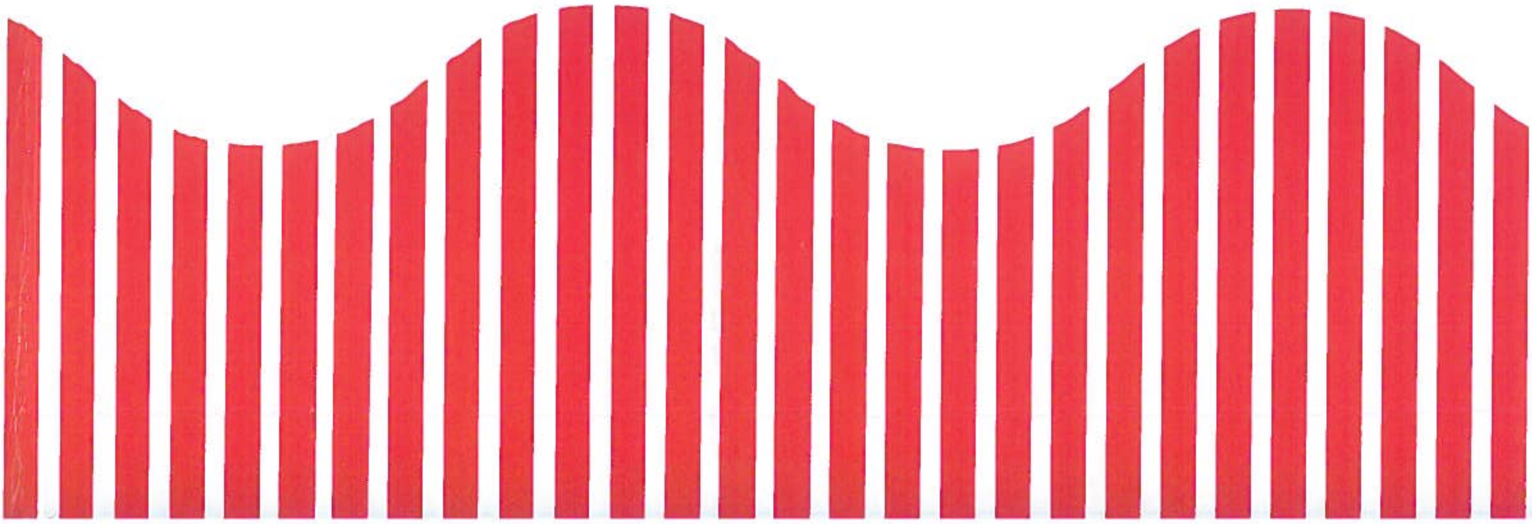
CNCRP

Project for Capacity Development on Natural Disaster Resistant
Techniques of Construction and Retrofitting for Public Buildings

MANUAL FOR
SEISMIC EVALUATION
OF EXISTING REINFORCED CONCRETE BUILDINGS

Public Works Department



MANUAL FOR SEISMIC EVALUATION 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 BUILDING (CNCRP)

A TECHNICAL COOPERATION PROJECT BETWEEN PWD AND JICA

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The contents of this book are related to retrofitting design and construction process generally undertaken by Public Works Department which have been described hereinafter in brief theoretical form as guidelines. As such no chapter, article, clause, sub-clause therefore, be referred to as VALID DOCUMENTS in the event of any arbitration, litigation, dispute, claim case, whatsoever secured, made or claimed by any person as the case may be under any circumstances.

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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 (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. 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 trained to adopt 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) is not included with any provisions of seismic evaluation and retrofit design. Throughout the project duration, the engineers of PWD studied the standard, guidelines and technical manuals for seismic evaluation and retrofitting design of RC buildings in Japan. The approach is a little unconventional in Bangladesh. 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 assumption to be taken based on local construction circumstances. This "Manual for Seismic Evaluation 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.

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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 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 Evaluation of Existing Reinforced Concrete Buildings**" has been prepared to supplement the Japanese Standard and Guidelines mentioned above incorporating the seismic load of BNBC. 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 Retrofit Design 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 Seismic Evaluation Manual and the building damage risk in Bangladesh will be mitigated.

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CHAPTER 4

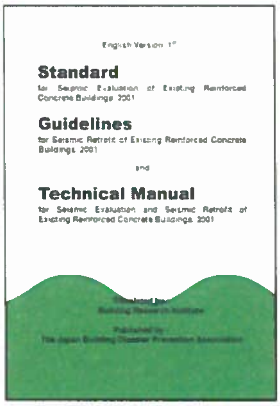
Judgment on Seismic Safety

CHAPTER 1. GENERAL

1.1 GENERAL PRINCIPAL

1) Composition of the Manual

This manual shall be used for seismic evaluation of existing Reinforced Concrete (RC) buildings. The manual has been prepared to supplement the following Japanese Standard (henceforth known as J. Standard), incorporating the characteristics of RC buildings in Bangladesh and seismic design load of Bangladesh National Building Code (henceforth known as BNBC). This manual narrates the evaluation methods and their explanations according to the Japanese Standard with examples using few actual buildings in Bangladesh. BNBC does not cover the structural evaluation of existing RC buildings.



English Version 1st

Standard

for Seismic Evaluation of Existing Reinforced Concrete Buildings 2001

Guidelines

for Seismic Retrofit of Existing Reinforced Concrete Buildings 2001

and

Technical Manual

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

"Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001 and Technical Manual for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001" translated in English and published by Japanese Building Disaster Prevention Association.

2) Fundamental Principal

The J. Standard explains seismic evaluation of existing reinforced concrete buildings. The seismic evaluation is based on: (i) Site inspection and (ii) Structural calculation. These two observations together represents the seismic performance of a building in terms of seismic index of structure I_S and seismic index of non-structural elements I_N. The overall safety of the building shall be judged based on standard for judgment on seismic safety wherein seismic performance demands are stated.

Flow chart for seismic evaluation is given in Figure 1.1. Relevant section of the J. standard and this application manual is given besides each step of flow chart.

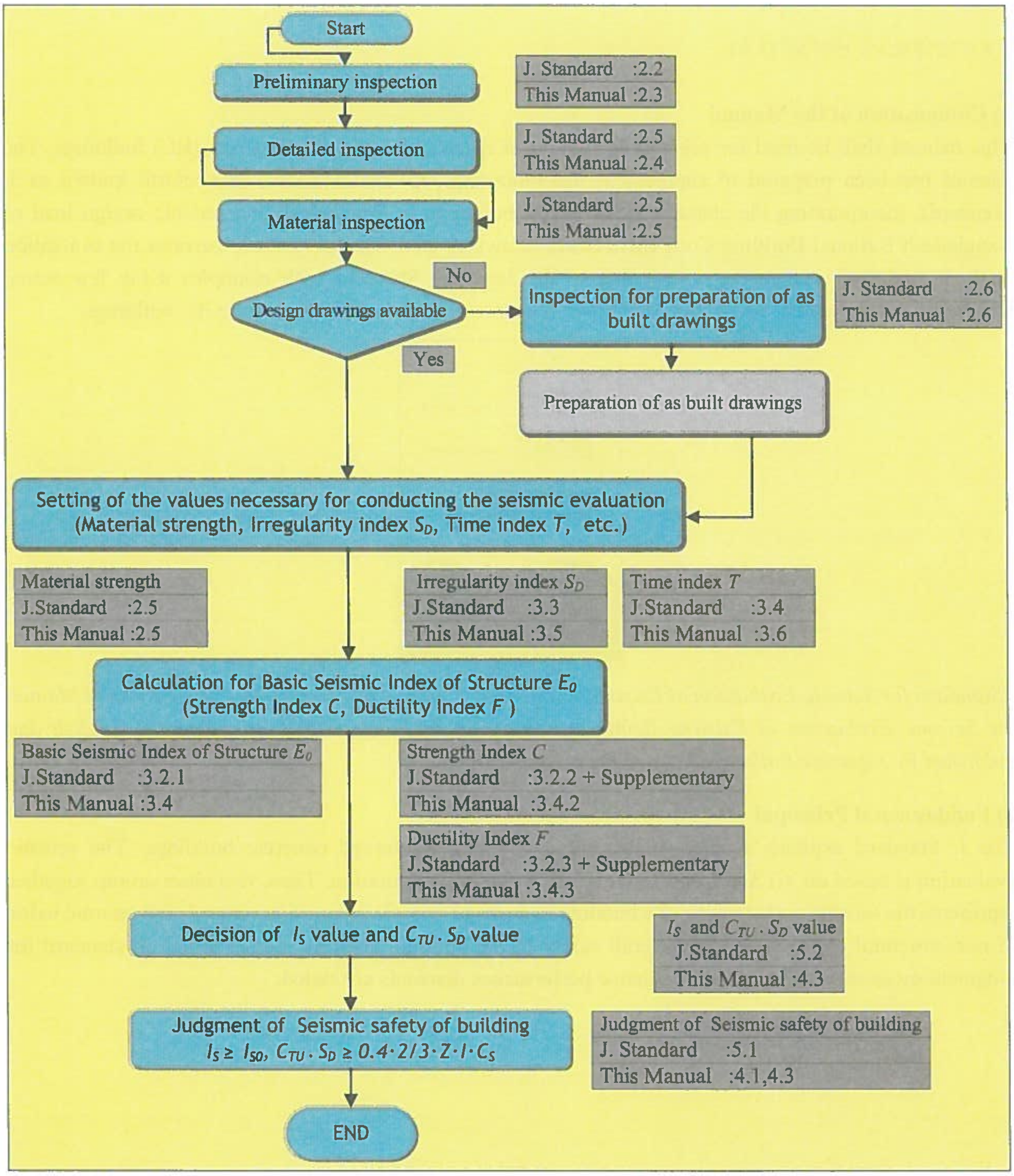


Figure 1.1 Standard Procedure for Seismic Evaluation

The application manual is prepared based on the flow chart of the seismic evaluation. In this application manual standard procedure for seismic evaluation is explained step by step by an example of existing building. Example of another building is provided in Supplement No.9 in concise from.

1.1.1 Seismic Index of Non-Structural Elements I_N

Evaluation of the non-structural element is one of the most important factors for the safety evaluation. But this manual gives higher priority to the evaluation of the seismic capacity of structure I_S. In a RC frame structure various types of materials and claddings are used as infill and partition walls including unreinforced brick masonry. During an earthquake these non-structural elements may cause problem. This manual does not cover detail procedure to calculate seismic index of non-structural elements $I_{N.}$ Therefore, it is recommended to further study the evaluation method of non-structural elements and to include it in the seismic evaluation of the buildings in future. For the present, it is recommended to see the J. standard (Chapter 4. Seismic Index of Non-structural Elements I_N) or the BNBC2015 (2.5.15 Non-structural components).

1.2 SCOPE OF APPLICATION

1)Elements that constitute a building

Main structural members shall be considered as a component of the seismic evaluation. Non-structural members are out of scope of this manual.

2) Type of structure

Reinforced concrete (RC) frame structure is covered in this manual. Flat plate (slab) RC structure is out of scope in principle. Brick masonry building is also out of scope and is not covered in this manual.

3) Number of stories

Mid to low-rise buildings with 6 stories and less are covered generally. Buildings higher than 6 stories shall be considered with special attention.

4) Concrete strength

Buildings with concrete strength not less than9.0N/mm^{2}are covered.

5)Others

This manual is not applicable for the significantly deteriorated buildings and the buildings with special structures.

1.2.1 Elements that Constitute a Building

The manual works only with the main structural elements excluding the non structural elements. Basically, the seismic evaluation for buildings are complete with the seismic evaluations of non-structural elements such as facade elements, finishing elements, brick walls, and machinery of building services. Issue of Non-structural elements remains as the subject of future study.

1.2.2 Type of Structure

This manual covers the seismic evaluation 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

Most of the RC flat plate (slab) buildings in Bangladesh are without RC shear walls. There are vulnerable structures, since horizontal stiffness and strength is low. Proper layout of seismic load resistant elements in plan and in elevation shall be considered and floor slabs shall be evaluated to transfer seismic load. Seismic assessment of flat plate structure is out of scope of this manual in principle.

B. Building with partial floor beams and as well as without grade beams

There are many RC buildings with partial floor beams and as well as without grade beams. Horizontal strength of a column without beams at bottom will be almost zero if foundation size is small. Horizontal load carrying capacity of a building shall be reduced considering this condition. In case of buildings without grade beams, it is required to evaluate effective column height at ground floor and connection condition to foundation footings for the evaluation of column strength.

1.2.3 Number of Stories

The original version of the Japanese Technical Manual has been developed and applied for many Reinforced Concrete (RC) buildings. Applicable limit of number of storey is 6 stories generally. This condition will cover majority of existing RC buildings in Bangladesh. In case of higher storied buildings, the flexural behavior of columns will change. This manual may be allowed to apply for evaluation of higher storey buildings by incorporating the change of axial force of columns by seismic load.

1.2.4 Concrete Strength

Brick chips aggregate have been used for concrete work of many existing RC buildings in Dhaka. The concrete strength with brick chips is low, and it has been observed that actual concrete strength of some of the buildings are less than $10N/mm^2$, according to the result of concrete core sampling test. Concrete strength with not more than $13.5N/mm^2$is defined as low strength concrete. Low strength concrete is not covered by the "Japanese Seismic Evaluation Standard 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 $9.0N/mm^2$. It is required to provide reduction coefficient for low strength concrete to evaluate shear strength of columns. Buildings with low strength concrete less than$9N/mm^2$ are out of scope of this manual. It will be required to evaluate separately in detail for buildings with concrete strength less than $9.0N/mm^2$.

1.2.5 Others

The applicability of this evaluation method needs to be examined for the following types of buildings.

-A building with significant differential settlement and structural cracks

-A building with cracks, falling of concrete slab etc. as a result of fire incidence.

- A building becoming decrepit in 50 years or more after its construction

The applicability of this manual to such buildings shall be carefully examined because it may be inappropriate to evaluate the seismic capacity based on the seismic index.

Corrosion of reinforcing bars in slab

Corrosion of reinforcing bars in beam

Diagonal crack in brick wall due to differential settlement

Spalling of slab concrete due to fire disaster

Figure 1.2.1 Examples of Deterioration of Building Elements Inappropriate for Seismic Evaluation

1.3 SUGGESTED MODIFICATION OF JAPANESE STANDARD FOR BANGLADESH

The Summary of modifications of Japanese Standard /Guidelines for its application in Bangladesh is shown in Table 1.3.1

Modifications of Japanese standard for its application in Bangladesh are summarized in Table 1.3.1, with respect to A: General, B: Ductility index, C: Strength index, D: Irregularity index, and E: 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 1.3.1 Suggested Modifications of Japanese Standard for its Application in Bangladesh

Item	Japan	Bangladesh
Title	The Standard for Seismic Evaluation of Existing RC Buildings of Existing RC Buildings 2001 (JBDPA)	Seismic Evaluation Manual of Existing RC Buildings (CNCRP)
A: General		
1. Status	"The Standard" and "Guidelines"	Technical **"Recommendations"**
2. Level of screening	1st, 2nd and 3rd level screening method. 2nd level is mainly used.	1st, 2nd and 3rd level screening method. **2nd level screening method** is applied, which is suitable and practical for buildings. 1st level screening method is not used for the judgment.(chap.3.3)
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. Detail building survey is required.(chap2) .Strength of concrete core(*) 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. *Ref. ACI 437*
4. Application: Concrete strength	Concrete strength F_C, not less than 13.5N/mm^2 (Not low strength concrete)	Concrete strength F_C, not less than9.0N/mm^2. Reduction factor Kr is used for column shear strength in case of concrete strength lower than *13.5 N/mm^2*.
5. Seismic index of structure, I_s	$I_S = E_o \times S_D \times T$ $E_O \propto \frac{n+1}{n+i} \times C \times F$	
6.Seismic demand index of structure, I_{so}	Seismic demand index of structure I_{SO}, (1) I_{SO} $I_{SO} = E_S \cdot Z \cdot G \cdot U$ E_S= Basic seismic demand index of structure E_S= 0.8, for 1st level screening E_S= 0.6, for 2nd level screening E_S= 0.6, for 3rd level screening Z = Seismic zone index G = Ground index U = Usage index (2) $C_{TU} \cdot S_D \geq 0.3 \cdot Z \cdot G \cdot U$ C_{TU} = Cumulative strength index at the ultimate deformation of structure. S_D = Irregularity index.	Proposed I_{SO} for 2nd and 3rd level screening, $I_{SO} = 0.8 \times \frac{2}{3} Z \cdot I \cdot C_s$ (*) (80% of elastic response shear force coefficient) Z : Seismic zone coefficient, as defined in Section 2.5.4.2 of BNBC2015 I : Structure importance factor C_S : Normalized acceleration response spectrum, which is a function of structure (building) period and soil type (site class) $C_{TU} \cdot S_D \geq 0.4 \times \frac{2}{3} \times Z \cdot I \cdot C_s$ C_{TU} = Cumulative strength index at the ultimate deformation of structure. S_D = Irregularity index. (Chap. 4)
7.Yield shear force coefficient and ductility ratio	$\frac{C_y}{C_E} = 0.75 \frac{1+0.05\mu}{\sqrt{2\mu-1}}$ (equ.3.2.3-2 of the Standard) is used. This is modified "Energy constant principle".	Same equation, which is modified "Energy constant principle", is applied. Note: BNBC follows displacement constant principle. (Supplement A1)

8.Evaluation of Ductility index F	$R_{mu} = {}_cR_{my} + {}_cR_{mu}$ $= {}_cR_{my} \cdot (1+(10{}_cQ_{su}/{}_cQ_{mu} - 1.1)$ $\leq R_{max}, R_{30}$ (A1.2.2& 3) $F = \dfrac{\sqrt{2\mu - 1}}{0.75 \cdot (1 + 0.5\mu)}$	(No change) q =1.1, in case interval of hoop, S >100mm. (Chap3.4.3)
B: Ductility index *F* related modification(*)(Chap3.4.3)		
1. Upper limit of deformation capacity 1.1 Column axial force ratio $N/(b.D.F_c)$ and Ductility index, F	Column tie interval > 100mm, $N/(b \cdot D \cdot F_c) > 0.4$, $F = 1.0$ Column tie interval ≤ 100mm, $N/(b \cdot D \cdot F_c) > 0.5$, $F = 1.0$ $N/(b \cdot D \cdot F_c) \leq 0.6$ is applicable range. b, D: Column width and depth F_c: concrete strength N: Axial force without load factor ($N/(b.D.F_c) < 0.4$ is common.)	**Ordinary concrete**: In case that shear reinforcement of column by BNBC, 2-10mm@ 150mm is satisfied or P_w (shear reinforcement ratio) ≥ 0.2% **0.4 < $N/(b \cdot D \cdot F_c) < 0.55$, $F = 1.27$** (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. **Low strength concrete**: In case that shear reinforcement 2-10mm@ 200mm is satisfied or $P_w \geq 0.15\%$ **0.4 < $N/(b \cdot D \cdot F_c) < 0.6$, $F = 1.27$** (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 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)	It needs further consideration related to BNBC. (*)
1.3 Tensile re-bar ratio	${}_cR_{max}(t) = {}_cR_{250}$ *for* $p_t > 1.0\%$ 1.0% (A1.2-8)	${}_cR_{max}(t) = {}_cR_{250}$ for $p_t > 1.3\%$ and Low strength concrete if $M/(Q \cdot D)$, $F = 1.0$ (Supplement 4)
1.4 Interval tie (hoop)	${}_cR_{max}(b) = {}_cR_{250}$ for $s/db > 8$ (A1.2-9)	(No change) It needs further consideration related to BNBC. Requirement of ($s/db \geq 6$) 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.

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, **Ordinary concrete, $F = 1.5$** (aim of storey deflection angle, R=1/124) ~ **1.75** (1/100) **Low strength concrete** (F_c< 13.5N/mm^2),**$F = 1.27$** (R=1/150) ~ **1.5** (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.)
3. Beam column joint and Ductility Index F	No specific requirement. (It's very rare of damage of beam column joint is very rare, since column size is relatively big.)	If no safety has been confirmed for the beam column joint, upper limit of column F, following is used. **Ordinary concrete, max. $F = 1.75$** (aim of storey deflection angle, R=1/100) **Low strength concrete, 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.)
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. (Chap3.4.2)
C: Strength index *C* related modification(*) (Chap3.4.2)		
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.
2.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 2nd level screening. Similar condition of F of B.3is applied.

D: Irregularity index S_D related modification(Chap3.5)		
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.
E: Others		
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

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

1.4 DEFINITIONS

(1) Indices for seismic performance of buildings

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

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

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

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

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

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

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

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

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

MATERIAL STRENGTH: Compressive strength of concrete and yield strength of reinforcing bar that are used to calculate the flexural and shear ultimate strengths of structural members.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

(3) Indices for judgment on seismic safety of buildings

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

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

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

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

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

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

1.5 NOTATION

This notation is stated in the J. Standard.

		Page No*
A_c	: Total cross-sectional area of columns (mm^2) 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^2).	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 storey concerned (mm^2).	1-16
a_t	: Total cross sectional area of tensile reinforcing bars in column (mm^2).	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^2).	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^2).	1-58
	Notice: Some equations use a_t. These meaning are different respectively.	
Σa_{wv}	: Total vertical reinforcing bars in the shear wall (mm^2).	1-50
A_{W1}	: Total cross-sectional area of walls with two boundary columns in the storey and effective to the direction concerned (mm^2).	1-15
A_{W2}	: Total cross-sectional area of walls with one boundary column in the storey and effective to the direction concerned (mm^2).	1-16
A_{W3}	: Total cross-sectional area of walls without columns in the storey and effective to the storey concerned (mm^2).	1-16
ΣA	: Sum of cross sectional areas of column and wing wall and wall (mm^2).	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 storey 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
${}_bQ_{mu}$	: 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_0	: 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

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		Page No*
${}_cR_{50}$	: 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^2), 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.	1-9,21,53
H_0	: 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 $\sum a_{tg}$.	1-51
h_{C0}	: 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-storey 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 storey 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

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		Page No*
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
	Notice: Some equations use N. These meaning are different respectively.	
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
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-story 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

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		Page No*
U	: Usage index.	1-39
W	: Total weight of the storey 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
α_1	: Effective strength factor of the columns at the ultimate deformation of the walls.	1-10
α_2	: Effective strength factor of the walls at the ultimate deformation of the extremely short columns,.	1-10
α_3	: Effective strength factor of the columns at the ultimate deformation of the extremely short columns.	1-10
α_j	: Effective strength factor in the *j*-th group at the ultimate deformation R_1 corresponding to the first group (ductility index of F_1).	1-12
α_m	: Effective strength factor of a flexural column.	1-13
α_S	: Effective strength factor of a shear column.	1-13
β	: Wing wall length in compressive side divided by D.	1-55
γ	: Factor on the precision in calculation of the uplift strength of the wall.	1-22
μ	: Ductility factor.	1-72
ΣA	: 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
$\sum A_f$	: Total floor area supported by the storey concerned (m^2).	1-16
$\sum W$	: Total weight (dead load plus live load for seismic calculation) supported by the storey concerned.	1-16,18
σ_B	:Compressive strength of concrete for evaluation	
σ_{sy}	: Yield strength of horizontal shear reinforcing-bars in the wing wall (N/mm^2).	1-50,51,55
σ_y	: Yield strength of tensile reinforcing bars (N/mm^2).	1-44,57,58
σ_{wy}	: Yield strength of shear reinforcing-bars in the column (N/mm^2).	1-55,50,59
σ_0	: Axial stress in column (N/mm^2).	1-44
σ_{0e}	: Axial stress in wall.	1-51,55
τ_C	: Shearing unit stress at the ultimate state of columns.	1-15
τ_{SC}	: Shearing unit stress at the ultimate state of extremely short columns.	1-15
τ_{W1}	: Shearing unit stress at the ultimate state of walls with two boundary columns.	1-15
τ_{W2}	: Shearing unit stress at the e ultimate state of walls with two boundary columns.	1-15
τ_{W3}	: Shearing unit stress at the ultimate state of walls without columns.	1-15

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CHAPTER 2. BUILDING INSPECTION

2.1 GENERAL

The inspection shall be conducted to check the structural characteristics of the building which is necessary to calculate the seismic index of structure I_S. Appropriate method for inspection such as site inspection, collection of design drawings and material test should be followed in accordance with the condition of building.

Building inspection is conducted to identify the historical records and current state of a building. Seismic capacity is to be evaluated by carrying out site inspection, actual measurement, test, etc. taking into account of irregularity, ageing of the structure mentioned in section 3.3 and 3.4 respectively as irregularity index and time index. At the same time the test of material strength and the sectional properties of each member shall also be checked. Building inspections for evaluating seismic performance should be conducted through preliminary and detailed inspection. Each of these inspections are carried out by following the standard procedure shown in Figure 2.1 and described in Table 2.1.The person responsible for the seismic evaluation shall determine the details of inspection to be conducted in accordance with the evaluation level and by considering the size, importance and inspection feasibility of the building.

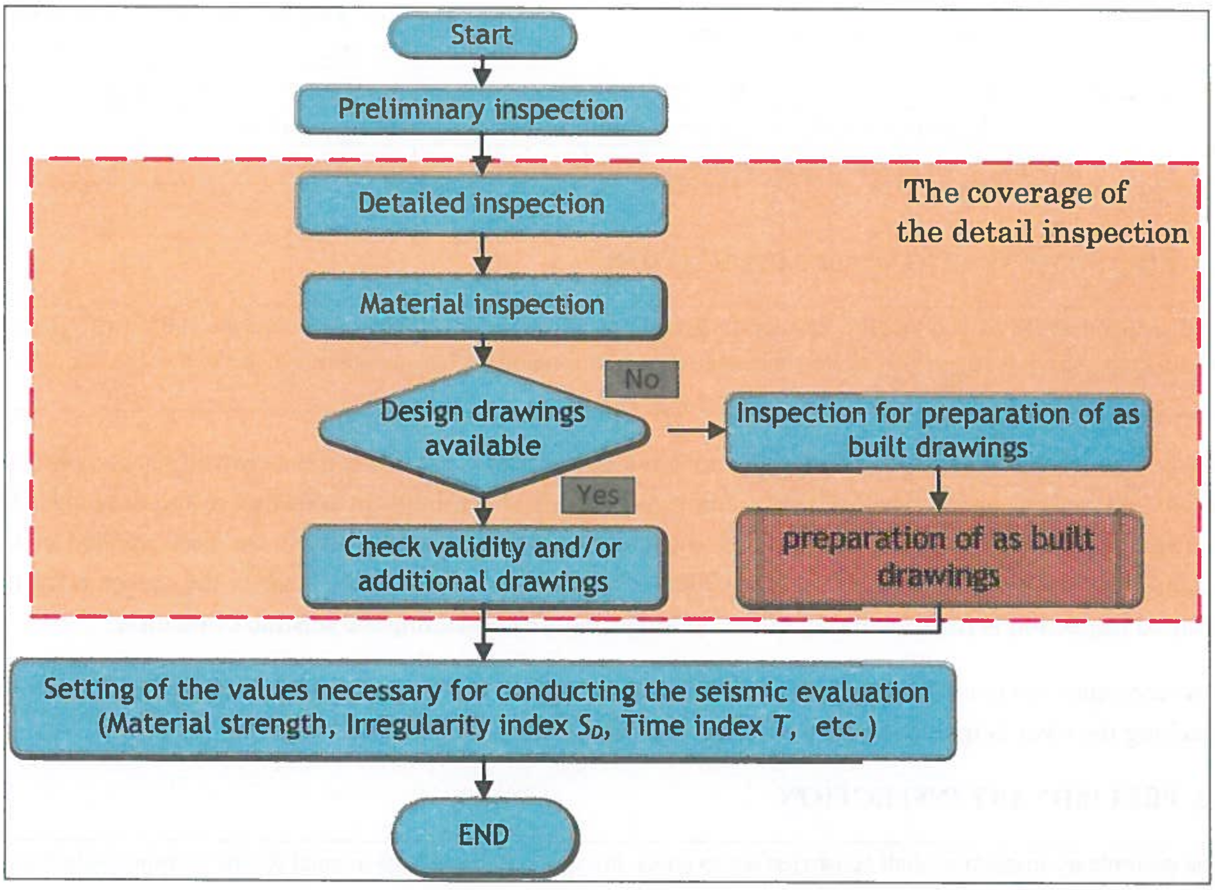


Figure 2.1 Standard Procedure for Building Inspection

Table 2.1 Inspection Items

Inspection Items		
Inspection Level	Detail Works	Remarks
Preliminary	Collection of general building information	
	Collection of drawings and necessary documentations	
	Confirmation of status of the buildings	Usage deterioration
	Confirmation of serious defects, if any.	Illegal construction (extension, incorrect usage), construction quality, structural configuration
	Confirmation of locations for sample collection (Concrete and reinforcing bars)	
Detailed	Verification of drawings	Storey height, span length, dimension of column/beam/slab, opening position of RC and brick wall
	Checking of construction quality	
	Checking of degradation	Check each floor
	Confirmation of main rebar of columns, beams and slabs by striping concrete cover.	
Material	Collection of concrete core samples	One sample from each floor or three samples
	Carry out concrete carbonation test	Each concrete core samples
	Collection of samples of reinforcing bars	Three samples
	Carry out masonry shear strength test on brick wall	Three tests

2.2 RECOMMENDATION FOR APPLICATION

It is recommended to conduct the detailed inspection to all the buildings that are to be the subject of seismic evaluation. Also it is suggested tofollow the instruction of this manual when conducting inspection of the buildings.

Most of the RC buildings in Bangladesh do not have as-built drawings. Even if the drawings are available it is very difficult to check whether the building has actually been built as indicated in the drawings. By knowing the year of construction, it is also difficult to confirm whether BNBC has been applied or the strength of reinforcing bar and concrete which was common at that time. That is the reason why the detailed inspection is required for all of the buildings before conducting the seismic evaluation.

It is recommended to create drawings of the framing elevation. It is necessary in the seismic evaluation for checking the clear height to see if an existing wall affects the deformation of column.

2.3 PRELIMINARY INSPECTION

The preliminary inspection shall be carried out to check the applicability of this manual for the seismic evaluation.

Objective

The preliminary inspection aims to obtain the basic information about the target building, to determine the applicability of this manual and to collect the information and documents necessary for conducting the on-site inspection.

(a) General building information

The name, location, current occupancy of the building, designer, builder, construction work supervisor, completion year,design year of the building are to be noted. Furthermore, the number of stories, height, main structure type and foundation type, building area, total floor area, floor height, plan and elevation, main interior and exterior characteristics, ground and geographical features of the site are mainly considered to prepare the summary of the building.

(b) Availability of the relevant documentions

Availabe records related to the design of the building (architectural drawings, structural drawings, structural calculation sheets, specifications, design modification drawings and ground inspection report) are collected in this phase of inspection. Detailed description of structural design is a very important issue for conducting accurate seismic evaluation of the building information.

If there is no available design drawings, the possibility of reproducing these document are explored . If these are not reproducible, the seismic evaluation may be deemed to be infeasible because of the lack of documents.

(c)Historical records

Records of occupancy, extension and alteration of building, deterioration due to aging, disaster records etc are noted.Interviews are conducted to examine the types of disasters suffered by the building so far.

(d)Feasibility of the on-site inspection

Feasibility of the on-site inspection is determined according to the condition of the building.

This sample office building has been taken as example for seismic evaluation in this manual. The building has been used in every step of evaluation from visual inspection to detail evaluation.

Table 2.3.1 Outline of Example Buildings (Specified in Drawings)

Component Type	Description
Usage	Office building
Number of storey	Five
Building height	15.24m (50′-00″)
Structural type	R.C Framed Structure
Foundation type	Shallow Foundation
Building area	377.38Sqm
Total floor area	1888.9 Sqm
Year of the design	1985
Year of the construction	1985
Concrete strength (Design)	F_c=13.7N/mm^2 (2,000 Psi)
Re-bar Yield Strength (Design)	σ_y= 275N/mm^2 (40,000 Psi)
Foundation capacity	1tsf (10.7t/m^2)
Soil type	S3

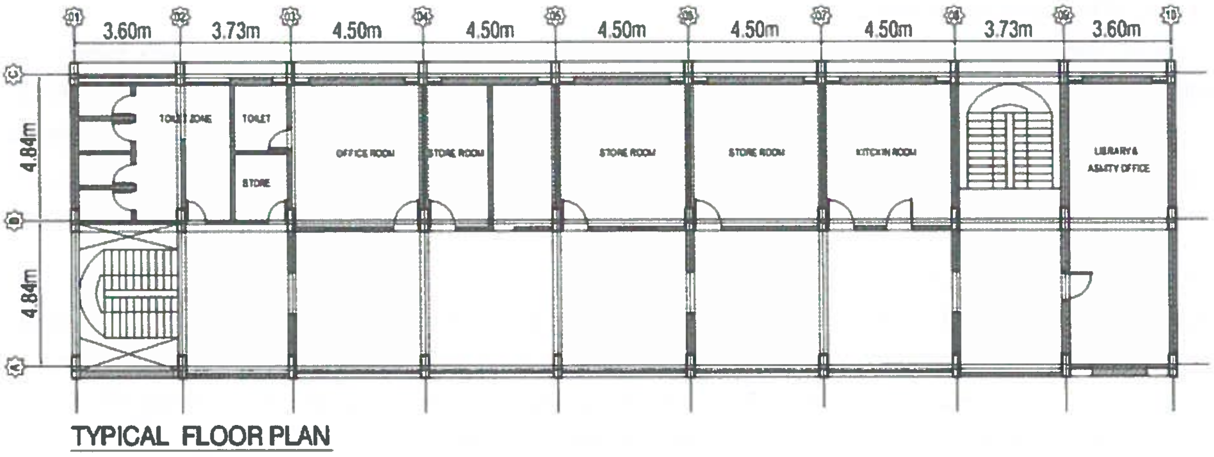


Figure 2.3.1 Typical Floor Plan

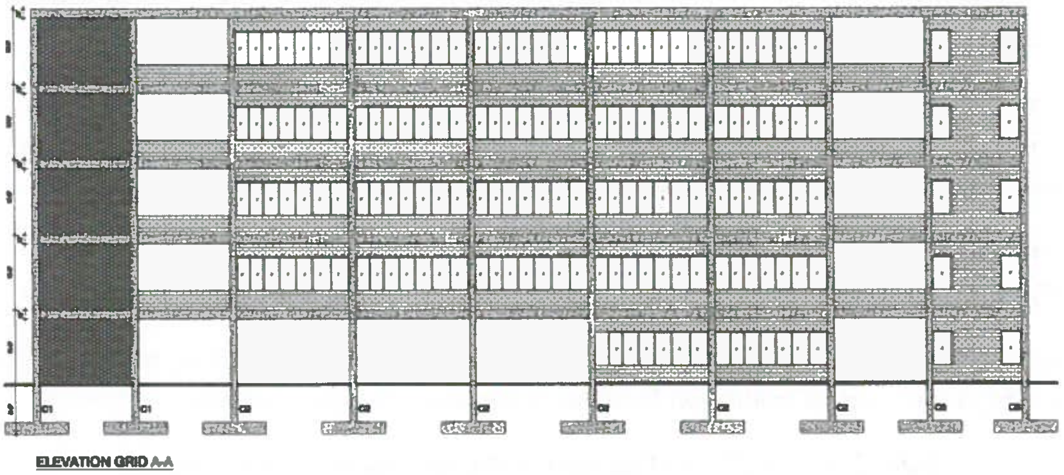
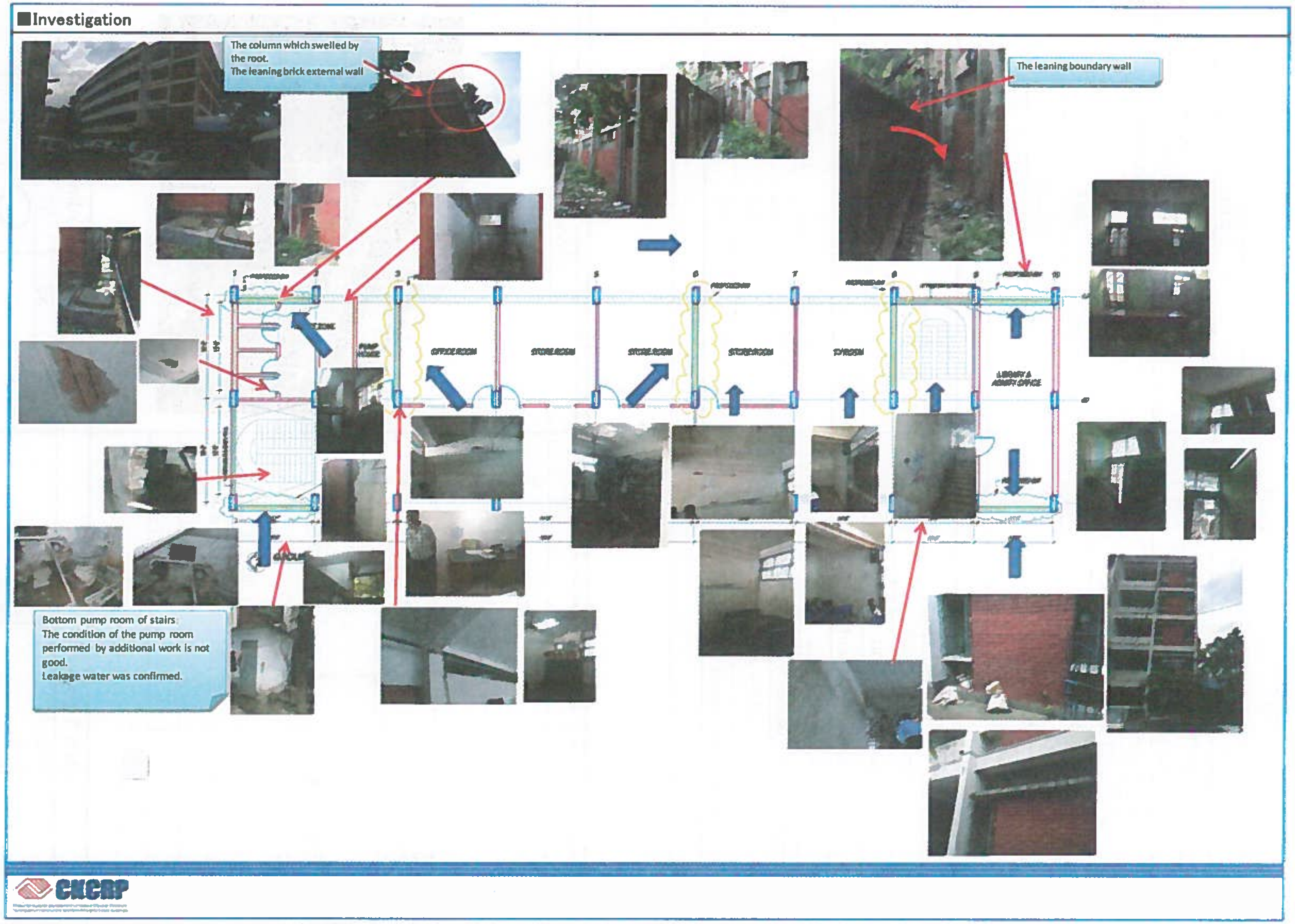


Figure 2.3.2 Elevation



■Investigation

The column which swelled by the root.
The leaning brick external wall

The leaning boundary wall

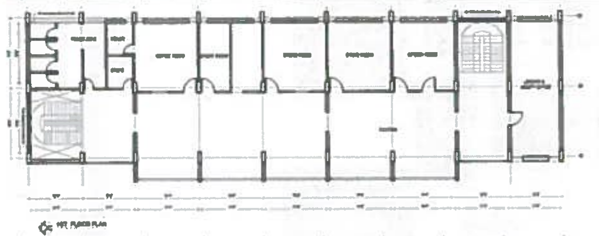
Bottom pump room of stairs:
The condition of the pump room performed by additional work is not good.
Leakage water was confirmed.

