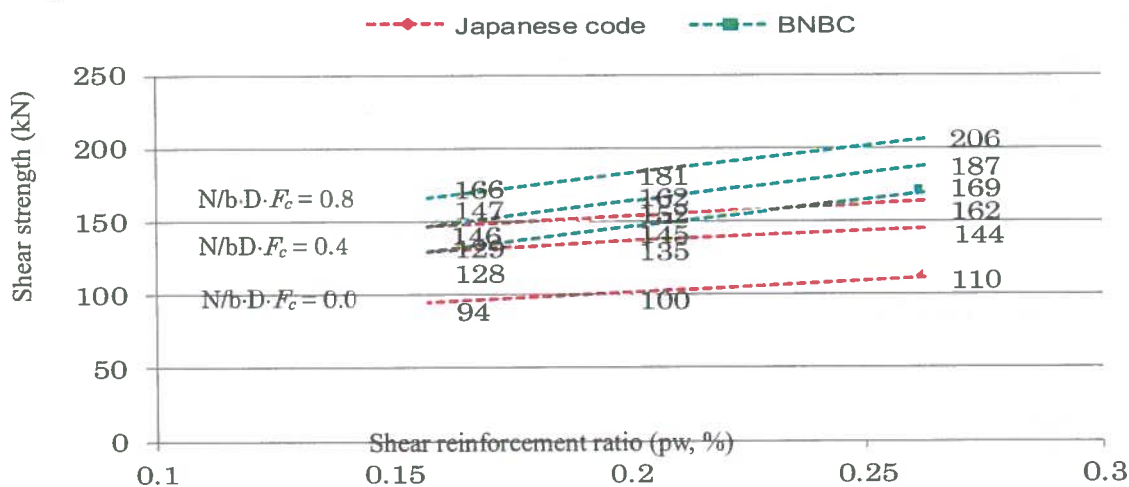


Figure SA 6.3 Comparison of Shear Strength of Column

#### 4) Ultimate Shear Strength of Column, By Japanese Standard

Following equation of the "Standard" was applied.

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

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

where:

$p_t$  = Tensile reinforcement ratio (%).

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

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

$\sigma_0$  = Axial stress in column (N/mm<sup>2</sup>).

$d$  = Effective depth of column.  $D-50\text{mm}$  may be applied.  $\sigma_0 \leq 8\text{N/mm}^2$

$\frac{M}{Q}$  = Shear span length. Default value is  $\frac{h_0}{2}$ .  $M/(Q \cdot d) \leq 3.0$

$h_0$  = Clear height of the column.

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

$b$  = Width of column (mm)

(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  $\sigma_0$  is greater than  $8\text{N/mm}^2$ , the value of  $\sigma_0$  shall be  $8\text{N/mm}^2$  in using Eq. (A1.1-2).

#### Shear Strength Calculation Procedure According to BNBC-93 [Sec-6.2.7, Part-6]

Design for shear shall be based on

$$V_u \leq \phi V_n$$

$$V_n = V_c + V_s$$

Where,

and  $\phi = 0.85$

For members subject to axial compression, in addition to flexure and shear

$$V_c = 0.17 \left( 1 + 0.073 \frac{N_u}{A_g} \right) \sqrt{f'_c} b_w d$$

$$V_s = \frac{A_v f_y d}{s}$$

$N_u$  = Axial load

$A_g$  = Gross area of section

$b_w$  = web width

$d$  = distance from extreme compression fiber to the centroid of tensile reinforcement

$A_v$  = Area of shear reinforcement within a distances

$S$  = Spacing of shear reinforcement

## SUPPLEMENT A7. SHEAR STRENGTH OF BEAM COLUMN JOINT

Shear strength of beam column joint sample calculations based on Japanese code and BNBC93 are shown below:

### 1) A Study Based on Japanese Design Guidelines

Architectural Institute of Japan, “Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings Based on Inelastic Displacement Concept” is used for sample calculation.

Shear strength of beam column joint is calculated using following equation,

$$V_{ju} = K \cdot \phi \cdot F_j \cdot b_j \cdot D_j \quad (8.3.1) \quad (\text{Design Guidelines})$$

Where,  $\kappa$ : Coefficient by the configuration of beam column joint

$$\kappa = 1.0 + \text{type}$$

$$\kappa = 0.7 \text{ T type}$$

$$\kappa = 0.5 \text{ L type}$$

$\phi$ : Adjustment coefficient with/ without orthogonal beam

$$\phi = 1.0 \text{ with orthogonal beam at both side}$$

$$\phi = 0.85 \text{ other cases}$$

$F_j$ : Standard strength of shear at beam column joint

$$F_j = 1.6 \times \sigma_B^{0.7} \text{ (kg/mm}^2\text{)}$$

$$F_j = 0.8 \times \sigma_B^{0.7} \text{ (N/mm}^2\text{)}$$

$\sigma_B$ : Compressive strength of concrete

$D_j$ : Column depth or horizontal projected length of 90 degree bending re-bar  $b_j$  is effective width of beam,

$$b_j = b_b + b_{a1} + b_{a2} \quad (8.3.2)$$

Where,  $b_b$  is beam width,  $b_{a1}$  is smaller value of  $b/2$  and  $D/4$ , and  $b_i$  is the parallel distance from beam surface to edge of column, and  $D$  is column depth.

#### a) Shear force at beam/column yield (Design shear force) $V_j$

$$V_j = T + C_s' + C_c' - V_c = T + T' - V_c \quad (\text{Exp 8.3.1})$$

Column shear force  $V_c$

Beam failure,

$$V_c = 2 \cdot \frac{M_b \cdot L_b / L + M_b' \cdot L_b' / L}{L_c + L_c'} = \frac{(M_b + M_b') \cdot L_b / L}{L_c} \quad (\text{Exp.8.3.2})$$

Where,  $M_b, M_b'$ : Flexural moment of beam at end of beam

$L_b, L_b'$ : Beam span length at each side

$L, L'$ : Beam clear span length at each side

$L_c, L_c'$ : Column length at upper and lower side

Column failure,

$$V_c = \frac{2 \cdot M_c}{L}$$

Where,  $L$  = clear length of column

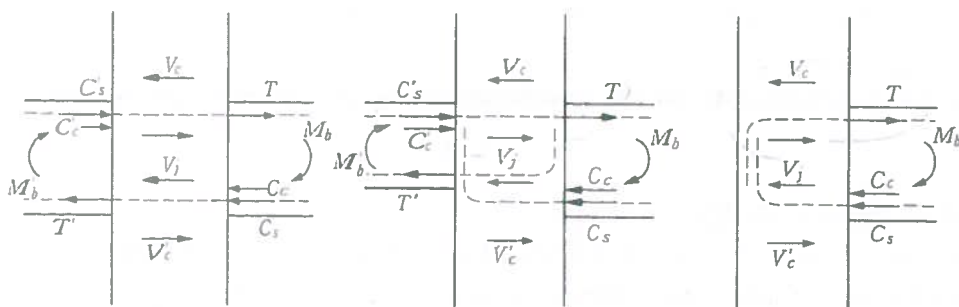


Figure SA 7.1 Shear Force at Joint (Fig. 8.3.1 of Design Guideline\*)

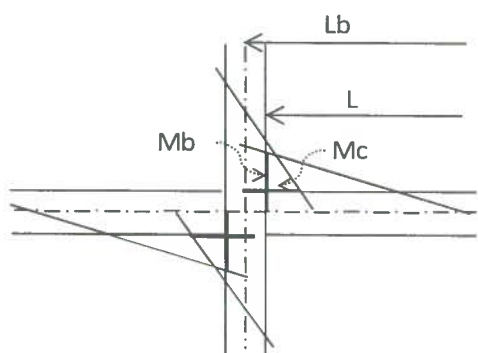


Figure SA 7.2 Flexural Moment of Beam and Column

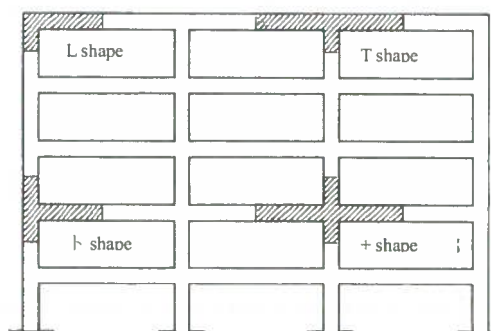


Figure SA 7.3 Classification of Beam Column Joint (Fig. 8.3.2 of Design Guidelines\*)

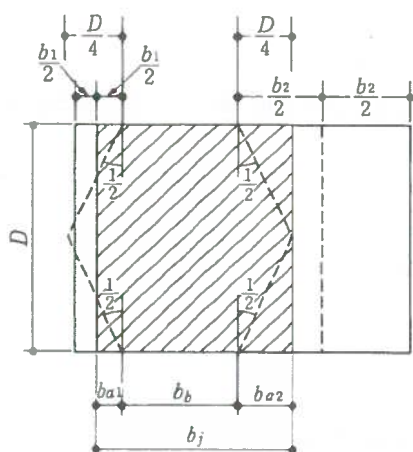


Figure SA 7.4 Effective Width of Joint (Fig. 8.3.3 of Design Guidelines\*)

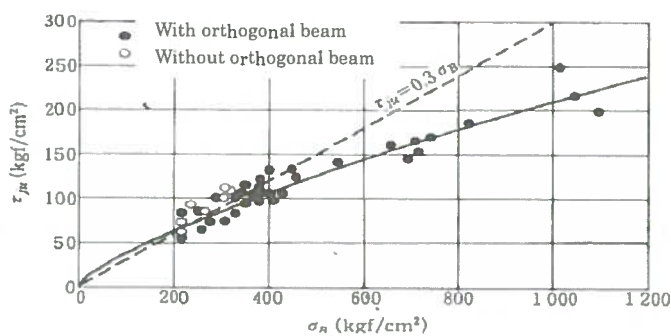
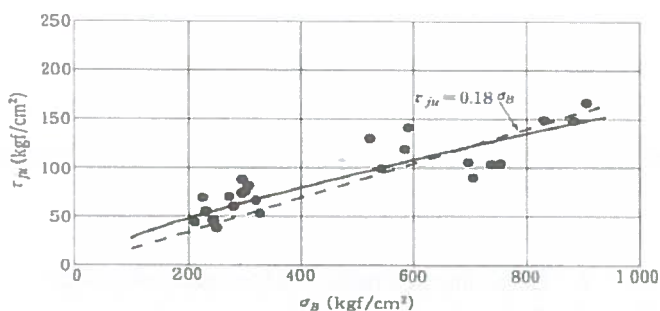
(a)  $\tau_{ju} - \sigma_B$  relation of + type joint (8.3.4)(b)  $\tau_{ju} - \sigma_B$  relation of - type joint

Figure SA 7.5 Shear Strength of Joint, which Cause Shear Failure (Design Guideline\*, Paper by Prof. Morita, others)

\* Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings Based on Inelastic Displacement Concept

Source: Architectural Institute of Japan.

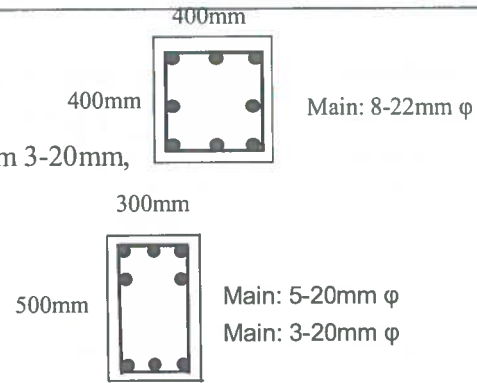
**Example member** (a typical mid-rise building)Column section, 400mm × 400mm, main 8-22mm $\phi$ ,

Beam section, 300mm × 500mm, top main bar 5-20mm, bottom 3-20mm,

Yield strength 400N/mm<sup>2</sup>Concrete strength:  $F_c = 14\text{kN/mm}^2$ ,  $F_c = 9\text{kN/mm}^2$ 

Storey height: 3m

Column span: 5m

**Example calculation**, one way direction only has been evaluated at here. $\kappa = 1.0$  (+ type), 0.7 (┐ type) $\phi = 1.0, 0.85$  $F_j = 0.8 \times \sigma B^{0.7} \text{ (N/mm}^2\text{)}$ 

$$= 0.8 \times 14^{0.7} = 0.8 \times 6.34 = 5.07 \text{ kN/mm}^2 \text{ (} F_c \text{ 14N/mm}^2\text{)}$$

 $b_j = b_b + b_{a1} + b_{a2}$ 

$$= 300 + 50/2 + 50/2 = 350 \text{ mm}$$

 $D_j = 400 \text{ mm}$  $F_c = 14 \text{ N/mm}^2$ ,  $F_j = 5.07 \text{ N/mm}^2$ 

$V_{ju} = \kappa \phi F_j b_j D_j$	$V_{ju} \text{ (kN)}$	$V_j = T + T' - V_c$	$V_j \text{ (kN)}$	$V_{ju} / V_j$
$\kappa = 1.0, \phi = 1.0$ (+ type, orthogonal beam)	709	Beam hinge ( $\frac{N}{b \cdot D \cdot F_c} = 0.4, 0.6$ )  Column hinge ( $\frac{N}{b \cdot D \cdot F_c} = 0.8$ )	859 783	$0.825 < 1.0$ $0.905 < 1.0$
$\kappa = 1.0, \phi = 0.85$ (+ type, no orthogonal beam)	603	Beam hinge ( $\frac{N}{b \cdot D \cdot F_c} = 0.4, 0.6$ )  Column hinge ( $\frac{N}{b \cdot D \cdot F_c} = 0.8$ )	859 783	$0.702 < 1.0$ $0.770 < 1.0$
$\kappa = 0.7, \phi = 1.0$ (┐ type, orthogonal beam)	496	Beam hinge	537	$0.924 < 1.0$
$\kappa = 0.7, \phi = 0.85$ (┐ type, no orthogonal beam)	422	Beam hinge	537	$0.786 < 1.0$

 $V_{ju}$ : Shear strength of beam column joint $\kappa$ : Coefficient by the configuration of beam column joint $\phi$ : Adjustment coefficient with/ without orthogonal beam $V_j$ : Shear force at beam/ column hinge formation $F_j$ : Standard strength of shear at beam column joint

$F_c = 9.0 \text{ N/mm}^2$ ,  $F_j = 3.72 \text{ N/mm}^2$ , assume equation  $V_{ju}$  can be applied for low strength concrete.