Figure 2.3.3 Condition Record of Building on Preliminary Inspection

Preliminary Inspection Sheet

Building Name	Government Office
Address	Dhaka
Above Ground Floor	Five
Basement Floor	-
Structrure Type	RC Framed structure
Foundation Type	Shallow Foundation
Design year	1985
Completion Year	1985
Building Area (m^2)	377.38
Total Floor Area (m^2)	1886.9
Building height (m)	15.24
Architectural Design	☑ Yes ☐ No
Structural Design	☑ Yes ☐ No
Soil test Report	☑ Yes ☐ No
Calculation Sheet	☐ Yes ☑ No
Approved Plan	☑ Yes ☐ No
Plan approved by (name of the Organization)	Rajuk
Approved for story	Five
Construction completed (floor)	Five

Comment

SKETCHES	PHOTOGRAPH
	
scale:	

OCCUPANCY

Assembly	Industrial	[✓ Office]
Commercial	Govt	Residential
Emer.Services	Historic	School

SOIL TYPE(according to BNBC-1993)

S1	S2	S3	S4
		○	

Figure 2.3.4 Preliminary Inspection Sheet

2.4 DETAILED INSPECTION

Detail inspection should be conducted based on the following inspection items, which are necessary for calculation of the seismic index of structure:

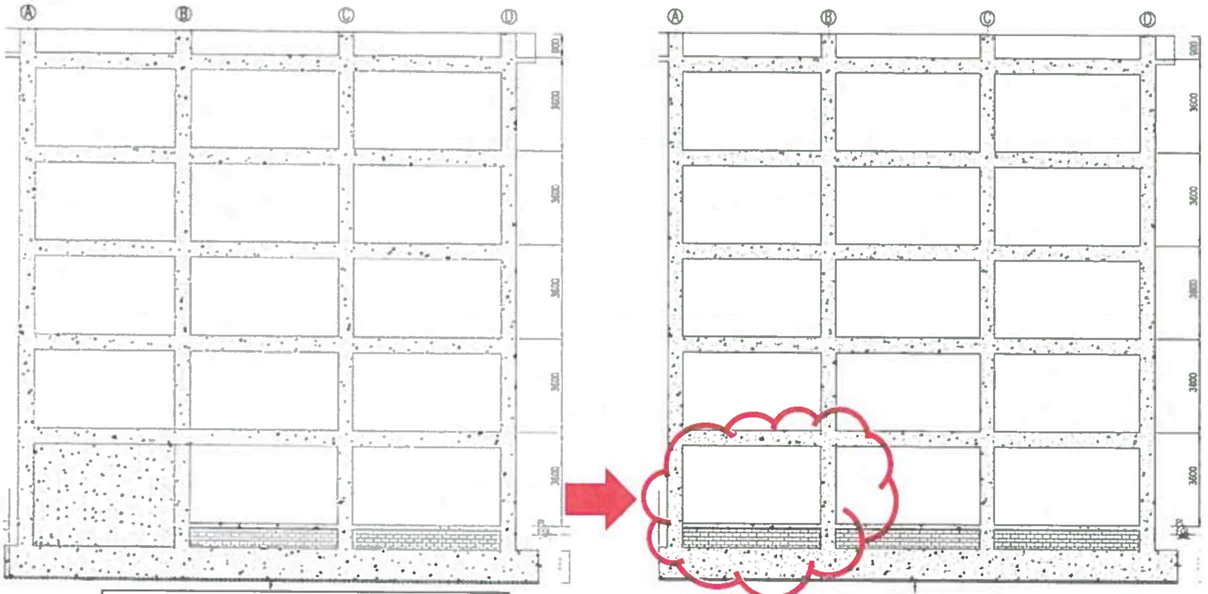
(1) Comparison of the current status of the building with the design drawings.
(2) Cross-sectional dimensions for calculation of strengths of structural members.
(3) Cracking and deterioration in concrete and deflections of structure for evaluation of time index.
(4) Building configuration for evaluation of irregularity index.
(5) Ground and geographical features of the site

Building inspector may conduct visual inspection or measurement without breaking the finishing materials. The finishing materials should be removed if necessary to ascertain grades of cracking and aging.

2.4.1 Comparison of the Current Status with the Design Drawings.

The changes in occupancy,removal of brick wallsetc are identified by referring to the original design drawings. In particular, wing walls, hanging walls, partitions etc. must be carefully inspected as they are removed or newly installed very often. Also, the shear walls, brick walls and openings must be carefully inspected as they significantly affect the evaluation results.

Locations of store houses, generators, and other heavy facilities have to be marked as these are considered to be the loads.



Grid9 (The design drawings)

Grid9 (The current status)

Figure 2.4.1 Comparison of the Current Status with the Design Drawings

2.4.2 Cross-Sectional Dimensions for Calculation of Strengths of Structural Members

The major column sections, wall opening shapes etc. are inspected in several places of each floor in order to compare them with design drawings. If there are no design drawings, all the members must be inspected, in principle.

2.4.3 Cracking in Concrete and Deformations of Structure for Evaluation of Time Index.

The cracks on the exterior and interior surfaces of the building and the corrosion of reinforcing bars are visually inspected. If any deflections, inclined column, differential settlement etc. are found, they must be specifically surveyed.

(a) Cracking inspection

Even if a reinforced concrete building is in good condition, it cannot avoid getting cracks with time. It is desirable to visually check the concrete surface of the structural frame. If it is impossible to directly check the structural frame, there is no option but to presume the condition by observing the finished surface. Cracks occur due to a number of complex factors. But it would be possible to find out the cracks that were caused by structural factors (shear cracks and flexural cracks), deformations (differential settlement etc.), drying shrinkage or deterioration by conducting visual observation. The inspection results are indicated in the framing elevation by using a crack scale or conducting visual observation to identify the crack width, length and factors. In case of a building which has been refurbished in the past, it is necessary to properly check and determine whether the cracks were repaired at the time of refurbishment. If the cracks were completely repaired at the time of refurbishment and no additional cracks have occurred, the building is considered to have no cracks. If a number of cracks which will seriously affect the structure are observed, it is necessary to conduct the crack inspection which is separately conducted in detail.

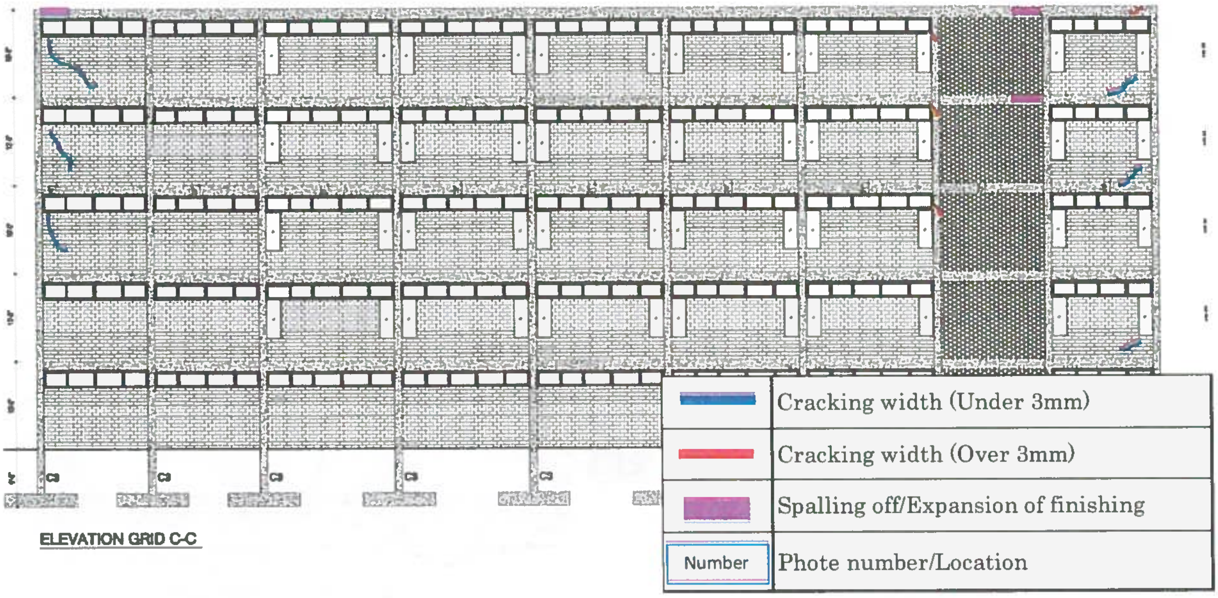


Figure 2.4.2 Drawing Showing Result of the Cracking Inspection

(b) Inspection of differential settlement

The cracks caused by differential settlement extend obliquely upward from the part where the settlement is insignificant to the part where it is significant. It is possible to estimate in which direction the settlement is extending depending on the crack pattern observed in the wall.

Figure 2.4.3 Inspection of Differential Settlement

If the differential settlement is determined to be present, the relative settlement amount is measured by using a level meter etc. In this case, the deformation in the entire building is identified by observing the deformation trend at each floor and calculating the average value of the settlement amount at each floor.

2.4.4 Building Configuration for Evaluation of Irregularity Index.

This inspection is carried out by observing the items shown in Table 6 of chapter 3[J.Standard 1-27] which are required for calculating the irregularity index.

Figure 2.4.4 Irregularity in Plan

With regard to the expansion joint the clearance is often not taken into consideration in the course of the designing process (approx. 1/100 as the interlayer deformation angle). As a result, the current conditions need to be inspected without fail even if details are shown in the design drawings.In figare 2.4.4 an U-shape structure is shown without expansion joint. One expansion joint is shown in figure 2.4.5 which is not well designed and well maintained.

Figure 2.4.5 Inspection of Expansion Joint

2.4.5 Inspection of Ground and Geographical Features of the Site

(a) Site class

Site class of the building is to be inspected according to provisions of site classification according to BNBC 2015 as shown in Table 2.4.1.

Table 2.4.1 Site Classification Based on Soil Porperties

Site classification based on soil properties				
Site Class	Description of soil profile up to 30 meters depth	Average soil properties in top 30 meters		
		Shear wave velocity V_s (m/s)	Standard penetration value N (blow/30cm)	Undrained shear strength S_u (kpa)
SA	Rock or other rock-like geological formation, including at most 5m of weaker material at the surface	>800	....	....
SB	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a grasual increase of mechanical properties with depth.	360-800	>50	>250
SC	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180-360	15-50	70-250
SD	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominatly soft-to-firm cohesive soil.	<180	<15	<70
SE	A soil profile consisting of a surface alluvium layer with V_s values of type C or D and thickness varying between about 5m and 20m, underlain by stiffer material with V_s>800m/s.	....	....	....

(b)Conditions of the geographical features

The conditions of the geographical features e.g. flat terrain, sloping land etc. are to be indicated. If the soil investigations report is unavailable, investigation to be conducted to find out the soil properties according to needs.

2.5 MATERIAL INSPECTION

The Material inspection should be conducted on the following investigation items:
(1) Strengths and Young's modulus of concrete.
(2) Arrangements, dimensions, and yield strengths of reinforcing bars.
(3) Material strengths considering carbonation, aging of concrete, and rust of reinforcing bars.

The material inspection should be conducted to collect actual data at presentincluding the concrete strength, the yield strength of reinforcing bar, shape and spacing of tie etc. By on-site inspection for accuracy of evaluations. At the same time, the inspection results provide useful data for future retrofitting design, if required. Damaged portion shall be repaired after survey work.

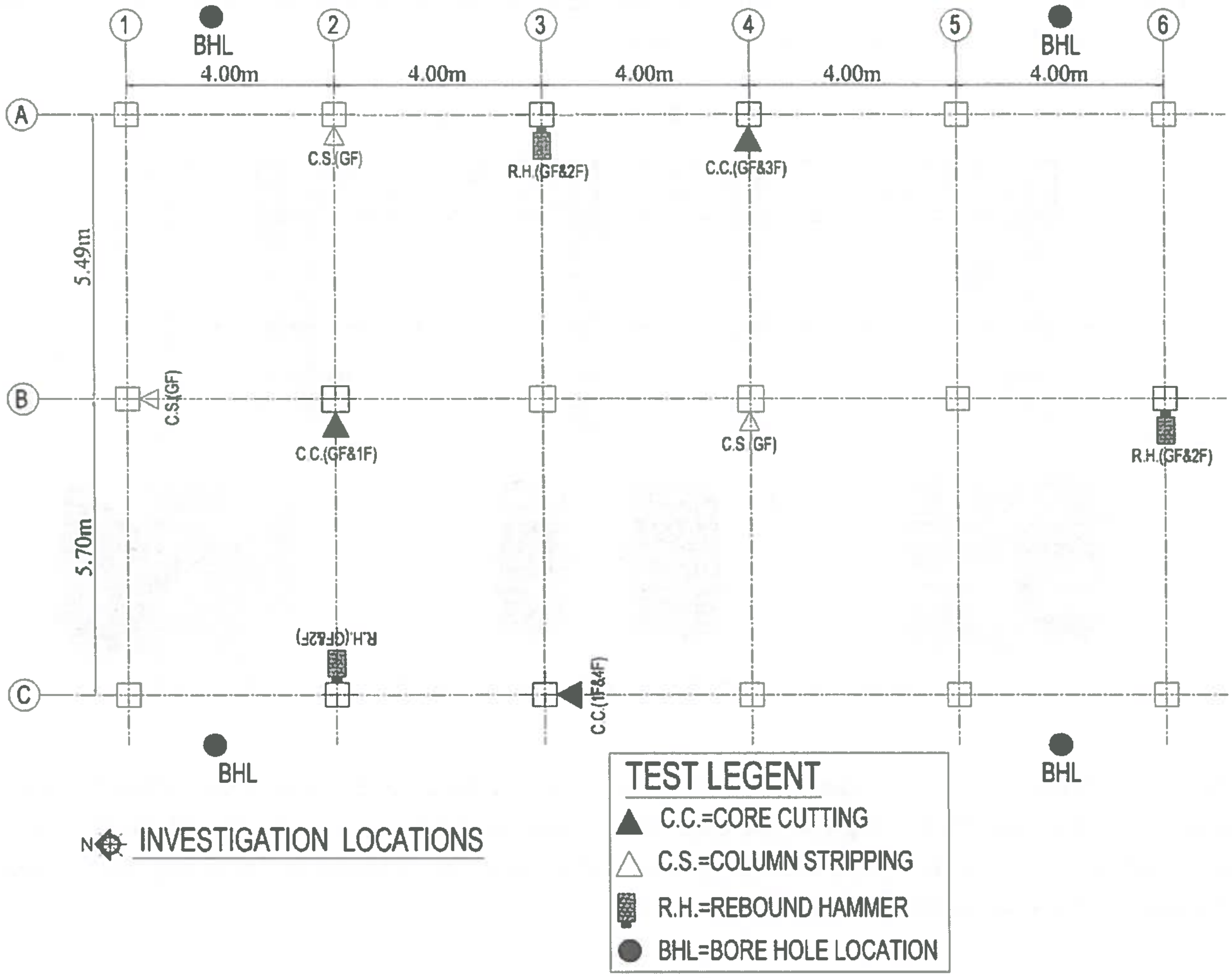


Figure 2.5.1 Building Plan Showing Investigation Locations.

2.5.1 Concrete Material Inspection

Many buildings were built with brick concrete, which used crushed brick as coarse aggregate. Characteristics of the concrete may vary widely. Also past data of test results are not available in many cases. So, inspection of concrete is necessary for the seismic evaluation.

2.5.1.1 The Compressive Strength Test of Concrete with Concrete CoreSampling

Even if the concrete strength is shown in the design drawings, it is necessary to conduct the compression test by sampling the core of the target building to check the strength. In this case, three or more concrete cores must be sampled at each floor of the building.

If the result of compressive strength test is lower than the specified design strength, the test result shall be used for the calculation of the seismic evaluation. Generally, as per construction practice in Bangladesh, concrete is cast in three segments in each floor, 2 segments in column and slab-beam as one segment. Moreover there is a thin kicker at the bottom of each column. Also, widely varied strength of concrete in each batch is found since the concrete is often prepared at the site. Ideally it is recommended to collect more than 3 cores from each batch of each floor casting during construction period. Concrete cores shall be collected from the columns, walls if any, and floors.

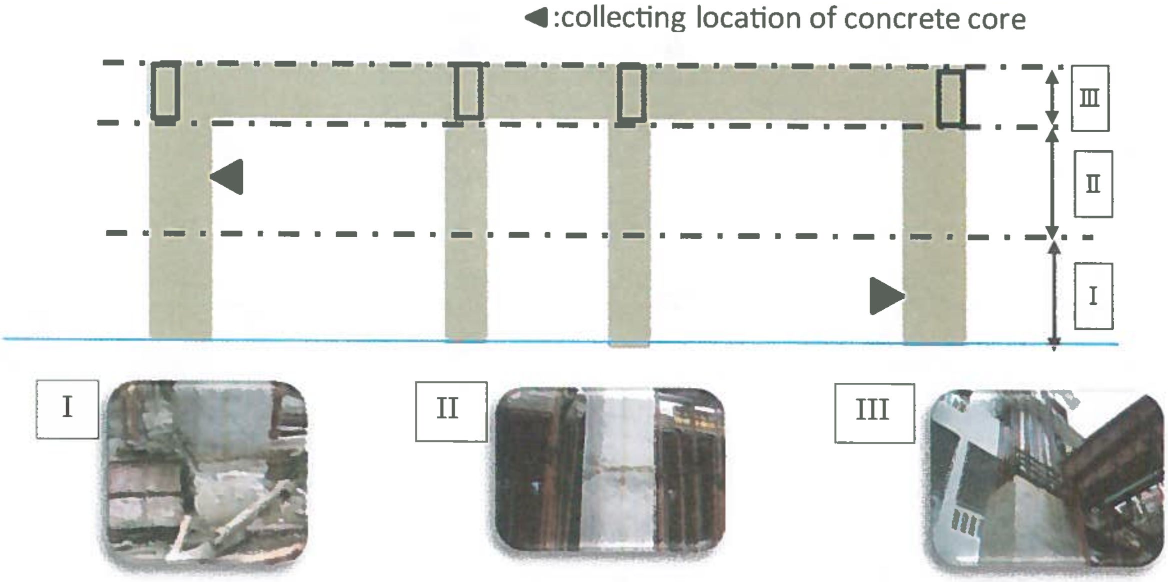


Figure 2.5.2 Sequence of Concrete Casting in Segments, Practiced in Bangladesh

Special attention must be paid when collecting cores from the column on the lower floor, which bears the high axial force, since the strength of the concrete, is expected to be low. It is also acceptable to collect concrete cores from at least three different places, and conduct the compression test using the Schmidt Hammer to make up for the shortage.

In this case, it is recommended to conduct core compression tests if retrofitting design is to be carried out after seismic evaluation. The detail of sampling methods is shown below:

- Core samples to be collected from column, beam, slab, and shear wall.
- One or more concrete cores must be sampled at each floor which represents each part of segment during casting.
- The core dimensions should be preferably 10cm (diameter) × 20cm (height). In no case the height to be less than 10cm
- Minimum 50 mm core diameter in general. (Ref. ACI 214.4R-03)
- Test samples shall not include any reinforcing-bars or cracks.
- The compression test must be conducted in accordance with ASTM C-42.
- The strength of tested value divided by 0.85 may be used. (Ref. ACI 437)

The concrete compressive strength test is conducted by a reliable institution or laboratory. The concrete compressive strength is figured out by assuming the value obtained by subtracting a half of the standard deviation (σ) from the average core value (X_{mean}) at each floor as the estimated strength (σ_B) at each floor. The core holes should be filled with non-shrink concrete or mortar of comparable strength of existing concrete.

$$\text{Average value: } X_{mean} = \frac{(X_1 + X_2 .... + X_n)}{n}$$

$$\text{Standard deviation} \quad : \sigma = \sqrt{\left(\frac{\Sigma (X_i - X_{avg})^2}{(n-1)} \right)}$$

[Example]

Floor	Core ID	Core length (h) mm	Core dia. (d) mm	Aspect ratio (*h/d*)	Compressive Strength from core test kN/mm^2	Correction factor based on aspect ratio	Correction factor based on sample (core/cylinder) Ref. ACI 437	Final compressive strength kN/mm^2	Average of Compressive Strength m N/mm^2	Standard deviation σ N/mm^2	Compressive Strength for evaluation σ_B' N/mm^2
3RD	01-L4C4	200	100	2.00	9.32	1.00	1/0.85	10.97	10.1	3.4	8.4
	02-L4C4	200	100	2.00	11.75	1.00	1/0.85	13.82			
	03-L4E2	200	100	2.00	4.83	1.00	1/0.85	5.68			
	04-L4E2	200	100	2.00	8.64	1.00	1/0.85	10.16			

$$X_{mean} = \frac{(10.97 + 13.82 + 5.68 + 10.16)}{4} = \mathbf{10.1}$$

$$\sigma \quad = \sqrt{(10.97 - 10.1)^2 + (13.87 - 10.1)^2 + (5.68 - 10.1)^2 + (10.16 - 10.1)^2} = \mathbf{3.4}$$

$$\sigma_B \quad = 10.1 - 3.\frac{4}{2} = \mathbf{8.4}$$

Building Name: **Completion Year:**

Result of concrete Compressive Strength

Floor	Core ID	Final Compressive Strength kN/mm^2	Average of Compressive Strength m N/mm^2	Standard deviation σ N/mm^2	Compressive Strength for evaluationσ_B' N/mm^2
3RD	01-L4C4	10.97	10.1	3.4	8.4
	02-L4C4	13.82			
	03-L4E2	5.68			
	04-L4E2	10.16			
2ND	05-L3D2	9.75	9.1	2.6	7.8
	06-L3F3	7.21			
	07-L3D5	5.68			
	08-L3D4	10.51			
	09-L3F6	12.19			
1ST	10-L2E6	5.73	5.7	-	5.7
	11-L2D6				
GF	12-L1G6	9.85	12.7	4.0	10.7
	13-L1G6	14.74			
	14-L1F1	7.41			
	15-L1H1	20.94			
	16-L1H7	13.21			
	17-L1D1	11.84			
	18-L1C1	11.99			
	19-L1C3	11.43			
B1					
Result of whole building			10.7	3.8	8.8
specified design strength			Aggregate: Brick		

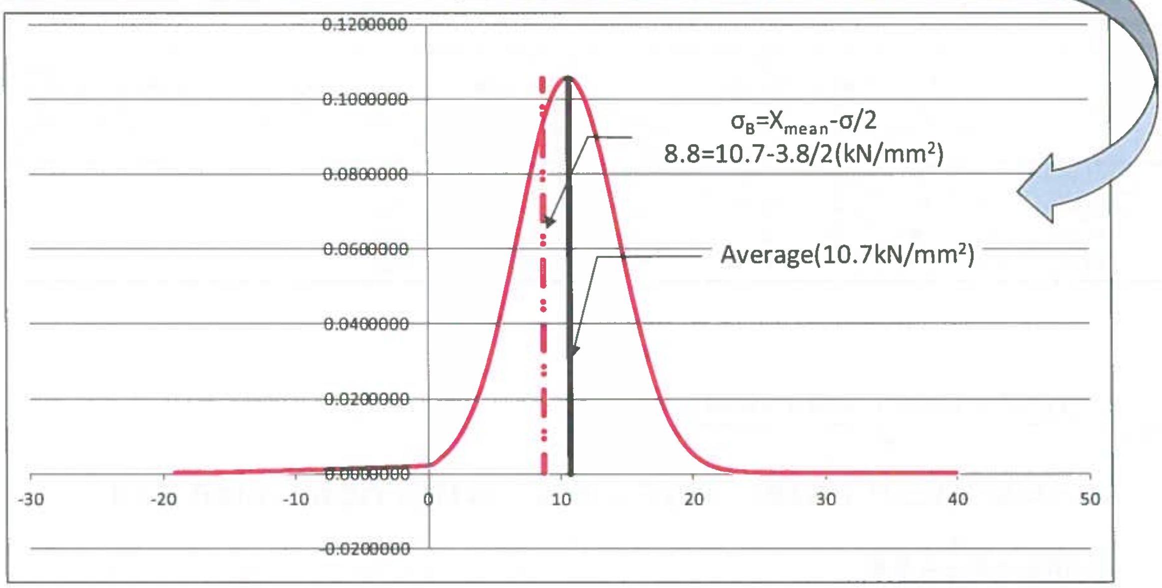


Figure 2.5.3 Example of Result of Compressive Strength of Concrete

Detecting re-bar arrangement

Marking re-bar arrangement

Setting core drill

Picking up core sample

Pulling out core sample

Filling up core hole with non-shrink mortar

Figure 2.5.4 Procedure of Core Sampling

2.5.1.2 Strength Test Conducted by Using the Schmidt Hammer

In principal, result of the core compressive strength test should be used as concrete compressive strength. The results of strength test conducted by using the Schmidt hammer are applied to complement the results of the core compressive strength test. To estimate the concrete compressive strength based on the test results obtained by using Schmidt Hammer, it is to be noted that there may be significant variations between tests results.

Schmidt hammer test shall be based on ASTM C805-97.

The following procedure shall be performed

•N-Type concrete Schmidt hammer shall be used.

•The 5cm spacing grids (20 × 20 cm surface) shall be drawn on the wall.

•The test surface shall be cleaned by sand paper,

•The rebound hammer shall be used in the horizontal direction.

This test shall be done on the surface after stripping the plaster.

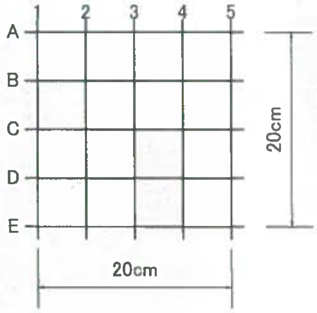
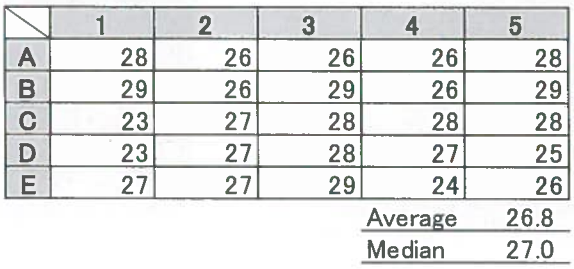


Figure 2.5.5 Grids



	1	2	3	4	5
A	28	26	26	26	28
B	29	26	29	26	29
C	23	27	28	28	28
D	23	27	28	27	25
E	27	27	29	24	26
				Average	26.8
				Median	27.0

Figure 2.5.6 Data Sheet (Results are Sample Data)

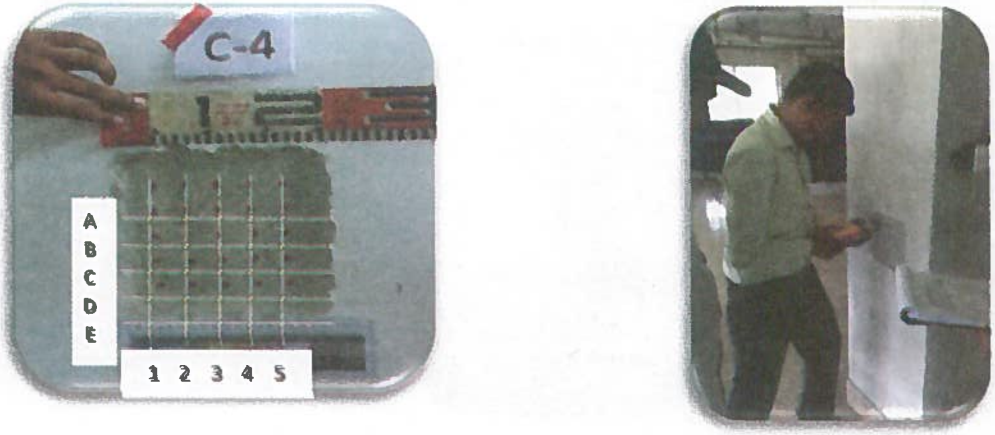


Figure 2.5.7 Schmidt Hammer Test after Stripping off Plaster

2.5.1.3 Concrete Carbonation Test

In principle, the carbonation test is carried out on the core samples at site. With regard to the carbonation depth, the maximum dimension is recorded from the splitting surfaces minus the finished material thickness. The result of carbonation evaluation shall be compared with the carbonation depth estimated by the age of the building and other factors based on the 40mm-thick of concrete cover. Then the deterioration and aging has to be determined before reflecting in the time index.

Figure 2.5.8 Concrete Carbonation Test

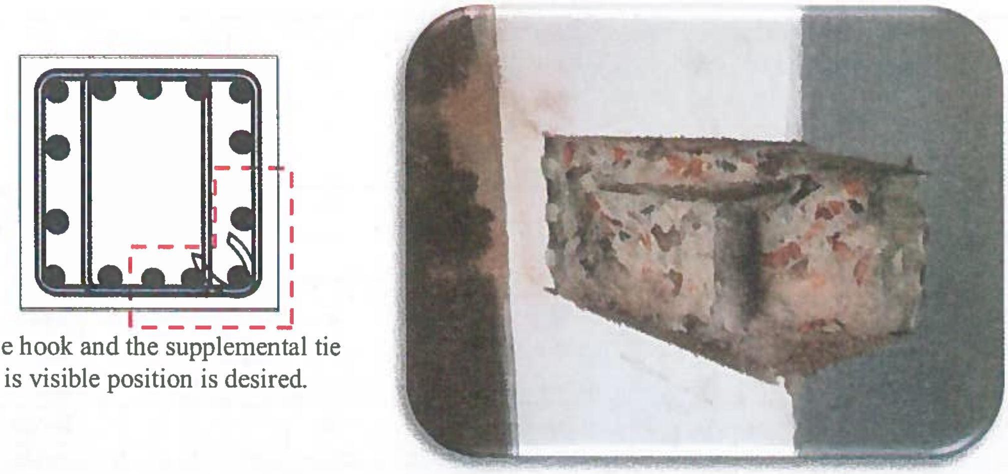
2.5.2 Inspection of Reinforcing Bars

Many buildings do not use the reinforcing bars which comply with the BNBC standard. Also, in many cases drawings have been lost and the types of reinforcing bars used are not known. It is necessary to conduct the tension test of those reinforcing bars. In case structural drawings are available and reinforcing bars used is ensured to comply with the standard, strength can be used as specified design strength without conducting an inspection.

2.5.2.1 Strength of the Reinforcing Bars

In principle, the type of reinforcing bars is judged according to the tension strength test after collection of reinforcing-bar sample. At the same time, the spacing of the stirrup and the tie, hook, condition etc. should be inspected. If the 90-degree hooks are observed during inspection, it is suggested to assume reduced shear capacity in evaluation.

Figure 2.5.9 Possible Place of Collection of Reinforcing-bar Sample



The hook and the supplemental tie is visible position is desired.

Figure 2.5.10 Location of Striping Inspection

2.5.2.2 Rust on the Reinforcing Bars

The degree of rusting is visually observed by checking the presence of rust and extent of damage due to expansion of concrete. The observation results are reflected in the time index. Also, if the reinforcing bar sections is damaged due to rust, the strength of each member is evaluated by anticipating the extent of damage in the reinforcing bar section.

2.5.3 Evaluation of the Concrete Compressive Strength

The concrete compression strength to be used for structural assessment is determined based on the results of the concrete core compression strength tests conducted as material inspection, the specified design strength shown in the design drawings etc.

(1) If the concrete core compressive strength is higher than the specified design strength, the specified design strength shall be adopted.

(2) If the concrete core compressive strength is lower than the specified design strength, the concrete core compressive strength shall be adopted.

(a) When the estimated strength $\sigma_B \geqq 9\text{N/mm}^2$

σ_B is adopted for the seismic evaluation. However, sometimes due to defects in the materials or work process considerable deterioration in concrete occurs.