$V_{ju} = \kappa \cdot \varphi \cdot F_j \cdot b_j \cdot D_j$	$V_{ju} \text{ (kN)}$	$V_j = T + T' \cdot V_c$	$V_j \text{ (kN)}$	$V_{ju} / V_j$
$\kappa = 1.0, \varphi = 1.0$ (+ type, orthogonal beam)	521	Beam hinge ( $\frac{N}{b \cdot D \cdot F_c} = 0.4, 0.6$ )  Column hinge ( $\frac{N}{b \cdot D \cdot F_c} = 0.8$ )	859 730	$0.606 < 1.0$ $0.714 < 1.0$
$\kappa = 1.0, \varphi = 0.85$ (+ type, no orthogonal beam)	443	Beam hinge ( $\frac{N}{b \cdot D \cdot F_c} = 0.4, 0.6$ )  Column hinge ( $\frac{N}{b \cdot D \cdot F_c} = 0.8$ )	859 730	$0.516 < 1.0$ $0.607 < 1.0$
$\kappa = 0.7, \varphi = 1.0$ (+ type, orthogonal beam)	365	Beam hinge	537	$0.680 < 1.0$
$\kappa = 0.7, \varphi = 0.85$ (+ type, no orthogonal beam)	310	Beam hinge	537	$0.577 < 1.0$

$F_c = 18 \text{ N/mm}^2$ ,  $F_j = 6.05 \text{ N/mm}^2$

$V_{ju} = \kappa \cdot \varphi \cdot F_j \cdot b_j \cdot D_j$	$V_{ju} \text{ (kN)}$	$V_j = T + T' \cdot V_c$	$V_j \text{ (kN)}$	$V_{ju} / V_j$
$\kappa = 1.0, \varphi = 1.0$ (+ type, orthogonal beam)	847	Beam hinge ( $\frac{N}{b \cdot D \cdot F_c} = 0.4, 0.6$ )  Column hinge ( $\frac{N}{b \cdot D \cdot F_c} = 0.8$ )	859 817	$0.986 < 1.0$ $1.04 > 1.0$
$\kappa = 1.0, \varphi = 0.85$ (+ type, no orthogonal beam)	720	Beam hinge ( $\frac{N}{b \cdot D \cdot F_c} = 0.4, 0.6$ )  Column hinge ( $\frac{N}{b \cdot D \cdot F_c} = 0.8$ )	859 817	$0.838 < 1.0$ $0.818 < 1.0$
$\kappa = 0.7, \varphi = 1.0$ (+ type, orthogonal beam)	593	Beam hinge	537	$1.10 > 1.0$
$\kappa = 0.7, \varphi = 0.85$ (+ type, no orthogonal beam)	504	Beam hinge	537	$0.939 > 1.0$

Column flexural strength,  $M_c = 284 \text{ kN.m}$  ( $\frac{N}{b \cdot D \cdot F_c} = 0.4$ ),  $228.4 \text{ kN.m}$  ( $= 0.6$ ),  $172.9 \text{ kN.m}$  ( $= 0.8$ )

#### b) Shear Strength of Beam Column Joint

$F_c = 14 \text{ N/mm}^2$

$$F_j = 0.8 \times \sigma_B^{0.7} \text{ (N/mm}^2\text{)}$$

$$= 0.8 \times 14^{0.7} = 0.8 \times 6.34 = 5.07 \text{ N/mm}^2 \text{ (} F_c = 14 \text{ N/mm}^2\text{)}$$

$$V_{ju} = \kappa \cdot \varphi \cdot F_j \cdot b_j \cdot D_j$$

$$= 1.0 \times 0.85 \times 5.07 \times 350 \times 400 = 603.3 \text{ kN (422.3 kN, in case } \kappa = 0.7\text{)}$$

$F_c = 9 \text{ N/mm}^2$ , assume same equation is can be applied for low strength concrete.

$$F_j = 0.8 \times \sigma_B^{0.7} \text{ (N/mm}^2\text{)}$$

$$= 0.8 \times 9^{0.7} = 0.8 \times 6.34 = 3.72 \text{ N/mm}^2$$

$$V_{ju} = \kappa \cdot \varphi \cdot F_j \cdot b_j \cdot D_j$$

$$= 1.0 \times 0.85 \times 3.72 \times 350 \times 400 = 443 \text{ kN (310 kN, in case } \kappa = 0.7\text{)}$$

**c) Shear Force at Beam/Column Yield (Design Shear Force)  $V_j$** 

$$V_j = T + C_s' + C_c' - V_c = T + T' - V_c \quad (\text{Explanation 8.3.1})$$

Beam failure, Column shear force  $V_c$

$$V_c = \frac{2(M_b \cdot L_b / L + M_b' \cdot L_b' / L')}{L_c + L_c'} = \frac{(M_b + M_b') \cdot L_b / L}{L_c} \quad (\text{Explanation 8.3.2})$$

$$V_c = (251 + 151) \times 5.0 / 4.6 / 3.0 \\ = 146 \text{ kN} \quad (251 \times 5.0 / 4.6 / 3 = 91 \text{ kN, } \vdash \text{ type})$$

Column failure,

$$V_c = 2 \times M_c / L \quad (\text{clear length of column})$$

**Flexural strength of beam**

$$M_b = 0.8 \times a_t \times \sigma_y \times D \\ = 251 \text{ kN} \cdot \text{m} \quad (5\text{-}20\text{mm}\phi) \\ = 151 \text{ kN} \cdot \text{m} \quad (3\text{-}20\text{mm}\phi)$$

**Flexural strength of column**, based on previous calculation of Supplement 6.

**Equilibrium at the center of joint**

$$F_c = 14.0 \text{ N/mm}^2$$

$$(M_b + M_b') \times L_b / L = 437 \text{ kN} \cdot \text{m} < 2 \cdot M_c \cdot L_c / L = 2 \times 253 \times 3.0 / 2.5 = 607 \text{ kN} \cdot \text{m} \quad (N = 0.4 \times b \cdot D \cdot F_c) \quad \begin{array}{l} \text{Beam hinge} \\ \text{Beam hinge} \\ \text{Column hinge} \end{array}$$

<	209	= 502	0.6	Beam hinge
>	165	= 396	0.8	Column hinge

(396/437 = 0.91)

**Shear force when beam/column yield**

$$F_c \quad 14 \text{ N/mm}^2$$

$$V_j = T + T' - V_c$$

= (628 (5- $\phi$ 20) + 377 (3- $\phi$ 20)) - 146	$M_c$ is calculated as $N = 0.4 \times b \cdot D \cdot F_c$
( ditto ) - 146	0.6
( ditto ) $\times 0.91$ - 132	0.8

$$= 1005 - 146$$

$$1005 - 146$$

$$915 - 132$$

$$= 859 \text{ kN} > 603 \text{ kN} \quad (V_{ju}, \text{ shear strength of joint of } \vdash \text{ type}) \quad 859/603 = 1.42, \text{ shear failure,}$$

$$859 >$$

$$783 >$$

In case of  $\vdash$  type (Beam hinge)

$$V_j = 628 - 91$$

$$= 537 > 422 \text{ kN} \quad (V_{ju}, \text{ shear strength of joint of } \vdash \text{ type}) \quad 537/422 = 1.27$$

$$537 >$$

## 2) Study on Beam-Column Joint According To BNBC 93

General requirements:

Joint shear capacity shall be checked for special moment frame (SMF) only. During joint shear calculation 25% over strength ( $1.25f_y$ ) of beam main rebar is considered. Strength reduction factor ( $\phi = 0.85$ ) due to shear shall also be implied. Joint shear capacity is also calculated for without over strength of rebar ( $1.0f_y$ ) and without strength reduction factor ( $\phi = 1.0$ ) for comparison with Japanese method. Calculation result is shown in the parenthesis.

A member that frames into a face is considered to provide confinement to the joint if at least three-quarters of the face of the joint is covered by the framing member. Effective joint area ( $A_j$ ) is shown in Figure-3

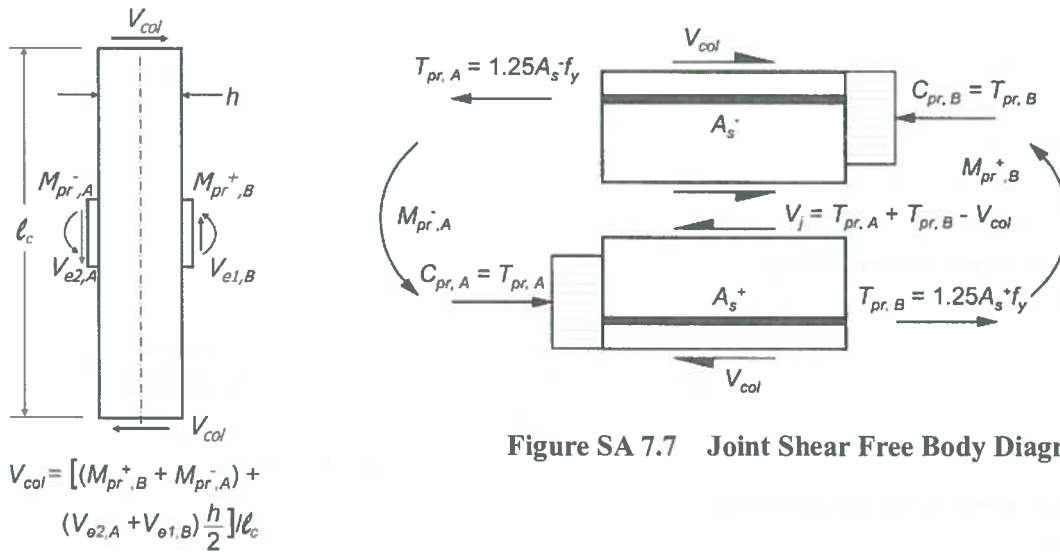


Figure SA 7.7 Joint Shear Free Body Diagram

Figure SA 7.6 Free Body Diagram of Column to Use Column Shear  $V_{col}$

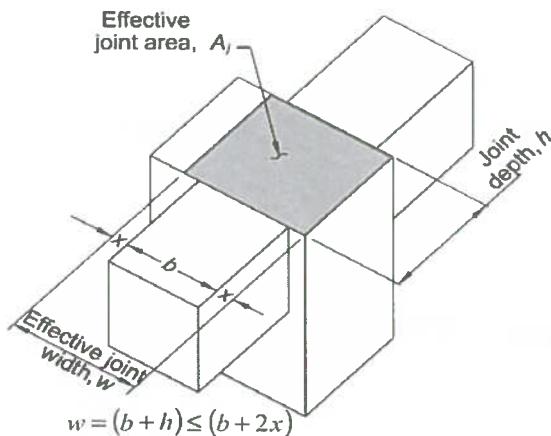


Figure SA 7.8 Definition of Beam-Column Dimension

Tensile Force of Beam Rebar:

$$T_{pr,A} = 5 \times 314 \times 1.25 \times 400 = 785 \text{ kN (628kN)}$$

$$T_{pr,A} = 3 \times 314 \times 1.25 \times 400 = 471 \text{ kN (376.8kN)}$$

Joint Shear for  $f'_c = 14 \text{ MPa}$ 

$$\text{For top bar, } a = 5 \times 314 \times 1.25 \times 400 / (0.85 \times 14 \times 300) = 220 \text{ mm (176mm)}$$

$$\text{So, } M_{pr,A} = 5 \times 314 \times 1.25 \times 400 (400 - 220/2) = 227650 \text{ kN-mm (196000 kN-mm)}$$

$$\text{For bottom bar, } a' = 3 \times 314 \times 1.25 \times 400 / (0.85 \times 14 \times 300) = 132 \text{ mm (105.5mm)}$$

$$\text{So, } M_{pr,B} = 3 \times 314 \times 1.25 \times 400 (400 - 132/2) = 176150 \text{ kN-mm (145900 kN-mm)}$$

$$\text{Beam end shear} = (227650 + 176150) / (5000 - 400) = 877.8 \text{ kN (743.3 kN)}$$

$$V_{col} = (227650 + 176150 + 877.8 \times 400) / 3000 = 146.1 \text{ kN (123.8 kN)}$$

So, Joint force if beam hinge form

$$V_j = 785 + 471 - 146.1 = 1109.9 \text{ kN (881 kN)}$$

Joint Shear for  $f'_c = 9 \text{ MPa}$ 

$$\text{For top bar, } a = 5 \times 314 \times 1.25 \times 400 / (0.85 \times 9 \times 300) = 342 \text{ mm (273.6mm)}$$

$$\text{So, } M_{pr,A} = 5 \times 314 \times 1.25 \times 400 (400 - 342/2) = 179760 \text{ kN-mm (165300 kN-mm)}$$

$$\text{For bottom bar, } a' = 3 \times 314 \times 1.25 \times 400 / (0.85 \times 9 \times 300) = 205 \text{ mm (164.2mm)}$$

$$\text{So, } M_{pr,B} = 3 \times 314 \times 1.25 \times 400 (400 - 205/2) = 158960 \text{ kN-mm (134900 kN-mm)}$$

$$\text{Beam end shear} = (179760 + 158960) / (5000 - 400) = 736.4 \text{ kN (652.6 kN)}$$

$$V_{col} = (179760 + 158960 + 736.4 \times 400) / 3000 = 122.7 \text{ kN (108.8 kN)}$$

So, Joint force if beam hinge form

$$V_j = 785 + 471 - 122.7 = 1133.3 \text{ kN (896 kN)}$$

Shear Strength of Joint

Shear strength capacity of the joint shall not be greater than as specified below –

$$V_{ju} = 1.66 \phi \sqrt{f'_c} \cdot A_j \text{ for joint confined on all four sides}$$

$$= 1.24 \phi \sqrt{f'_c} \cdot A_j \text{ for joint confined on three faces or two opposite faces}$$

$$= 1.0 \phi \sqrt{f'_c} \cdot A_j \text{ for others}$$

Since  $0.75 \times 400$  (column width)  $\leq 300$  (beam width), so the joint is confined.