Therefore, it is necessary to properly evaluate this influence by time index.

(b) When the estimated strength $\sigma_B < 9\text{N/mm}^2$

The core is resample and retested. The core must be sampled from the structural members (shear walls, beams and column). The number of test specimen to be collected shall be 3 or more. If the test result is $\sigma_B < 9\text{N/mm}^2$ again, it is necessary to seek alternate measures, which includes the plan for demolishing the building or change of use.

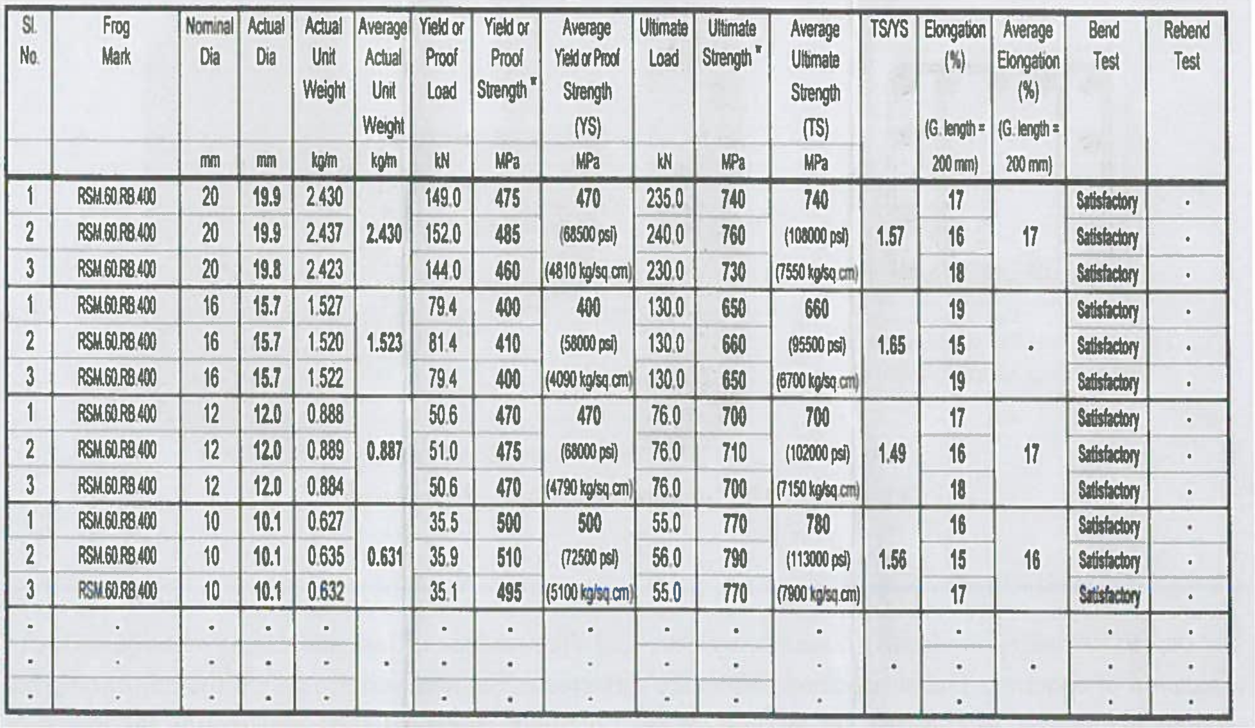
(3) Precautions

Mechanical property of the brick concrete, which includes the crushed brick for the aggregate, is not very clear. It is hoped that further study of brick concrete will create the most suitable evaluation method.

2.5.4 Evaluation of Reinforcing Bar Yield Strength Based on Inspection

In principal, the Reinforcing bar yield strength used for structural assessment is determined from the results of the tension strength tests carried out as part of the material inspection.

Table 2.5.1 Example of Tension Test Result



Sl. No.	Frog Mark	Nominal Dia mm	Actual Dia mm	Actual Unit Weight kg/m	Average Actual Unit Weight kg/m	Yield or Proof Load kN	Yield or Proof Strength * MPa	Average Yield or Proof Strength (YS) MPa	Ultimate Load kN	Ultimate Strength * MPa	Average Ultimate Strength (TS) MPa	TS/YS	Elongation (%) (G. length = 200 mm)	Average Elongation (%) (G. length = 200 mm)	Bend Test	Rebend Test
1	RSM.60.RB 400	20	19.9	2.430	2.430	149.0	475	470 (68500 psi) (4810 kg/sq cm)	235.0	740	740 (108000 psi) (7550 kg/sq cm)	1.57	17	17	Satisfactory	•
2	RSM.60.RB 400	20	19.9	2.437		152.0	485		240.0	760			16		Satisfactory	•
3	RSM.60.RB 400	20	19.8	2.423		144.0	460		230.0	730			18		Satisfactory	•
1	RSM.60.RB 400	16	15.7	1.527	1.523	79.4	400	400 (58000 psi) (4090 kg/sq cm)	130.0	650	660 (95500 psi) (6700 kg/sq cm)	1.65	19	•	Satisfactory	•
2	RSM.60.RB 400	16	15.7	1.520		81.4	410		130.0	660			15		Satisfactory	•
3	RSM.60.RB 400	16	15.7	1.522		79.4	400		130.0	650			19		Satisfactory	•
1	RSM.60.RB 400	12	12.0	0.888	0.887	50.6	470	470 (68000 psi) (4790 kg/sq cm)	76.0	700	700 (102000 psi) (7150 kg/sq cm)	1.49	17	17	Satisfactory	•
2	RSM.60.RB 400	12	12.0	0.889		51.0	475		76.0	710			16		Satisfactory	•
3	RSM.60.RB 400	12	12.0	0.884		50.6	470		76.0	700			18		Satisfactory	•
1	RSM.60.RB 400	10	10.1	0.627	0.631	35.5	500	500 (72500 psi) (5100 kg/sq cm)	55.0	770	780 (113000 psi) (7900 kg/sq cm)	1.56	16	16	Satisfactory	•
2	RSM.60.RB 400	10	10.1	0.635		35.9	510		56.0	790			15		Satisfactory	•
3	RSM.60.RB 400	10	10.1	0.632		35.1	495		55.0	770			17		Satisfactory	•
•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•
•	•	•	•	•		•	•	•	•	•	•		•		•	•
•	•	•	•	•		•	•	•	•	•	•		•		•	•

2.6 INSPECTION OF BUILDINGS ON THE BASIS OF AVAILABILITY OF DESIGN DRAWINGS

Investigation items differ according to the availability of the design drawings. When the design drawings are available, the work is to verify the drawings with the existing building to see whether the building has been built as designed. When the design drawings are not available, it is required to prepare the as-built drawings.

1) Design drawings available

When the design drawings are available, it is to be checked whether the building has been constructed according to the design drawings. If not, then the drawings are not to be trusted. In that case step 2) to be followed to do the investigation. Items to be checked are shown in the Table 2.6.1.

Table 2.6.1 List of the Items for Comparison of Design Drawings at Site

Items	Application methods		Evaluation items affect seismic performance of buildings
Storey height	Compare actual measurement with the drawings.		Building weight, Shear span ratio of column, Natural period of buildings
Span length	Same as above		Building weight, Shear span ratio of beam
Section, size of members (beams, columns, slabs, etc)	Same as above (All type of members)		Strength and ductility of members
Reinforcing-bar arrangement of members (beams, columns, slabs, etc)	By scanning	All of each members	Strength and ductility of members
	By stripping	One of each members	Strength and ductility of members,
Standing, hanging and wing wall dimensions	Actual measurement		Shear span ratio of columns and beams Stiffness of building (eccentricity/S_D) Soft storey (eccentricity/S_D)
Wall opening size	Same as above		Same as above Strength of brick wall (Excluded in this manual)
Foundation section, size	Compare actual measurement with the drawings (If necessary)		Bearing capacity of soil
Soil investigations or static plate load test	Compare actual test and available report (If necessary)		Bearing capacity of soil and pile

It is to be checked whether the cross sectional shapes and the arrangement of reinforcing-bars are constructed as per design drawings. Reinforcing bar detector should be used for checking the arrangement of reinforcing-bars. Stripping of concrete to be conducted for at least one area of column, beam, and floor respectively to check the bar arrangements visually.

Framing elevation drawings are to be created during the investigation. Framing elevation is useful when studying the brick walls and their influence on deformation and stress of columns and beams. Framing elevation is also used in retrofitting design to indicate the locations of additional reinforcing bars. Table 2.6.2 shows the list of drawings, which are generally recognized to be needed for seismic evaluation.

Table 2.6.2 List of the Drawings Required for Seismic Evaluation

Drawings	Drawings title
Architectural Drawings	Site plan
	All floor plan
	All elevation
Structural Drawings	Column layout plan
	Footing layout plan
	Footing Schedule
	Column Schedule
	Beam layout plan of all floors
	Stair Details
	Beam details of all floors
	Reinforcement plan of all floors
	All frame elevations
	Soil investigation report

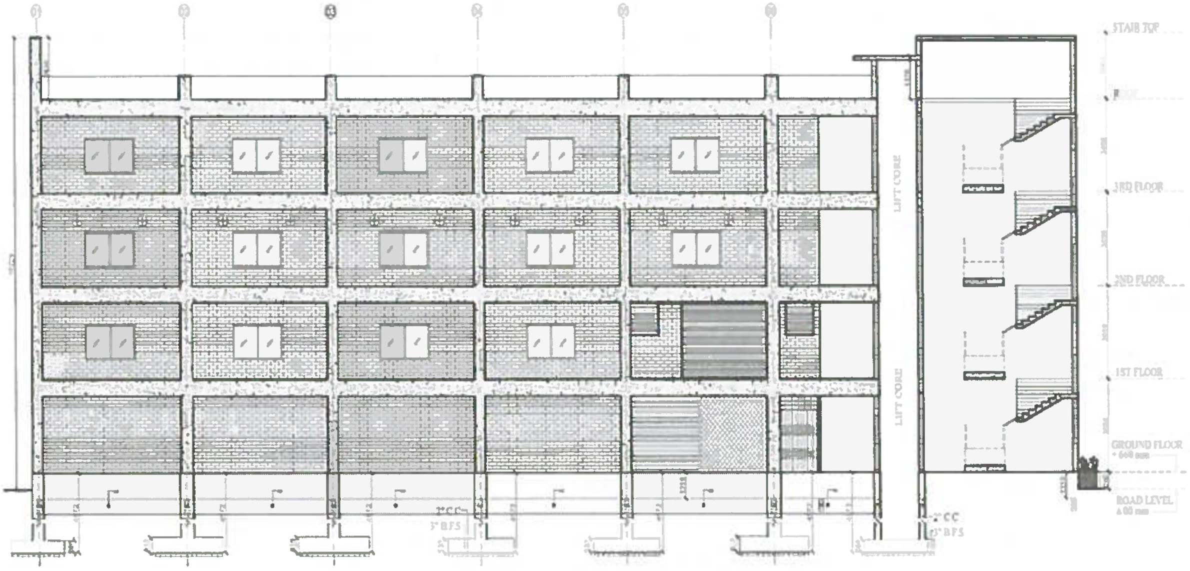


Figure 2.6.1 Example of Frame Elevation (Before Retrofit)

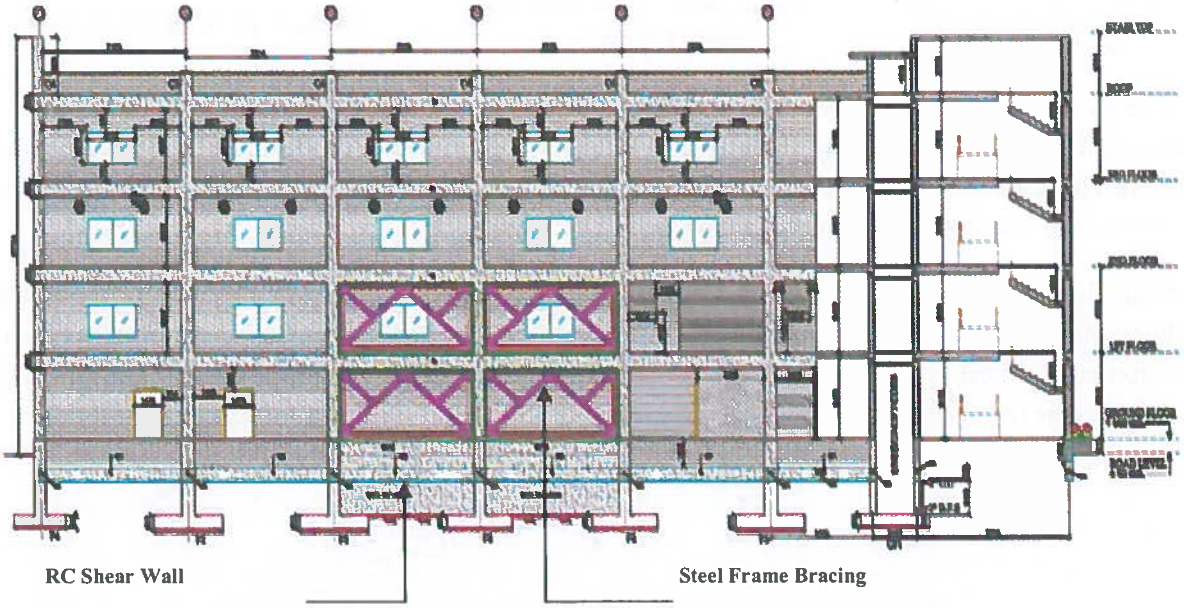


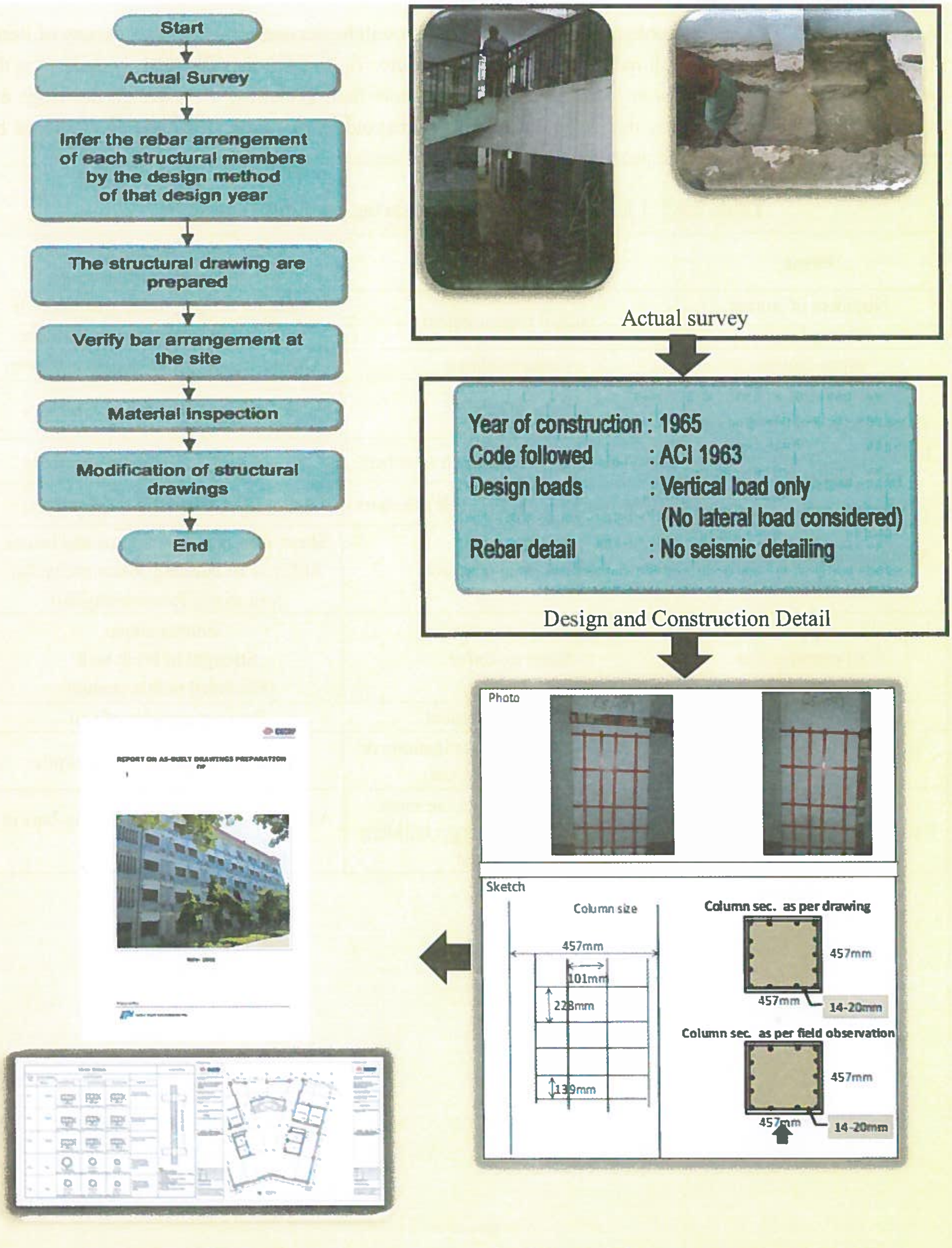
Figure 2.6.2 Example of Frame Elevation (After Retrofit)

2)Design drawings not available

If no design drawings are available for seismic evaluation, it will be necessary to inspect a variety of items in order to calculate the strength index, the ductility index etc. Table 2.6.3 lists the items required for the comparison of design drawings at site. It is important to note that, generally, if no design drawings are available, the inspection cost is increased because it is unavoidable to inspect a number of places by stripping in order to prepare the necessary drawings for the seismic evaluations.

Table 2.6.3 List of the Items for Preparing As-Built Drawings

<table>
<tr><th>Items</th><th colspan="2">Application methods</th><th>Evaluation items affect seismic performance of buildings</th></tr>
<tr><td>Numbers of stories
Storey height</td><td colspan="2">Actual measurement</td><td>Building weight, Shear span ratio of column, Natural period of building</td></tr>
<tr><td>Span length</td><td colspan="2">Same as above</td><td>Building weight, Shear span ratio of beam</td></tr>
<tr><td>Section, size of members (beams, columns, slabs, etc)</td><td colspan="2">Same as above</td><td>Strength and ductility of members</td></tr>
<tr><td rowspan="2">Reinforcing-bar arrangement of members (beams, columns, slabs, etc)</td><td>By scanning</td><td>All of each members</td><td>Strength and ductility of members</td></tr>
<tr><td>By stripping</td><td>One of each members</td><td>Strength and ductility of members,</td></tr>
<tr><td>Standing, hanging and wing wall dimensions</td><td colspan="2">Actual measurement</td><td>Shear span ratio of columns and beams
Stiffness of building (eccentricity/S_D)
Soft storey (eccentricity/S_D)</td></tr>
<tr><td>Wall opening size</td><td colspan="2">Same as above</td><td>Same as above
Strength of brick wall
(Excluded in this manual)</td></tr>
<tr><td>Foundation section, size</td><td colspan="2">Actual measurement</td><td>Bearing capacity of soil</td></tr>
<tr><td>Soil investigations or static plate load test</td><td colspan="2">By conducting soil investigations or static plate load test</td><td>Bearing capacity of soil and pile</td></tr>
<tr><td>Back up structural calculation</td><td colspan="2">Calculate according to the same method by which the target building was designed</td><td>Assuming number of reinforcing-bars in each members</td></tr>
</table>



Actual survey

Year of construction : 1965
Code followed : ACI 1963
Design loads : Vertical load only
(No lateral load considered)
Rebar detail : No seismic detailing

Design and Construction Detail

Prepared as-built drawings

Verification of bar arrangement at the site

Figure 2.6.3 Procedure for Inspection in case Design Drawings are not Available

CHAPTER 3. SEISMIC INDEX OF STRUCTURE I_S

3.1 GENERAL

The seismic index of structure $I_s$is calculated by Equation (1) of J. standard. The calculation is done at each storey and in each principal horizontal direction of a building.

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

Where:

E_o = Basic seismic index of structure.

S_D = Irregularity index.

T = Time index.

The seismic index $I_s$is calculated in either the first, the second, or the third level screening procedure with appropriate considerations specified in J. Standard for any of the screening procedure.

3.2 SCREENING PROCEDURES

As per Japanese standard, there are three levels of screening procedures. Of the three screening level procedures, the use of the second level screening method is recommended for Bangladesh. The same sub-indices will be used in calculating I_s for E_o, S_D, and T. General idea of the three screening level methods and images of building failure mode are shown below.

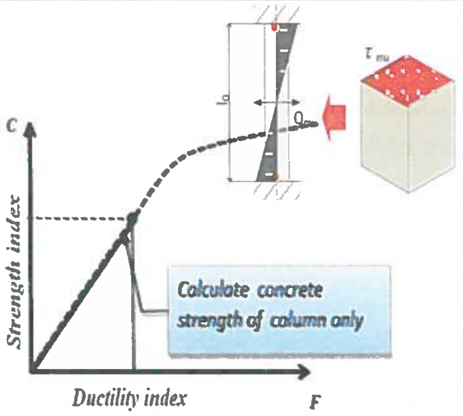
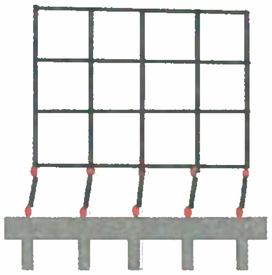
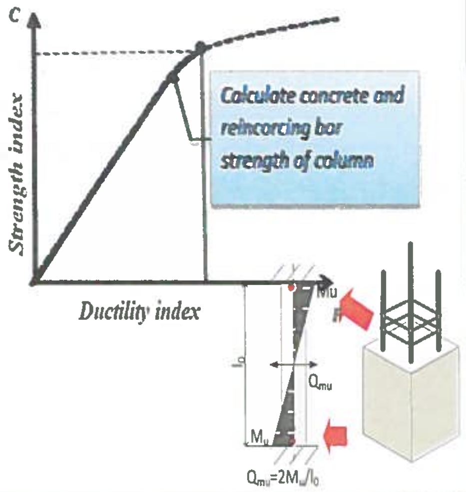
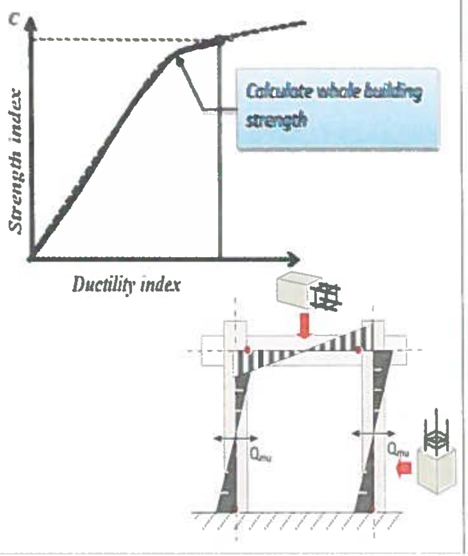
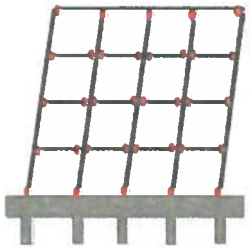
Evaluation methods	Images as a failure mode of building
First Level Screening Procedure 	This failure mode is premised on story collapse or partial collapse in the vertical direction of the building.  First Level Screening Procedure The column strength is calculated in a simplified manner using the average columnar stress of a general strong-beam-weak-column behavior. This procedure is premised on the top and bottom flexural failure of slender-columns.
Second Level Screening Procedure 	Second Level Screening Procedure Evaluation is performed by obtaining the failure mode and strength of column on the assumption of the strong-beam-weak-column behavior.
Third Level Screening Procedure 	Estimate the failure mode of a building from static analysis.  Third Level Screening Procedure Estimate the failure mode and the horizontal load-carrying capacity of a building in consideration of all members.

Figure 3.2.1 Three Screening Level Methods and Images of Building Failure Mode


1990_Luzon Earthquake

1990_Luzon Earthquake

1990_Luzon Earthquake

Figure 3.2.2 Storey Collapse in Luzon (Philippines) Earthquake of 1990

3.3 RECOMMENDATION FOR APPLICATION

Of the three screening level procedures, the use of the second level screening method is recommended for Bangladesh.

Most of the existing buildings in Bangladesh are designed considering gravity loads only, ignoring the seismic load. When such buildings collapse from an earthquake, the strong beam-weak column behavior is predicted and the application of the Second Level Screening Procedure is considered as an appropriate screening procedure since the strength of beam is larger than that of the column. Accordingly, this chapter of the manual describes recommendations in applying the second level screening procedure. The survey results obtained so far indicate that buildings will likely collapse from the shortage of the axial strength as the largest characteristic of the building damage resulting from an earthquake. With the axial force ratio being high (ground floor $N/(b{\cdot}D{\cdot}F_c) >$ approximately 0.6) under the significant effect of the low-strength concrete, if a building deforms as a result of an earthquake, it will likely collapse because the columns cannot support the vertical load. In applying this evaluation method, what remains unclear is the effect of the problems as shown below on the strength and deformability that are directly relevant with the seismic performance of buildings. Due consideration needs to be given to this point. Consideration should also be given to the low-strength concrete resulting from the extremely poor construction quality. However, it is difficult to properly evaluate those problems in the present situation. Evaluation that is conservative from the technical point of view is recommended in this manual for the issues, which are difficult to evaluate correctly at this point.

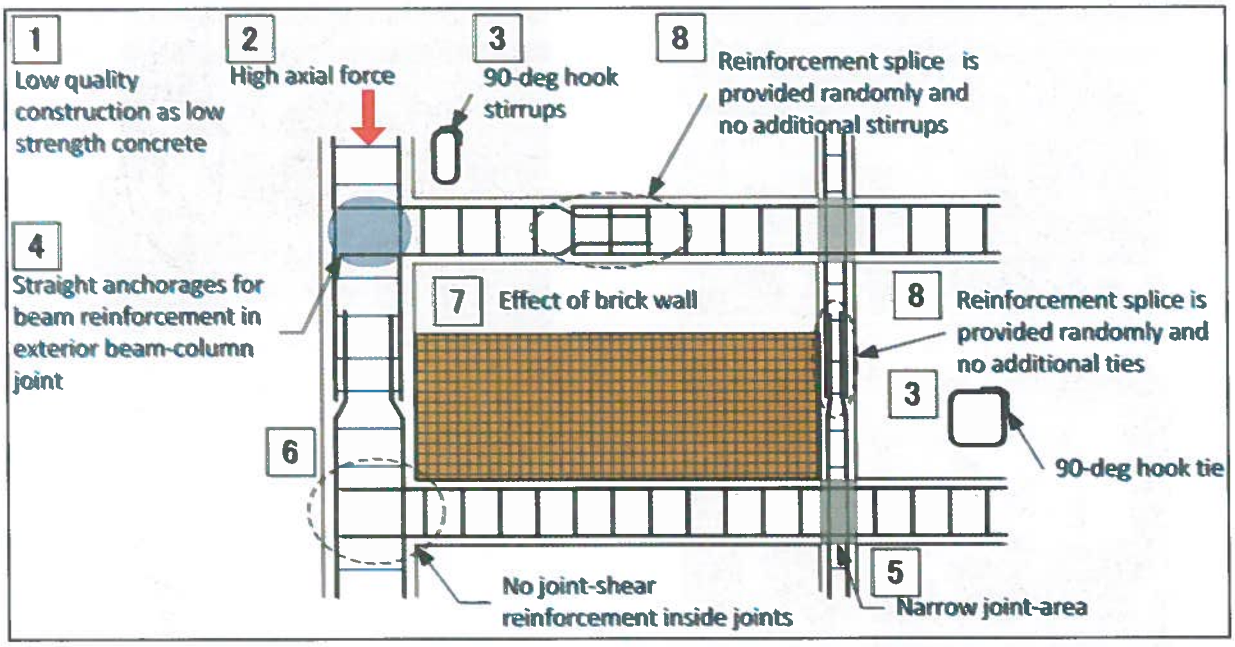


Figure 3.3.1 Issues to be Considered for Seismic Evaluation of Existing Buildings

3.4 BASIC SEISMIC INDEX OF STRUCTURE E_0

3.4.1 Calculation of E_0

The basic seismic index of structure E_o, is necessary to evaluate the basic seismic performance of the building. E_o is calculated for each storey and each major direction considering the ultimate strength, failure mode and ductility of the building. The basic seismic index of structure E_o of the i-th storey in a n-storey building is given as a product of the strength index C defined in 3.2.2 (of J. Standard) and the ductility index F defined in 3.2.3 (of J. Standard). In addition, the storey-shear modification factor, which is expressed as $(n+1)/(n+i)$ in Equation (4) and (5) of J. Standard, may be changed accounting for the lateral earthquake force distribution along the building height.

Second level screening procedure

The vertical structural members shall be classified into five categories as listed in Table 3.4.1 in the second level screening procedure, where the basic seismic index of structure E_o shall be calculated based on the detailed evaluation of the strength index C and the ductility index F. The strength index C and the ductility index F shall be evaluated in accordance with the provisions in 3.2.2 and 3.2.3 respectively of J. Standard. The effective strength factor α is used to consider the difference in stiffness of vertical members during calculation of strength index C at a certain level of deflection of the structure. The effective strength factor α may be taken as given in Table 3.4.1.

The basic seismic index of structure E_o shall be taken as the larger one from Equations (4) and (5) of J. Standard. Here, the index E_o shall be evaluated within the limitation of the minimum ductility index of the second-class prime elements (see the translator's note 3 of J. Standard) defined in the item (4) of J. Standard in case the storey consist of these elements.

Table 3.4.1 Classification of Vertical Members Based on Failure Modes in the Second Level Screening Procedure

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

(a) Ductility-dominant basic seismic index of structure (Equation 4 of J. Standard).

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

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

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 5of J. Standard).

For the calculation of E_o by Equation (5), the ductility index of the first group F_1 shall be selected as the cumulative point of strength, and the contribution of strength indices of only the vertical members with larger ductility indices than that of the first group shall be considered. The index F_1 of the first group shall be less than that corresponding to the ultimate deformation of the story given in the item (4) of J. Standard, and may be selected so that the index E_o by Equation (5) would be evaluated as maximum. The effective strength factor α in the second and higher groups should be calculated considering the effects of yield deformations and clear heights of vertical members on the relationships between the storey shear forces and the drift angles. The values of α given in Table 3.4.2 may be used in case no special verification. The minimum effective strength factor of the vertical members should be used in each group.

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

Where:

α_j= Effective strength factor in the j-th group at the ultimate deformation R_1 corresponding to the first group (ductility index of F_1), given in Table 3.4.2.

Table 3.4.2 (a) Effective Strength Factor for 1st Group, $F_1 = 0.8$

Ductility index of 1st group	F_I	F_I= 0.8
Deflection angle of 1st group	R_I	R_I=R_{500}
Effective strength factor for second and higher groups	Shear (R_{SU}=R_{500})	α_S
	Shear (R_{250}=R_{SU})	α_S
	Flexural (R_{my}=R_{250})	0.65
	Flexural (R_{250}<R_{my}<R_{150})	α_m
	Flexural (R_{my}=R_{150})	0.51
	Flexural and shear walls	0.65

Table 3.4.2 (b) Effective Strength Factor for 1st Group, $F_1 \geq 1.0$

Ductility index of 1st group	F_1	F_1=1.0	1.0<F_1<1.27	1.27≤ F_1
Deflection angle of 1st group	R_1	R_{250}	R_{250}< R_I< R_{150}	R_{150}≤ R_1
Effective strength factor for second and higher groups	Shear (R_{SU}=R_{250})	1.0	0.0	0.0
	Shear (R_I <R_{SU})	α_s	α_{ss}	0.0
	Flexural (R_{my}<R_I)	1.0	1.0	1.0
	Flexural (R_I<R_{my})	α_m	α_m	1.0
	Flexural (R_{my}=R_{150})	0.72	α_m	1.0

(Note)

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

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

R_{my}= Drift angle at flexural yielding, calculated by Equation (A1. 3-1) in the supplementary provision 1.

R_{su}= Drift angle at shear strength, calculated by Equation (A1. 2-11) in the supplementary provision 1.

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

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

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

(c) Ultimate state of a structure (for a storey) [Relate "Translators' Note 3" in J. Standard]

One of the most important issues of the seismic evaluation is to predict the process of structure collapsing .In general, one of the causes of building collapse is, the axial load capacity of column was deteriorated due to the flexural or the shear failure of the column, or increase of axial force from P-δ effect caused by the residual deformations. Then the column that is not able to support axial force leads to collapse buildings. In seismic evaluation, it is important to decide at which point this collapse is evaluated.

When deformability $C1$ to $C3$ are equal as shown Figure 3.4.1, all columns reach the ultimate state simultaneously when $F = 2.0$, and the building reaches ultimate state.

Then, if the situation is as shown Figure 3.4.2, would the building be collapse at deformation of F =1.5?In this case, if the axial force that had been supported by C1 can be relocated to the surrounding columns, a building shall not collapse. In this evaluation method, it is important to decide at which F value the buildings finally collapse. The example C1 is called the second class prime element (collapsing of this column does not affect the state of the entire building).First-class prime element is the element, which immediately leads to the collapsing of the entire building (or part of it). Investigations found that many of the buildings in Bangladesh have low strength concretes, and the columns on ground floors do not have enough axial load capacity against the axial force. Therefore, it is recommended to assume that relocation is not allowed, and to set the minimum F value of the floor as the ultimate state of buildings.

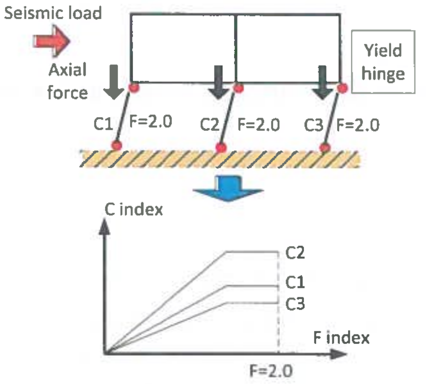


Figure 3.4.1Each column with Same ductility index

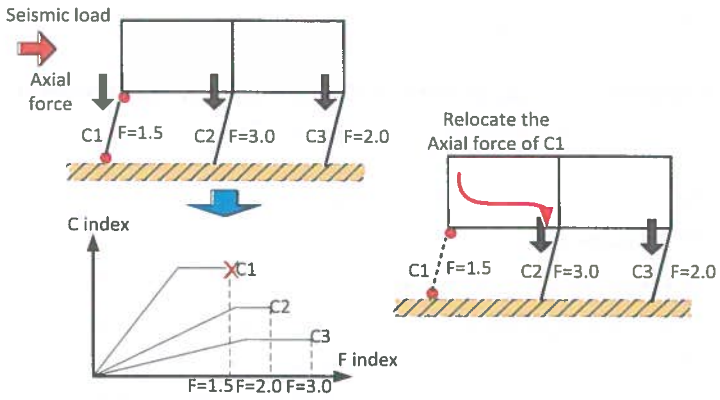


Figure 3.4.2 Each column with different ductility index

It is recommended not to consider Residual axial load capacity N_R. Residual axial load capacity N_R is that the column has axial load capacity after collapsing itself.