Since the beam passes through the centre of the column

$$b_j = 400 \text{ mm}$$

$$d_j = 400 \text{ mm}$$

$$A_j = 400 \times 400 = 160000 \text{ mm}^2$$

For  $f'_c = 14 \text{ MPa}$ :

$$V_{ju} = 844.7 \text{ kN (993.8 kN) for joint confined on all four sides}$$

$$= 631.0 \text{ kN (742.4 kN) for joint confined on three faces or two opposite faces}$$

$$= 508.8 \text{ kN (598.6 kN) for others}$$

For  $f'_c = 9 \text{ MPa}$ :

$V_{ju} = 677.3 \text{ kN}$  (796.8 kN) for joint confined on all four sides  
 $= 505.9 \text{ kN}$  (595.2 kN) for joint confined on three faces or two opposite faces  
 $= 408.0 \text{ kN}$  (480.0 kN) for others

For  $f'_c = 14 \text{ MPa}$ :

Joint Type	Joint Capacity, $V_{ju}$ (kN)	Column Axial Force Ratio	Joint Force, $V_j$ (kN)		$V_{ju}/V_j$
			For Beam Hinge	For Column Hinge	
⊕ Type With orthogonal beam	844.7 (993.8)	0.4	1109.9 (881)	×	0.76 (1.12)
		0.6	1109.9 (881)	×	0.76 (1.12)
		0.8	×	757.9 (881)	1.11 (1.12)
⊕ Type Without orthogonal beam	631.0 (742.4)	0.4	1109.9 (881)	×	0.57 (0.84)
		0.6	1109.9 (881)	×	0.57 (0.84)
		0.8	×	757.9 (881)	0.83 (0.84)
⊥ Type With orthogonal beam	844.7 (993.8)	0.4	703.3 (557.7)	×	1.20 (1.78)
		0.6	703.3 (557.7)	×	1.20 (1.78)
		0.8	703.3 (557.7)	×	1.20 (1.78)
⊥ Type Without orthogonal beam	631.0 (742.4)	0.4	703.3 (557.7)	×	0.90 (1.33)
		0.6	703.3 (557.7)	×	0.90 (1.33)
		0.8	703.3 (557.7)	×	0.90 (1.33)

Note:

1. Calculation result shown in the parenthesis is for Japanese method with  $f_y$  in main bar and  $\phi = 1.0$

For  $f'_c = 9 \text{ MPa}$ :

Joint Type	Joint Capacity, $V_{ju}$ (KN)	Column Axial Force Ratio	Joint Force, $V_j$ (KN)		$V_{ju}/V_j$
			For Beam Hinge	For Column Hinge	
⊕ Type With orthogonal beam	677.3 (796.8)	0.4	1133.3 (896)	×	0.60 (0.89)
		0.6	1133.3 (896)	×	0.60 (0.89)
		0.8	×	1071.4 (896)	0.63 (0.89)
⊕ Type Without orthogonal beam	505.9 (595.2)	0.4	1133.3 (896)	×	0.45 (0.66)
		0.6	1133.3 (896)	×	0.45 (0.66)
		0.8	×	1071.4 (896)	0.47 (0.66)
└ Type With orthogonal beam	677.3 (796.8)	0.4	720.8 (568.5)	×	0.94 (1.41)
		0.6	720.8 (568.5)	×	0.94 (1.41)
		0.8	720.8 (568.5)	×	0.94 (1.41)
└ Type Without orthogonal beam	505.9 (595.2)	0.4	720.8 (568.5)	×	0.70 (1.04)
		0.6	720.8 (568.5)	×	0.70 (1.04)
		0.8	720.8 (568.5)	×	0.70 (1.04)

Note:

1. Calculation result shown in the parenthesis is for Japanese method with  $f_y$  in main bar and  $\phi = 1.0$

### 3) Summary

A sample calculation of strength and design shear force for beam column joint was done using Japanese code and BNBC for comparison. The result shows similar trends of shear failure at beam column joint, especially in case of low strength concrete.

#### Beam Column Joint,

1. Shear strength of joint is affected by concrete strength and is proportional to the size of effective width and column depth. It is easy to cause shear failure in case of small column (depth).
2. Shear failure is easy to occur, in case that the quantity (Area of beam main-bar  $\times$  yield strength of re-bar) is big and size of joint is small. Strength will decrease if shear failure or deterioration of the bond of main-bar occurs.

#### From the sample calculation,

1. Shear failure at the joint is easy to occur, in case of low concrete strength.
2. Shear failure of the joint is easy to occur, in case of + type joint compared with └ type joint, and joint without orthogonal beam.
3. (In case that floor slab is ignored), there is difference of beam hinge and column hinge formation by the column strength due to axial force ratio ( $N/b \cdot D \cdot F_c = 0.4, 0.6, 0.8$ ), but there is no clear difference of shear force at the joint.

**Countermeasures,**

1. In case of low strength concrete ( $F_c < 13.5\text{N/mm}^2$ ) or small size column, it will be required to evaluate shear strength of joint and shear force of joint when beam/column hinge occurs for typical members.
2. If the failure of the joint is supposed, the reduction of strength of frames by 2<sup>nd</sup> level screening, which suppose column failure, will be considered.
3. It has been said that the storey deflection angle will be  $R=1/100$  when shear failure of beam column joint occurs. It is recommended to provide strength oriented retrofit such as RC in-filled wall and steel framed brace to reduce horizontal deflection of frames. (AIJ, Chapter 6 Recommendation of assessment for beam column joint, "Seismic Design of RC Structure after the Hanshin. Awaji Earthquake Disaster, 1998", written in Japanese)

Storey deflection angle will be controlled, in case of shear failure type RC in-filled wall, which is  $R=1/250$  ( $F=1.0$ ), and in case of steel framed brace which is  $R=1/124\sim$  ( $F=1.5\sim$ ). It is recommended to control storey deflection angle within  $1/124$  ( $F=1.5$ ) or less in case of low strength concrete.

## SUPPLEMENT A8. COMPARISON OF COMPRESSIVE STRENGTH CALCULATION OF STEEL BRACE BY JAPANESE GUIDELINES AND BNBC

Comparison between Japanese Guidelines and BNBC93 was made.

### Example;

Size: H-200×200×8×12 ( $A = 63.53\text{cm}^2$ ,  $W = 49.9\text{kg/m}$ ,  $i_x = 8.62\text{cm}$ ,  $i_y = 5.02\text{cm}$  (radius of gyration)

$I_x, I_y$  (moment of inertia),

C-260 ×130×12 ( $A = 59.52\text{cm}^2$ ,  $W = 46.8\text{kg/m}$ ,  $i_x = 10.12\text{cm}$ ,  $i_y = 3.96\text{cm}$ ).

Effective length: 1,800mm for  $i_y$ , and 3,600 for  $i_x$ ,

2,200mm for  $i_y$  and 4,400mm for  $i_x$

Material:  $F_y = 345\text{N/mm}^2$

Check  $i_x, i_y$  (radius of gyration), and slenderness ratio  $\lambda_x, \lambda_y$ , then strength evaluation.

**Table SA 8.1 Comparison of Compressive Strength of Steel Brace**

Effective length	H-200×200×8×12					C-260×130×12				
	$\lambda$	Comp. strength(kN)				$\lambda$	Comp. strength(kN)			
		$F_{cr}$ (N/mm <sup>2</sup> )	J. code (kN)	$F_{cr}$ (N/mm <sup>2</sup> )	BNBC (kN)		$F_{cr}$ (N/mm <sup>2</sup> )	J. code (kN)	$F_{cr}$ (N/mm <sup>2</sup> )	BNBC (kN)
Weak axis L=180cm	35.9	326.6	2076	314.7	1661 (1954)	45.5	315.8	1879	297.8	1507 (1773)
Strong axis L= 360cm	41.8	320.3	2035	303.6	1602 (1885)	35.6	327.1	1947	308.5	1561 (1836)
Weak axis L=220cm	43.8	317.9	2019	300.7	1587 (1867)	55.6	301.4	1793	277.0	1401 (1648)
Strong axis L= 440cm	51.0	308.3	1958	285.0	1504 (1769)	43.5	318.3	1894	292.0	1477 (1738)

Note: 1) Critical compressive stress was calculated by Japanese code. See Figure S8.1.

2) Buckling strength using LRFD method for steel section by BNBC.

3) Strength reduction factor 0.85 was used for the strength calculation by BNBC.

4) Inside of ( ) shows the value by BNBC without reduction factor.

5) Section area  $A = 62.08\text{cm}^2$  was used for H-200×200×8×12 by BNBC.

### Compressive Strength of Steel Brace by the Japanese “Guidelines for Seismic Retrofit”

Limit compressive stress of steel bracing is calculated as follows.

$$f_{cr} = \{1 - 0.4(\lambda/\Lambda)^2\} \cdot F \quad \text{for } \lambda \leq \Lambda$$

$$f_{cr} = 0.6F/(\lambda/\Lambda)^2 \quad \text{for } \lambda > \Lambda$$

The J. Guidelines (3.4.5-1)

where:

$f_{cr}$  = Limit compressive stress (N/mm<sup>2</sup>).

$\Lambda$  = Limit aspect ratio ( $\Lambda = \sqrt{(\pi^2 \cdot E)/(0.6F)}$ ).

$\lambda$  = Effective aspect ratio.

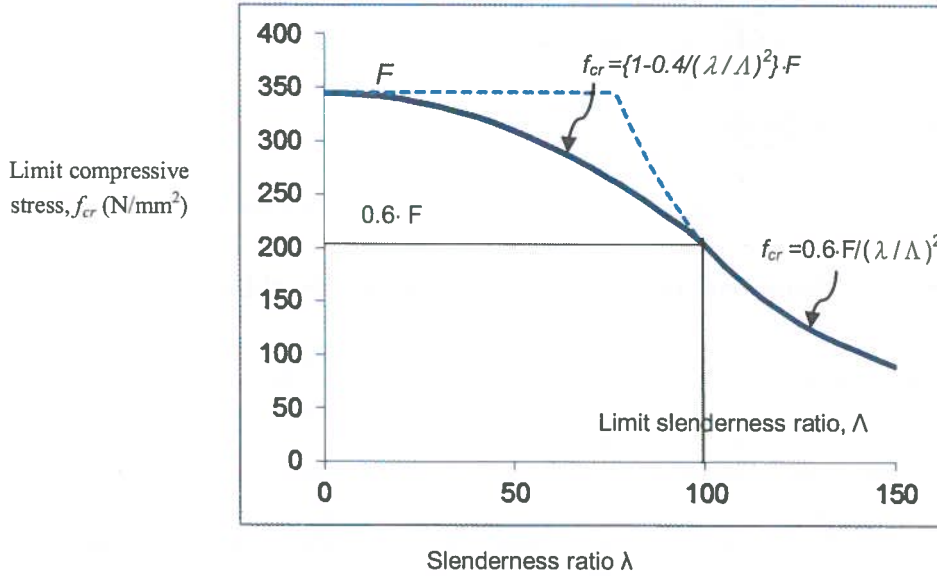
$F$  = Specified strength of steel(N/mm<sup>2</sup>).

$E$  = Young's modulus of steel (N/mm<sup>2</sup>).

Note: 1) Above “Aspect ratio” will also be expressed by “Slenderness ratio”.

Slenderness ratio  $\lambda$  is expressed by effective length / radius of gyration of a member.

- 2) “ $F$  value” will be specified yield strength of steel ( $\text{N/mm}^2$ ),  $F = 345 \text{ N/mm}^2$ .
- 3) Compressive strength is calculated by  $f_{cr} = \{1 - 0.4 \cdot F / (\lambda / \Lambda)^2\} \cdot F$  section area of a compressive member.
- 4) In case of  $F = 345 \text{ N/mm}^2$ ,  $\Lambda$  will be  $\sqrt{(\pi^2 \cdot 205000) / (0.6 \cdot 345)} = 98.9$



**Figure SA 8.1 Limit Compressive Stress of Steel Bracing by Japanese Guideline**

(Note: The idea of Euler's elastic buckling stress is the basis, and the range between  $0.6 \cdot F$  (yield stress) to  $F$  is approximated by a parabolic curve considering yield stress, unavoidable eccentricity and residual stress etc.)

### Buckling Strength of Steel Column by BNBC (LRFD)

#### CALCULATION FOR BUCKLING STRENGTH, BNBC93-Chapter 10, Part 6

$P_n$ ; nominal compressive strength (kN)

$$\phi_c = 0.85$$

$$P_n = 0.001 A_g F_{cr}$$

For  $\lambda_c \leq 1.5$

$$F_{cr} = (0.658 \lambda_c^2) \cdot F_y, \quad \lambda_c = \frac{kl}{r\pi} \sqrt{\frac{F_y}{E}}$$

For  $\lambda_c > 1.5$

$$F_{cr} = \left[ \frac{0.877}{\lambda_c^2} \right] \cdot F_y$$

Where;

$A_g$  = gross area ( $\text{mm}^2$ )

$F_{cr}$  = critical stress ( $\text{N/mm}^2$ )

$\Phi_c$  = resistance factor for compression

$\lambda_c$  = column slenderness parameter

$l$  = actual unbraced length of a member (mm)

$r$  = radius gyration (mm)

$K$  = effective length factor for prismatic member

$E$  = modulus of elasticity of steel ( $200,000 \text{ N/mm}^2$ )

$F_y$  = specified minimum yield strength of the type of steel being used, ( $\text{N/mm}^2$ )

Example:

1. Section: H 200×200×8×12 & C 260×130×12
2. Length: 1800mm for  $i_y$  and 3600 for  $i_x$ , 2200mm for  $i_y$  and 4400 for  $i_x$
3.  $F_y = 345 \text{ N/mm}^2$
4.  $E = 200000 \text{ N/mm}^2$

$A$  = Area of the section

$L_x, L_y$  = Effective length in X and Y direction

$i_x, i_y$  = Moment of inertia in X and Y direction

$r_x, r_y$  = Radii of gyration in X and Y direction

Section	A ( $\text{mm}^2$ )	$L_x$ (mm)	$L_y$ (mm)	$i_x$ ( $\text{mm}^4$ )	$i_y$ ( $\text{mm}^4$ )	$r_x$ (mm)	$r_y$ (mm)	$kix/rx$	$kly/ry$	$\lambda_{cx}$	$\lambda_{cy}$	$F_{cr}(x)$ ( $\text{N/mm}^2$ )	$F_{cr}(y)$ ( $\text{N/mm}^2$ )	$\phi P_n(x)$ (KN)	$\phi P_n(y)$ (KN)
H-200x200x8x12	6208	3600	1800	46047317	16007509	86.12	50.78	41.80	35.45	0.55	0.47	303.6	314.7	1602.1	1660.6
	6208	4400	2200	46047317	16007509	86.12	50.78	51.09	43.32	0.68	0.57	285.0	300.7	1504.1	1586.9
C-260x130x12	5952	3600	1800	50488576	9595585	92.10	40.15	39.09	44.83	0.52	0.59	308.5	297.8	1560.9	1506.8
	5952	4400	2200	50488576	9595585	92.10	40.15	47.77	54.79	0.63	0.72	292.0	277.0	1477.0	1401.3

Note: 1) Strength reduction factor 0.85 was applied.