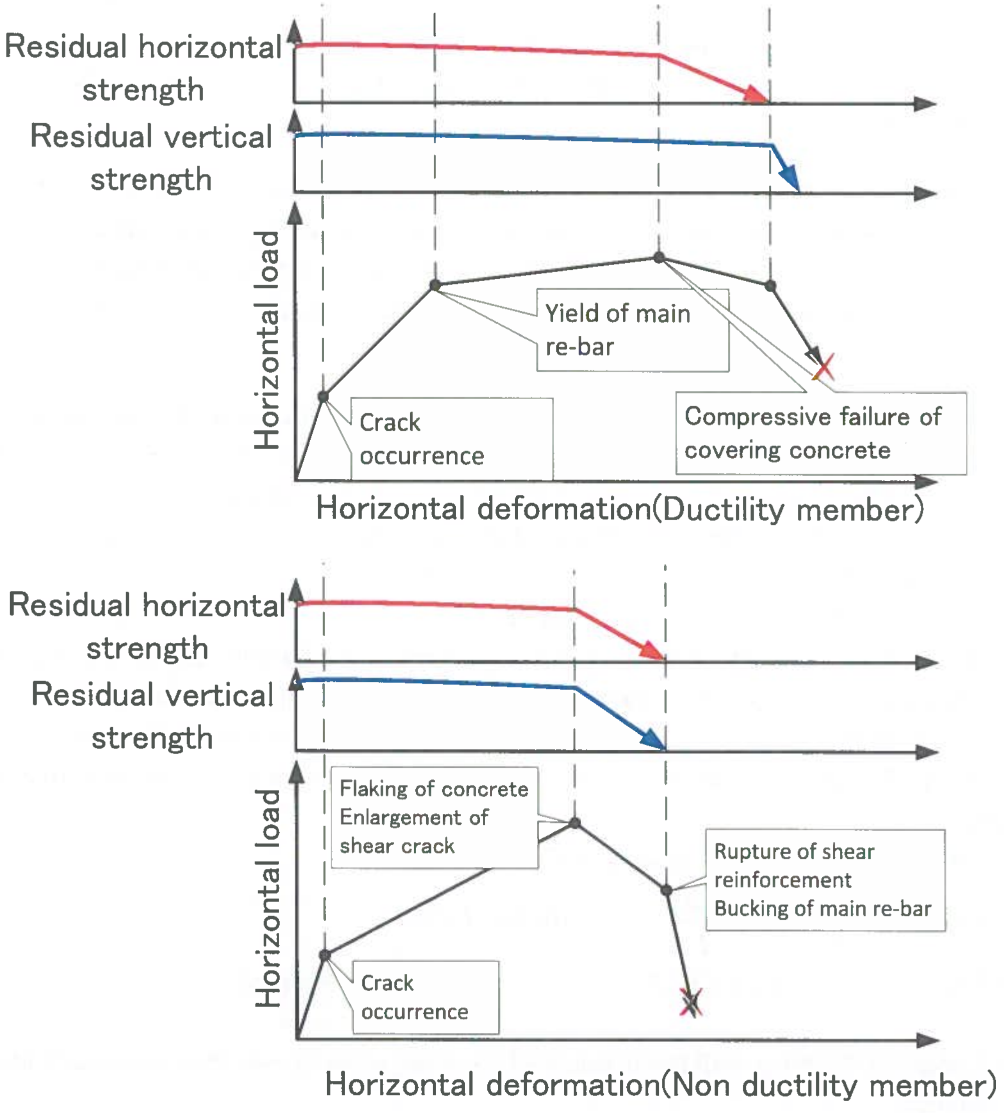


Figure 3.4.3 Concept of Residual strength

3.4.2 Strength Index *C*

The methods of calculating the strength index *C* of vertical members in each storey of a building for the second level screening procedure is as follows.

Second level screening procedure

(a) Principles

The strength index *C* in the second level screening procedure shall be calculated from the ultimate lateral load-carrying capacity of vertical members (columns and walls) in principle based on the assumption that the beams are strong enough. The failure modes of the vertical members shall be classified in accordance with Table 3.4.1 by comparing the ultimate shear strength Q_{su} and the shear at the ultimate flexural failure Q_{mu}. Published methods, which have reliable accuracy, may be used for the calculation of the ultimate shear strength Q_{su} and the ultimate flexural strength M_u. The inflection heights for calculations of Q_{su} and Q_{mu} should be used as specified in the following item (c) in case no special considerations.

(b) Calculation of ultimate strength of members

The formulas or methods estimating the lower bound of the actual strength should be used in calculation of the ultimate shear strength Q_{su} while those estimating the average should be used in calculation of the ultimate flexural strength M_u. The formulas given in the Supplementary Provisions may be used in case no special considerations.

It is recommend using the material strengths based on material test for the compressive strength of concrete and the yield strength of reinforcing bars to calculate the ultimate strength of a vertical member. When calculating the flexural strength and shear strength of a column, a vertical load may be used. When the compressive strength is less than 13.5N/mm^2, it is advised to revise the shear strength using the shear reduction factor for low strength concrete.

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

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

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

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

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

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

$$h_{CWO} = h_{CO} + \left(h_{WO} - h_{CO}\right)\cdot\frac{L_W}{L} \qquad (0<L_W<L-D_C)$$

$$h_{CWO} = h_{WO} \qquad (L_W \geq L-D_C) \qquad \text{(11) of J. Standard}$$

Where:

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

D_c = Column depth.

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

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

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

A clear height of a column with a standing wall of brick may be estimated by disregarding the effect of the brick wall; however, in the case of the clear height $h_o/d \leq 2.0$ when considering the standing wall of brick, the clear height of the column will be evaluated in consideration of the effect of the brick wall.

(d) Calculation of strength index

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

$$C = \frac{Q_u}{\sum W} \quad \text{(12) of J. Standard}$$

Where:

Q_u = Ultimate lateral load-carrying capacity of the vertical members in the storey concerned
$\sum W$ = The weight of the building including live load for seismic calculation supported by the storey concerned.

3.4.2.1 Recommendation for Application of strength index

> As mentioned in 3.3 it needs to be careful about various things when conducting the evaluation using the Japanese Standard. Those issues need consideration when evaluating the strength.

There are eight items shown in figure 3.3.1, which may greatly affect the evaluation. Structural experiments (Refer to Manual for Seismic Retrofit Design of Existing Reinforced Concrete Buildings) were conducted in 2012 and 2013 to understand those issues. The results showed that items 3(90-deg-hook tie), 6(No joint-shear reinforcement inside joints) had insignificant effects. However, the experiment was done only once for each item, further investigations and studies (seismic evaluation standards and damage reports of various countries) are required to obtain more reliable results. Effects of straight anchorages of beam reinforcement in exterior beam-column joint and the narrow area of column- beam-joint are not clearly understood, and needs further investigations/ studies as well.

(1) Reduction of strength of member in case of low concrete strength.

Japanese Standard defines the minimum strength of concrete in the applicable range as 13.5N/mm^2.Results of compressive strength tests showed that the majority of the concretes used for the existing buildings in Bangladesh have the strength less than 13.5N/mm^2 (Refer to Figure 2.5.2). 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 as low as9.0N/mm^2. It recommends providing reduction coefficient, k_r for low strength concrete to evaluate shear strength of columns. In case of evaluating a building with the concrete strength less than 13.5N/mm^2, it is recommended to adopt the reduction factor with the lower limit of 9N/mm^2 in order to reduce the shear strength. (The lower limit 9N/mm^2)

Ultimate shear strength of columns shall be calculated with equation.

$$Q_{su} = k_r \cdot \left\{ \frac{0.053 p_t^{0.23}(18+\sigma_B)}{M/(Q \cdot d)+0.12} + 0.85\sqrt{p_w \cdot \sigma_{wy}} + 0.1\sigma_o \right\} \cdot b \cdot j \qquad \text{(A 1.1-2)}$$

[Supplementary Provisions 1 Column with Modified to consider low Strength Concrete]

($k_r \leq 1.0$)

Where:

k_r : Reduction factor of low strength concrete ($\sigma_B < 13.5\text{N/mm}^2$) $k_r = 0.244 + 0.056\sigma_B$

p_t : Tensile reinforcement ratio (%).

a_w : Cross sectional area of shear reinforcement.

s : Spacing of shear reinforcement (In case tie is 90 degree hook; $s = 2\times s$ (Twice spacing length))

P_w : Shear reinforcement ratio, $P_w = 0.012$ for $P_w \geq 0.012$. $P_w = \dfrac{a_w}{(b \times s)}$

${}_s\sigma_{wy}$: Yield strength of shear reinforcing bars (N/mm^2)$\cdot \sigma_o$: Axial stress in column (N/mm^2).

d : Effective depth of column. D·50mm may be applied.

M/Q : Shear span length. Default value is $h_o/2$.

h_o : Clear height of column.

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

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 and if the value of σ_o is greater than 8N/mm^2, the value of 0.6 shall be 8N/mm^2 in using Equation.

As the low strength concrete with a plain bar (as a main bar) provides smaller bond strength, bonding between a re-bar and concrete is decreased so that the strength of re-bar becomes less than expected. As a result the strength of members assumed to be lower than the evaluation equations. In this case, it is recommended to evaluate with the reduced flexural strength of column (about 20% less) to be safe.

φ_{lp} (plain)= 0.80 for low-concrete-strength and plain-bar

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

Beam bars on the outer edge of the existing buildings may have small column width, or the main bar is insufficiently anchored due to the inappropriate form of the anchor. In case of short anchor length of beam main bar at an external column, it is suggested to reduce flexural strength of column by 25%.

(For columns of small depth such as 250mm; 50%of flexural strength of the column)

In case of short anchor length of beam main bar at an external column, it is used to reduce 25%. 50% (for small depth column such as 250mm) of flexural strength of the column by 2nd level screening.

(3) Reduction factor of flexural strength of column φ_r

For seismic evaluation of the buildings in Bangladesh, flexural strength of column shall be set. Minimum reduction of flexural strength for each factor is used for (1) and (2).It is preferable to conduct an experiment, to determine the reduction factors to be used in calculating flexural strength of columns. However, there is not a reliable data available so that we set the minimum value of redaction factor obtained from different influences as a flexural strength redaction factor of a column

$\varphi_r = \min\{\varphi_{lp}\,(\text{plain}), \varphi_a\,(\text{anchor})\}$

- φ_{lp} (plain)= 0.80 for low-concrete-strength and plain-bar simultaneously
- φ_a (anchor)= 0.75 for normal depth column

  φ_a (anchor)= 0.50 for small depth column such as 250mm

3.4.3 Ductility Index *F*

3.4.3.1 Basic Principles in Calculation of Ductility Index *F*

The ductility index of a vertical member shall be evaluated in consideration of the screening level, failure mode and member deformation capacity, and response to earthquakes. A standard value of ductility index shall be defined as the ductility index of the shear wall, in which shear failure precedes other failure modes. The ductility indices of the other members shall be determined as a relative value to this standard value. The ductility index of the member shall be evaluated by the methods specified as in the following items (2) - (4) according to the screening level and the classification by the failure mode of the member (as shown in Table 3.4.1), in case no special investigations.

Second Level Screening Procedure

The ductility index of a vertical member in the second level screening procedure shall be calculated as follows according to the classification of the member listed in Table 3.4.1.

Vertical members are classified based on the characteristic seismic damages observed in Japan. Therefore, it is required to reset it with the failure of the long column with high axial force taken into consideration, before using this procedure. Other factors that may affect the seismic capacity shall be taken into consideration as well to determine the ductility. A clear height of a column with a standing wall of brick may be estimated by disregarding the effect of the brick wall; however, in the case of the clear height $h_o/d \leq 2.0$ when considering the standing wall of brick, the clear height of the column will be evaluated considering the effect of the brick wall. Classification of vertical members in different failure modes used for the second level screening by J. standard.

Flexural column is the main failure mode of column of seismic evaluation for buildings in Bangladesh. Shear walls are not actually used for any of the target buildings. Calculation formula of ductility factor *F* of the column is as shown below.

(a) Shear Wall

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

(b) Flexural Wall

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

If ${}_{W}Q_{SU}/\,{}_{W}Q_{mu} = 1.0$ then $F = 1.0$ (13) of J. Standard

If ${}_{W}Q_{SU}/\,{}_{W}Q_{mu} \geq 1.3$ then $F = 2.0$

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

If $1.0 < {}_{W}Q_{SU}/\,{}_{W}Q_{mu} < 1.3$ then *F* should be calculated by interpolation.

Where:

${}_{W}Q_{SU}$ = Ultimate shear strength of the wall, calculated by Equation (A2. 1-2) in the supplementary provisions of J. Standard.

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

(c) Shear Column

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

$$F = 1.0 + 0.27 \frac{R_{SU} - R_{250}}{R_Y - R_{250}} \qquad \text{(14) of J. Standard}$$

Where:

R_{su} : Inter-storey deformation angle at the ultimate deformation capacity in shear failure of the column member

$$R_{su} = \frac{{}_cQ_{su} / {}_cQ_{mu} - 0.3}{0.7} \cdot R_{my} \geq R_{250} \text{ for } {}_c\alpha \cdot {}_cQ_{mu} < {}_cQ_{su}$$

[Supplementary Provisions 1.2 Ultimate Deformation A1.2-11of J. Standard]

$R_{su} = R_{250}$ for ${}_c\alpha \cdot {}_cQ_{mu} \geq {}_cQ_{su}$

${}_c\alpha = 0.3 + 0.7\ (R_{250}/R_{my})$

[Supplementary Provisions 1.2 Ultimate Deformation A1.2-12of J. Standard]

Where:

${}_cQ_{su}$: Ultimate shear strength of the column

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

${}_c\alpha$: Effective strength factor of the column

R_{my} : Yield inter-storey deformation angle

$R_{my} = (h_o/H_o) \cdot {}_cR_{my} \geq R_{250}$

[Supplementary Provision 1.3 Yield Deformation Flexural Columns A1.3-1of J. Standard]

${}_cR_{my}$: Yield deformation angle of column (measured in clear height of column)

(d) Flexural Column

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

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

$$F = 1.0 + 0.27 \frac{R_{mu} - R_{250}}{R_y - R_{250}} \qquad \text{(15) of J. Standard}$$

(i) In case $R_{mu} \geq R_y$

$$F = \frac{\sqrt{2R_{mu} / R_y - 1}}{0.75 \cdot (1 + 0.05 R_{mu} / R_y)} \leq 3.2 \qquad \text{(16) of J. Standard}$$

Where:

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

R_{250} = Standard inter-storey drift angle (corresponding to the ductility index of a 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 Equation (A1.2-1) in the supplementary provision 1.2(1) of J. Standard.

*Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001. English version published by: The Japan Building Disaster Prevention Association.

$R_{mu} = (h_o / H_o) \cdot {}_cR_{mu} \geq R_{250}$

Where:

$h_o / H_o \leq 1.0$

${}_cR_{mu} = {}_cR_{my} + {}_cR_{mp} \leq {}_cR_{30}$

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 deformation angle of column (measured in clear height of column)
$_cR_{mu}$: Deformation angle at the ultimate flexural strength of column (measured in the clear height of column).
$_cR_{mp}$: Plastic deformation angle of column (measured in the clear height of column)
$_cR_{30}$: Standard deformation angle of the column (measured in the clear height of column), $R = 1/30$.

$_cR_{my} = {}_cR_{150}$ for $h_o/D \geq 3.0$
$_cR_{my} = {}_cR_{250}$ for $h_o/D \leq 2.0$

$_cR_{my}$ is set by interpolation for $2.0 < h_o/D < 3.0$

$_cR_{mp} = 10\ ({}_cQ_{su}/{}_cQ_{mu} - q)\cdot{}_cR_{my} \geq 0$
$q = 1.0$ for $s \leq 100\text{mm}$
$q = 1.1$ for $s > 100\text{mm}$

Where:
$_cQ_{su}$: Ultimate shear strength of the column
$_cR_{mu}$: Shear force at the ultimate flexural strength of the column
s : Spacing of ties

The value of $_cR_{my}$ shall not be greater than that of $_cR_{max}$.

(e) Extremely brittle column

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

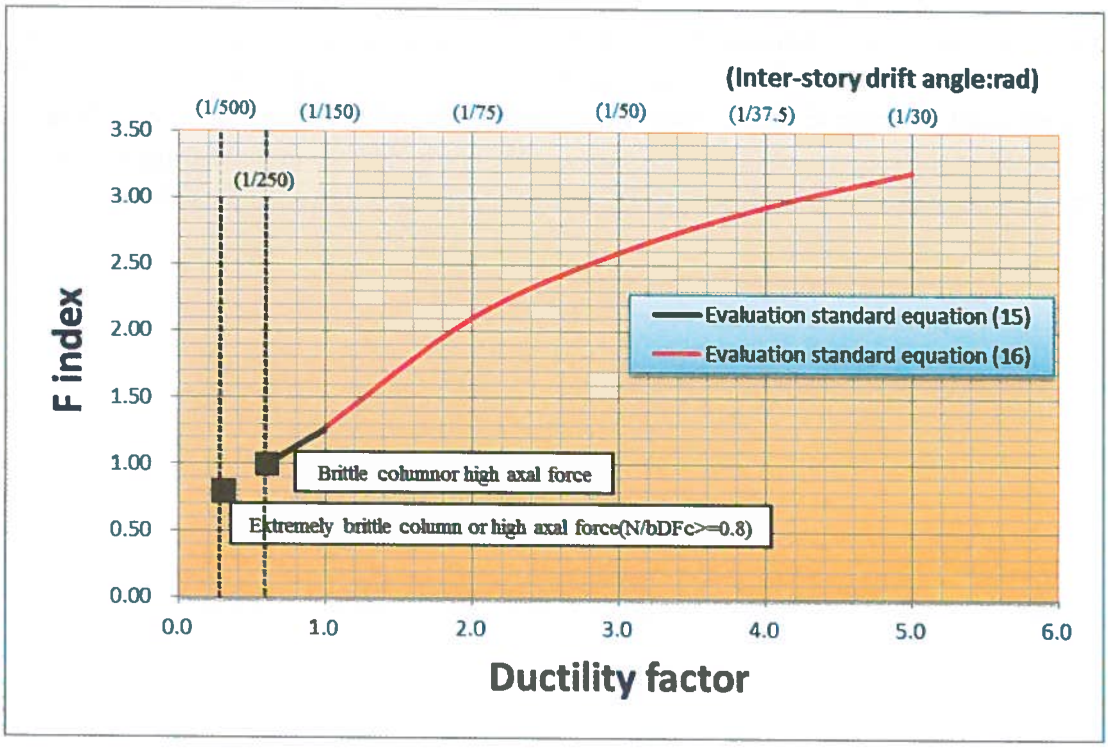


Figure 3.4.4 Relationship between Horizontal Force and Horizontal Displacement of RC Buildings

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

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

(i) Wall (Wall with a column)

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

(ii) Column with wing wall(s)

The index shall be calculated as follows:

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

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

Where:

h_o = Clear height of the column.

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

(iii) Column

The index shall be calculated according to the above items (c)-(e).

However, the ductility index should be calculated by reducing the plastic drift angle ${}_cR_{mp}$ to 0.5 times as specified in the Supplementary Provisions 1.2(2)of J. Standard, and should not exceed 1/150, in case of a flexural column with wing walls.

3.4.3.2 Additional Provisions for Maximum Deflection Angle

It is recommended to modify and add the items of evaluation of maximum angle based on the explanations in 3.3, which is the difference of structural characteristics between buildings in Japan and Bangladesh. Considering those differences additional provision have been described below.

(1) High axial force of column

Columns of the buildings in Bangladesh have high axial load compared to the ones in Japan. It is even higher if the concrete strength is lower. Axial force ratio ($N/b{\cdot}D{\cdot}F_c$) of it can be around 0.6. CNCRP carried out an experiment to understand the deformation capacity of columns with high axial force and low strength. Based on the outcome of that experiment and the one conducted in Japan (Supplementary 2).The following parameters is recommended regarding the maximum deformation of the column.

[Ordinary concrete ($\sigma_B \geq 13.5\ N/mm^2$)]

Shear reinforcement of column by BNBC, 2-10mm@150 with 135 degree hook is satisfied.

$$_cR_{\max(n)} = {_cR_{500}}\ for\quad \eta \geq 0.80$$
$$_cR_{\max(n)} = {_cR_{250}}\ for\quad 0.80 > \eta \geq 0.55$$
$$_cR_{\max(n)} = {_cR_{150}}\ for\quad 0.55 > \eta > 0.40$$
$$_cR_{\max(n)} = {_cR_{30}} \cdot \left(\frac{_cR_{150}}{_cR_{30}}\right)^{n'} \leq {_cR_{30}}\quad for\ other\ case$$

In case the reinforcement is not satisfied.

$$_cR_{\max(n)} = {_cR_{500}}\ for\quad \eta \geq 0.80$$
$$_cR_{\max(n)} = {_cR_{250}}\ for\quad 0.80 > \eta \geq 0.40$$
$$_cR_{\max(n)} = {_cR_{30}} \cdot \left(\frac{_cR_{250}}{_cR_{30}}\right)^{n'} \leq {_cR_{30}}\quad for\ other\ case$$

$n' = (\eta - \eta_L)/(\eta_H - \eta_L)$
$\eta = N_s/(b \cdot D \cdot F_c).$
$\eta_L = 0.20\ \ and\ \eta_H = 0.4$

[Low strength concrete ($\sigma_B < 13.5\ N/mm^2$)]

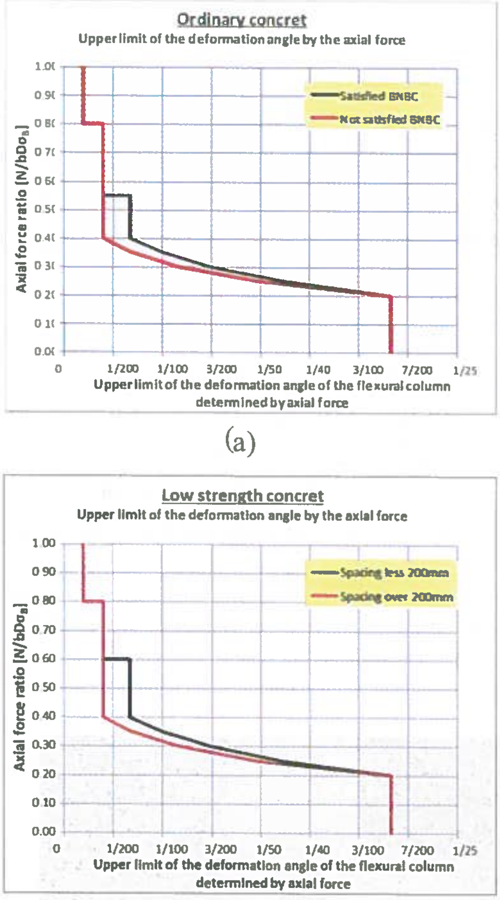
In case shear reinforcement, 2-10mm@200 or less,

$$_cR_{\max(n)} = {_cR_{500}}\ for\quad \eta \geq 0.80$$
$$_cR_{\max(n)} = {_cR_{250}}\ for\quad 0.80 > \eta \geq 0.60$$
$$_cR_{\max(n)} = {_cR_{150}}\ for\quad 0.60 > \eta > 0.40$$
$$_cR_{\max(n)} = {_cR_{30}} \cdot \left(\frac{_cR_{150}}{_cR_{30}}\right)^{n'} \leq {_cR_{30}}\quad for\ other\ case$$

In case 2-10mm@more than200,

$$_cR_{\max(n)} = {_cR_{500}}\ for\quad \eta \geq 0.80$$
$$_cR_{\max(n)} = {_cR_{250}}\ for\quad 0.80 > \eta \geq 0.40$$
$$_cR_{\max(n)} = {_cR_{30}} \cdot \left(\frac{_cR_{250}}{_cR_{30}}\right)^{n'} \leq {_cR_{30}}\quad for\ other\ case$$

$n' = (\eta - \eta_L)/(\eta_H - \eta_L)$
$\eta = N_s/(b \cdot D \cdot F_c).$
$\eta_L = 0.20\ \ and\ \eta_H = 0.4$



(a)

(b)

Figure 3.4.5 Relation between Axial Force Ratio and Upper Limit of Deformation Angle, (a) For Ordinary Concrete (b) For Low Strength Concrete

(2) Tensile reinforcing-bar ratio

In case there is a possibility of a bond split failure of the main reinforcement, the upper limit of ultimate deformation angle of a column $_cR_{max(t)}$ will restrict the upper limit of the deformation angle as its deformability will decrease. From the results of past experiment in Japan, it has been confirmed that bond split failures tend to increase when tensile reinforcement bar quantity exceeds 1%.

However, in the diagnosis, pillars with a shear span ratio ($M/(Q{\cdot}D)$) of 3 or more or with round steel bar main reinforcement are excluded from the subject of study, since the possibility of a bond split failure is low.

Shear span ratios of almost all the buildings in Bangladesh are 3.0 or more (ignoring the effects of brick walls) leading to believe the possibility of destruction from a bond split failure to be low. However, the upper limit $_cR_{max(t)}$ is set as follows to include the influence of low strength concrete after studying the supplement 5.

The study results shows that when $\frac{M}{Q \cdot D}$ is less than 3 and low strength concrete is used, there is a possibility of bond split failure when p_t is 1.3 or less. This case is applied to the building, which rebar diameter is thick and the grade of rebar is more than 400W. Further study by referring supplement 5 is recommended when such a case is encountered.

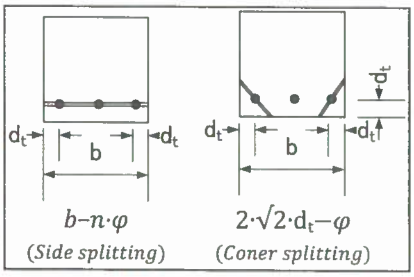


Figure 3.4.6 Presupposition of Splitting Mode

$_{C}R_{max(t)} = {}_{C}R_{250}$ for $P_t > 1.3$ and Low strength concrete ($\sigma_B < 13.5\text{N/mm}^2$) if $\dfrac{M}{Q \cdot D} < 3$

(3) 90 degree hook of column

It is a known fact that when the 90degrees hook is used as a shear reinforcement bar, the ductility capacity is lower than when the 135degrees hook is used. Some buildings in Bangladesh still have the 90degrees hooks. For those buildings, it is recommended to evaluate using the lower toughness of the column. In this evaluation, shear margin ratio (Q_{su}/Q_{mu}) affects ductility factor. Therefore, it is suggested to reduce the ductility index by setting the shear reinforcing ratio p_w ($a_w/b \cdot @$) as $0.5p_w$

(4) Beam-column-joint

Japanese seismic evaluation manual does not include the evaluation of the beam-column joint. It is because the Japanese buildings have larger section area of column. Damages of the beam-column joint were not found much in the buildings damaged by the past earthquakes. Buildings in Bangladesh have smaller section area of column, so the beam-column joint and its bearing force are smaller. Many buildings built in a certain period do not have the shear reinforcement bars at their beam-column joints. Therefore, this application manual recommends studying the beam-column joint by referring to BNBC. Pending further research, the following upper limit of column F is suggested:

Ordinary concrete, max. $F = 1.75$ (aim of storey deflection angle, $R = 1/100$)
Low strength concrete, max. $F = 1.5$ ($R = 1/124$)

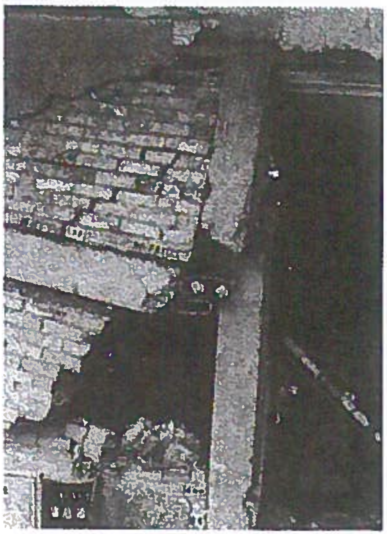
1990_Luzon Earthquake 1985_Mexico Earthquake

Figure 3.4.7 Failure of Beam-Column Joint

(5) Short anchor length of beam main-bar at external column

Beam bars on the outer edge of the existing buildings may have small column width, or the main bar is insufficiently anchored due to the inappropriate form of the anchor. Insufficient anchoring limits the allowance of the rotational angle of member. In that case, it is recommended to set the maximum of the deformation angle. Pending further research, the following values are recommended.

$_{C}R_{max(a)} = {}_{c}R_{100}$ for Ordinary-strength concrete ($\sigma_B \geq 13.5$ N/mm^2)
$_{C}R_{max(a)} = {}_{c}R_{124}$ for Low-strength concrete ($\sigma_B < 13.5$ N/mm^2)



1979_Montenegro Earthquake

The source: Data for Ultimate Strength Design of Reinforced Concrete Structures (Architectural Institute of Japan)

Figure 3.4.8 Collapse of Pulling out of Main Reinforcement

(6) Infill Brick Wall

It is known that the standing wall of brick affects the deformation and the strength of a column. However, the columns of the buildings have characteristically extremely small flexural strength compared with the shear strength and the failure mode of the columns is flexural failure. Accordingly, consideration of the effect of the standing wall will not likely affect the evaluation result significantly. In consideration of the labor in calculation, the clear height will be assumed with the effect of the standing wall disregarded. However, given the building damage caused by collapse of a short column observed in earthquake damage cases in other countries, it is recommended to evaluate the failure mode in the case of $h_0/D \leq 2.0$.

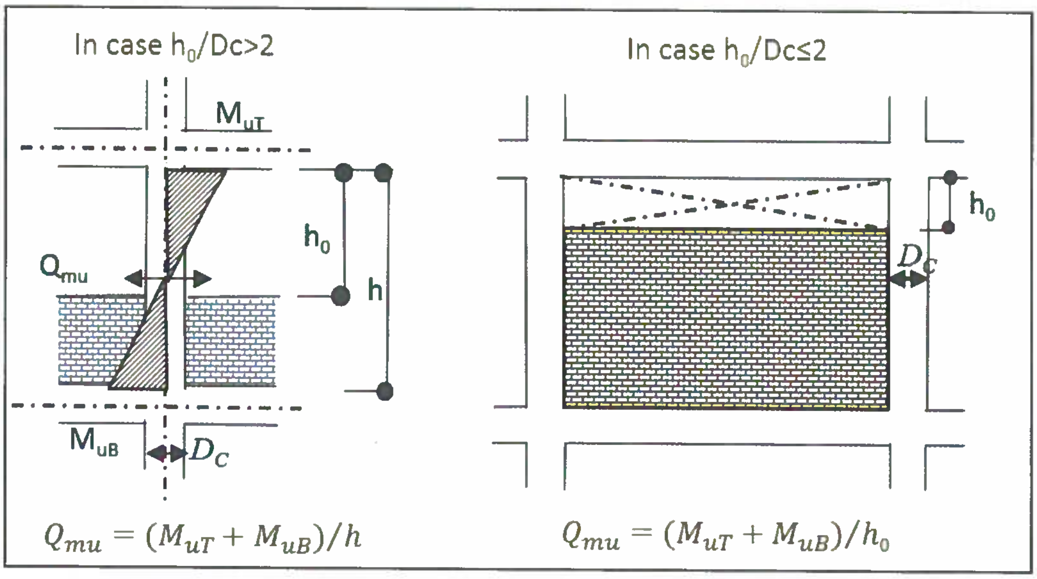
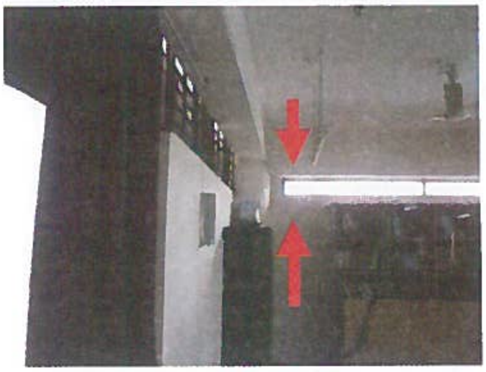


Figure 3.4.9 Relationship between Clear Height of Brick Wall and Column Width



Hospital in Dhaka

Shear failure of a short column with brick walls

The source: Report on the Damage Investigation of the 1999 Chi-Chi Earthquake (Architectural Institute of Japan)

Figure 3.4.10 Short Column

In case of short column effect due to brick standing wall has not been studied in detail, upper limit of column F

【Ordinary-strength concrete ($\sigma_B \geq 13.5\text{N/mm}^2$)】

$_cR_{max(B)} = {}_cR_{100}$ for $P_w \geq 0.2\%$

$_cR_{max(B)} = {}_cR_{124}$ for $P_w < 0.2\%$

【Low-strength concrete ($\sigma_B < 13.5\text{N/mm}^2$)】

$_cR_{max(B)} = {}_cR_{124}$ for $P_w \geq 0.2\%$

$_cR_{max(B)} = {}_cR_{150}$ for $P_w < 0.2\%$

3.4.3.3 Upper Limit of the Drift Angle of Flexural Columns $_cR_{max}$

Recommendations for calculation of maximum deflection angle $_cR_{max}$ is summarized below.

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

$_cR_{max} = min\{{}_cR_{max(n)}, {}_cR_{max(s)}, {}_cR_{max(t)}, {}_cR_{max(b)}, {}_cR_{max(h)}, {}_cR_{max(p)}, {}_cR_{max(a)}, {}_cR_{max(B)}\}$

[Modified supplementary Provisions 1.2 Ultimate Deformation A1.2-5]

● $_cR_{max(n)}$: Upper limit of the deformation angle of the flexural column determined by the axial force

[Ordinary concrete ($\sigma_B \geq 13.5$ N/mm^2)]

Shear reinforcement of column by BNBC, 2-10mm@150 with 135 degree hook is satisfied.

$_cR_{max\,(n)} = {}_cR_{500}$ for $\eta \geq 0.80$

$_cR_{max\,(n)} = {}_cR_{250}$ for $0.80 > \eta \geq 0.55$

$_cR_{max\,(n)} = {}_cR_{150}$ for $0.55 > \eta \geq 0.40$

$$_cR_{max(n)} = {}_cR_{30} \cdot \left(\frac{{}_cR_{150}}{{}_cR_{30}} \right)^{n'} \leq {}_cR_{30} \text{ for other case}$$

In case the reinforcement is not satisfied.

$_cR_{max\,(n)} = {}_cR_{500}$ for $\eta \geq 0.80$

$_cR_{max\,(n)} = {}_cR_{250}$ for $0.80 > \eta \geq 0.40$

$$_cR_{max(n)} = {}_cR_{30} \cdot \left(\frac{{}_cR_{250}}{{}_cR_{30}} \right)^{n'} \leq {}_cR_{30} \text{ for other case}$$

[Low strength concrete ($\sigma_B < 13.5$ N/mm^2)]

In case shear reinforcement, 2-10mm@200 or less,

$_cR_{max\,(n)} = {}_cR_{500}$ for $\eta \geq 0.80$

$_cR_{max\,(n)} = {}_cR_{250}$ for $0.80 > \eta \geq 0.60$

$_cR_{max\,(n)} = {}_cR_{150}$ for $0.60 > \eta \geq 0.40$

$$_cR_{max(n)} = {_cR_{30}} \cdot \left(\frac{_cR_{150}}{_cR_{30}}\right)^{n'} \leq {_cR_{30}} \text{ for other case}$$

In case 2-10mm@more than200mm,

$_cR_{max\,(n)} = {_cR_{500}}$ for $\eta \geq 0.80$

$_cR_{max\,(n)} = {_cR_{250}}$ for $0.80 > \eta \geq 0.40$

$$_cR_{max(n)} = {_cR_{30}} \cdot \left(\frac{_cR_{250}}{_cR_{30}}\right)^{n'} \leq {_cR_{30}} \text{ for other case}$$

Where:

$$n' = \frac{\eta - \eta_L}{\eta_H - \eta_L}$$

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

$\eta_L = 0.20$ and $\eta_H = 0.4$

[Modified Supplementary Provisions 1.2 Ultimate Deformation A1.2-6]

- $_cR_{max\,(n)}$: Upper limit of the deformation angle of the flexural column determined by the shear force

quote $_cR_{max(s)} = {_cR_{250}}$ for $_c\tau_u / F_c > {_cR_{max(s)}} = {_cR_{250}}$ for $_c\tau_u / F_c > 0.2$

$_cR_{max\,(s)} = {_cR_{30}}$ for other case

[Supplementary Provisions 1.2 Ultimate Deformation A1.2-7]

- $_cR_{max\,(t)}$:Upper limit of the deformation angle of the flexural column determined by the tensile reinforcement ratio

$_cR_{max(t)} = {_cR_{250}}$ for $p_t > 1.3\%$

$_cR_{max\,(s)} = {_cR_{30}}$ for other case

[Modified supplementary Provisions 1.2 Ultimate Deformation A1.2-8]

- $_cR_{max\,(b)}$:Upper limit of the deformation angle of the flexural column determined by the spacing of hoop

$_cR_{max\,(b)} = {_cR_{50}}$ for $s/d_b > 8$

$_cR_{max\,(b)} = {_cR_{30}}$ for other case

[Supplementary Provisions 1.2 Ultimate Deformation A1.2-9]

- $_cR_{max\,(h)}$:Upper limit of the deformation angle of the flexural column determined by the clear height

$_cR_{max(h)} = {_cR_{250}}$ for $h_o/D \leq 2$

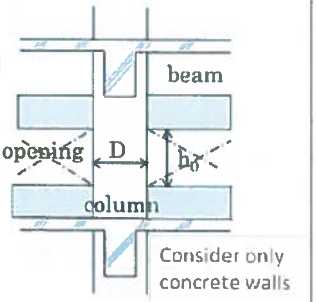
$_cR_{max\,(h)} = {_cR_{30}}$ for other case

[Supplementary Provisions 1.2 Ultimate Deformation A1.2-10]

- $_cR_{max\,(p)}$:Upper limit of the deformation angle of the flexural column determined by the beam-column joint

$$_cR_{max(p)} = {_cR_{100}} \text{ for Ordinary} - \text{Strength Concrete} \left(\sigma_B \geq 13.\frac{5N}{mm^2}\right)$$

$$_cR_{max(p)} = {_cR_{124}} \text{ for Low} - \text{Strength Concrete} \left(\sigma_B < 13.\frac{5N}{mm^2}\right)$$



- $_cR_{max\,(a)}$:Upper limit of the deformation angle of the flexural column determined by the anchor length

$$_cR_{max(a)} = {_cR_{100}} \text{ for Ordinary} - \text{Strength Concrete} \left(\sigma_B \geq 13.\frac{5N}{mm^2}\right)$$

$$_cR_{max(a)} = {_cR_{124}} \text{for Low} - \text{Strength Concrete} \left(\sigma_B < 13.\frac{5N}{mm^2}\right)$$

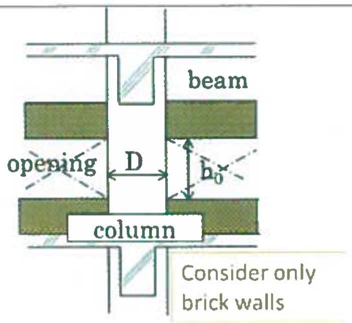
- $_cR_{max\,(h)}$:Upper limit of the deformation angle of the flexural column determined by the clear height

$h_o/D \leq 2$ (consider brick walls)

[Ordinary-strength concrete ($\sigma_B \geq 13.5N/mm^2$)]

$_cR_{max(B)} = {_cR_{100}}$ for $P_w \geq 0.2\%$

$_cR_{max(B)} = {_cR_{124}}forP_w \geq 0.2\%$



[Low strength concrete (σ_B<13.5N/mm^2)]

$$_cR_{max(B)} = {}_cR_{100} \text{ for } P_w \geq 0.2\%$$
$$_cR_{max(B)} = {}_cR_{124} \text{ for } P_w \geq 0.2\%$$

[In case of h_0>2]

$$_cR_{max(B)} = {}_cR_{30}$$

Where, N_s: Additional axial force of column due to earthquakes

$_c\tau_u$:Shear stress at the column strength $= min\left\{\frac{_cQ_{mu}}{(b \cdot j)}, \frac{_cQ_{su}}{(b \cdot j)}\right\}$

s:Spacing of hoops

d_b :Diameter of the flexural reinforcing bar of the column

3.4.4 Example of Calculation of Column Strength

SEISMIC ASSESSMENT OF A 5 STOREY OFFICES BUILDING IN DHAKA

Exercise on seismic assessment of an office building was done by CNCRP. The building is a 5 storey office building. The summary of the assessment is shown below.

3.4.4.1 Description of Building

a) 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.

b) Outline of Building

Usage	Office Building
Number of storey	Five
Structural type	R.C. Framed Structure
Foundation type	Shallow Foundation
Year of design	1985
Year of construction	1985

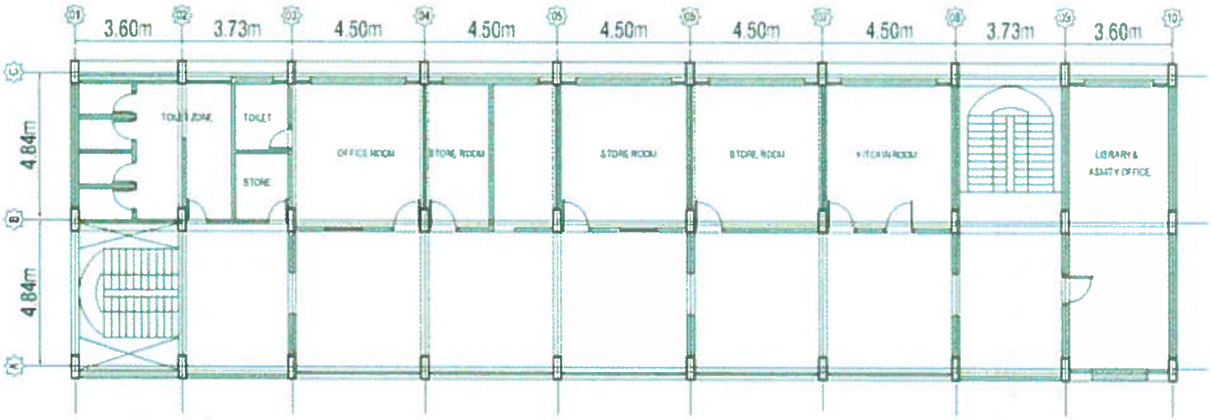
c) Summary of Findings

Concrete Strength	9.2 N/mm^2 (MPa) (Design strength $f'c$ =13.7MPa)
Re-bar Yield Stress	275 N/mm^2 (MPa)
Low Strength Concrete (<13.5N/mm^2) or not	Low Strength Concrete

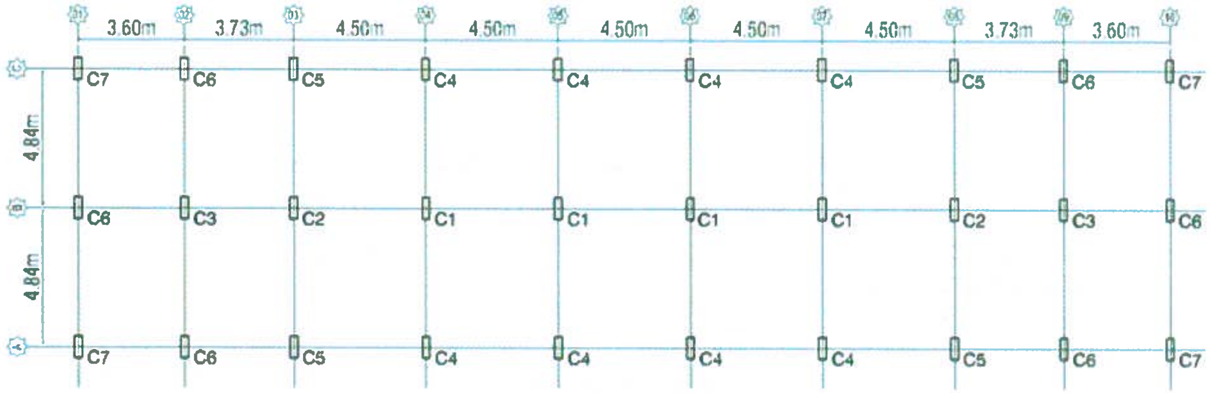
3.4.4.2 Calculation of Gravity Load

a) Calculation of Unit Weight of Structure

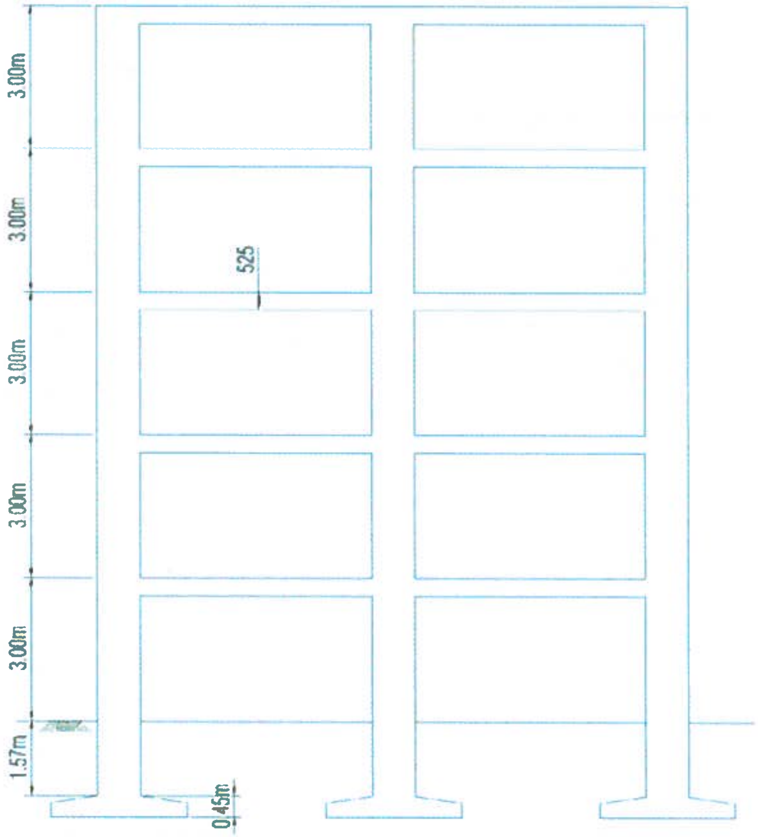
The unit weight of structure is calculated based on actual weight of brick wall, self weight of structural members (Column, Beam and Slab), floor finish and actual live load for unit area.



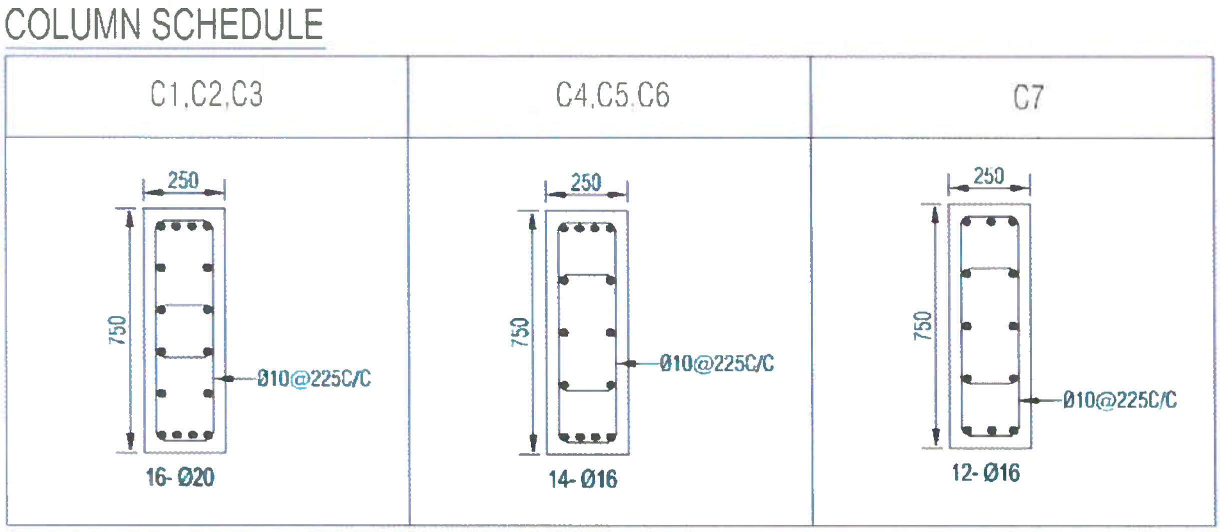
TYPICAL FLOOR PLAN



COLUMN LAYOUT PLAN



FRAME SECTION ALONG Y DIRECTION



Unit Area Weight (Typical Floor):		
1. Live Load	0.80	kN/Sqm
2. Brick Wall	4.00	kN/Sqm
3. Slab weight & Floor Finish	3.85	kN/Sqm
5. SW (Column + Beam)	2.15	kN/Sqm
W =	10.8	kN/Sqm

Unit Area Weight (Roof):		
1. Live Load	0.30	kN/Sqm
2. Brick Wall	0.00	kN/Sqm
3. Slab weight & Floor Finish	3.85	kN/Sqm
5. SW (Column + Beam)	2.15	kN/Sqm
W =	6.3	kN/Sqm

b) Summary of Floor Weight of Structure

Structural Weight			
Storey	Floor Area (m^2)	Floor Weight W_i (kN)	ΣW_i (kN)
5	405.5	2554.9	2555
4	405.5	4379.8	6935
3	405.5	4379.8	11315
2	405.5	4379.8	15694
1	405.5	4379.8	20074

c) Summary of Sustaining Force of Columns

Sustaining force of Columns				
COL. ID	**Storey**	**Tributary Area (m^2)**	**Unit Weight (kN/m^2)**	**Sustaining force N (kN)**
C1	5	22.55	6.3	142
	4	22.55	10.8	386
	3	22.55	10.8	629
	2	22.55	10.8	873
	1	22.55	10.8	1116
C2	5	20.60	6.3	130
	4	20.60	10.8	352
	3	20.60	10.8	575
	2	20.60	10.8	797
	1	20.60	10.8	1020
C3	5	18.35	6.3	116
	4	18.35	10.8	314
	3	18.35	10.8	512
	2	18.35	10.8	710
	1	18.35	10.8	908
C4	5	13.00	6.3	82
	4	13.00	10.8	222
	3	13.00	10.8	363
	2	13.00	10.8	503
	1	13.00	10.8	644
C5	5	11.90	6.3	75
	4	11.90	10.8	203
	3	11.90	10.8	332
	2	11.90	10.8	461
	1	11.90	10.8	589
C6	5	10.60	6.3	67
	4	10.60	10.8	181
	3	10.60	10.8	296
	2	10.60	10.8	410
	1	10.60	10.8	525
C7	5	5.56	6.3	35
	4	5.56	10.8	95
	3	5.56	10.8	155
	2	5.56	10.8	215
	1	5.56	10.8	275

3.4.4.3 Calculation of Member Strength

Calculation procedure of C_1 column at ground floor is shown below.

Assessment Direction : X
Location : Grid B5
Floor : Ground
Column : C_1
Storey height : h = 15'-0" = 4.57m
Clear height : h_o =13'-3" = 4.0m
Standard clear height of column, H_o = 4.0m

Material Properties

Compressive strength of existing concrete (F_c) = 9.2 N/mm^2
Tensile strength of existing rebar (σ_y) = 275 N/mm^2 (40 Grade)
Axial force N = 1116 kN
Column size, bD = 750mm × 250mm

Reinforcing Bar

Total reinforcing bars, 16-ϕ20, a_g = 5026.5mm^2
Tensile reinforcing bars, 6-ϕ20, a_τ =1885mm^2
Tie 4-ϕ10 @ 225 a_w = 314.2mm^2

Ultimate Flexural Strength

$0.4bDF_c = 0.4 \times 750 \times 250 \times 9.2$ N

$= 690{,}000$ N

$= 690$ kN

$N_{max} = a_g \cdot \sigma_y + bDF_c$

$= 5026.5 \times 275 + 750 \times 250 \times 9.2$ N

$= 3107287.5$ N

$= 3107.28$ kN

$N_{min} = a_g \cdot \sigma_y$

$= 5026.5 \times 275$ N

$= 138.2$ kN

Here, $N_{max} > \mathrm{N} > 0.4bDF_c$

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

$$= (0.8 \times 1885 \times 275 \times 250 + 0.12 \times 750 \times 250^2 \times 9.2)\left(\frac{3107.28 - 1116}{3107.28 - 690}\right)$$

$= 127914775$ N·mm

$= 127.9$ kN·m

The shear force at the ultimate flexural strength Q_{mu} can be calculated on the assumption that the M_u at the top and bottom of the column are the same.

$$Q_{mu} = \frac{2M_u}{h_o} = \frac{2 \times 127.9}{4.0} \text{kN}$$

$= 63.95$ kN

Considering provision for low strength concrete and plain bar {Sec- 3.4.2.1(3)} flexural strength should be reduced by φ_r.

Therefore $Q_{mu} = \varphi_r \times 63.95 = 0.8 \times 63.95 = 51.16$ kN

Ultimate Shear Strength

Shear strength of column [Modified supplementary provision 1.1 ultimate strength A1.1-2]

$$Q_{su} = k_r \cdot \left\{ \frac{0.053 p_t^{\,0.23} (18 + \sigma_B)}{M / (Q \cdot d) + 0.12} + 0.85\sqrt{p_w \cdot \sigma_{wy}} + 0.1\sigma_o \right\} \cdot b \cdot j \qquad \text{(A 1.1-2)}$$

[Supplementary Provisions 1 Column with Modified to consider low Strength Concrete]

($k_r \leq 1.0$)

Where:

k_r : Reduction factor of low strength concrete ($\sigma_B < 13.5\text{N/mm}^2$) $k_r = 0.244 + 0.056\sigma_B$

p_t : Tensile reinforcement ratio (%).

a_w : Cross sectional area of shear reinforcement.

s : Spacing of shear reinforcement (In case tie is 90 degree hook; $s = 2 \times s$ (Twice spacing length))

P_w : Shear reinforcement ratio, $P_w = 0.012$ for $P_w \geq 0.012$. $P_w = \dfrac{a_w}{(b \times s)}$

${}_s\sigma_{wy}$: Yield strength of shear reinforcing bars (N/mm^2)$\cdot \sigma_o$: Axial stress in column (N/mm^2).

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

M/Q : Shear span length. Default value is $h_o/2$.

h_o : Clear height of column (mm).

$k_r = 0.244 + 0.056 \times F_c$

$= 0.244 + 0.056 \times 9.2$

$= 0.76$

$$p_t = \frac{a_t}{b \cdot D} \times 100\% \quad = \frac{1885}{750 \times 250} \times 100\%$$

$$= 1.005\%$$

$$\frac{M}{Q} = \frac{h_o}{2} = \frac{4.0 \times 1000\ \text{mm}}{2} = 2000\ \text{mm}$$

$$\frac{M/Q}{d} = \frac{2000}{250 - 50} = 10.0 > 3$$

$$\therefore \frac{M}{Q \cdot d} = 3$$

$$p_w = \frac{a_w}{b \cdot s(\text{spacing})} = \frac{314.2}{750 \times 225} = 0.00186$$

$$\sigma_o = \frac{N}{b \cdot D} = \frac{1116}{250 \times 750}\ \text{kN/mm}^2 = 5.95\ \text{N/mm}^2 < 8\text{N/mm}^2$$

$$\therefore \sigma_o = 5.95\ \text{N/mm}^2$$

$$Q_{su} = 0.76 \left\{ \frac{0.053 \times 1.005^{0.23}(18 + 9.2)}{3 + 0.12} + 0.85\sqrt{0.00186 \times 275} + 0.1 \times 5.95 \right\} 750 \times 0.8 \times 250$$

$$= 189866.43\text{N} = 189.86\ \text{kN}$$

The Strength of Column

$Q_{mu} < Q_{su}$ (flexural failure type)

$\therefore Q_u = 63.95$kN

The Summary of Strength of Columns

The Strength of Members						
COL. ID	**Storey**	**h_o/D**	**Q_{mu}(kN)**	**Q_{su}(kN)**	**Q (kN)**	**Failure Type**
C1	5	9.84	78	131	78	Flexural
	4	9.84	92	145	92	Flexural
	3	9.84	100	160	100	Flexural
	2	9.84	93	175	93	Flexural
	1	16.00	51	190	51	Flexural
C2	5	9.84	77	130	77	Flexural
	4	9.84	90	143	90	Flexural
	3	9.84	99	157	99	Flexural
	2	9.84	97	170	97	Flexural
	1	16.00	54	184	54	Flexural
C3	5	9.84	76	129	76	Flexural
	4	9.84	88	141	88	Flexural
	3	9.84	97	153	97	Flexural
	2	9.84	100	165	100	Flexural
	1	16.00	57	177	57	Flexural
C4	5	9.84	42	120	42	Flexural
	4	9.84	52	128	52	Flexural
	3	9.84	59	137	59	Flexural
	2	9.84	65	145	65	Flexural
	1	16.00	42	154	42	Flexural
C5	5	9.84	42	119	42	Flexural
	4	9.84	51	127	51	Flexural
	3	9.84	58	135	58	Flexural
	2	9.84	63	143	63	Flexural
	1	16.00	42	151	42	Flexural
C6	5	9.84	41	119	41	Flexural
	4	9.84	49	126	49	Flexural
	3	9.84	56	133	56	Flexural
	2	9.84	61	140	61	Flexural
	1	16.00	40	147	40	Flexural
C7	5	9.84	39	117	39	Flexural
	4	9.84	43	121	43	Flexural
	3	9.84	47	124	47	Flexural
	2	9.84	51	128	51	Flexural
	1	16.00	34	132	34	Flexural

3.4.4.4 Calculation of Column Ductility Index *F*

Calculation example of *F*-index is as shown below.
To calculate ductility index of each vertical member following flowchart is follows

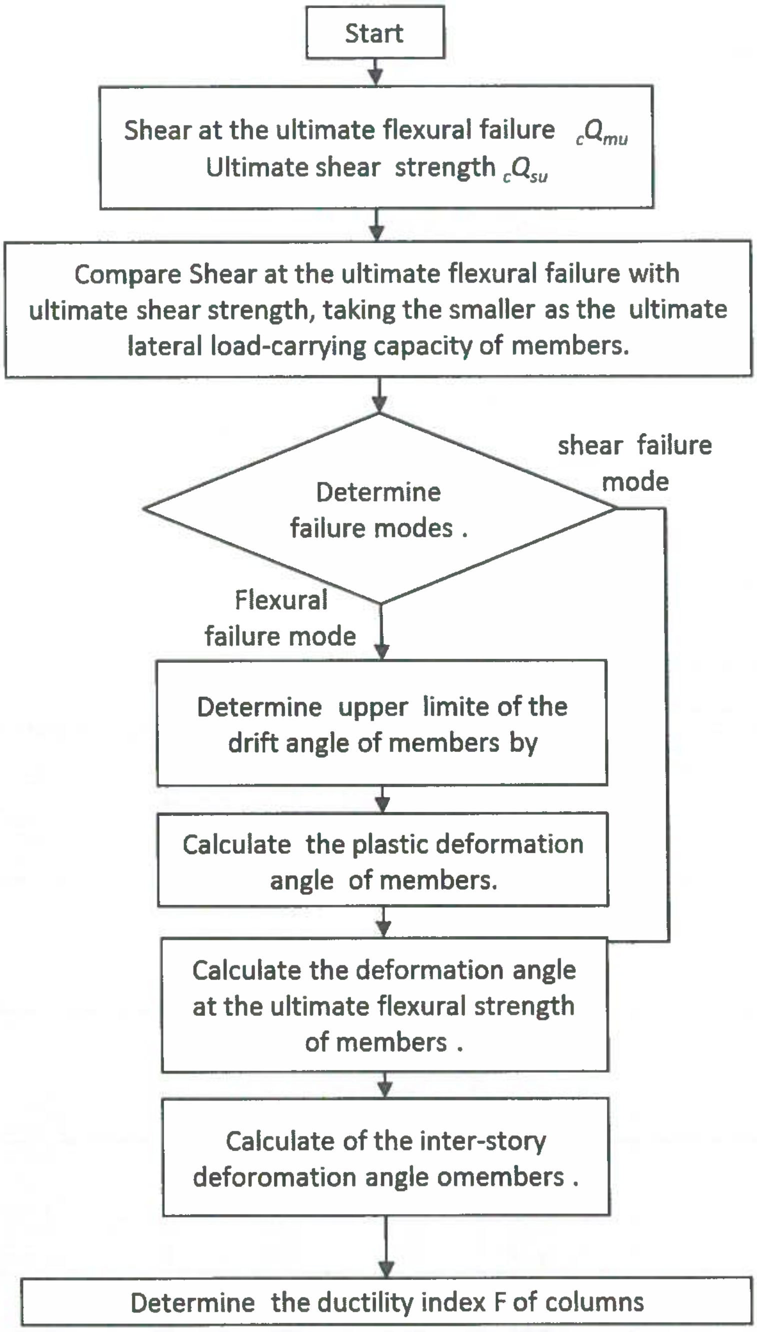


Figure 3.4.11 Flow Diagram of Ductility Index Calculation

The ductility index
Calculation procedure of ductility index of C_1 column at ground floor in X-direction is shown below.

a) Upper limit of deflection angle of flexural column $_cR_{max}$

$_cR_{max}=min\{_cR_{max(n)}, {}_cR_{max(s)}, {}_cR_{max(t)}, {}_cR_{max(b)}, {}_cR_{max(h)}, {}_cR_{max(p)}, {}_cR_{max(a)}, {}_cR_{max(b)}\}$

[$_cR_{max(n)}$] Upper limit of the deformation angle of the flexural column determined by the axial force

$$\eta=\frac{1116\times10^3}{(750\times250\times9.2)}=0.64$$

Concrete : Low strength concrete

Tie spacing : @225 with 90 Degrees hook

$$\eta_l = 0.20 \qquad \eta_H = 0.40$$

$$\eta > \eta_H, \therefore {}_cR_{max(n)} = \frac{1}{250}$$

[${}_cR_{max(s)}$] Upper limit of the deformation angle of the flexural column determined by the shear force

$${}_c\tau_u = \frac{min\ (51{,}119) \times 10^3}{(750 \times 0.8 \times 250)} = 0.34$$

$$\frac{{}_c\tau_u}{F_c} = \frac{0.426}{9.2} = 0.036 < 0.2$$

$${}_cR_{max(s)} = \frac{1}{30}$$

[${}_cR_{max(t)}$] Upper limit of the deformation angle of the flexural column determined by the tensile

$$p_t = 1.005 < 1.3$$

$${}_cR_{max(t)} = \frac{1}{30}$$

[${}_cR_{max(b)}$] Upper limit of the deformation angle of the flexural column determined by the spacing of hoop

$$s = @225 \text{ and } d_b = 20\phi$$

$$\frac{s}{d_b} = \frac{225}{20} = 11.25 > 8.0$$

$${}_cR_{max(b)} = \frac{1}{50}$$

[${}_cR_{max(h)}$] Upper limit of the deformation angle of the flexural column determined by the clear height

$$\frac{h_o}{D} = \frac{4000}{250} = 16 > 2.0$$

$${}_cR_{max(h)} = \frac{1}{30}$$

[${}_cR_{max(p)}$] Upper limit of the deformation angle of the flexural column determined by the poor beam-column joint for low strength concrete

$${}_cR_{max(p)} = \frac{1}{124}$$

[${}_cR_{max(o)}$] Upper limit of the deformation angle of the flexural column determined by the anchor length for low strength concrete

$${}_cR_{max(o)} = \frac{1}{124}$$

[${}_cR_{max(B)}$] Upper limit of the deformation angle of the flexural column determined by the clear height.

In this building there are no standing wall which makes $\frac{h_o}{D} \leq 2$

So ${}_cR_{max(B)} = \frac{1}{30}$

Therefore [${}_cR_{max}$] = $\frac{1}{250}$

b) Yield deflection angle of the Column ${}_cR_{my}$

$$\frac{h_o}{D} = \frac{4000}{250} = 16 \geq 3$$

$${}_cR_{my} = \frac{1}{150} \text{ but } {}_cR_{my} \ngtr {}_cR_{max}$$

$$\text{So, } {}_cR_{my} = \frac{1}{250}$$

c) The inter storey drift angle at the flexural yielding of column R_{my}.

$$\frac{h_o}{H_o}=\frac{4000}{4000}=1$$

$$\begin{aligned} R_{my} &= \left(\frac{h_o}{H_o}\right)\cdot {}_cR_{my} \\ &= 1\times\frac{1}{250} \\ &= \frac{1}{250}\geq\frac{1}{250} \end{aligned}$$

So, $R_{my}=\frac{1}{250}$

d) Plastic deformation angle of column $_cR_{mp}$

s =225 >100(mm)

$q = 1.1$

The strength margin for shear failure

$$\frac{{}_cQ_{su}}{{}_cQ_{mu}}=\frac{190}{51}=3.71$$

The plastic deflection angle of the column

$${}_cR_{mp}=10(3.71-1.1)\times\frac{1}{250}=\frac{1}{9.57}$$

e) The deformation angle at ultimate flexural strength of column $_cR_{mu}$

$${}_cR_{mu}=\frac{1}{250}+\frac{1}{9.57}=\frac{1}{9.25}\not>{}_cR_{max}=\frac{1}{250}\text{ (Lowest of above consideration)}$$

So, ${}_cR_{mu}=\frac{1}{250}$

f) The inter storey drift angle at the ultimate flexural strength of column R_{mu}

$$\frac{h_o}{H_o}=\frac{4000}{4000}=1.00$$

$$R_{mu}=\frac{h_o}{H_o}\times{}_cR_{mu}=1\times\frac{1}{250}=\frac{1}{250}\geq\frac{1}{250}$$

So, $R_{mu}=\frac{1}{250}$

g) The ductility index

Since $R_{mu}<R_y=\frac{1}{150}$

$$\begin{aligned} F &= 1.0+0.27\frac{R_{mu}-R_{250}}{R_y-R_{250}} \\ &= 1.0+0.27\frac{\frac{1}{250}-\frac{1}{250}}{\frac{1}{150}-\frac{1}{250}} \\ &= 1.0 \end{aligned}$$

The summary of ductility indices in X-direction is given below

The Ductility Indices													
COL. ID	**Storey**	Q_{mu} **(kN)**	Q_{su}**(kN)**	Q **(kN)**	**Failure Type**	${}_cR_{max}$	${}_cR_{my}$	R_{my}	${}_cR_{mp}$	${}_cR_{mu}$	R_{mu}	R_{su}	F
C1	5	78	131	78	Flexural	1/124	1/150	1/150	1/26	1/124	1/124	-	1.50
	4	92	145	92	Flexural	1/124	1/150	1/150	1/31	1/124	1/124	-	1.50
	3	100	160	100	Flexural	1/172	1/172	1/172	1/34	1/172	1/172	-	1.18
	2	93	175	93	Flexural	1/250	1/250	1/250	3/97	1/250	1/250	-	1.00
	1	51	190	51	Flexural	1/250	1/250	1/250	5/48	1/250	1/250	-	1.00
C2	5	77	130	77	Flexural	1/124	1/150	1/150	1/26	1/124	1/124	-	1.50
	4	90	143	90	Flexural	1/124	1/150	1/150	3/92	1/124	1/124	-	1.50
	3	99	157	99	Flexural	1/124	1/150	1/150	2/61	1/124	1/124	-	1.50
	2	97	170	97	Flexural	1/250	1/250	1/250	1/38	1/250	1/250	-	1.00
	1	54	184	54	Flexural	1/250	1/250	1/250	4/43	1/250	1/250	-	1.00
C3	5	76	129	76	Flexural	1/124	1/150	1/150	3/76	1/124	1/124	-	1.50
	4	88	141	88	Flexural	1/124	1/150	1/150	1/30	1/124	1/124	-	1.50
	3	97	153	97	Flexural	1/124	1/150	1/150	1/31	1/124	1/124	-	1.50
	2	100	165	100	Flexural	1/250	1/250	1/250	11/503	1/250	1/250	-	1.00
	1	57	177	57	Flexural	1/250	1/250	1/250	3/37	1/250	1/250	-	1.00
C4	5	42	120	42	Flexural	1/124	1/150	1/150	3/26	1/124	1/124	-	1.50
	4	52	128	52	Flexural	1/124	1/150	1/150	7/76	1/124	1/124	-	1.50
	3	59	137	59	Flexural	1/124	1/150	1/150	5/62	1/124	1/124	-	1.50
	2	65	145	65	Flexural	1/124	1/150	1/150	6/79	1/124	1/124	-	1.50
	1	42	154	42	Flexural	1/188	1/188	1/188	5/37	1/188	1/188	-	1.13
C5	5	42	119	42	Flexural	1/124	1/150	1/150	2/17	1/124	1/124	-	1.50
	4	51	127	51	Flexural	1/124	1/150	1/150	5/53	1/124	1/124	-	1.50
	3	58	135	58	Flexural	1/124	1/150	1/150	8/97	1/124	1/124	-	1.50
	2	63	143	63	Flexural	1/124	1/150	1/150	1/13	1/124	1/124	-	1.50
	1	42	151	42	Flexural	7/941	1/150	1/150	15/89	7/941	7/941	-	1.40
C6	5	41	119	41	Flexural	1/124	1/150	1/150	5/42	1/124	1/124	-	1.50
	4	49	126	49	Flexural	1/124	1/150	1/150	4/41	1/124	1/124	-	1.50
	3	56	133	56	Flexural	1/124	1/150	1/150	4/47	1/124	1/124	-	1.50
	2	61	140	61	Flexural	1/124	1/150	1/150	4/51	1/124	1/124	-	1.50
	1	40	147	40	Flexural	1/124	1/150	1/150	12/71	1/124	1/124	-	1.50
C7	5	39	117	39	Flexural	1/124	1/150	1/150	11/86	1/124	1/124	-	1.50
	4	43	121	43	Flexural	1/124	1/150	1/150	9/80	1/124	1/124	-	1.50
	3	47	124	47	Flexural	1/124	1/150	1/150	8/79	1/124	1/124	-	1.50
	2	51	128	51	Flexural	1/124	1/150	1/150	4/43	1/124	1/124	-	1.50
	1	34	132	34	Flexural	1/124	1/150	1/150	3/16	1/124	1/124	-	1.50

3.4.4.5 Calculation of the Effective Strength Factor

The effective strength factor indicates the ratio of the restoring force at the ultimate deflection angle of the first group (R_1) to the ultimate strength. The practical calculation method is as follows; the effective strength factor is calculated using the ratio of $R_1$to R_{my} where R_{my} is the deflection angle at yielding. The effective strength factors for the column on the ground floor are calculated as follows. In this floor, failure mode of all columns is flexural and minimum ductility index 1.