### Horizontal Stiffness of Steel Brace

Simplified calculation of horizontal elastic stiffness of steel brace only is shown below.

Horizontal deflection of steel brace is  $\delta$ , then elongation of brace is  $\Delta S = \delta \cdot \cos\theta$

Horizontal force is  $P$ , then force of brace is  $P_B = P / \cos\theta$

Length of brace is  $S$ , section area is  $A$ , Young's Modulus is  $E$ , then

$$E = (P_B / A) / (\Delta S / S) = P \cdot S / (A \cdot \delta \cdot \cos\theta \cdot \cos\theta) = P \cdot L / (A \cdot \delta \cdot \cos^3\theta)$$

$$\cos\theta = L / S$$

Horizontal stiffness  $K$  is expressed by,

$$K = P / \delta = A \cdot E \cdot \cos^3\theta / L$$

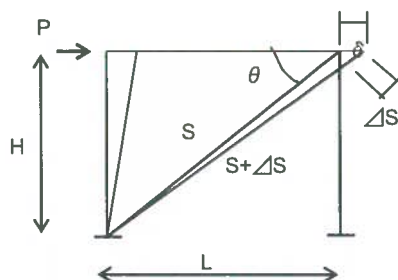


Figure SA 8.2 Load And Deflection of Steel Brace (1)

In case tension brace and compression brace is provided like V shape, K is twice of above calculation.

< Example >

V shape brace with tension and compression member,  $H = 2.85\text{m}$ ,  $L = 3.23\text{m}$ ,  $\theta = 41.47^\circ$   $\cos 41.47 = 0.749$ ,  
 $2\text{L}-130 \times 13 \times 12$  ( $A = 29.76 \text{ cm}^2$ ),

$$K = P/\delta = 2 \cdot A \cdot E \cdot \cos^3 \theta / L = 2 \times 29.76 \times 2 \times 20,580 \times 0.749^3 / 323 = 3,192 \text{ kN/cm} = 319.2 \text{ kN/mm}$$

In case of  $2\text{L}-100 \times 100 \times 12$  ( $A = 2 \times 22.54 \text{ cm}^2$ ),

$$K = 241.8 \text{ kN/mm}$$

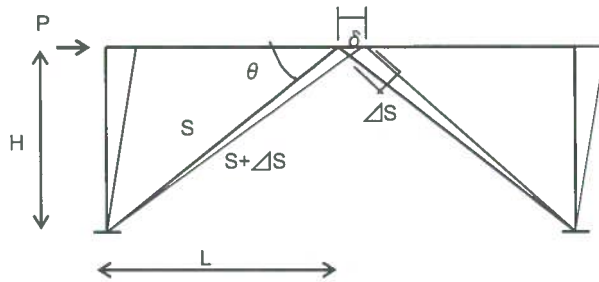


Figure SA 8.3 Load And Deflection of Steel Brace (2)

## SUPPLEMENT A9. RESPONSE MODIFICATION FACTOR AND OVER-STRENGTH FACTOR

Response modification factor  $R$  and over strength factor  $\Omega_o$  used for the calculation of design seismic load is indicated in ASCE7-10 and UBC.  $R$  value is shown in case of reinforced concrete moment frame in the Table SA9.1. Over-strength factor of 3 is taken in this table, but it will be required to investigate this value for buildings in Bangladesh. In this figure, the value  $R_d (= R/\Omega_o)$  will be similar concept and can be compared with Ductility index  $F$  of Japanese code.

**Table SA 9.1 Response Modification Factor:  $R$  and over-Strength Factor:  $\Omega_o$**   
Reinforced Concrete Moment Frames

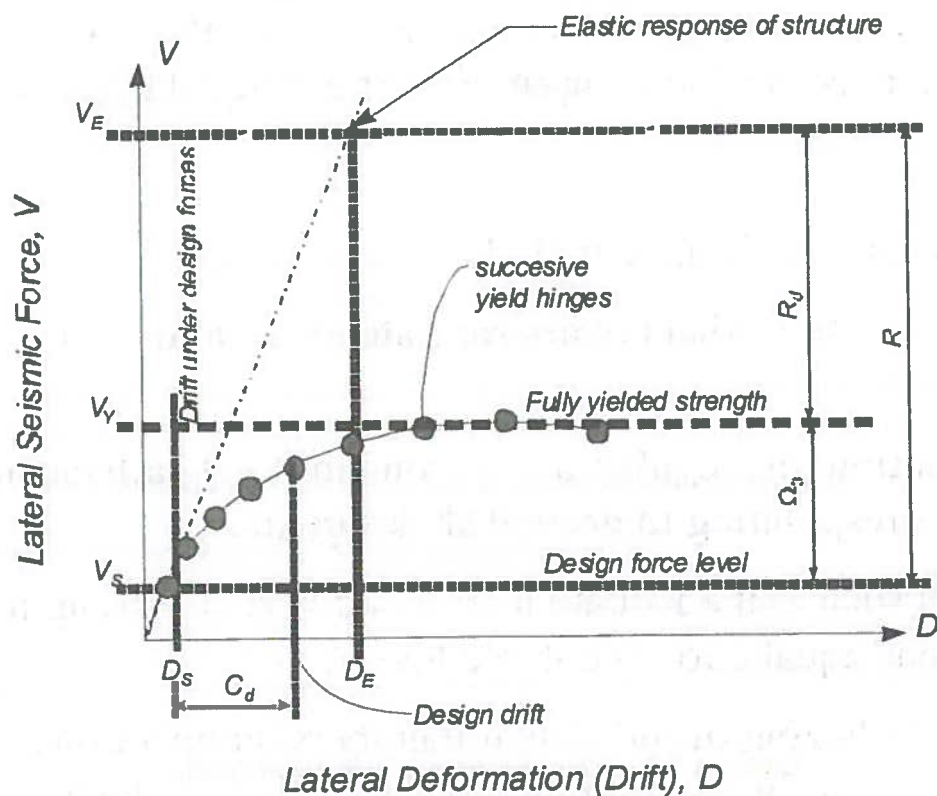
	UBC94	UBC97			ASCE 7-10		
	$R^1$	$R$	$\Omega_o$	$R_d = R/\Omega_o$	$R^2$	$\Omega_o^3$	$R_d = R/\Omega_o^4$
Special moment frame	12	8.5	2.8	3.06	8	3	2.67
Intermediate moment frame	8	5.5	2.8	1.96	5	3	1.67
Ordinary moment frame	5	3.5	2.8	1.25	3	3	1.0

Notes: 1. Value of  $R$  is same in the BNBC 1993.

2. Value of  $R$  is same in the BNBC 2015.

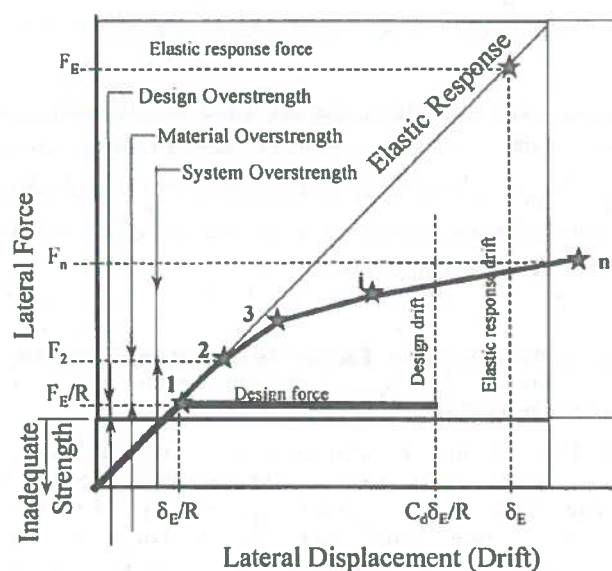
3.  $\Omega_o$  Needs further study to use in Bangladesh.

4.  $R_d$  and Ductility Index  $F$  of Japanese code is a similar concept.



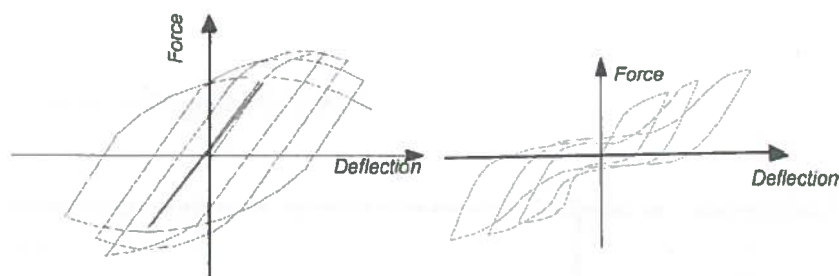
Source: FEMA 450-2/ 2003 Part 2 Commentary Figure C 4.2-1

**Figure SA 9.1 Inelastic Force-Deformation Curve.**



Source: FEMA 450-2/ 2003 Part 2 Commentary Figure C 4.2-3

**Figure SA 9.2 Factors Affecting over Strength.**



a. Ductile hysteris loops

b. Pinched hysteris loops

Source: FEMA 450-2/ 2003 Part 2 Commentary Figure C 4.4-2

**Figure SA 9.3 Typical Hysteretic Curves**

According to BNBC2015, Response Reduction Factor,  $R$  and deflection amplification factor,  $C_d$  is shown in Table 6.2.19. Allowable Storey Drift Limit (part) is shown in Table 6.2.21 for information.

**Table SA 9.1 Response Reduction Factor,  $R$  and Deflection Amplification Factor,  $C_d$  (part)**

[From part 6, BNBC 2015 Table 6.2.19]

	Response Reduction Factor, $R$	Deflection Amplification Factor, $C_d$
Special reinforced concrete frames	8	5.5
Intermediate reinforced concrete frames	5	4.5
Ordinary reinforced concrete frames	3	2.5

**Table SA 9.2 Allowable Storey Drift Limit (part).**

[From part 6, BNBC 2015 Table 6.2.21]

	Importance Category I	II	III
Structure, 4 stories or less	$0.025 h_{sx} = h_{sx} / 40$	$0.020 h_{sx} = h_{sx} / 50$	$0.015 h_{sx} = 1/67$
Masonry shear wall structure	$0.007 h_{sx} = h_{sx} / 143$	$0.007 h_{sx} = h_{sx} / 143$	$0.007 h_{sx} = h_{sx} / 143$
Typical structures	$0.020 h_{sx} = h_{sx} / 50$	$0.015 h_{sx} = 1/67$	$0.01 h_{sx} = 1/100$

Notes:  $h_{sx}$  is the storey height below Level  $x$

**SUPPLEMENT A10. SUMMARY OF TEST WORK**

Sample construction of several retrofit methods was done by CNCRP in 2011 and 2012. Outline of each method has been introduced in Table SA10.1.

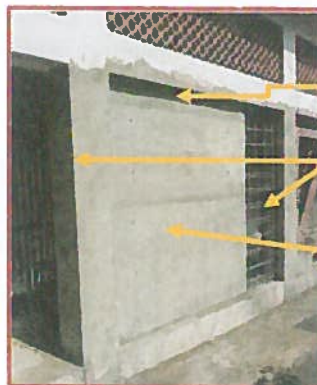
**Table SA 10.1 Summary of Retrofit Methods Executed by CNCRP as “Test Work”**

SL	Retrofit method	Short dissipation
1	Concrete jacketing of column (Figure S10.1, Method-1)	Jacketing on existing RC column done increase in size. Surface of the column is stripped of plaster, re-bars (both vertical shear) are added around and concreting is done. The top of the column is finished with mortar grouting to avoid shrinkage. Increases strength and ductility of column.
2	RC shear wall (Figure S10.1, Method-2)	An open frame may be fully filled by reinforced concrete wall. Post-installed anchors are installed first on all around the existing frame. Re-bars are placed as per design then casting is done. It increases strength.
3	RC wing wall (Method-3)	Partial RC wall is added to on existing column in the same manner as described above to increase the strength of column.
4	Steel braced frame (In frame) Method-4	Steel frame braces are inserted into an existing R.C frame. This is an alternate of R.C shear wall. Post installed in the R.C frame and bolted studs are provided in the steel frame brace. The gap is filled by mortar grout. It is good practice to provide steel spiral or ladder in the gap between R.C frame and steel brace where anchor bolts overlap. This spiral helps prevent splitting of infill mortar grout increases strength.
5	Carbon fiber sheet wrapping	Existing column may be jacketed with carbon fiber sheets to improve ductility. Column to be jacketed is first stripped off mortar, corners of the columns are made smooth to increase proper wrapping chemical adhesive is applied on the surface and carbon fiber is wrapped tightly around. Usually mortar finishing is given for protection of fiber.
6	Slits on the brick wall	Slits (crevices) are made between columns and attached standing walls to prevent short columns. It improves ductility of the column and may give better stability to the whole structure. Steel angle supporting member is provided on brick walls to prevent overturning of the wall itself.
7	External steel braced frame (Figure 7)	Sometime steel frame bracing is installed at the outside of building frame for architectural reason such as to maintain an existing window. There are two methods: i) Direct connection using anchor bolt. ii) Indirect connection using grout mortar with post installed embedded bolt in the building frame and studded steel frame structural consideration for eccentricity is necessary.
8	Concrete jacketing of R.C column including slab. (Figure 8)	Concrete jacketing of existing R.C column up to the floor slab is also done. The procedure is same as described in Sl. 1. Here added column re-bars may get through the slab or may be embedded sufficiently into the member above increases both strength and ductility.
9	Concrete jacketing on beam (Method 8& 9)	Concrete jacketing on existing R.C beam is done to increase strength and ductility. Basic approach of work is same as jacketing of column.
10	New beam on floor slab (Method-10)	New R.C beams can be provided under existing slab. This addition of new beams sometime becomes necessary to complete a beam-column frame for structural necessity. These beams are connected with existing columns and slab usually anchor bolt and concrete grouting.