The effective strength factor for flexural column, $\alpha_m = 0.3 + 0.7\dfrac{R_1}{R_{my}}$

At ductility index F=1.0, $R_1 = \dfrac{1}{250}$

$$\alpha_m \text{ for column } C_4 = 0.3 + 0.7 \times \dfrac{\frac{1}{250}}{\frac{1}{188}} = 0.83$$

$$\alpha_m \text{ for column } C_5, C_6, C_7 = 0.3 + 0.7 \times \dfrac{\frac{1}{250}}{\frac{1}{150}} = 0.72$$

At ductility index F=1.13, $R_1 = \dfrac{1}{188}$

$$\alpha_m = 0.3 + 0.7 \times \dfrac{\frac{1}{188}}{\frac{1}{150}} = 0.72$$

At ductility index F = 1.13, $R_l = \dfrac{1}{188}$

$$\alpha_m \text{ for column } C_5, C_6, C_7 = 0.3 + 0.7 \times \dfrac{\frac{1}{188}}{\frac{1}{150}} = 0.85$$

Summery of effective strength factor in X-direction is given below:

The Effective strength factors										
							Effective strength factor for various 1st Group			
Storey	COL.ID	Q_{mu}(kN)	Q_{su}(kN)	Failure Mode	R_{my}	F	F=0.8	F=1.0	1.0<F<1.27	1.27≤F
5	C1	78	131	Flexural	1/150	1.50	-	-	-	1.00
	C2	77	130	Flexural	1/150	1.50	-	-	-	1.00
	C3	76	129	Flexural	1/150	1.50	-	-	-	1.00
	C4	42	120	Flexural	1/150	1.50	-	-	-	1.00
	C5	42	119	Flexural	1/150	1.50	-	-	-	1.00
	C6	41	119	Flexural	1/150	1.50	-	-	-	1.00
	C7	39	117	Flexural	1/150	1.50	-	-	-	1.00
4	C1	92	145	Flexural	1/150	1.50	-	-	-	1.00
	C2	90	143	Flexural	1/150	1.50	-	-	-	1.00
	C3	88	141	Flexural	1/150	1.50	-	-	-	1.00
	C4	52	128	Flexural	1/150	1.50	-	-	-	1.00
	C5	51	127	Flexural	1/150	1.50	-	-	-	1.00
	C6	49	126	Flexural	1/150	1.50	-	-	-	1.00
	C7	43	121	Flexural	1/150	1.50	-	-	-	1.00
3	C1	100	160	Flexural	1/172	1.18	-	-	1.00	-
	C2	99	157	Flexural	1/150	1.50	-	-	0.91	1.00
	C3	97	153	Flexural	1/150	1.50	-	-	0.91	1.00
	C4	59	137	Flexural	1/150	1.50	-	-	0.91	1.00
	C5	58	135	Flexural	1/150	1.50	-	-	0.91	1.00
	C6	56	133	Flexural	1/150	1.50	-	-	0.91	1.00
	C7	47	124	Flexural	1/150	1.50	-	-	0.91	1.00
2	C1	93	175	Flexural	1/250	1.00	-	1.00	-	-
	C2	97	170	Flexural	1/250	1.00	-	1.00	-	-
	C3	100	165	Flexural	1/250	1.00	-	1.00	-	-
	C4	65	145	Flexural	1/150	1.50	-	0.72	-	1.00
	C5	63	143	Flexural	1/150	1.50	-	0.72	-	1.00
	C6	61	140	Flexural	1/150	1.50	-	0.72	-	1.00
	C7	51	128	Flexural	1/150	1.50	-	0.72	-	1.00
1	C1	51	190	Flexural	1/250	1.00	-	1.00	-	-
	C2	54	184	Flexural	1/250	1.00	-	1.00	-	-
	C3	57	177	Flexural	1/250	1.00	-	1.00	-	-
	C4	42	154	Flexural	1/188	1.13	-	0.83	1.00	-
	C5	42	151	Flexural	1/150	1.40	-	0.72	0.86	1.00
	C6	40	147	Flexural	1/150	1.50	-	0.72	0.86	1.00
	C7	34	132	Flexural	1/150	1.50	-	0.72	0.86	1.00

3.4.4.6 The Strength Index

$$C = \frac{Q_u}{\Sigma W}$$

Q_u = The strength of column

ΣW = The weight of building supported by the storey concerned

The calculation procedure of strength index for C_1 column at ground floor are shown here.

Q_u = 51kN

W = 20074kN

No of C_1 column = 4

$$\therefore C = \frac{51 \times 4}{20074} = 0.01 \text{ for column } C_1$$

***C,F* Indices and Effective strength factors**										
Storey	**COL.ID**	**No of Column**	**ΣW_i (kN)**	**Q_u (kN)**	***C***	***F***	**Effective strength factor for various 1st Group**			
							F=0.8	**F=1.0**	**$1.0<F<1.27$**	**$1.27\leq F$**
5	C1	4	2555	78	0.12	1.50	-	-	-	1.00
	C2	2		77	0.06	1.50	-	-	-	1.00
	C3	2		76	0.06	1.50	-	-	-	1.00
	C4	8		42	0.13	1.50	-	-	-	1.00
	C5	4		42	0.07	1.50	-	-	-	1.00
	C6	6		41	0.10	1.50	-	-	-	1.00
	C7	4		39	0.06	1.50	-	-	-	1.00
4	C1	4	6935	92	0.05	1.50	-	-	-	1.00
	C2	2		90	0.03	1.50	-	-	-	1.00
	C3	2		88	0.03	1.50	-	-	-	1.00
	C4	8		52	0.06	1.50	-	-	-	1.00
	C5	4		51	0.03	1.50	-	-	-	1.00
	C6	6		49	0.04	1.50	-	-	-	1.00
	C7	4		43	0.02	1.50	-	-	-	1.00
3	C1	4	11315	100	0.04	1.18	-	-	1.00	-
	C2	2		99	0.02	1.50	-	-	0.91	1.00
	C3	2		97	0.02	1.50	-	-	0.91	1.00
	C4	8		59	0.04	1.50	-	-	0.91	1.00
	C5	4		58	0.02	1.50	-	-	0.91	1.00
	C6	6		56	0.03	1.50	-	-	0.91	1.00
	C7	4		47	0.02	1.50	-	-	0.91	1.00
2	C1	4	15694	93	0.02	1.00	-	1.00	-	-
	C2	2		97	0.01	1.00	-	1.00	-	-
	C3	2		100	0.01	1.00	-	1.00	-	-
	C4	8		65	0.03	1.50	-	0.72	-	1.00
	C5	4		63	0.02	1.50	-	0.72	-	1.00
	C6	6		61	0.02	1.50	-	0.72	-	1.00
	C7	4		51	0.01	1.50	-	0.72	-	1.00
1	C1	4	20074	51	0.01	1.00	-	1.00	-	-
	C2	2		54	0.01	1.00	-	1.00	-	-
	C3	2		57	0.01	1.00	-	1.00	-	-
	C4	8		42	0.02	1.13	-	0.83	1.00	-
	C5	4		42	0.01	1.40	-	0.72	0.86	1.00
	C6	6		40	0.01	1.50	-	0.72	0.86	1.00
	C7	4		34	0.01	1.50	-	0.72	0.86	1.00

Basic Seismic Index in X-direction is given below:

Storey	$\frac{(n+1)}{(n+i)}$	**Strength Dominant** $E_0 = (C_1 + \Sigma \acute{a}_i . C_i) F_i$							**Ductility Dominant** $E_0 = \sqrt{(C_1 \times F_1)^2 + (C_2 \times F_2)^2 + (C_3 \times F_3)^2}$							**E_0**
		1st group		**2nd group**		**3rd group**		**E_{01}**	**1st group**		**2nd group**		**3rd group**		**E_{02}**	
		F_1	C_1	$\acute{a}_2$	C_2	$\acute{a}_3$	C_3		C_1	F_1	C_2	F_2	C_3	F_3		
5	0.60	1.50	0.60					0.54	0.60	1.50					0.54	0.54
4	0.67	1.50	0.26					0.26	0.26	1.50					0.26	0.26
3	0.75	1.18	0.04	0.91	0.14			0.15	0.04	1.18	0.14	1.5			0.16	0.16
		1.5	0.14					0.16								
2	0.86	1.0	0.05	0.72	0.09			0.09	0.05	1.0	0.09	1.5			0.12	0.12
		1.5	0.09					0.11								
1	1.00	1.0	0.02	0.83	0.02	0.72	0.03	0.05	0.02	1.0	0.02	1.13	0.03	1.4	0.05	0.05
		1.13	0.02	0.86	0.03			0.05	0.04	1.0	0.01	1.4	0.02	1.5	0.05	0.05
		1.4	0.03					0.04								

The Assessment Direction: Y
Seismic assessment in Y-direction has been given below following similar steps shown in X-direction.
The summery of strength of column in Y-direction

The Strength of Members						
COL. ID	**Storey**	**h_o/D**	**Q_{mu} (kN)**	**Q_{su}(kN)**	**Q (kN)**	**Failure Type**
C1	5	3.28	302	187	187	Shear
	4	3.28	343	202	202	Shear
	3	3.28	367	217	217	Shear
	2	3.28	343	231	231	Shear
	1	5.33	188	212	188	Flexural
C2	5	3.28	299	186	186	Shear
	4	3.28	338	200	200	Shear
	3	3.28	363	213	213	Shear
	2	3.28	354	227	227	Shear
	1	5.33	197	206	197	Flexural
C3	5	3.28	296	185	185	Shear
	4	3.28	332	197	197	Shear
	3	3.28	358	209	209	Shear
	2	3.28	368	222	222	Shear
	1	5.33	207	199	199	Shear
C4	5	3.28	148	169	148	Flexural
	4	3.28	177	177	177	Flexural
	3	3.28	199	186	186	Shear
	2	3.28	216	194	194	Shear
	1	5.33	140	174	140	Flexural
C5	5	3.28	147	168	147	Flexural
	4	3.28	173	176	173	Flexural
	3	3.28	195	184	184	Shear
	2	3.28	212	192	192	Shear
	1	5.33	97	142	97	Flexural
C6	5	3.28	172	176	172	Flexural
	4	3.28	191	183	183	Shear
	3	3.28	207	190	190	Shear
	2	3.28	220	197	197	Shear
	1	5.33	96	142	96	Flexural
C7	5	3.28	141	170	141	Flexural
	4	3.28	153	173	153	Flexural
	3	3.28	163	177	163	Flexural
	2	3.28	173	181	173	Flexural
	1	5.33	75	136	75	Flexural

Summary of ductility indices in Y-direction is given below

The Ductility Indices													
COL. ID	Storey	Q_{mu} (kN)	Q_{su} (kN)	Q (kN)	Failure Type	${}_cR_{max}$	${}_cR_{my}$	R_{my}	${}_cR_{mp}$	${}_cR_{mu}$	R_{mu}	R_{su}	F
C1	5	302	187	187	Shear	1/250	1/250	1/250	-	-	-	1/250	1.00
	4	343	202	202	Shear	1/250	1/250	1/250	-	-	-	1/250	1.00
	3	367	217	217	Shear	1/250	1/250	1/250	-	-	-	1/250	1.00
	2	343	231	231	Shear	1/250	1/250	1/250	-	-	-	1/250	1.00
	1	188	212	188	Flexural	1/250	1/250	1/250	0	1/250	1/250	-	1.00
C2	5	299	186	186	Shear	1/250	1/250	1/250	-	-	-	1/250	1.00
	4	338	200	200	Shear	1/250	1/250	1/250	-	-	-	1/250	1.00
	3	363	213	213	Shear	1/250	1/250	1/250	-	-	-	1/250	1.00
	2	354	227	227	Shear	1/250	1/250	1/250	-	-	-	1/250	1.00
	1	197	206	197	Flexural	1/250	1/250	1/250	0	1/250	1/250	-	1.00
C3	5	296	185	185	Shear	1/250	1/250	1/250	-	-	-	1/250	1.00
	4	332	197	197	Shear	1/250	1/250	1/250	-	-	-	1/250	1.00
	3	358	209	209	Shear	1/250	1/250	1/250	-	-	-	1/250	1.00
	2	368	222	222	Shear	1/250	1/250	1/250	-	-	-	1/250	1.00
	1	207	199	199	Shear	1/250	1/250	1/250	-	-	-	1/250	1.00
C4	5	148	169	148	Flexural	1/124	1/150	1/150	0	1/124	1/124	-	1.50
	4	177	177	177	Flexural	1/124	1/150	1/150	0	1/150	1/150	-	1.27
	3	199	186	186	Shear	1/124	1/150	1/150	-	-	-	1/167	1.20
	2	216	194	194	Shear	1/124	1/150	1/150	-	-	-	1/176	1.17
	1	140	174	140	Flexural	1/188	1/188	1/188	0	1/188	1/188	-	1.13
C5	5	147	168	147	Flexural	1/124	1/150	1/150	0	1/124	1/124	-	1.50
	4	173	176	173	Flexural	1/124	1/150	1/150	0	1/150	1/150	-	1.27
	3	195	184	184	Shear	1/124	1/150	1/150	-	-	-	1/163	1.22
	2	212	192	192	Shear	1/124	1/150	1/150	-	-	-	1/174	1.18
	1	97	142	97	Flexural	1/124	1/150	4/369	2/83	4/305	4/305	-	2.08
C6	5	172	176	172	Flexural	1/124	1/150	1/150	0	1/150	1/150	-	1.27
	4	191	183	183	Shear	1/124	1/150	1/150	-	-	-	1/161	1.22
	3	207	190	190	Shear	1/124	1/150	1/150	-	-	-	1/172	1.18
	2	220	197	197	Shear	1/124	1/150	1/150	-	-	-	1/177	1.17
	1	96	142	96	Flexural	1/124	1/150	1/150	2/79	1/124	1/124	-	1.50
C7	5	141	170	141	Flexural	1/124	1/150	1/150	3/442	1/124	1/124	-	1.50
	4	153	173	153	Flexural	1/124	1/150	1/150	2/871	1/124	1/124	-	1.50
	3	163	177	163	Flexural	1/124	1/150	1/150	0	1/150	1/150	-	1.27
	2	173	181	173	Flexural	1/124	1/150	1/150	0	1/150	1/150	-	1.27
	1	75	136	75	Flexural	1/124	1/150	1/150	25/524	1/124	1/124	-	1.50

Summary of effective strength factor in Y-direction is given below

The Effective strength factors										
Storey	COL. ID	Q_{mu} (kN)	Q_{su} (kN)	Failure Mode	R_{my}	F	Effective Strength Factors for various 1st Group			
							$F = 0.8$	$F = 1.0$	$1.0 < F < 1.27$	$1.27 \leq F$
5	C1	302	187	Shear	1/250	1.00	-	1.00	-	-
	C2	299	186	Shear	1/250	1.00	-	1.00	-	-
	C3	296	185	Shear	1/250	1.00	-	1.00	-	-
	C4	148	169	Flexural	1/150	1.50	-	0.72	-	1.00
	C5	147	168	Flexural	1/150	1.50	-	0.72	-	1.00
	C6	172	176	Flexural	1/150	1.27	-	0.72	-	1.00
	C7	141	170	Flexural	1/150	1.50	-	0.72	-	1.00
4	C1	343	202	Shear	1/250	1.00	-	1.00	-	-
	C2	338	200	Shear	1/250	1.00	-	1.00	-	-
	C3	332	197	Shear	1/250	1.00	-	1.00	-	-
	C4	177	177	Flexural	1/150	1.27	-	0.72	0.95	1.00
	C5	173	176	Flexural	1/150	1.27	-	0.72	0.95	1.00
	C6	191	183	Shear	1/150	1.22	-	0.75	1.00	-
	C7	153	173	Flexural	1/150	1.50	-	0.72	0.95	1.00
3	C1	367	217	Shear	1/250	1.00	-	1.00	-	-
	C2	363	213	Shear	1/250	1.00	-	1.00	-	-
	C3	358	209	Shear	1/250	1.00	-	1.00	-	-
	C4	199	186	Shear	1/150	1.20	-	0.77	1.00	-
	C5	195	184	Shear	1/150	1.22	-	0.76	1.00	-
	C6	207	190	Shear	1/150	1.18	-	0.79	1.00	-
	C7	163	177	Flexural	1/150	1.27	-	0.72	1.00	-
2	C1	343	231	Shear	1/250	1.00	-	1.00	-	-
	C2	354	227	Shear	1/250	1.00	-	1.00	-	-
	C3	368	222	Shear	1/250	1.00	-	1.00	-	-
	C4	216	194	Shear	1/150	1.17	-	0.80	1.00	-
	C5	212	192	Shear	1/150	1.18	-	0.80	1.00	-
	C6	220	197	Shear	1/150	1.17	-	0.81	1.00	-
	C7	173	181	Flexural	1/150	1.27	-	0.72	1.00	-
1	C1	188	212	Flexural	1/250	1.00	-	1.00	-	-
	C2	197	206	Flexural	1/250	1.00	-	1.00	-	-
	C3	207	199	Shear	1/250	1.00	-	1.00	-	-
	C4	140	174	Flexural	1/188	1.13	-	0.83	1.00	-
	C5	97	142	Flexural	1/92	2.08	-	0.56	0.64	1.00
	C6	96	142	Flexural	1/150	1.50	-	0.72	0.86	1.00
	C7	75	136	Flexural	1/150	1.50	-	0.72	0.86	1.00

C,F Indices and Effective strength factors										
Storey	COL.ID	No of Column	ΣW_i (kN)	Q_u (kN)	*C*	*F*	1st Group			
							F=0.8	*F*=1.0	1.0<*F*<1.27	1.27≤*F*
5	C1	4	2555	302	0.47	1.00	-	1.00	-	-
	C2	2		296	0.23	1.00	-	1.00	-	-
	C3	2		148	0.12	1.00	-	1.00	-	-
	C4	8		147	0.46	1.50	-	0.72	-	1.00
	C5	4		172	0.27	1.50	-	0.72	-	1.00
	C6	6		141	0.33	1.27	-	0.72	-	1.00
	C7	4		343	0.54	1.50	-	0.72	-	1.00
4	C1	4	6935	338	0.20	1.00	-	1.00	-	-
	C2	2		332	0.10	1.00	-	1.00	-	-
	C3	2		177	0.05	1.00	-	1.00	-	-
	C4	8		173	0.20	1.27	-	0.72	0.95	1.00
	C5	4		191	0.11	1.27	-	0.72	0.95	1.00
	C6	6		153	0.13	1.22	-	0.75	1.00	-
	C7	4		367	0.21	1.50	-	0.72	0.95	1.00
3	C1	4	11315	363	0.13	1.00	-	1.00	-	-
	C2	2		358	0.06	1.00	-	1.00	-	-
	C3	2		199	0.04	1.00	-	1.00	-	-
	C4	8		195	0.14	1.20	-	0.77	1.00	-
	C5	4		207	0.07	1.22	-	0.76	1.00	-
	C6	6		163	0.09	1.18	-	0.79	1.00	-
	C7	4		343	0.12	1.27	-	0.72	1.00	-
2	C1	4	15694	354	0.09	1.00	-	1.00	-	-
	C2	2		368	0.05	1.00	-	1.00	-	-
	C3	2		216	0.03	1.00	-	1.00	-	-
	C4	8		212	0.11	1.17	-	0.80	1.00	-
	C5	4		220	0.06	1.18	-	0.80	1.00	-
	C6	6		173	0.07	1.17	-	0.81	1.00	-
	C7	4		188	0.05	1.27	-	0.72	1.00	-
1	C1	4	20074	197	0.04	1.00	-	1.00	-	-
	C2	2		207	0.02	1.00	-	1.00	-	-
	C3	2		140	0.01	1.00	-	1.00	-	-
	C4	8		97	0.04	1.13	-	0.83	1.00	-
	C5	4		96	0.02	2.08	-	0.56	0.64	1.00
	C6	6		75	0.02	1.50	-	0.72	0.86	1.00
	C7	4		0	0.00	1.50	-	0.72	0.86	1.00

Basic seismic index of each floor is given below

Storey	$\frac{(n+1)}{(n+i)}$	Strength Dominant $E_0 = (C_1 + \Sigma \acute{a}_i . C_i) F_i$							Ductility Dominant $E_0 = \sqrt{(C_1 \times F_1)^2 + (C_2 \times F_2)^2 + (C_3 \times F_3)^2}$							E_0
		1st group		2nd group		3rd group		E_{01}	1st group		2nd group		3rd group		E_{02}	
		F_1	C_1	$\acute{a}_2$	C_2	$\acute{a}_3$	C_3		C_1	F_1	C_2	F_2	C_3	F_3		
5	0.60	1.00	0.82	0.72	1.60			1.18	0.82	1.00	0.33	1.27	1.27	1.50	1.27	1.27
		1.27	1.60					1.22	1.60	1.27					1.22	
4	0.67	1.00	0.34	0.72	0.65			0.54	0.34	1.00	0.44	1.22	0.21	1.50	0.47	0.57
		1.22	0.13	0.95	0.52			0.51								
		1.27	0.52					0.44	0.47	1.00	0.31	1.27	0.21	1.50	0.57	
3	0.75	1.00	0.23	0.72	0.12	0.76	0.30	0.41	0.23	1.00	0.30	1.18	0.12	1.27	0.33	0.37
		1.18	0.42					0.37								
2	0.86	1.0	0.16	0.80	0.23	0.72	0.05	0.33	0.16	1.0	0.23	1.17	0.05	1.27	0.28	0.33
		1.17	0.28					0.28	0.28	1.17					0.28	
1	1.00	1.0	0.07	0.72	0.06	0.56	0.02	0.13	0.07	1.0	0.04	1.13	0.04	1.5	0.08	0.13
		1.13	0.04	0.64	0.02	0.86	0.02	0.07	0.04	1.13	0.02	1.5	0.02	2.08	0.07	
		1.5	0.04					0.06	0.04	1.5					0.06	

3.5 IRREGULARITY INDEX S_D

3.5.1 General

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

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

3.5.2 Items to be Considered

Item to be considered are listed below:

(a) Items related to floor plan (to the structural integrity of floor plan) regularity, aspect ratio, narrow part, expansion joint, well-style hall (size and location)

(b) Items related to sectional plan (to the structural integrity of sectional plan) existence of basement, uniformity of storey height, existence of pilotis.

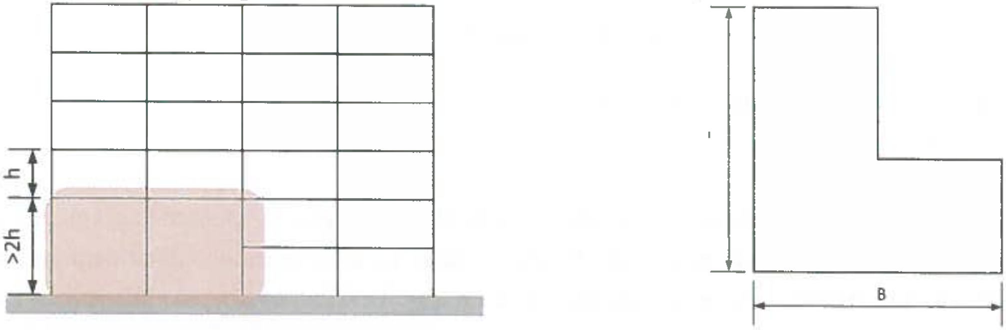
(c) Items related to floor plan-distance between centroid of gravity and center of lateral stiffness.

(d) Items related to sectional plan-ratio of stiffness of lower storey to upper storey.

3.5.3 Recommendation for Application

Eccentricity and stiffness in second level screening procedure are done by the simplified method since the evaluation has a precondition of allowing manual /hand calculation. This fact limits the evaluation capacity, such as not being able to fully address when evaluating a complex type of building. But it may use BNBC 2015 as an alternative method instead of these cases.

This standard targets the buildings which have relatively simple structures. Therefore, the evaluation of the stiffness is done by using only the section area of columns. It may cause inaccurate evaluation of the stiffness of a building with an atrium. Another issue is the evaluation of building with complex plan, because the irregularity of plan determines the rotational stiffness.



Portion of double height column **Irregular plan**

Figure 3.5.1 Examples of Irregularity in Buildings

In such a case, it is recommended to study BNBC 20152.5.7.3.1 Plan Irregularity i) torsion irregularity or 2.7.7.3.2 Vertical Irregularity, and conduct the evaluation as suggested below.

Table 3.5.1 Case of Using Torsion Irregularity in BNBC 2015

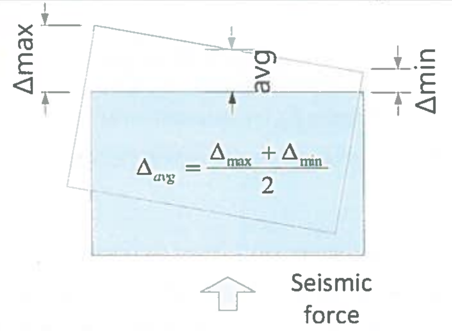
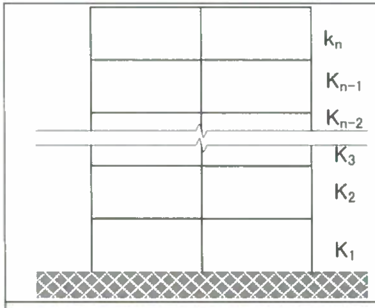
	$\frac{\Delta \max}{\Delta \text{avg}}$	G_l
Eccentricity (Plan Irregularity)	<1.2	1.0
	=1.2	0.9
	=1.4	0.8
Remarks		
If $1.2 < \frac{\Delta \max}{\Delta \text{avg}} < 1.4$ then G_l should be calculated by interpolation. Set G_a=*1.0* when using the result of the calculation.		

Table 3.5.2 Case of Using Stiffness Irregularity in BNBC 2015

 $k_i < 0.7k_{i+1}$ $or\ k_i < 0.8\left(\frac{k_{i+1}+k_{i+2}+k_{i+3}}{3}\right)$	$\frac{k_i}{k_{i+1}} \times \frac{3 \cdot k_i}{k_{i+1}+k_{i+2}+k_{i+3}}$	G_n
Stiffness Irregularity	>0.7 (>0.8)	1.0
	=0.7 (=0.8)	0.9
	=0.6 (=0.7)	0.8
Remarks		
If $0.6(0.7) < \frac{k_i}{k_{i+1}} \times \frac{3 \cdot k_i}{k_{i+1}+k_{i+2}+k_{i+3}} < 0.7(0.8)$ then G_n should be calculated by interpolation. Set G_i=*1.0* and G_j=*1.0* when using the result of the calculation.		

3.5.4 Calculation Procedure

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

(1) Calculation method for index

Second level screening

$S_{D2} = q_{2a} \times q_{2b} \times ..... \times q_{2n}$ (29) of J. Standard

Where:

q_{2i} = [1-(1-G_i)×R_{2i}] ......$i=a,b,c,d,e,f,i.j,l,n$

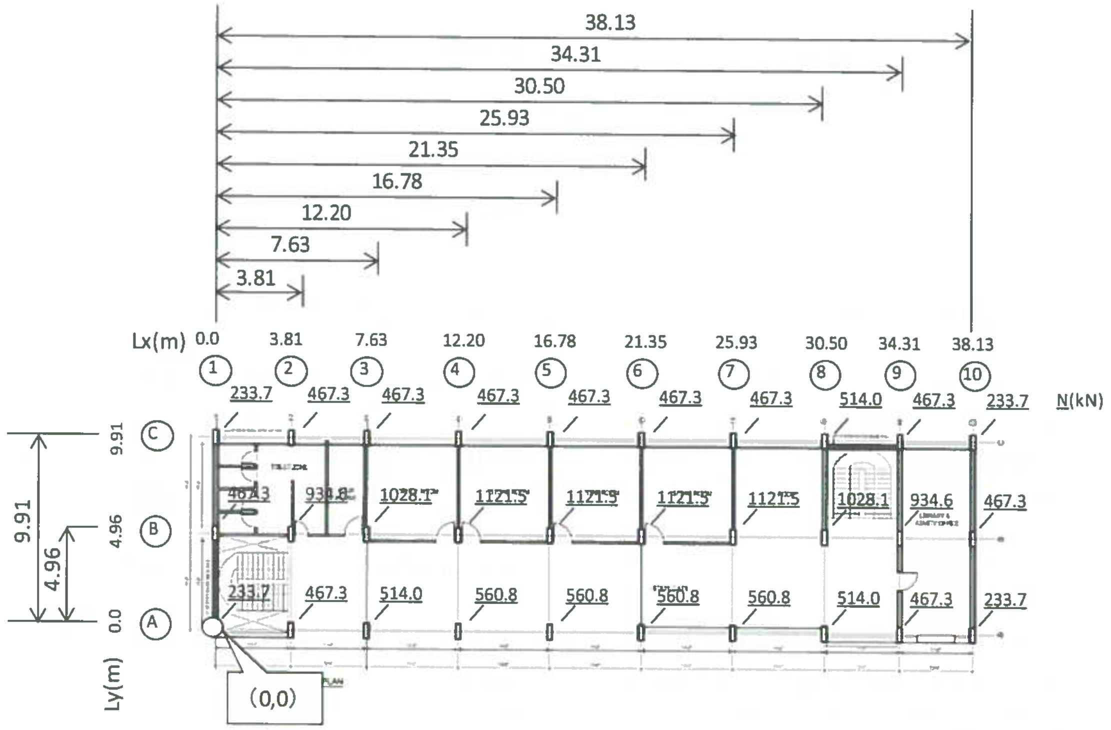
q_{2i} = [1.2-(1-G_i)×R_{2i}] ......$i=h$

3.5.5 Example of Calculation of Stiffness/Mass Ratio of above and below Stories, Eccentricity and S_D Index

Here are calculation examples of stiffness/mass ratio and eccentricity based on J-Standard.

(1)The center of gravity

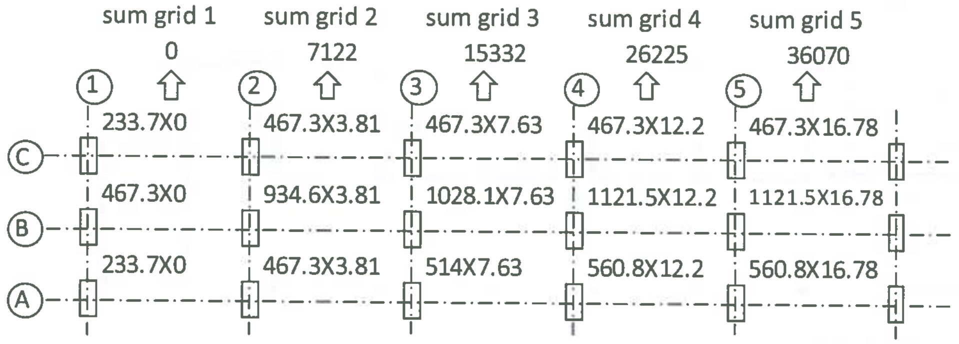
A-1 is set as an origin of coordinates. Below shows the distances from the origin and the axial forces of the columns.

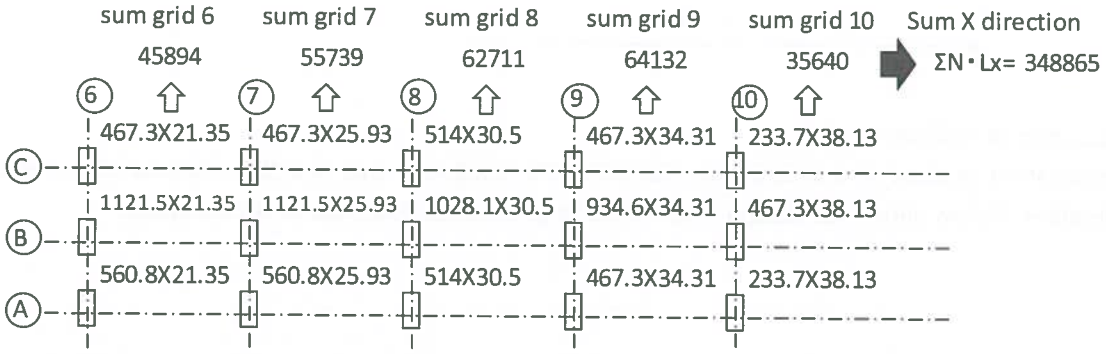


ΣN= 18271.2 kN

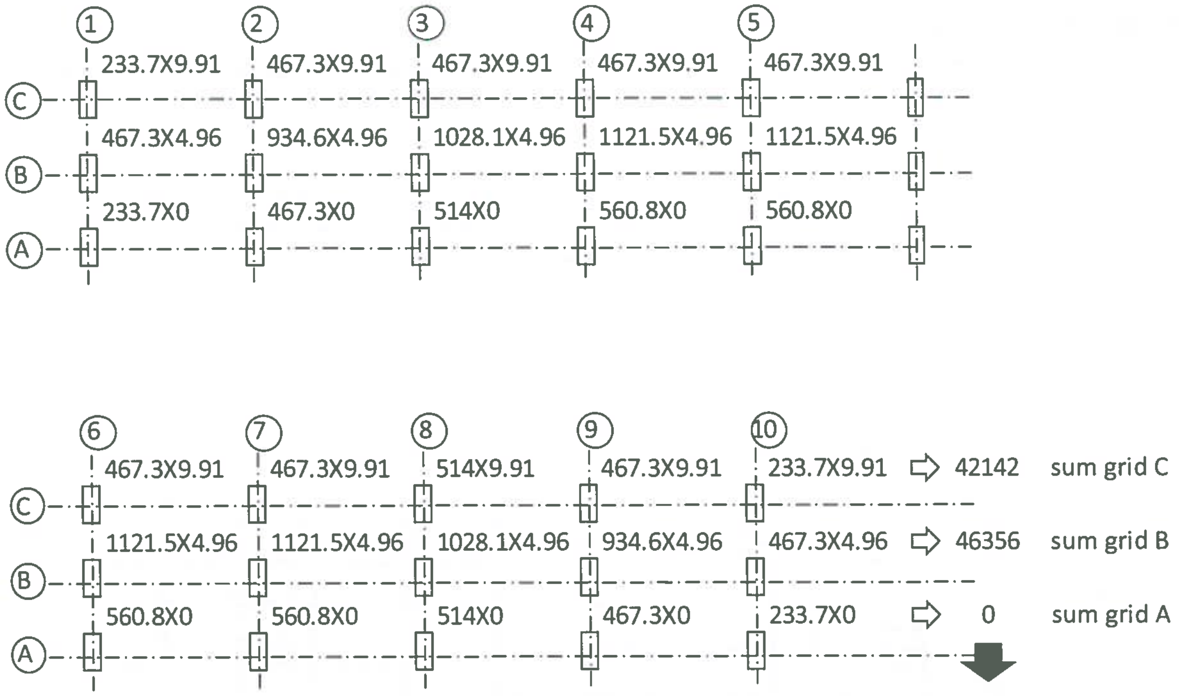
Multiply axial force and distance in X and Y direction

■N・Lx





■N・Ly

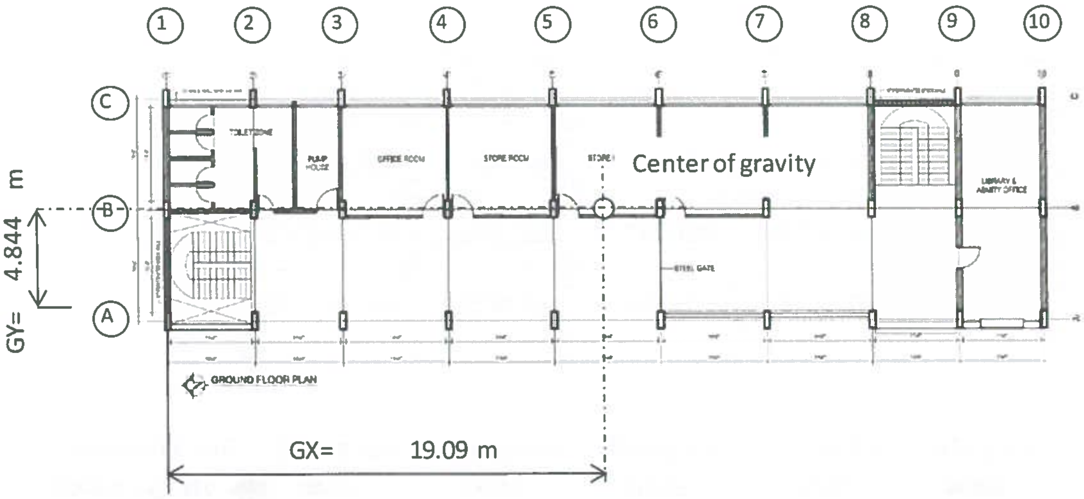


Sum Y direction ΣN・Ly= 88498

Distance to the center of gravity from the origin of each direction is obtained by dividing the total value of all directions with the total weight, which the floor is holding.

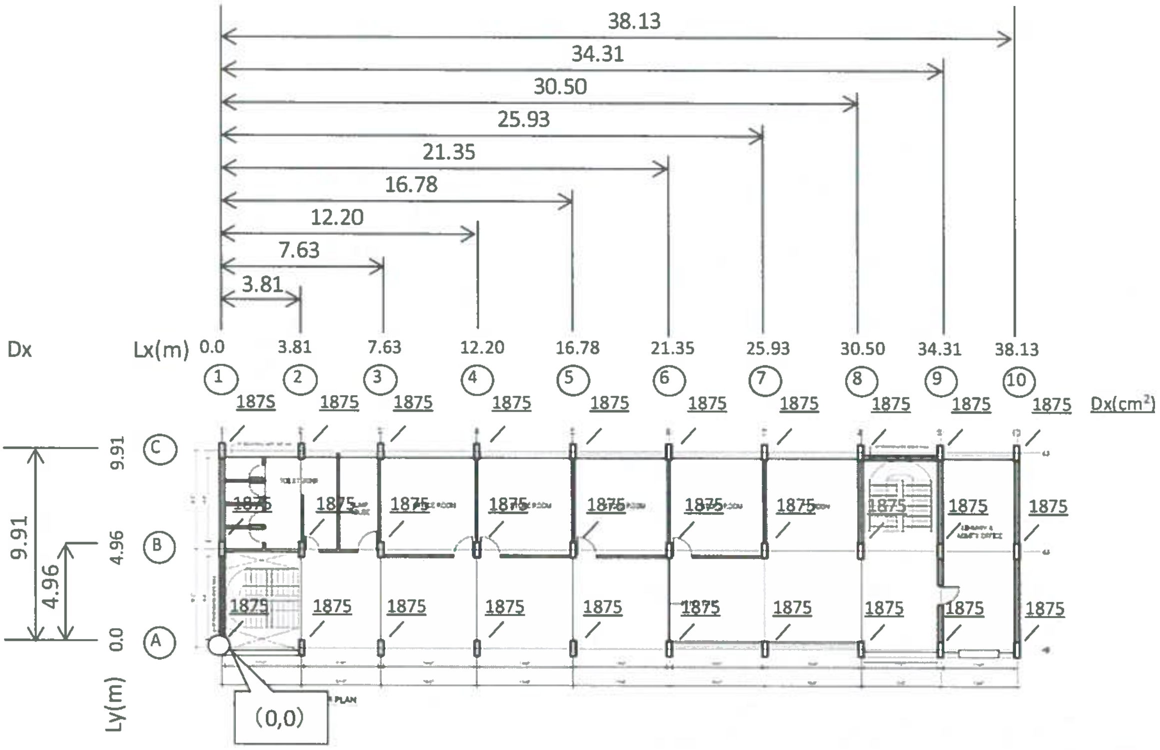
GX= ΣN×Lx/ΣN= 348865 / 18271.2 = 19.09 m

GY= ΣN×Ly/ΣN= 88498 / 18271.2 = 4.844 m

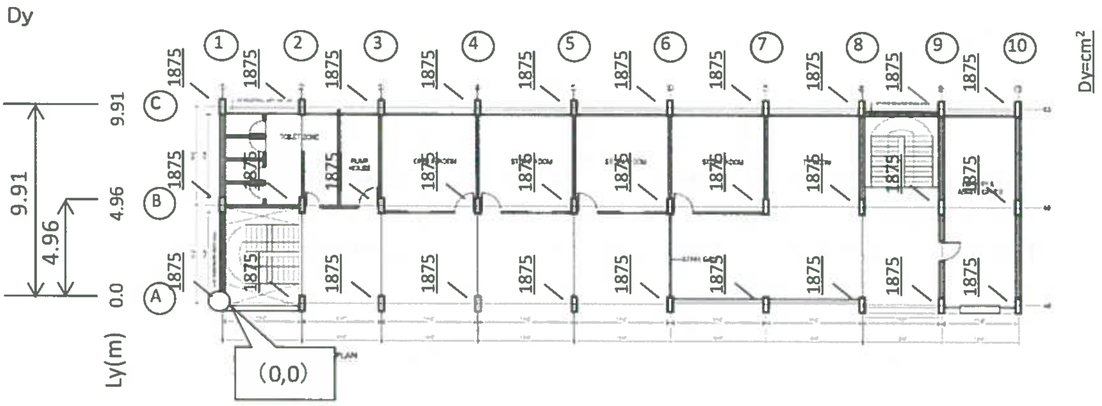


(2) The center of stiffness

This calculation method is a simple one that takes the sectional area and the stiffness of column into consideration. Below shows the distances from the origin and the stiffness of the columns.



ΣDx= 56250

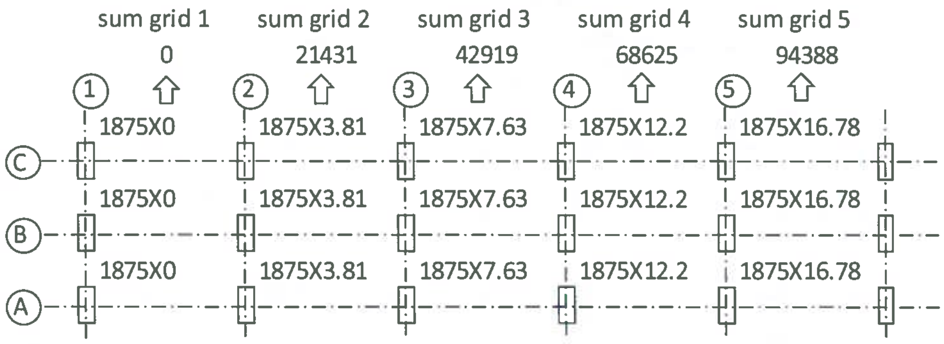


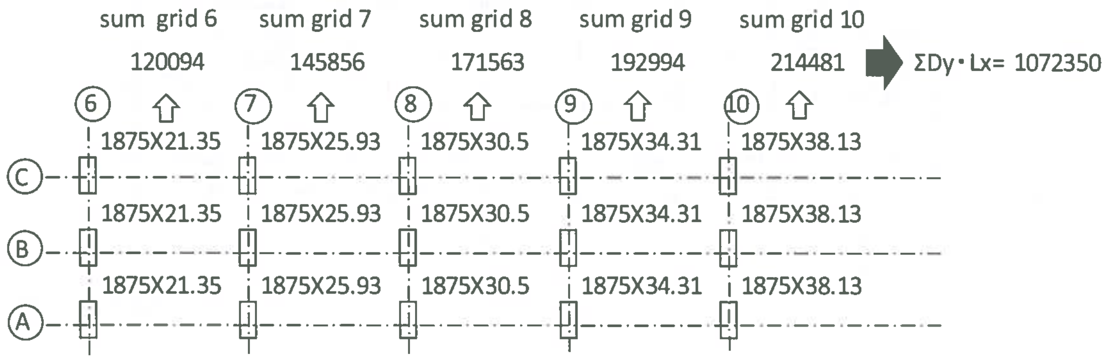
ΣDy= 56250

Multiply stiffness (section area of column) and distance in X and Y direction respectively and then sum them in each X and Y direction separately. Note that when calculating the center of stiffness that affects in X direction, it is to multiply the stiffness in X direction and the distance in Y direction. Vice versa for Y direction.

Effect of Y direction

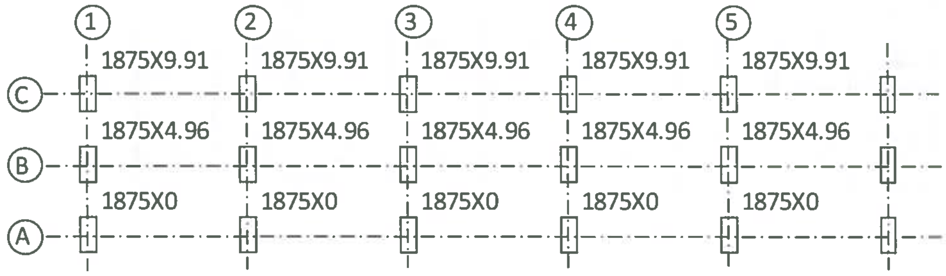
■Dy・Lx

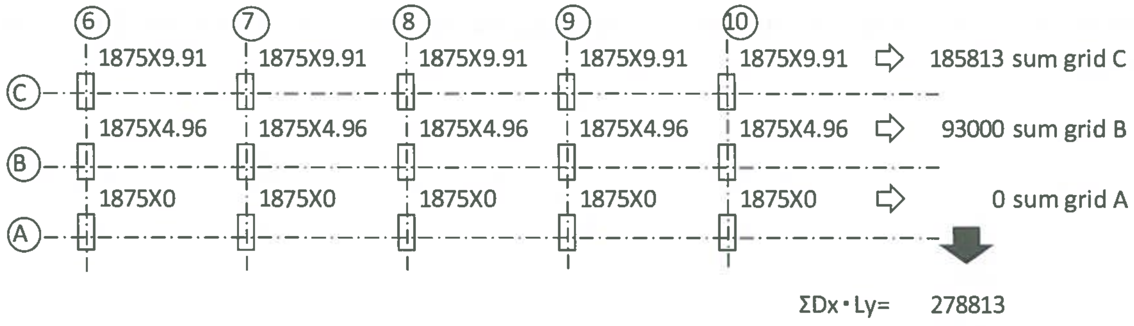




Effect of X direction

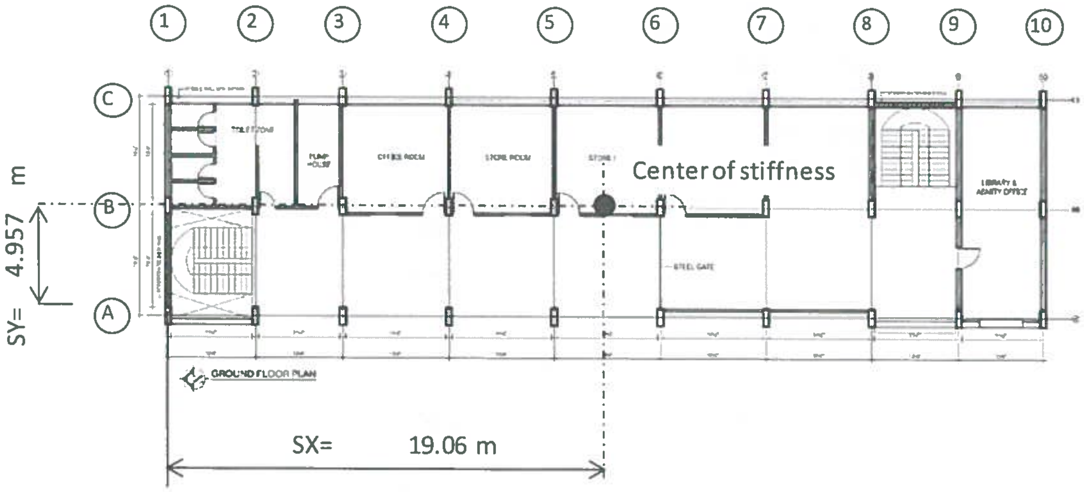
■Dx・Ly





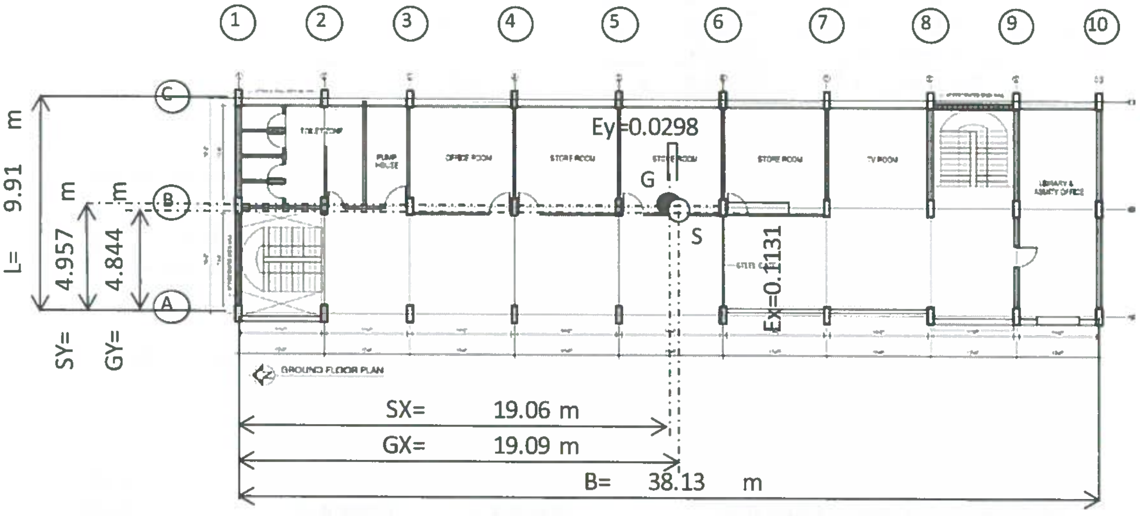
Distance from the origin of each direction to the center of rigidity of a building is obtained by dividing the total value of one direction with the total rigidity of that direction.

SX= ΣDy×Lx/ΣDy= 1072350 / 56250 = 19.06 m
SY= ΣDx×Ly/ΣDx= 278813 / 56250 = 4.957 m



(3) Eccentricity

Eccentric distance in a direction is obtained from the gravity center and the rigidity center of each direction. Eccentricity is evaluated by the ratio of eccentric distance and spring radius that is obtained by the simplified calculation using the shape of the building. Calculation example is as shown below. This method of obtaining the spring radius is useful only for buildings in regular shape.



ℓ: eccentricity
S: Center of stiffness
G: Center of gravity
E: Distance between the center of gravity and the center of rigidity
B: longitudinal direction length
L: transverse direction length

$\ell = E/\sqrt{B^2 + L^2}$

B= 38.13 m
L= 9.91 m
G: the center of gravity
GX= 19.09 m
GY= 4.844 m
S : the center of stiffness
SX= 19.06 m
SY= 4.957 m

X-direction

Ex= |GY-SY|= | 4.844 - 4.957 | = 0.113 m

$l = 0.113/\sqrt{38.13^2 + 9.91^2}$ = 0.0029 ≦1.0

↓

$\boxed{\text{Gn=1.0}}$

Y-direction

Ey= |GX-SX|= | 19.09 - 19.06 | = 0.030 m

$l = 0.030/\sqrt{38.13^2 + 9.91^2}$ = 0.0008 ≦1.0

↓

$\boxed{\text{Gn=1.0}}$

(4) (Stiffness/mass) Ratio of above and below stories

Stiffness balance of upper and lower floors of the building can be evaluated by comparing the stiffness/mass-ratio of upper and lower floors.

Calculation example is as shown below.

(Stiffness/mass)Ratio of above and below stories

	ΣWi(kN)	N	β (N-1)/N
4th Floor 5	2325.4	1	2.000
3rd Floor 4	6311.9	2	0.500
2nd Floor 3	10298.3	3	0.667
1st Floor 2	14284.8	4	0.750
Ground Floor 1	18271.2	5	0.800

N: The number of floors sustained by the story concerned.

n=(the ratio of the stiffness to the weight of the story above) / (the ratio of the stiffness to the weight of the story concerned) x β

β:(N-1)/N

(Stiffness/mass)Ratio =(Stiffness of the story)/(the weght of the story above)

Stiffness of the story =(the sum of column area +the sum of wall area x α)/the story height

Stiffness of the ground floor= 6250/4. = 14062.5
Stiffness of the first floor= 6250/2. = 20833.3

(Stiffness/mass)Ratio of ground floor 14062.5 / 18271.2 = 0.770
(Stiffness/mass)Ratio of first floor 20833.3 / 14284.8 = 1.458

n= 1.458 / 0.770 × 0.800 = 1.516 1.2 < n ≦ 1.7

↓

$\boxed{\text{GI=0.9}}$

(Both direction is same)

[The eccentricity of another method]
Example of the evaluation method of the eccentricity ratio of the building in complex shape.
First, obtain the torsional stiffness in Y and X directions.
Calculation example of torsional stiffness is as shown below.

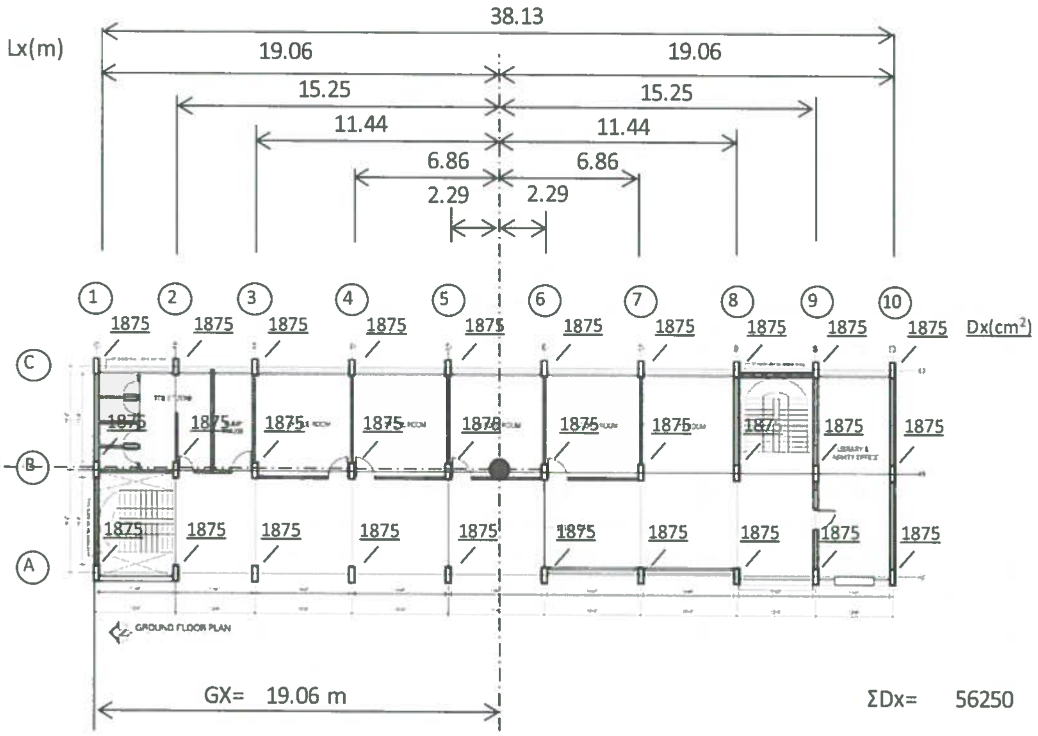
(1)Torsional stiffness

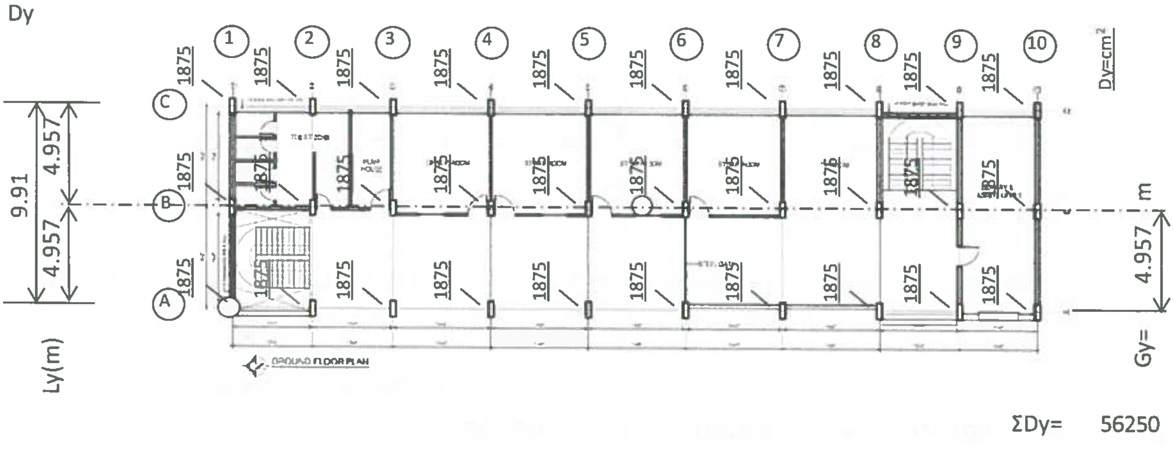
K_T= Ix+Iy

K_T: Torsional stiffness

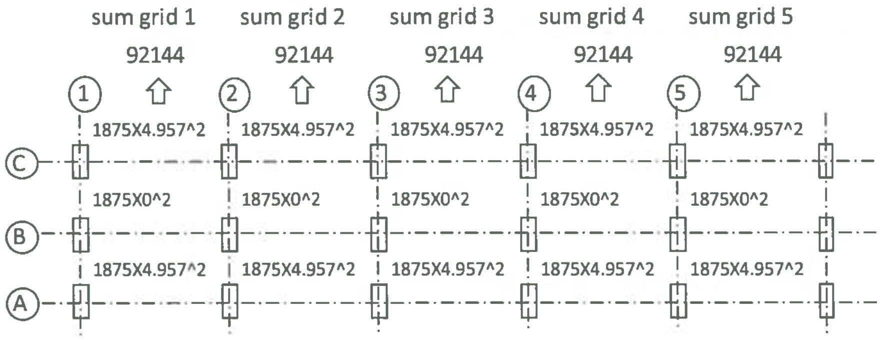
Ix: ΣDx・Ly^2

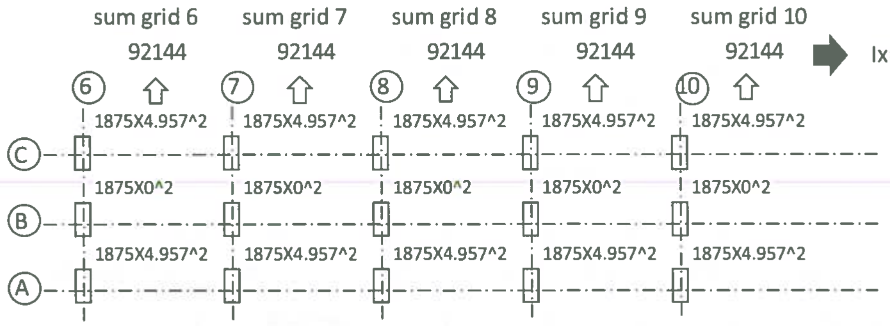
Iy: ΣDy・Lx^2 column size=B(cm)XD(cm)= 25 x 75 = 1875 cm^2





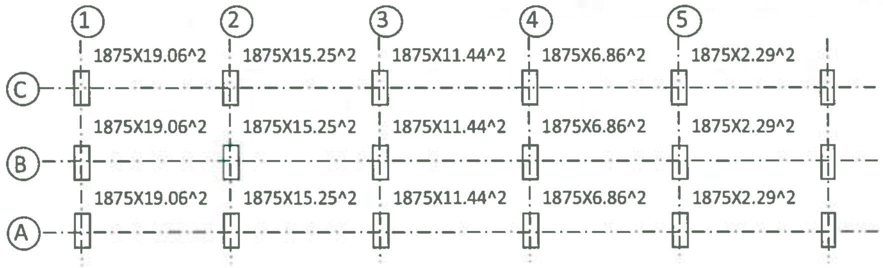
■Dx・Ly²

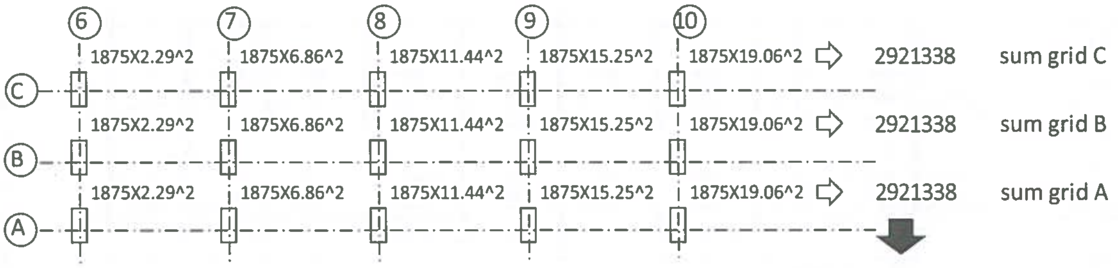




➡ Ix=ΣDx・Ly²= 921444

■Dy・Lx²





Iy=ΣDy・Lx²= 8764013

K_T= Ix+Iy= 921444 + 8764013 = 9685458

(2) Spring radius

Obtain the spring radius to be used for evaluating the earthquake-load of X and Y directions from the torsional stiffness.

$r_{ex} = \sqrt{K_T/\Sigma Dx}$ $r_{ey} = \sqrt{K_T/\Sigma Dy}$

Dx= 56250

Dy= 56250

r_{ex}: Spring radious for X direction

r_{ey}: Spring radious for Y direction

$r_{ex} = \sqrt{9685458/56250}$ = 13.122 m

$r_{ey} = \sqrt{9685458/56250}$ = 13.122 m

(3) Eccentricity

Eccentricity is evaluated by the ratio of eccentric distance and spring radius that obtained. Calculation example is as shown below.

R_{ex}= ex/r_{ex}= 0.113 / 13.122 = 0.0086 ≤0.15

R_{ey}= ey/r_{ey}= 0.030 / 13.122 = 0.0023 ≤0.15

R_{ex}: Eceentricity ratio for Xdirection

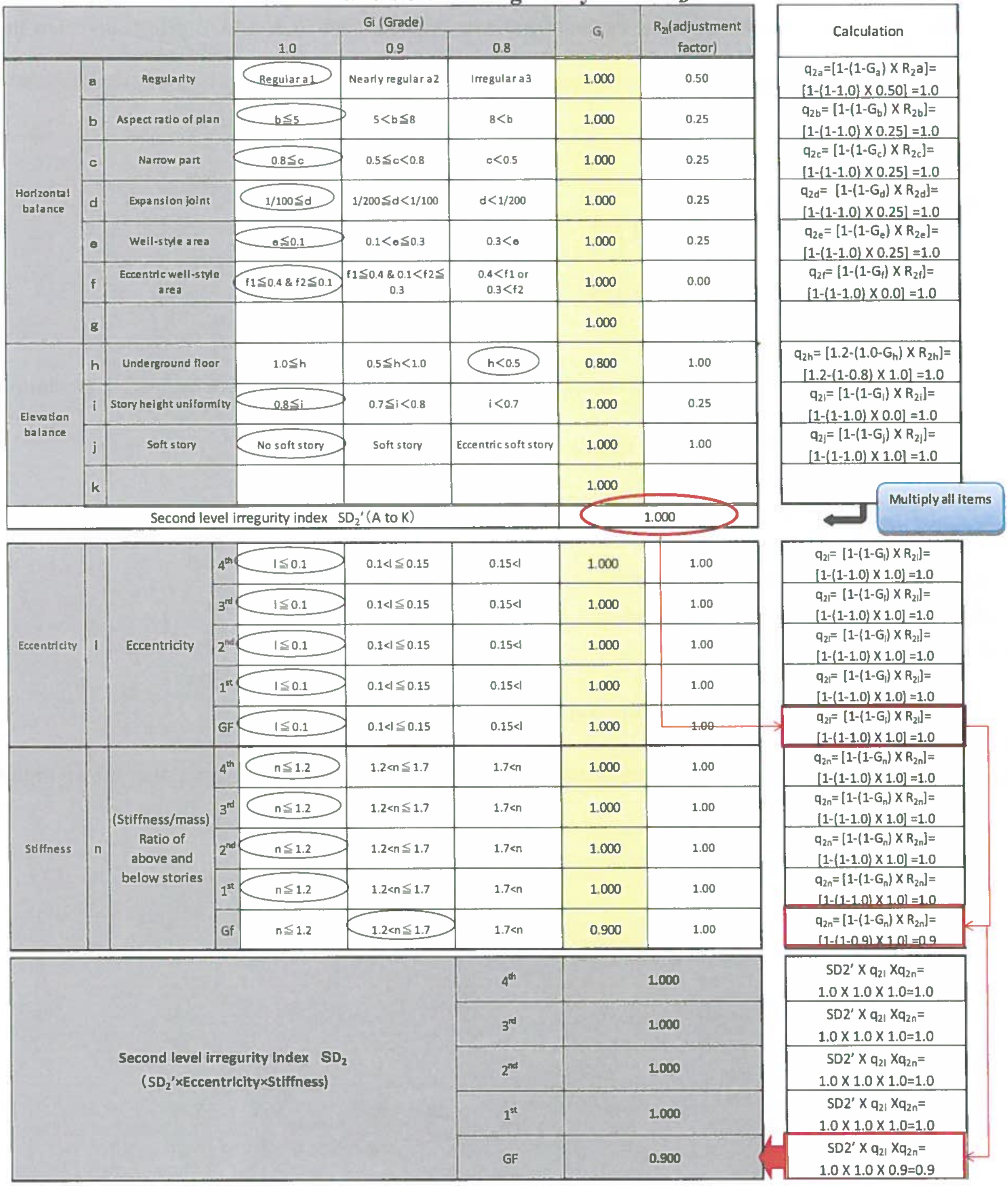
R_{ey}: Eceentricity ratio for Ydirection

Eccentricity i Gi=1.0 for Re≤0.15
Gi=0.8 for Re≥0.30
Gi is set by interporation for 0.15<Re<0.3

(5) Irregularity index S_D

Irregularity index that is obtained using result of eccentricity (item l) and stiffness (item n) is shown in the table 3.5.3.

Table 3.5.3 Irregularity Index S_D



				Gi (Grade)			G_i	R_{2i}(adjustment factor)	Calculation
				1.0	0.9	0.8			
Horizontal balance	a	Regularity		[✓ Regular a1]	Nearly regular a2	Irregular a3	1.000	0.50	q_{2a}=[1-(1-G_a) X R_2a]= [1-(1-1.0) X 0.50] =1.0
	b	Aspect ratio of plan		[✓ b≦5]	5<b≦8	8<b	1.000	0.25	q_{2b}= [1-(1-G_b) X R_{2b}]= [1-(1-1.0) X 0.25] =1.0
	c	Narrow part		[✓ 0.8≦c]	0.5≦c<0.8	c<0.5	1.000	0.25	q_{2c}= [1-(1-G_c) X R_{2c}]= [1-(1-1.0) X 0.25] =1.0
	d	Expansion joint		[✓ 1/100≦d]	1/200≦d<1/100	d<1/200	1.000	0.25	q_{2d}= [1-(1-G_d) X R_{2d}]= [1-(1-1.0) X 0.25] =1.0
	e	Well-style area		[✓ e≦0.1]	0.1<e≦0.3	0.3<e	1.000	0.25	q_{2e}= [1-(1-G_e) X R_{2e}]= [1-(1-1.0) X 0.25] =1.0
	f	Eccentric well-style area		[✓ f1≦0.4 & f2≦0.1]	f1≦0.4 & 0.1<f2≦0.3	0.4<f1 or 0.3<f2	1.000	0.00	q_{2f}= [1-(1-G_f) X R_{2f}]= [1-(1-1.0) X 0.0] =1.0
	g						1.000		
Elevation balance	h	Underground floor		1.0≦h	0.5≦h<1.0	[✓ h<0.5]	0.800	1.00	q_{2h}= [1.2-(1.0-G_h) X R_{2h}]= [1.2-(1-0.8) X 1.0] =1.0
	i	Story height uniformity		[✓ 0.8≦i]	0.7≦i<0.8	i<0.7	1.000	0.25	q_{2i}= [1-(1-G_i) X R_{2i}]= [1-(1-1.0) X 0.0] =1.0
	j	Soft story		[✓ No soft story]	Soft story	Eccentric soft story	1.000	1.00	q_{2j}= [1-(1-G_j) X R_{2j}]= [1-(1-1.0) X 1.0] =1.0
	k						1.000		
Second level irregurity index SD_2'(A to K)							1.000		Multiply all items
Eccentricity	l	Eccentricity	4th	[✓ l≦0.1]	0.1<l≦0.15	0.15<l	1.000	1.00	q_{2l}= [1-(1-G_l) X R_{2l}]= [1-(1-1.0) X 1.0] =1.0
			3rd	[✓ l≦0.1]	0.1<l≦0.15	0.15<l	1.000	1.00	q_{2l}= [1-(1-G_l) X R_{2l}]= [1-(1-1.0) X 1.0] =1.0
			2nd	[✓ l≦0.1]	0.1<l≦0.15	0.15<l	1.000	1.00	q_{2l}= [1-(1-G_l) X R_{2l}]= [1-(1-1.0) X 1.0] =1.0
			1st	[✓ l≦0.1]	0.1<l≦0.15	0.15<l	1.000	1.00	q_{2l}= [1-(1-G_l) X R_{2l}]= [1-(1-1.0) X 1.0] =1.0
			GF	[✓ l≦0.1]	0.1<l≦0.15	0.15<l	1.000	1.00	q_{2l}= [1-(1-G_l) X R_{2l}]= [1-(1-1.0) X 1.0] =1.0
Stiffness	n	(Stiffness/mass) Ratio of above and below stories	4th	[✓ n≦1.2]	1.2<n≦1.7	1.7<n	1.000	1.00	q_{2n}=[1-(1-G_n) X R_{2n}]= [1-(1-1.0) X 1.0] =1.0
			3rd	[✓ n≦1.2]	1.2<n≦1.7	1.7<n	1.000	1.00	q_{2n}=[1-(1-G_n) X R_{2n}]= [1-(1-1.0) X 1.0] =1.0
			2nd	[✓ n≦1.2]	1.2<n≦1.7	1.7<n	1.000	1.00	q_{2n}=[1-(1-G_n) X R_{2n}]= [1-(1-1.0) X 1.0] =1.0
			1st	[✓ n≦1.2]	1.2<n≦1.7	1.7<n	1.000	1.00	q_{2n}=[1-(1-G_n) X R_{2n}]= [1-(1-1.0) X 1.0] =1.0
			Gf	n≦1.2	[✓ 1.2<n≦1.7]	1.7<n	0.900	1.00	q_{2n}=[1-(1-G_n) X R_{2n}]= [1-(1-0.9) X 1.0] =0.9
Second level irregularity index SD_2 (SD_2'×Eccentricity×Stiffness)			4th				1.000		SD2' X q_{2l} Xq_{2n}= 1.0 X 1.0 X 1.0=1.0
			3rd				1.000		SD2' X q_{2l} Xq_{2n}= 1.0 X 1.0 X 1.0=1.0
			2nd				1.000		SD2' X q_{2l} Xq_{2n}= 1.0 X 1.0 X 1.0=1.0
			1st				1.000		SD2' X q_{2l} Xq_{2n}= 1.0 X 1.0 X 1.0=1.0
			GF				0.900		SD2' X q_{2l} Xq_{2n}= 1.0 X 1.0 X 0.9=0.9

[Example] Calculation of S_{D2} at Ground floor is shown below

$$S_{D2}=q_{2a}\times q_{2b}\times q_{2c}\times q_{2d}\times q_{2e}\times q_{2f}\times q_{2i}\times q_{2j}$$

$$=[1-(1-G_a)\times R_{2a}]\times[1-(1-G_b)\times R_{2b}]\times[1-(1-Gc)\times R_{2c}]\times[1-(1-G_d)\times R_{2d}]\times[1-(1-G_e)\times R_{2e}]\times[1-(1-G_f)\times R_{2f}]\times[1-(1-G_i)\times R_{2i}]\times[1-(1-G_j)\times R_{2j}]$$

$$=[1-(1-1.0)\times 0.50]\times[1-(1-1.0)\times 0.25]\times[1-(1-1.0)\times 0.25]\times[1-(1-1.0)\times 0.25]\times[1-(1-1.0)\times 0.25]\times[1-(1-1.0)\times 0.00]\times[1-(1-1.0)\times 1.0]\times[1-(1-0.9)\times 1.0]=\boxed{0.90}$$

Note: The I_{SO} value has been calculated with incorporating above Irregularity Index S_D of every floor in section 4.3.1.

Criteria of item a to item *k* in J. Standard are show below.