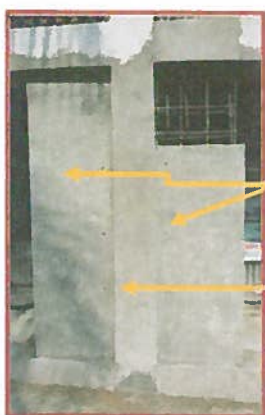
Column jacketing  
(Re-bar work  
and concreting  
after removal of  
finishing mortar)

Method1: Column jacketing



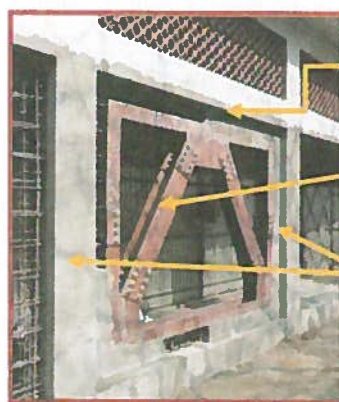
Chemical anchor,  
spiral bar and grout  
mortar  
Existing column  
RC shear wall

Method 2: RC shear wall



RC wing wall  
Existing column

Method 3: RC wing wall



Chemical anchor,  
spiral bar, and grout  
mortar  
Steel braced frame  
with stud  
Existing column

Method 4: Steel bracing



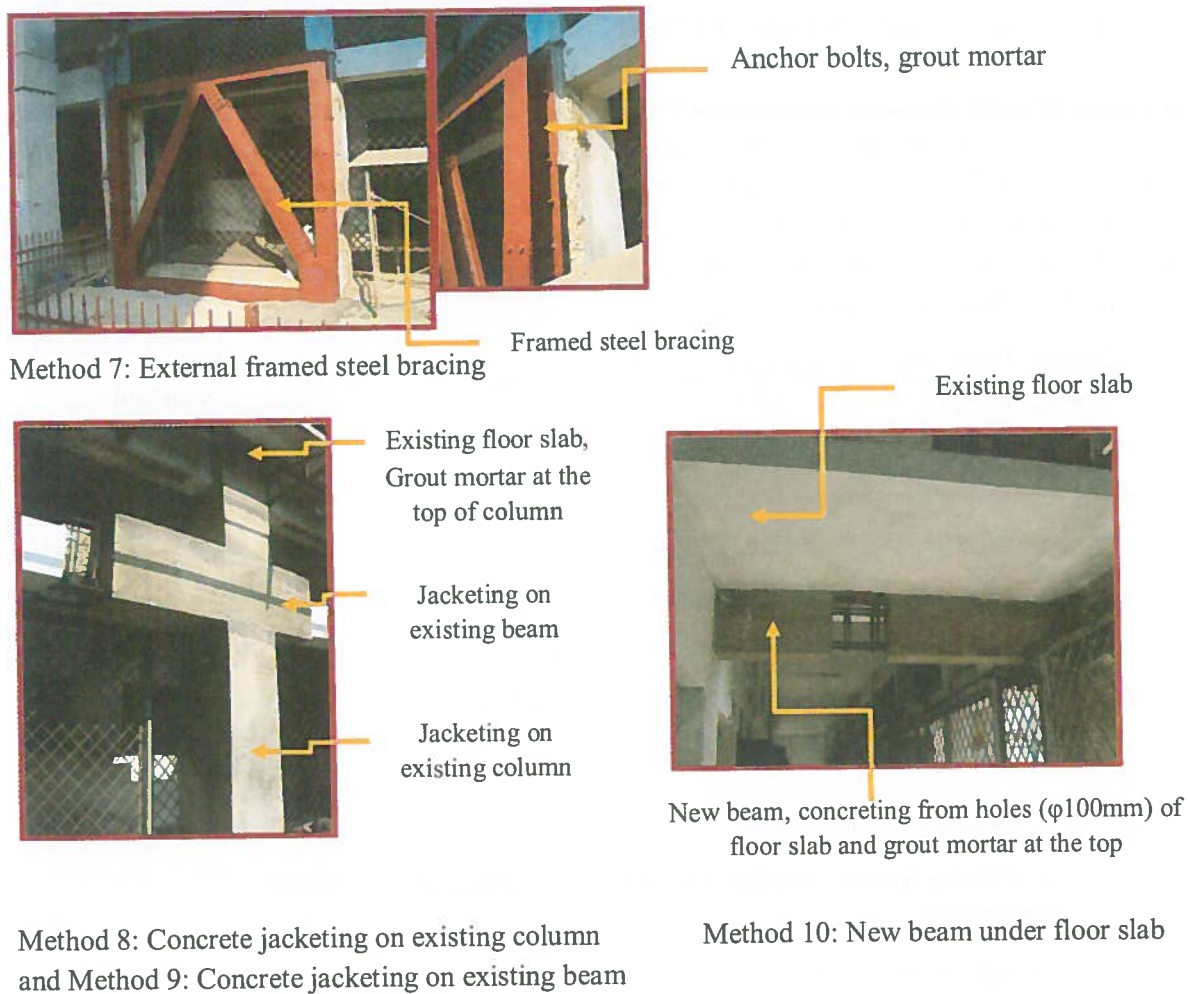
Carbon sheet  
wrapping  
Mortar finish

Method 5: Carbon fiber wrapping



Structural slit  
Reinforcement  
by steel angle  
member

Method 6: Seismic slit on brick standing wall



**Figure SA 10.1 Construction Methods of Test Work**

(Construction sequence is shown by exposing each steps of construction such as anchoring, re-bar work, concreting, and mortar grouting etc.)

## SUPPLEMENT A11. FORMULA OF ULTIMATE SHEAR STRENGTH FOR COLUMN

Ultimate shear strength of column is calculated by Eq. (A1.1-2) of the J. Standard and Eq. (3.3.4-2) of the J. Guidelines for retrofitted members. Background of the Eq. (A1.1-2) is explained as follows. Eq. (A1.1-2) is called as Prof. Arakawa's equation. 1<sup>st</sup> term is the factor of concrete strength, tensile re-bar ratio and shear span ratio. 2<sup>nd</sup> term is the factor of the amount of shear reinforcement. These are derived from a lot of experimental studies with 1,200 no's specimens of beams. Then 3<sup>rd</sup> term of axial stress is introduced by Prof. Hirose to apply to columns.

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

where:

- $p_t$  = Tensile reinforcement ratio (%).
- $p_w$  = Shear reinforcement ratio,  $p_w = 0.012$  for  $p_w \geq 0.012$ .
- $\sigma_{wy}$  = Yield strength of shear reinforcing bars (N/mm<sup>2</sup>).
- $\sigma_o$  = Axial stress in column (N/mm<sup>2</sup>).  $\sigma_o \leq 8 \text{ N/mm}^2$
- $d$  = Effective depth of column.  $D-50 \text{ mm}$  may be applied.
- $\frac{M}{Q}$  = Shear span length. Default value is  $\frac{h_o}{2}$ .  $M/(Q \cdot d) \leq 3.0$
- $h_o$  = Clear height of the column.
- $j$  = Distance between centroids of tension and compression forces, default value is  $0.8D$ .

$b$  = Width of column (mm)

Shear strength of beam is expressed by the following formula from experimental studies. Coefficient 2.7 and 180 of the formula is converted to 0.85 and 18 respectively due to the unit change from (kg/cm<sup>2</sup>) to (N/mm<sup>2</sup>). Then basic formula of the Eq. (A1.1-2) was derived.

$$\frac{Q_u}{b \cdot j} = \tau_u = \frac{\beta \cdot k_u \cdot k_p (180 + F_c)}{\left( \frac{M}{Q \cdot d} \right) + 0.12} + 2.7 \sqrt{p_w \cdot \sigma_{wy}} \quad (\text{Eq. SA 11.1})$$

Where;

- $\tau_u$  = Lower limit shear stress at shear failure (unit; kg/cm<sup>2</sup>)
- $\beta$  = Coefficient to provide lowest value from the experimental results, 0.092 is used.
- $k_u$  = Modification coefficient by the size of section, 0.70 is used.
- $k_p$  = Modification coefficient by tensile reinforcement ratio,  $k_p = 0.82 \cdot P_t^{0.23}$

Following Figure SA11.1 shows the relationship between experimental values and analytical values on ultimate shear strength of beam for ordinary concrete. Horizontal axis is  $\frac{M}{(Q \cdot d)}$  (ratio of shear span length

to effective depth, simply called as shear span ratio). Vertical axis is  $\frac{\text{test } \tau_u - 2.7 \sqrt{p_w \cdot \sigma_y}}{\{k_u \cdot k_p (180 + \sigma_B)\}}$ . The range

of concrete strength ( $\sigma_B = F_c$ ) of the specimen is 11.4~79 N/mm<sup>2</sup>.

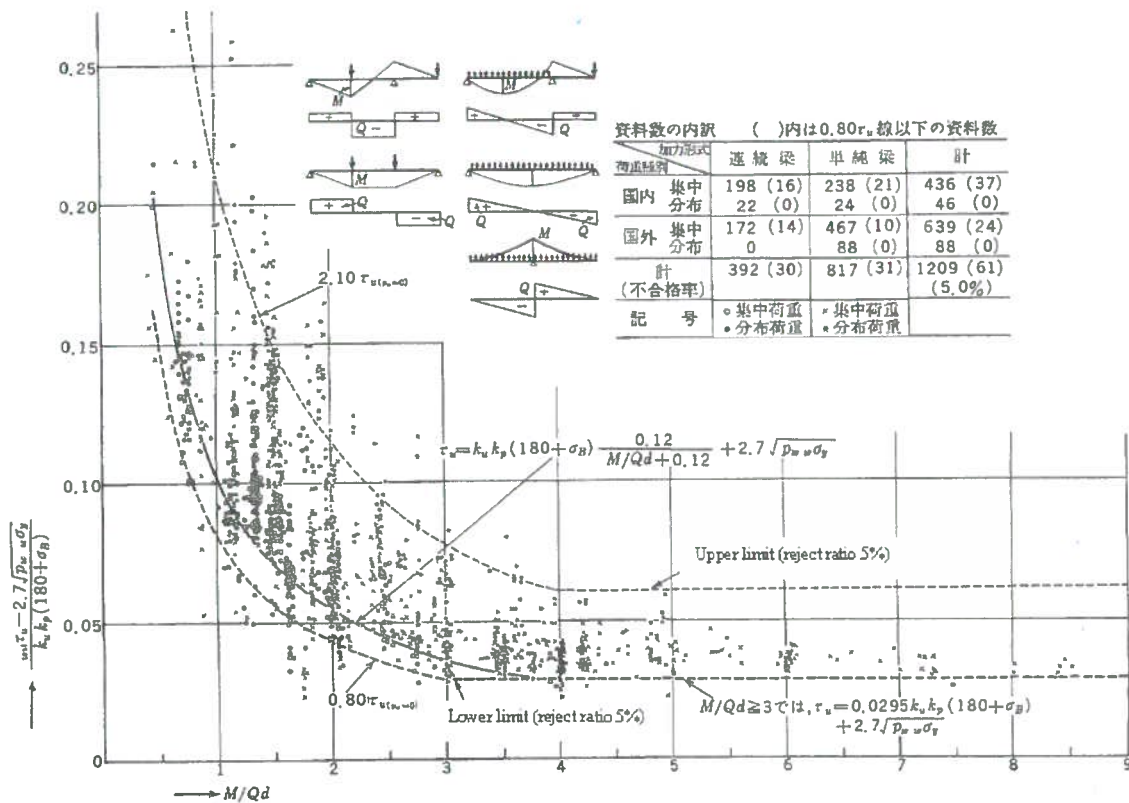


Figure SA 11.1 Relationship Between Experimental Values and Analytical Values on Ultimate Shear Strength of Beam for Ordinary Concrete

(Exp. figure 15.2, AIJ Standard for Structural Calculation of Reinforced Concrete Structures 2010, in Japanese)

Following Figure SA11.2 shows a recent analytical study of the evaluation of shear strength for columns. Shear strength is evaluated as the summation of strength by beam mechanism and arch (truss) mechanism. This is an example of analytical approach and for information only.

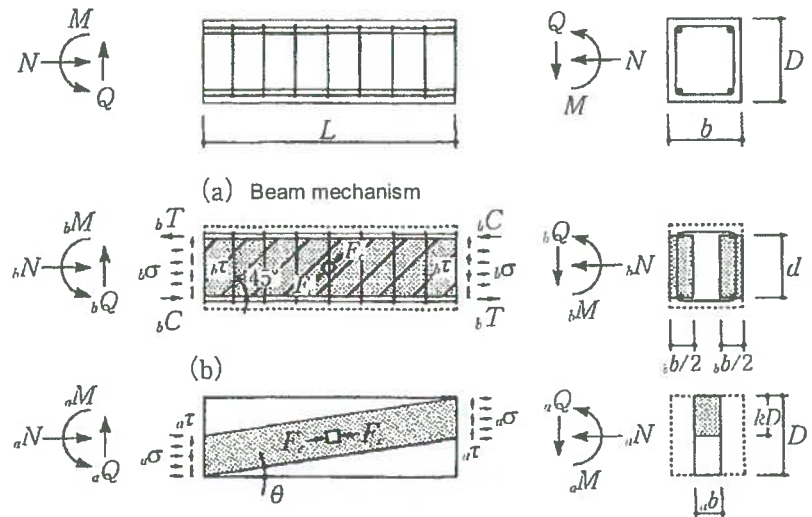


Figure SA 11.2 Shear Resistance Mechanism by Prof. Wakabayashi and Prof. Minami's Equation

Prof. Kuramoto & Prof. Minami, AIJ Structural paper report, "Practical shear strength formula of RC members applying plastic theory", No.417, 1990.11, in Japanese)



## SUPPLEMENT B

### SUPPLEMENT B1. EARTHQUAKE DAMAGES OF BUILDINGS

#### a) Structural damages, Japan

Main causes of collapse are shear failure of short columns, irregularity such as soft storey, etc. In Japan non-structural walls such as standing walls are RC walls and not brick walls. It is to be noted that construction quality including concrete strength is generally satisfactory.



1階の崩壊 (Storey collapse at 1st floor)



1階の崩壊 (First storey collapse of three storey)



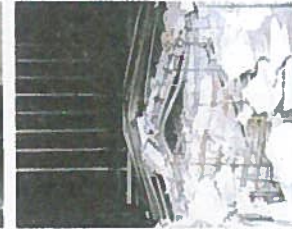
RC柱の剪断破壊 (Shear failure of column with widely spaced shear reinforcement)



短柱のRC柱の剪断破壊 (Shear failure of short column)



柱中央に於けるRC柱の剪断破壊 (Failure at mid height column)



2階のRC柱の剪断破壊 (Shear failure of 2nd storey column)



集合住宅1階の崩壊 (Collapse of roof in 1st storey building)



L形平面角部梁の損傷 (Damage of beams at corner of L-shaped plan)



剪断時のRC柱の剪断破壊 (Shear failure of column with aggregate)



2階のRC柱の剪断破壊 (Shear failure of 2nd storey column)

Collapse of soft story and other weak portion

Shear failure of RC columns

(a) Damages of RC Buildings by HyogokenNanbu EQ. (Kobe) 1995

Figures from "Damage Investigation Report " by Architectural Institute of Japan and other Institutions

b) Non-structural damages, Turkey

Damage and fall of non-structural element such as brick wall will cause human casualties.



Overturning of brick infill wall



Drop of infill brick wall



Failure of infill brick wall

Seismic Damages of Non-structure Elements, Kocaeli 1999 Turkey, report by Architectural Institute of Japan

(b) Seismic Damages of Non-structure Elements

c) Building Services Damages (M/E works), Japan

Building Services Damages (M/E works) are shown. Overturning and sliding of M/E equipment will damage the building function, and its recovery is not easy.



Sliding of water tank



Overturning of elec. transformer



Drop of elec. cable rack

(c) Building services damages

Building Center of Japan, 1997 "Guideline for Seismic Design and Construction of building Services (in Japanese)"

d) Office Damage, Japan

Overturning and damage of office furniture are shown.

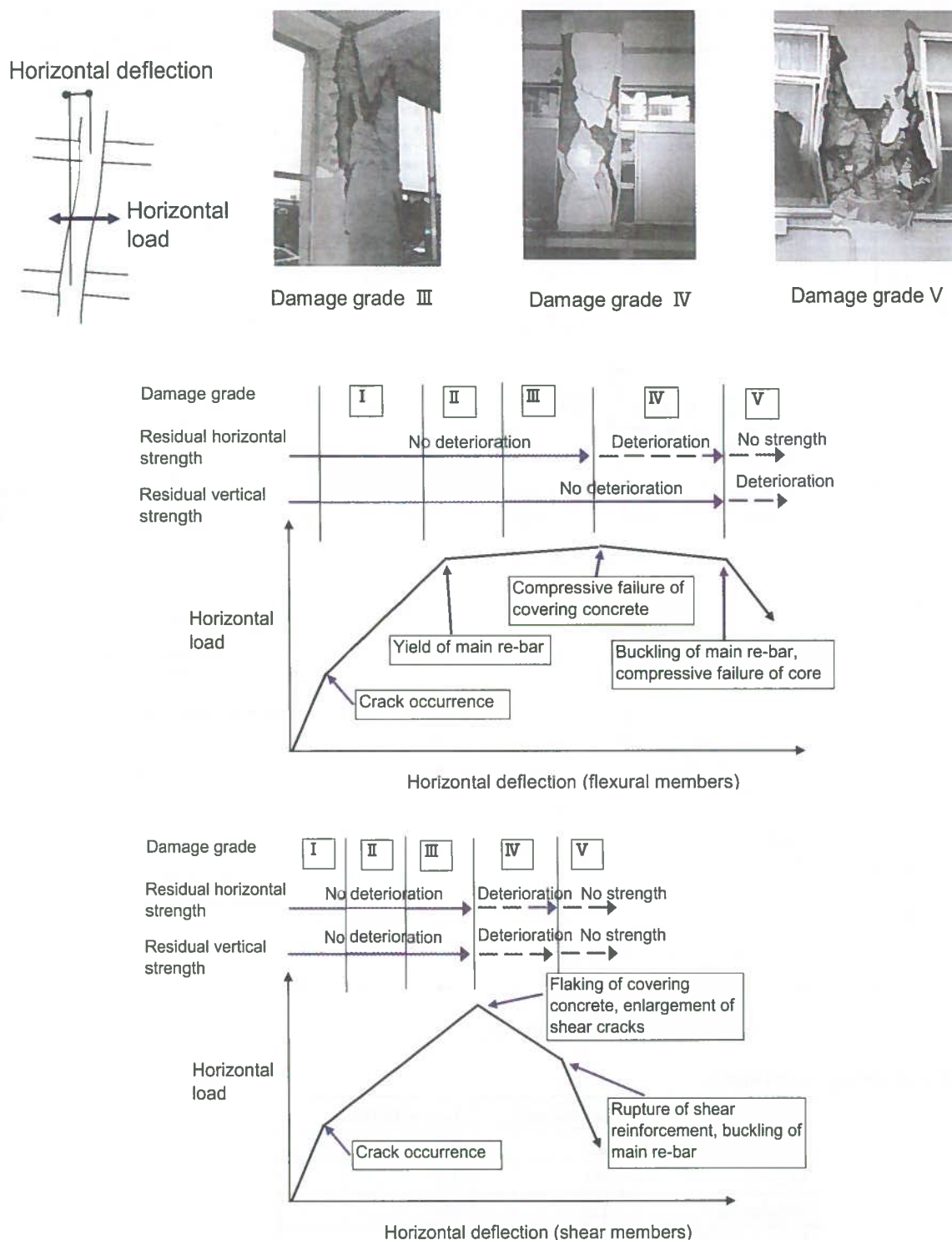


(d) Damages at Offices after Earthquakes,  
BCP for Earthquake, Tokyo Metropolitan Government (in Japanese), Japan 2008

**Figure SB 1.1 Earthquake Damages of Buildings**

## SUPPLEMENTB2. DAMAGE GRADE OF RC COLUMNS, JAPAN

Concept of structural damage grade and load-deflection curve of RC columns in Japan is shown. Five damage grades are shown. It is said that restoring work is difficult at grade IV and V.

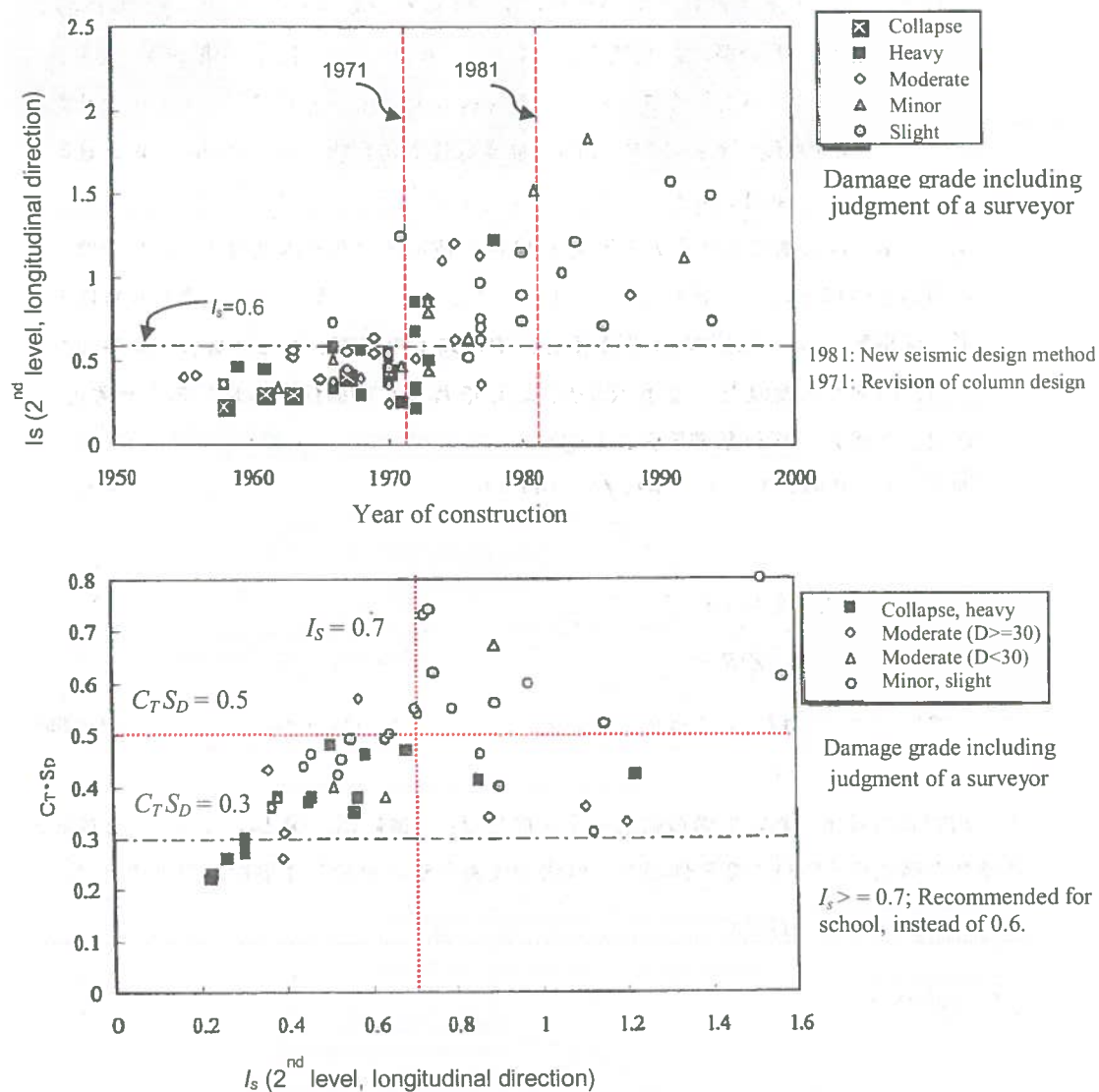


**Figure SB 2.1 Damage Grade of RC Columns**

[Source: "Standard of Judgment of Damage Grade and Guidelines of Recovery Engineering for Damaged Buildings, 2001",  
The Japan Building Disaster Prevention Association (written in Japanese)]

### SUPPLEMENT B3. SEISMIC INDEX OF STRUCTURE $I_s$ AND EARTHQUAKE DAMAGES

Damage survey for 700 school buildings, including seismic evaluation of 67 buildings after 1995 Kobe earthquake, by the Architectural Institute of Japan is shown below for information. It is noted that Seismic index of structure  $I_s$  requires 0.6 and more and  $C_T S_D$  (cumulative strength index  $\times$  irregularity index) requires 0.3 and more for existing medium height RC buildings in Japan. As shown in the Figure, a few buildings with  $I_s$  more than 0.6 were evaluated as heavy damage, which were ductility oriented structure.



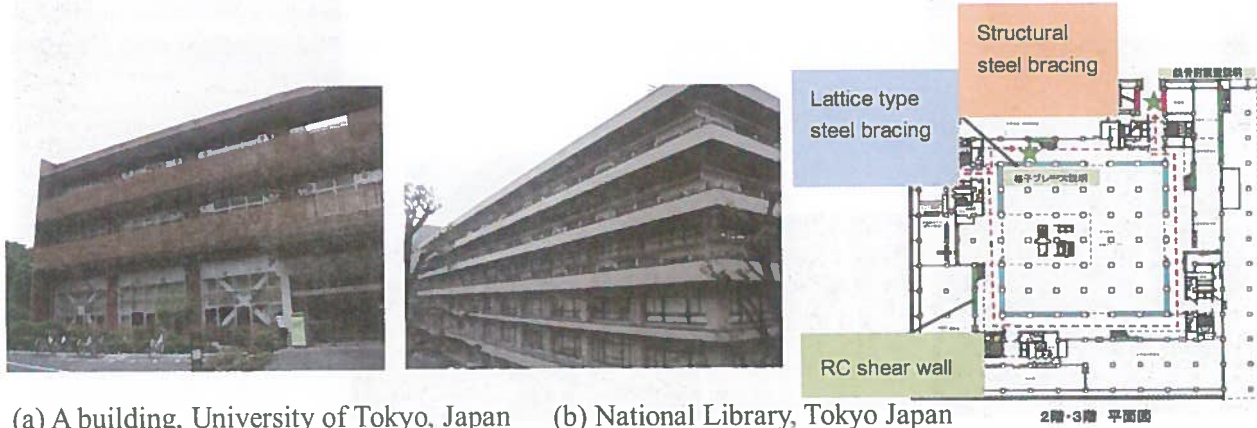
Break down of surveyed buildings

	Damaged	$I_s$ , evaluated
Collapse	15	10
Heavy	30	17
Moderate	126	40
Minor, Slight	529	---
Total	700	67

Source; Architectural Institute of Japan, "Earthquake damage report for RC buildings, II School buildings, 1995 Hyogoken Nambu Earthquake", 1997. 3 (in Japanese)

Figure SB 3.1 Seismic Index of Structure  $I_s$  and Earthquake Damages

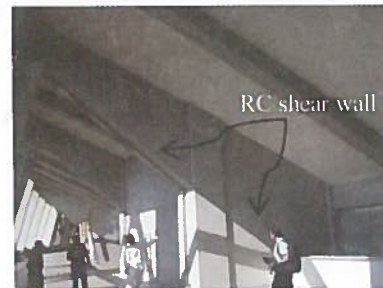
**SUPPLEMENT B4. EXAMPLES OF SEISMIC RETROFITTED BUILDINGS**



(a) A building, University of Tokyo, Japan      (b) National Library, Tokyo Japan



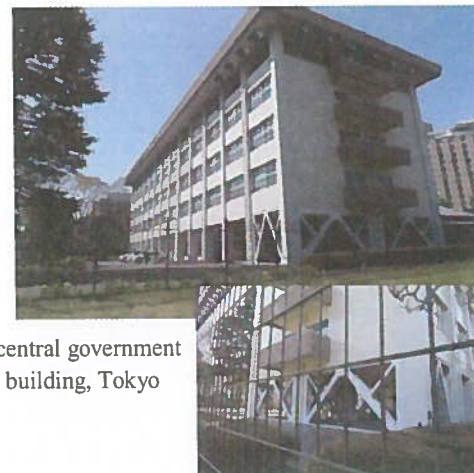
(c) Steel Framed Brace at a high school      (d) Retrofit work completed for a school building, Chiba Japan



(e) Gymnastic Field, Chiba Japan      (f) Steel plate jacketing      (g) RC shear wall



(h) A local government office building, Tokyo



(i) A central government office building, Tokyo



(j) Medical University Building, Tokyo



(k) Residential building, Tokyo



(l) RC wall and steel framed bracing

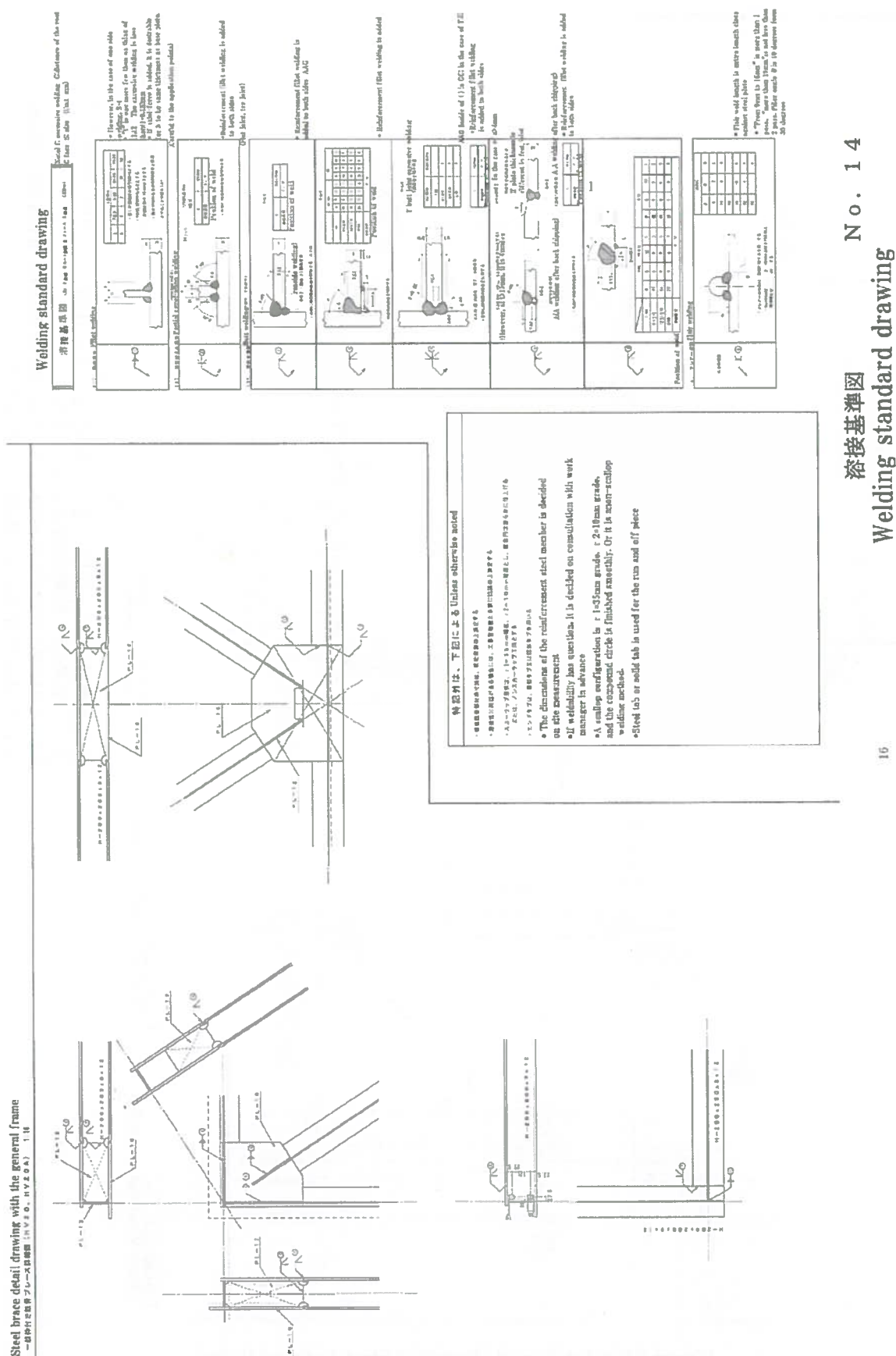


(m) Steel framed brace of a school building, at E-defense, Hyogo

**Figure SB 4.1 Example of Seismic Retrofitted Buildings**



Welding detail using butt welding is shown below for information only.



**Figure SB 4.3 A Sample of Standard Welding Detail of Steel Framed Brace**

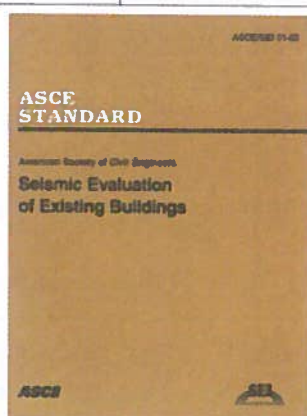
## SUPPLEMENT B5. SUMMARY OF COMPARISON OF SEISMIC EVALUATION METHOD

(Reference only)

	Japanese Standard, 2001 (JBDPA)	ASCE 31-03												
Concept	<b>Diagnosis, how the building collapse.</b> <b>Result can be provided by manual calculation.</b>	<b>Use of Check list and Analytical approach.</b>												
Methodology	<p>3 levels of seismic screening</p> <p>1<sup>st</sup> level, simple calculation for the seismic capacity.</p> <p>2<sup>nd</sup> level, supposing column collapse, which is commonly used.</p> <p>3<sup>rd</sup> level, including beam collapse, is also possible to apply.</p> <p>1) Building Survey</p> <p>2) <b>Classification of column (flexural or shear column)</b></p> <p>3) Grouping of columns</p> <p>4) <b>Seismic Index of Structure <math>I_s</math></b> is calculated for each direction and each storey.  <math>I_s = \text{Basic seismic index of structure } (E_o) \times \text{Irregularity Index } (S) \times \text{Time Index } (T)</math>  <math>E_s = \text{Strength Index } (C) \times \text{Ductility Index } (F)</math></p> <p>5) <b>Seismic Demand Index of Structure</b>  <math>I_{so} = \text{Basic Seismic Demand Index of Structure } (E_s) \times \text{Zone Index } (Z) \times \text{Ground Index } (G) \times \text{Usage Index } (U)</math></p> <p>6) Judgment:  <math>I_s &gt; I_{so}</math>, and requirement of <math>C_r S_d</math>, then OK  <math>I_s</math> value shows the seismic performance.</p>	<p>Requirement of checking for 3 tiers</p> <p>Building survey</p> <p><b>1<sup>st</sup> Tier:</b></p> <p>1) <b>Level of Intensity (Low, Moderate, High)</b></p> <p>2) <b>Level of performance (Life safety, Immediate occupancy)</b></p> <p>Check list using above 1 and 2. Basic and Supplementary checking is required. Check items, structure, non-structure, geological,...Check list for various structure</p> <table border="1" style="margin-left: auto; margin-right: auto;"> <thead> <tr> <th>Compliance</th><th>Non Compliance</th><th>Not</th></tr> </thead> <tbody> <tr> <td>Applicable</td><td></td><td></td></tr> <tr> <td>Load pass</td><td></td><td></td></tr> <tr> <td>Soft storey✓ *****</td><td style="text-align: center;">↓</td><td></td></tr> </tbody> </table> <p style="text-align: center;">Go to 2nd Tier</p> <p>Checking of each element, such as column beam/joint.</p> <p>Requirement: capacity of each element is checked.</p> <p><b>Force control action</b></p> <p><b>Deformation control action</b></p> <p>Deficiency of each element is checked. Each element is O.K., then the structure is O.K.</p> <p><b>3rd Tier</b>, to follow ASCE (41-06, Rehabilitation), FEMA 356, ATC-40, or to follow new buildings (ASCE7-02, IBC 2000).</p>	Compliance	Non Compliance	Not	Applicable			Load pass			Soft storey✓ *****	↓	
Compliance	Non Compliance	Not												
Applicable														
Load pass														
Soft storey✓ *****	↓													
Issues to apply (local conditions)	<p>1) Proposed <math>I_{so}</math> for buildings in Dhaka and Sylhet, equivalent to new buildings.</p>	<p>1) Change of default value for materials. 2) Selection of Level of Intensity (3 zones in BNBC93, 4 zones in BNBC 2015)</p> <p>3) Selection of Building type (Concrete moment frame with low strength concrete using brick chip concrete)</p>												
	<p>Structural experiment, 2012 and 2013 by CNCRP</p>	<p>1) Low strength concrete 2) Re-bar detail (lap joint, anchorage, 90 degree hook)</p> <p>3) High axial force ratio 4) Poor quality control at sites (honey-comb, etc.)</p> <p>5) In-filled brick-wall, etc.</p>												
Remarks	<p>1) Relation of Strength Index- Ductility Index shows the performance of a building and target.</p> <p>2) Data of relation between <math>I_s</math> value and damages by past earthquakes has been accumulated.</p>	<p>1) Numerical value such as <math>I_s</math> is NOT shown.</p> <p>2) Collapse mode of the building is NOT shown clearly.</p>												

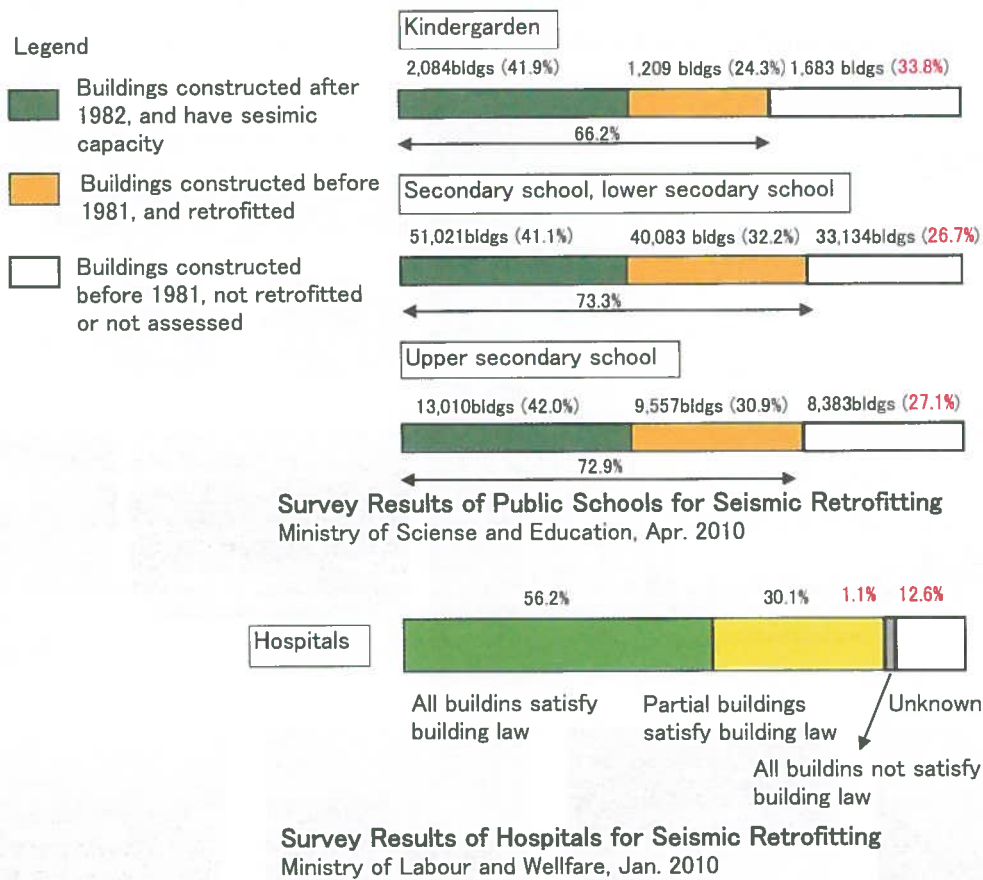
## Summary of Comparison of Seismic Retrofit Design (Reference only)

	Japanese Guidelines, 2001 (JBDPA)	ASCE 41-06 Standard (FEMA P-420)
<b>Concept</b>	<b>Strength oriented, Ductility oriented and the mix is considered for the retrofit.</b> Strength and ductility of each retrofit member is introduced. Result can be provided by manual calculation.	<b>Performance-based rehabilitation design approach.</b>
<b>Methodology</b>	Seismic Index of Structure after retrofit $I_s$ is calculated, and compared with Seismic Demand Index $I_{so}$ . Strength and ductility evaluation of following member/ element 1) to 6) is introduced. Connection detail to existing member including post-installed anchor 7) is shown. 1) RC wall 2) RC wing wall 3) Column 3.1, RC jacketing, 3.2 Steel plate jacketing, 3.3 Carbon fiber wrapping 4) Steel framed brace 5) Beam strengthening 6) Foundation 7) Post-installed anchor	Performance objective selected from a range of performance levels (Collapse, Collapse Prevention, Life Safety, Immediate Occupancy, and Operational) at any specific seismic hazard level. Engineering analysis is based on a series of four options: (1) Linear static procedure (2) Linear dynamic procedure (3) Nonlinear static procedure (4) Nonlinear dynamic procedure Increasing levels of effort and greater confidence in the design. Simplified rehabilitation is applicable to structural and nonstructural components in buildings within a specified range of buildings types and characteristics Prioritization of rehabilitation measures, such as structural, use and integration.
<b>Issues to apply (local conditions)</b>	1) Evaluation of ductility for retrofitted members against existing low strength concrete members 2) Connection detail by post-installed anchor against low strength concrete member. 3) To control the damage of non-structural elements, such as brick wall.	1) Application to non-engineered RC buildings in Bangladesh is within the scope? 2) Judgment criteria of performance design, especially deformation criteria or acceptable deformation is not easy to establish.
<b>Remarks</b>		1) Performance based approach will require high level of Engineers for the execution.



**SUPPLEMENT B6. PROGRESS OF SEISMIC RETROFIT FOR SCHOOLS IN JAPAN,**  
Reference only

Total 50,849 buildings of existing public schools have been seismic retrofitted, as of 2010, Japan.



**Figures SB 6.1 Progress of Seismic Retrofit for Schools in Japan**

## SUPPLEMENT B7. SEISMIC RETROFIT USING BASE ISOLATION SYSTEM, REFERENCE ONLY

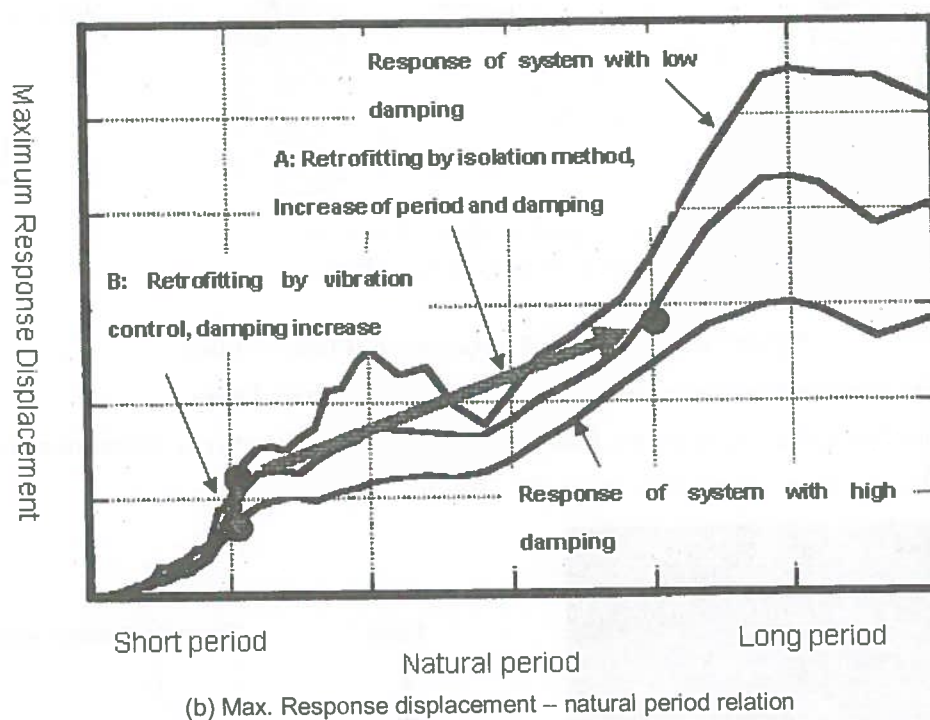
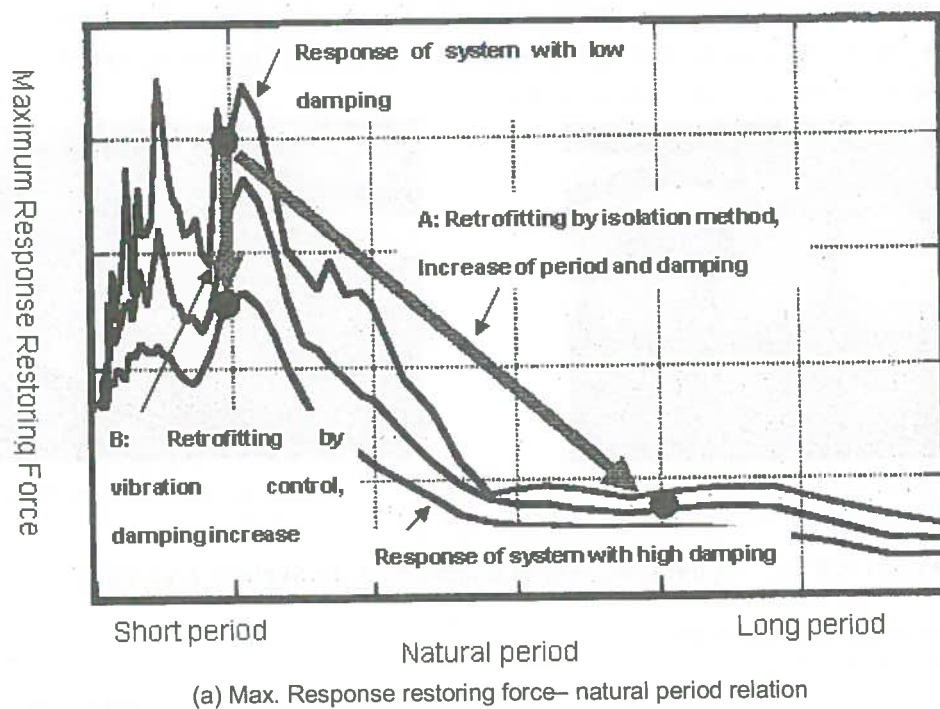
An example of seismic retrofit using base isolation system for the central government building in Japan is shown for information. Long natural period has been provided for the building through the installation of isolators, and dampers are also provided to reduce the response such as displacement. It is noted that this method is expensive construction while the office can maintain their function without evacuation.



Central Government Building No.3, Isolator at the basement Expansion Joint at ground floor level  
(Courtesy: Ministry of Land, Transport, Infrastructure and Tourism, Japan)

**Figure SB 7.1 Example of Retrofit by Base Isolation System, Tokyo Japan**

Maximum response restoring force and natural period relation, and maximum response displacement and natural period relation by A) Isolation method and B) Vibration control method are shown below. The change of the response before and after retrofit of a building, which is the main concept of this retrofit, is indicated for information only.



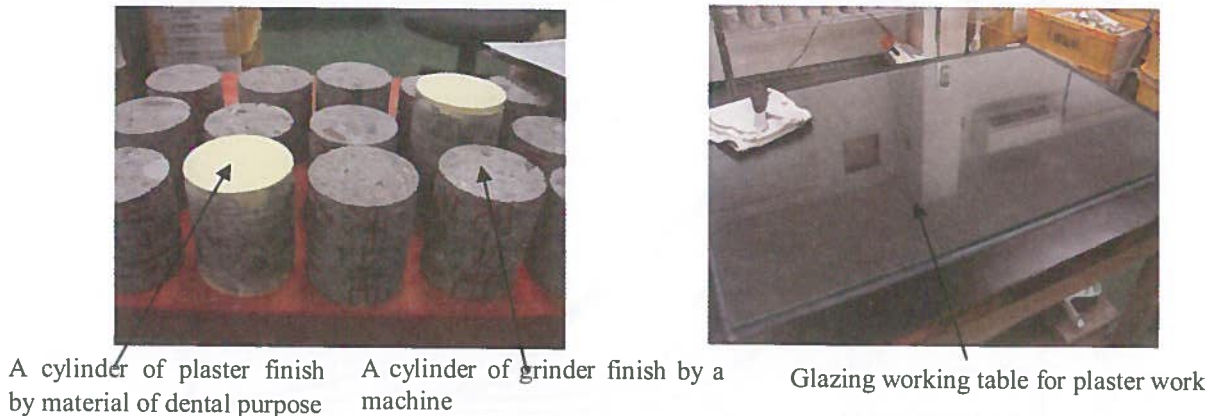
References: "Guideline for Seismic Retrofitting by Isolation and Vibration Control for Existing reinforced Concrete Buildings", Japan Disaster Prevention Association, Japan 2006 (written in Japanese)

**Figure SB 7.2 Concept of Retrofit Using Isolator and Damper System**

## SUPPLEMENT B8. CONCRETE CORE SAMPLING AND STRENGTH EVALUATION

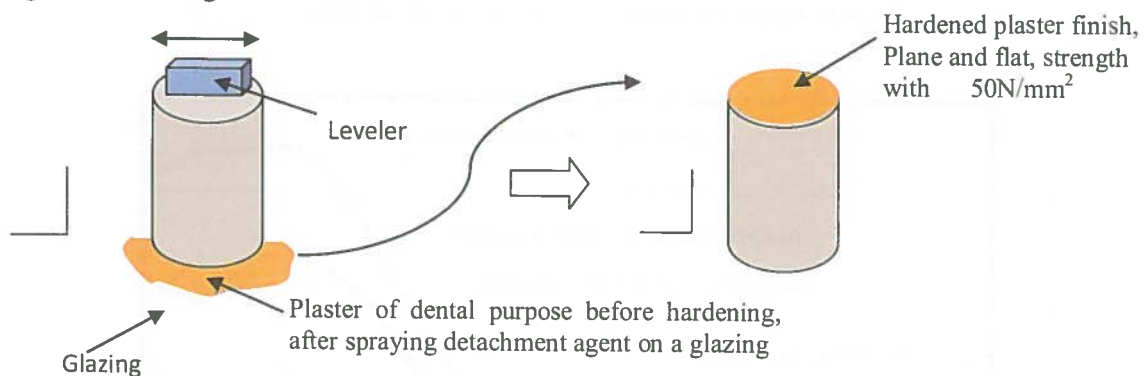
### 1) Preparation of concrete core sampling for the test

(1) In order to get concrete strength by compressive test, flatness and verticality of top and bottom of cored drilled sample is important. Two typical methods of surface treatment have been introduced in Urawa Laboratory-Building Material Testing Center, Japan. (1) Grinder finishes by a machine, (2) Plaster finish. Plaster finish capping method is shown at here.



**Figure SB 8.1 Cylinders (100mm diameter) after Surface Treatment**

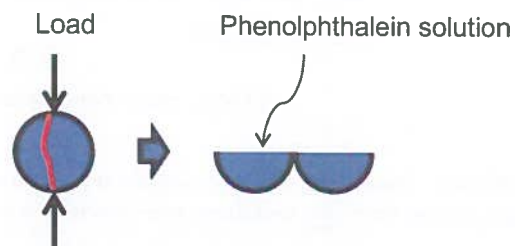
To shake cylinder to provide alignment, through the monitoring of a leveler



**Figure SB 8.2 Typical Sequence of Plaster Finish**

### (2) An example of carbonation test of core drilled sample at a lab in Japan.

In this case, after splitting the concrete core, Phenolphthalein solution is sprayed. If the color is changed to purple, the area has not been neutralized chemically. This test can also be done at site.



**Figure SB 8.3 Carbonization Test**

## 2) Strength evaluation of sampled concrete cores

It has been said that concrete strength of core sampling is lower than cylinder strength generally. Especially the use of small diameter cores results in lower and more erratic strengths, related to the size of course aggregate.

### (1) ACI 437R03

According to the Sec 5.1.1 of ACI 437R03 (Strength Evaluation of Existing Concrete Buildings), it has been stated that the average concrete compressive strength obtained by testing concrete cores may be divided by **0.85** to arrive at the in-place concrete strength value to be used in strength calculations (Bloem 1968). It is also noted that there is the requirement of sampling numbers.

### (2) Comparison of strength between cylinder and concrete core by structural test 2013 CNCRP

Same concrete was casted for cylinder and a plate with thickness of 150mm for core sampling. Diameters of core sampling were 50mm and 100mm. Concrete was low strength concrete using **small size brick chips**. Strength of core sampling is shown below.

**Table SB 8.1 Concrete Strength of Cores**

**Diameter; 50mm** (diameter is more than 3 times of course aggregate size)

	Height (mm)	Diameter (mm)	Crushed load (kN)	Crushed strength (kN/mm <sup>2</sup> )	L/D	Correction factor	Corrected strength (kN/mm <sup>2</sup> )	Average (kN/mm <sup>2</sup> )
1	96.8	49.5	18.1	9.4	2.0	1.0	9.4	9.0
2	101.2	49.4	17.1	8.9	2.0	1.0	8.9	
3	101.0	49.6	16.8	8.7	2.0	1.0	8.7	

**Diameter; 100mm**

	Height (mm)	Diameter (mm)	Crushed load (kN)	Crushed strength (kN/mm <sup>2</sup> )	L/D	Correction factor	Corrected strength (kN/mm <sup>2</sup> )	Average (kN/mm <sup>2</sup> )
1	150.4	99.3	63.7	8.2	1.5	0.96	7.9	9.0
2	151.8	99.3	88.0	11.3	1.5	0.96	10.9	
3	150.0	99.2	66.7	8.6	1.5	0.96	8.3	



**Figure SB 8.4 A Concrete Plate for Core Sampling**

Result of average strength of concrete cylinder (diameter 100mm) was 10.6N/mm<sup>2</sup> at 4 weeks. Strength was compared. Strength of concrete core was **15%** lower than cylinder strength. ( $9.0/10.6 = 0.849$ ). Similar result to ACI 437 was obtained. Other statement of ACI437 will be followed, and it may be accepted to adjust the strength of cores in Bangladesh. The accumulation of further data will be suggested.



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## REFERENCES

- 1) The Japan Building Disaster Prevention Association, “Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001, and Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001, English version, 1st”
- 2) The Japan Building Disaster Prevention Association, “Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001, in Japanese”
- 3) The Japan Building Disaster Prevention Association, Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001, in Japanese”

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