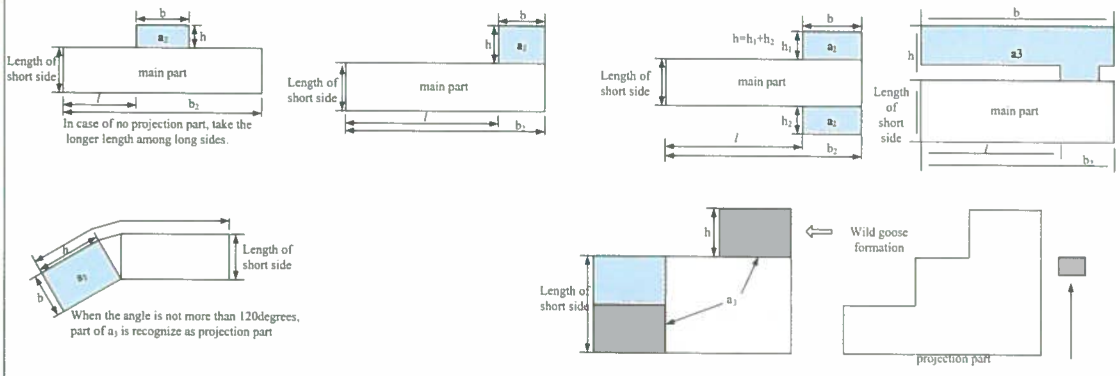
Remarks

a_1: Structural balance is good, and the area of a projection part is not more than 10% of the floor area.

a_2: Structural balance is worse than a_1, or the area of a projection part is not more than 30% of the floor area with L, T or U shaped plan.

a_3: Structural balance is worse than a_2, or the area of a projection part is larger than 30% of the floor area with L, T or U shaped plan.

If the aspect ratio (h/b) of the projection part is less than 1/2, it may not be accounted in this item. The projection part should be defined as the smaller part, while the larger rest as the main part.

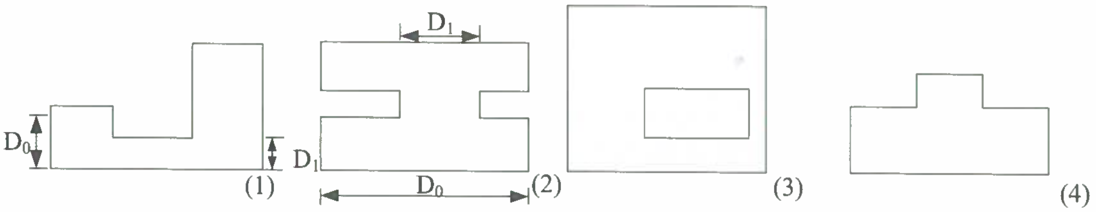


In case of no projection part, take the longer length among long sides.

When the angle is not more than 120degrees, part of a_3 is recognize as projection part

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

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



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

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

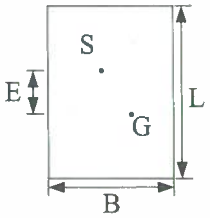
f: f_1= (the distance between the center of the floor area and the center of the well-style area / the length of the short side of the building) = r/y,

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

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

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

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



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

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

3.6 TIME INDEX *T*

3.6.1 General

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

3.6.2 Recommendation for Application

It is recommended to use Time Index *T* for the First level screening procedure. Regarding the age of the building, seismic evaluation requires the material inspection in advance. Evaluation shall include the status of main reinforcements of members and carbonation of concrete.

Many buildings have stone floors and mortar and plaster columns/ beams. These make it difficult to know the cracks just by observing at the surface. On the other hand, we see many cases of the spilling of the cover concrete, one of the deterioration, which is caused by the corrosion of reinforcement. Evaluation of deterioration is therefore difficult even when the results of detailed inspection (Table8) are available. Since the maintenance actions are rarely performed, first level screening procedure is recommended as an evaluation on the safety side. Age of the building shall be found out by the carbonation inspection of concrete and striping inspection of the main reinforcement. And the results of these inspections shall be reflected to the ageing evaluation.

(1) First, Second & Third Level Screening Procedure

Table 7 shown in the J. Standard shall be replaced with Table 3.6.1 in the application manual when applied to an evaluation.

The rust on reinforcing bars at the striping site to be checked visually. It is recommended to grade the corrosion on the surface and deduce some points according to the grade.

Concretes are graded depending on the carbonation depth. Carbonation itself does not affect the strength or the ductility. Problem is the loss of suppression effect on corrosion caused by carbonation. Therefore, it is not regarded as an issue as long the reinforcing bars are not corroded even if the carbonation is progressed further than the location of reinforcing bars. But a couple of investigations are not enough to prove that the reinforcing bars are corrosion free. So it is recommended to deduct certain points in evaluation to be on the safe side when the carbonation is found. In case of advanced carbonation, it is highly recommended to take measure against corrosion.

Occupation (chemical has been used, etc.) has excluded from the evaluation item since it is evaluated in other items in the suit inspection and the material inspection. Table 3.6.2 lists the evaluation items.

Table 3.6.1 Time Index T

Item to be checked		Degree	T value (Check circle at relevant degree)
I.Deflection		Tilting of a building or obvious uneven settlement is observed	0.70
		Landfill site of former rice field	0.90
		Deflection of beam or column is observed visually	0.90
		No correspondence to the foregoing	1.00
II.Cracking in walls and columns		Rain leak with rust of reinforcing bar is observed	0.80
		Inclined cracking in columns is obviously observed	0.90
		Countless cracking is observed in external wall	0.90
		Rain leak without rust of reinforcing bar is observed	0.90
		No correspondence to the foregoing	1.00
III.Fire experience		Trace	0.70
		Experience but traceless	0.80
		No experience	1.00
IV.Deterioration	IV-i.Rust of reinforceing bar	Grade iii or more	0.90
		Grade iior more	0.95
		Grade i	1.00
	IV-ii.Depth of carbonation	Over the thickness of concrete(Grade ii)	0.90
		Same or less the thickness of cover concrete(Grade i)	0.95
		No correspondence to the foregoing	1.00
V.Finishing condition		Significant spalling of external finishing due to aging is observed	0.90
		Significant spalling and deterioration of internal finishing is observed	0.90
		No problem	1.00

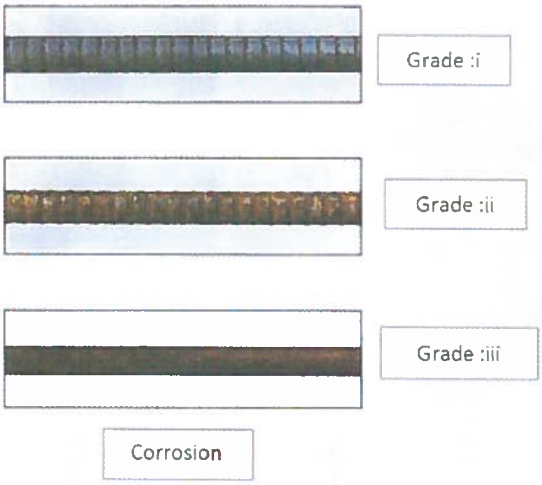


Figure 3.6.1 Example of Deterioration of Reinforcement-Bar

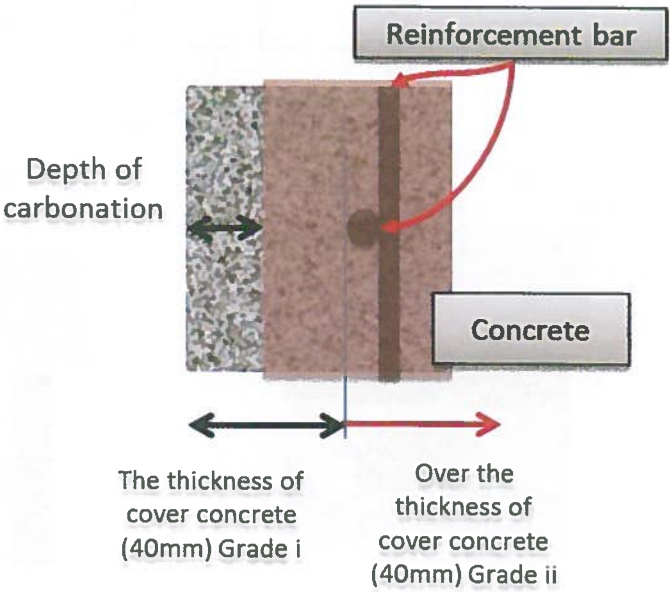


Figure 3.6.2 Example for Carbonation of Concrete

Second and third level screening procedures in 3.4.3-3.4.4 is not explained here since the first level screening procedure is recommended.

[Example]

Calculation example of the Time-index for the sample building is shown below.

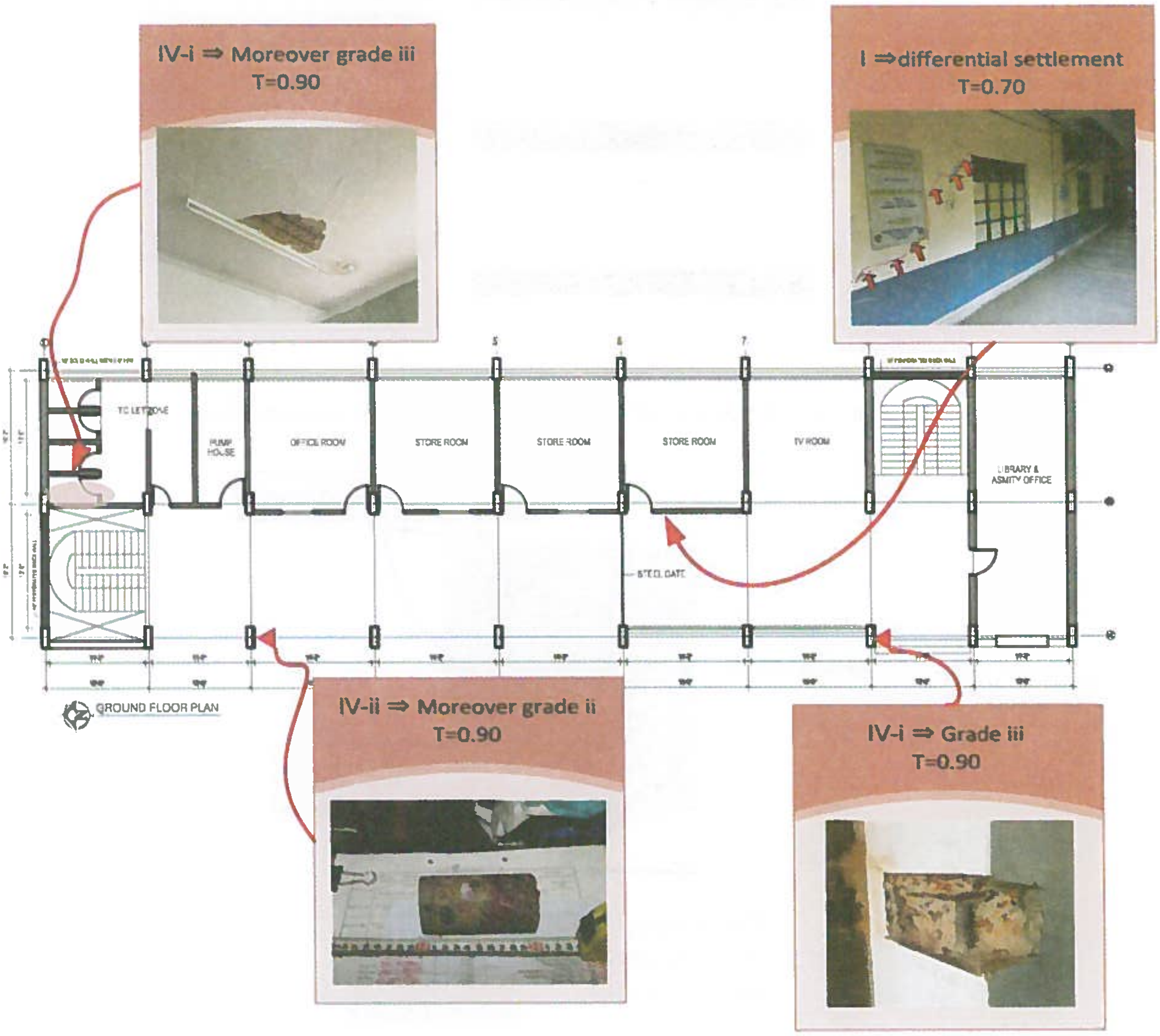


Figure 3.6.3 Example of Site Inspection Result

Table 3.6.2 Example for Evaluating Time Index *T*

Item to be checked		Degree	T value (Check circle at relevant degree)	check ○ at relevant degree
I.Deflection		Tilting of a building or obvious uneven settlement is observed	0.70	○
		Landfill site of former rice field	0.90	
		Deflection of beam or column is observed visually	0.90	
		No correspondence to the foregoing	1.00	
II.Cracking in walls and columns		Rain leak with rust of reinforcing bar is observed	0.80	
		Inclined cracking in columns is obviously observed	0.90	
		Countless cracking is observed in external wall	0.90	
		Rain leak without rust of reinforcing bar is observed	0.90	
		No correspondence to the foregoing	1.00	○
III.Fire experience		Trace	0.70	
		Experience but traceless	0.80	
		No experience	1.00	○
IV.Deterioration	IV-i.Rust of reinforceing bar	Grade iii or more	0.90	○
		Grade iior more	0.95	
		Grade i	1.00	
	IV-ii.Depth of carbonation	Over the thickness of concrete(Grade ii)	0.90	○
		Same or less the thickness of cover concrete(Grade i)	0.95	
		No correspondence to the foregoing	1.00	
V.Finishing condition		Significant spalling of external finishing due to aging is observed	0.90	
		Significant spalling and deterioration of internal finishing is observed	0.90	
		No problem	1.00	○

Time index T by the first level inspection T_1	0.7

Note: The *Iso* value has been calculated with incorporating above Time Index *T* in section 4.3.1.

CHAPTER 4. JUDGMENT ON SEISMIC SAFETY

4.1 BASIC PRINCIPLES

Seismic safety of a building shall be judged by detail and extensive assessment based on the seismic evaluations conducted on the structural as well as the non-structural elements. Seismic safety of structure shall be judged by the following Equation

$I_s \geq I_{so}$ Equation (37) of J. Standard

Where:

I_s = Seismic index of structure

I_{so} = Seismic demand index of structure

If Equation stated above is satisfied, the building may be assessed to be 'Safe'. Otherwise, the building should be assessed to be 'Uncertain' in seismic safety.

> The Japanese standard suggests to evaluate the seismic index of non-structural elements I_N along with the seismic index of structure I_S. However, it covers only the main structural parts of a building in this manual. The seismic capacity of non-structural elements may be checked with provisions of BNBC 2015 (Part 6 sec2.5.18)

4.2 RECOMMENDATION FOR APPLICATION

> Seismic Demand Index must be set to ensure the equivalent performance as the seismic capacity required by BNBC 2015 (Bangladesh National Building Code). It is highly recommended that the Cumulative strength index at the ultimate deformation of structure shall be set to ensure the strength of minimum requirement required by BNBC 2015.

Seismic Demand Index has been set based on the response result of the Artificial Earthquake Waves that was created based on the Normalized acceleration response spectrum used for the calculation of seismic design load in BNBC 2015. (Refer to Supplement1 Proposed Seismic Demand Index "I_{S0}").

Cumulative strength index is measured at the ultimate deformation of structure with ductile frame. It is highly recommended to ensure the minimum strength required for the ductility type of building by BNBC 2015.

4.2.1 Relationship between BNBC 2015 and I_{so}

According to BNBC 2015, Design base shear, $V = S_a \cdot W$

Where:

S_a : Design spectral acceleration (in units of g) /Lateral seismic force coefficient

$$S_a = \frac{2}{3} \times \frac{Z \times I}{R} \times C_S$$

Where:

Z : Seismic zone coefficient

I : Structure importance factor

R : Response reduction factor which depends on the type of structural system.

C_s : Normalized acceleration response spectrum, which is a function of structure (building) period and soil type (site class)

So, $V = \frac{2}{3} \times \frac{Z \times I}{R} \times C_S \times W$

The response reduction factor or above equation can be subdivided into ductility reduction factor R_d and over-strength factor, Ω.

i.e. $R = R_d \times \Omega$

The ductility reduction factor, R_d is the ratio between elastic force capacity and force at structural yield level. According to Japanese standard this R_d is expressed as ductility index *F*. The other parameter over-strength factor Ω include design over-strength, material over-strength and system over-strength.

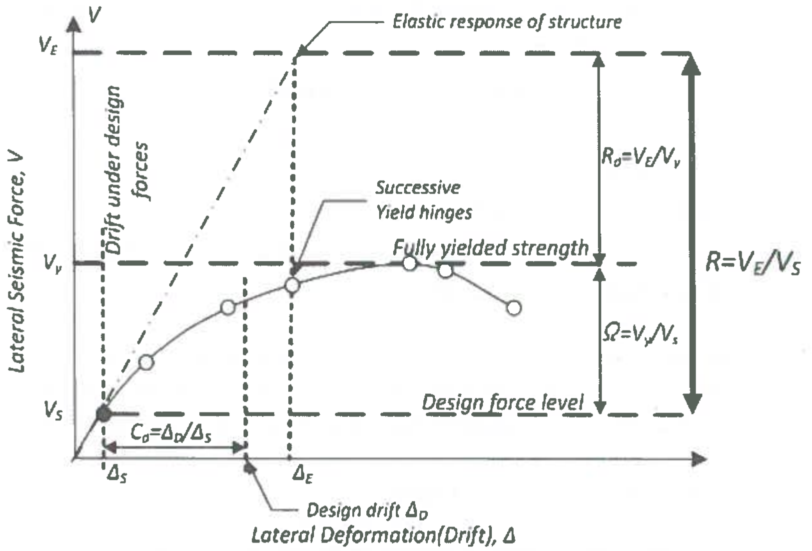


Figure 4.2.1 Inelastic Force-Deformation Curve

*NEHRP Recommended Provisions (FEMA P-750) 2009 Edition

Now, the equation for base shear can be rewritten.

$$V = \frac{2}{3} \times \frac{Z \times I}{R_d \times \Omega} \times C_S \times W$$

$$\frac{V}{W} \times R_d \times \Omega = \frac{2}{3} \times Z \times I \times C_S$$

$V \times \Omega$ is the yield strength of the structure. $V \times \frac{\Omega}{W}$ is expected as strength index *C* according to Japanese standard of seismic evaluation.

Above equation can be expressed as $C \times F = \frac{2}{3} \times Z \times I \times C_S$

In the above equation left side provides an index about capacity of a structure based on its strength and ductility. The right side of the equation provides an index about seismic demand considering local seismicity, sub-soil condition, and structural period. The capacity of the structure shall be equal or more than the demand.

So, $C \times F \geq \frac{2}{3} \times Z \times I \times C_S$

According to Japanese standard, seismic capacity index is noted as I_s and seismic demand index is noted as I_{so}.

Therefore, $I_s \geq I_{so}$

Where, $I_s = C \times F$

and $I_{SO} = \frac{2}{3} \times Z \times I \times C_S$

Considering nonlinear behavior due to crack occurrence, $I_{SO} = 0.8 \times \frac{2}{3} \times Z \times I \times C_S$

4.3 SEISMIC DEMAND INDEX I_{S0}

(1) The seismic demand index of structure I_{S0} should be calculated by Equation (4.2.1) The 2^{nd} level screening method is applied.

$$I_{SO} = 0.8 \times \frac{2}{3} \times Z \times I \times C_S \qquad (4.2.1)$$

Where:

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

I: Structure importance factor, as defined in Section 6.2.35a to 6.2.35d 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 2.5.5a-d of BNBC 2015

(2) In case the seismic safety of a structure is judged by Eq. (4.2.1) in the second level screening procedure and assessed to be "Safe," Eq. (4.2.2) shall also be satisfied.

$$C_{TU} \cdot S_D \geq 0.4 \times \frac{2}{3} \times Z \times I \times C_S \qquad (4.2.2)$$

where:

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

S_D : Irregularity Index (Refer to 3.3)

4.3.1 Seismic Demand Index of Structure I_{S0}

Seismic Demand index must be set to meet the seismic capacity that is required by the current Building code. However, the current standard has a precondition, which is to secure the sufficient deformation capacity. All the targeted buildings in this standard are considered known not to have the sufficient deformation capacity. For that reason, Artificial Earthquake Waves based on the design acceleration spectrum in BNBC 2015 have been simulated and the Seismic Demand Index is set based on the response results due to earthquake waves. Seismic Demand Index I_{S0} in different seismic zones is shown below.

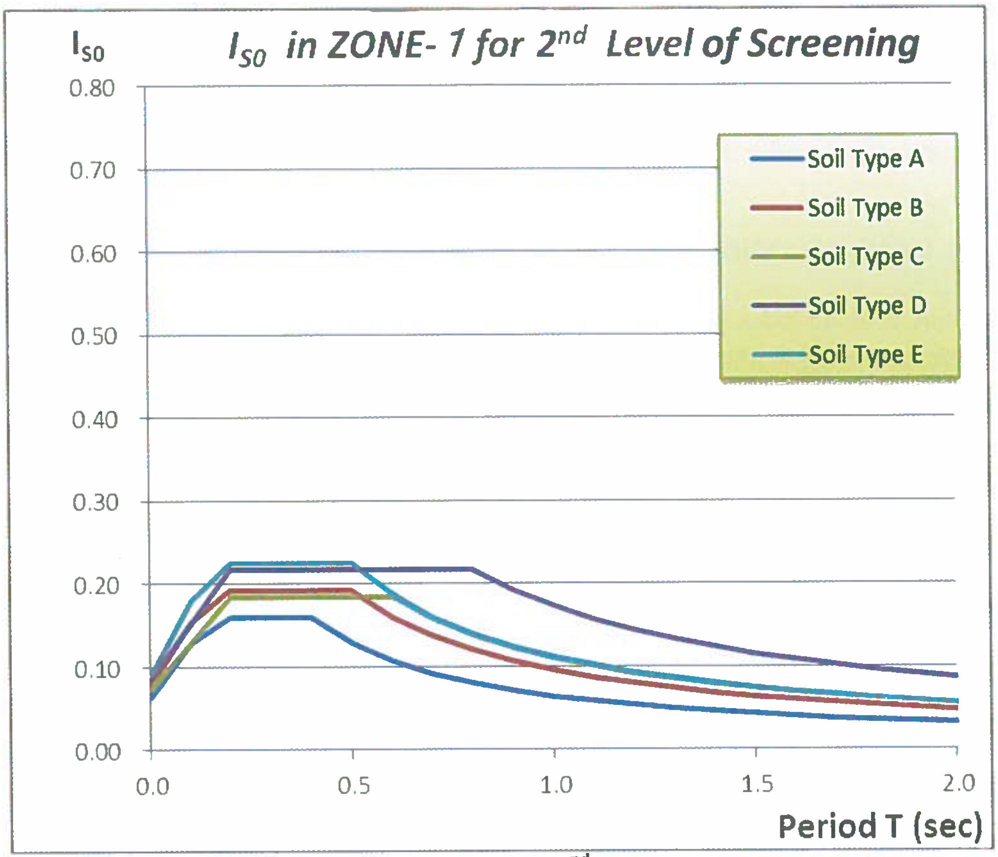


Figure 4.3.1I_{S0} in ZONE-1 for 2nd Level of Screening

Table 4.3.1 Quick-Reference of I_{S0} in ZONE-1 Relationship with Building Height

Building Height(m)	Soil Type				
	A	B	C	D	E
30.0	0.06	0.10	0.11	0.17	0.11
27.0	0.07	0.11	0.12	0.19	0.12
24.0	0.08	0.12	0.14	0.21	0.14
21.0	0.09	0.13	0.15	0.22	0.16
18.0	0.10	0.15	0.18	0.22	0.18
15.0	0.12	0.18	0.18	0.22	0.21
12.0	0.15	0.19	0.18	0.22	0.22
9.0	0.16	0.19	0.18	0.22	0.22
6.0	0.16	0.19	0.18	0.22	0.22
3.0	0.14	0.17	0.14	0.17	0.20

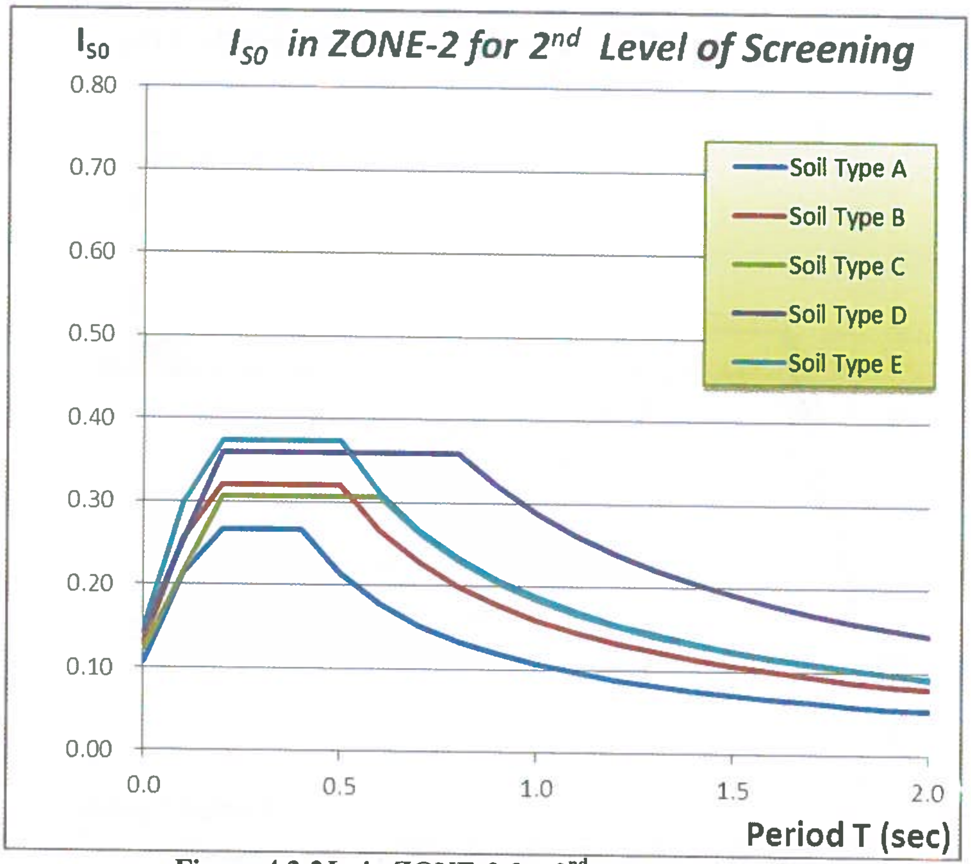


Figure 4.3.2I_{S0} in ZONE-2 for 2ndLevel of Screening

Table 4.3.2 Quick-Reference of I_{S0} in ZONE-2 Relationship with Building Height

Building Height(m)	Soil Type				
	A	B	C	D	E
30.0	0.11	0.16	0.18	0.29	0.19
27.0	0.12	0.18	0.20	0.32	0.21
24.0	0.13	0.20	0.23	0.35	0.23
21.0	0.15	0.22	0.25	0.36	0.26
18.0	0.17	0.25	0.29	0.36	0.30
15.0	0.20	0.30	0.31	0.36	0.35
12.0	0.24	0.32	0.31	0.36	0.37
9.0	0.27	0.32	0.31	0.36	0.37
6.0	0.27	0.32	0.31	0.36	0.37
3.0	0.24	0.29	0.24	0.28	0.34

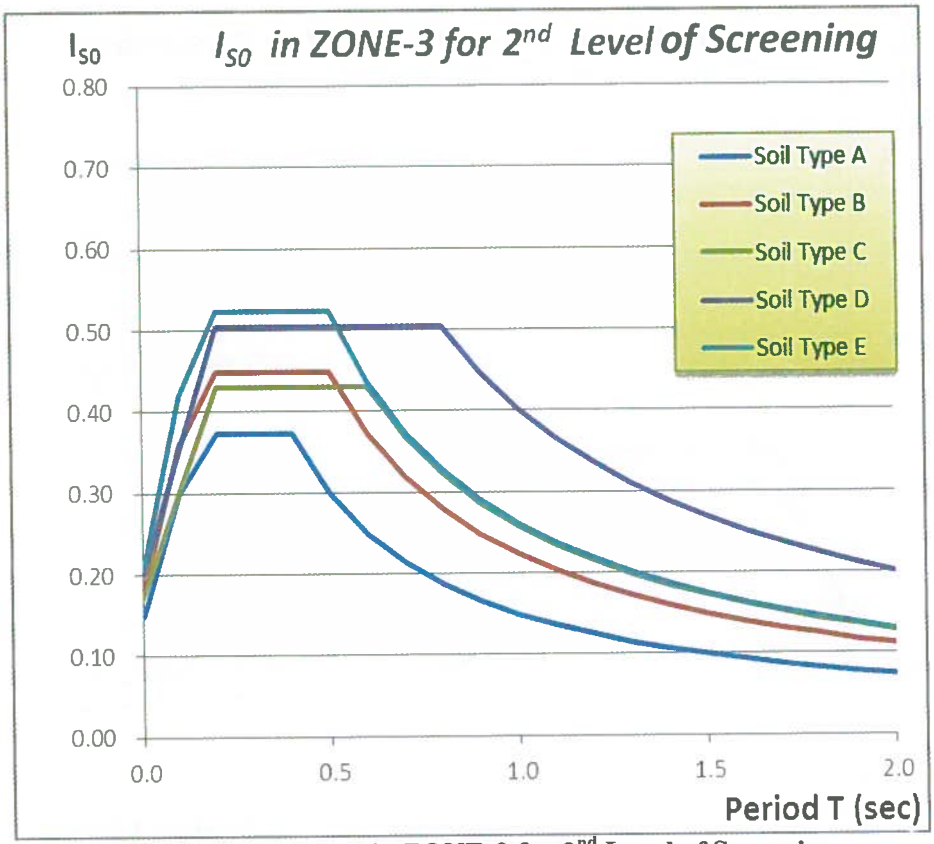


Figure 4.3.3 I_{S0} in ZONE-3 for 2nd Level of Screening

Table 4.3.3 Quick-Reference of I_{SO} in ZONE-3 Relationship with Building Height

Building Height(m)	Soil Type				
	A	B	C	D	E
30.0	0.15	0.23	0.26	0.41	0.26
27.0	0.17	0.25	0.28	0.45	0.29
24.0	0.18	0.28	0.32	0.50	0.32
21.0	0.21	0.31	0.36	0.50	0.36
18.0	0.24	0.36	0.41	0.50	0.42
15.0	0.28	0.42	0.43	0.50	0.49
12.0	0.34	0.45	0.43	0.50	0.52
9.0	0.37	0.45	0.43	0.50	0.52
6.0	0.37	0.45	0.43	0.50	0.52
3.0	0.34	0.40	0.33	0.39	0.47

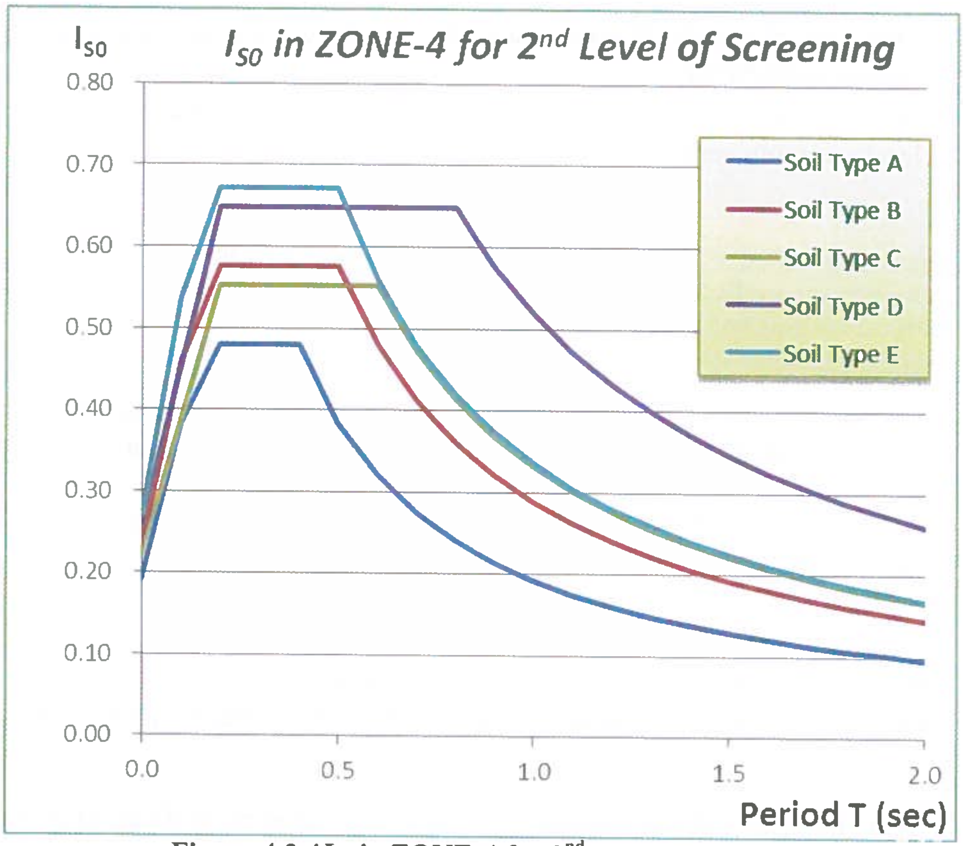


Figure 4.3.4 I_{S0} in ZONE-4 for 2nd Level of Screening

Table 4.3.4 Quick-Reference of I_{SO} in ZONE-4 Relationship with Building Height

Building Height(m)	Soil Type				
	A	B	C	D	E
30.0	0.19	0.29	0.33	0.52	0.34
27.0	0.21	0.32	0.37	0.57	0.37
24.0	0.24	0.35	0.41	0.64	0.41
21.0	0.27	0.40	0.46	0.65	0.47
18.0	0.31	0.46	0.53	0.65	0.53
15.0	0.36	0.54	0.55	0.65	0.63
12.0	0.44	0.58	0.55	0.65	0.67
9.0	0.48	0.58	0.55	0.65	0.67
6.0	0.48	0.58	0.55	0.65	0.67
3.0	0.43	0.52	0.43	0.50	0.61

Similarly here is the case of seismic demand index, I_{S0} in different seismic zone applying 2nd level screening method based on BNBC93.

$$I_{so} = 0.8 \cdot Z \cdot C \cdot I \qquad (4.2.4)$$

In case of the 1st level screening method is applied.

$$I_{so} = Z \cdot C \cdot I \qquad (4.2.5)$$

Where:

Z : Seismic zone coefficient, as defined in Section 2.5.6.1 of BNBC93

I : Structure importance factor, as defined in Section 2.5.6.1 of BNBC93

C : Numerical coefficient given by the relation,

$$C = 1.25 \cdot S / T^{\frac{2}{3}}$$

S :Site coefficient for soil characteristics, as defined in Section 2.5.6.1 of BNBC93

T : Fundamental period of vibration in seconds, the value of C need not exceed 2.75, as defined in Section 2.5.6.2 of BNBC93

In case the seismic safety of a structure is judged by Eq. (4.2.4) in the second and assessed to be "Safe," Eq. (4.2.6) shall also be satisfied.

$$C_{TU} \cdot S_D \geq 0.4 \cdot Z \cdot C \cdot I \qquad (4.2.6)$$

Where:

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

S_D : Number of the storey for evaluation, where the first storey is numbered as *1* and the top storey as *n*.

The proposed Seismic Demand Index of Structure I_{S0} to meet the condition of BNBC93 is derived using same concept of that of BNBC 2015.

Seismic Demand Index I_{S0} in different seismic zones based on BNBC93 are shown below.

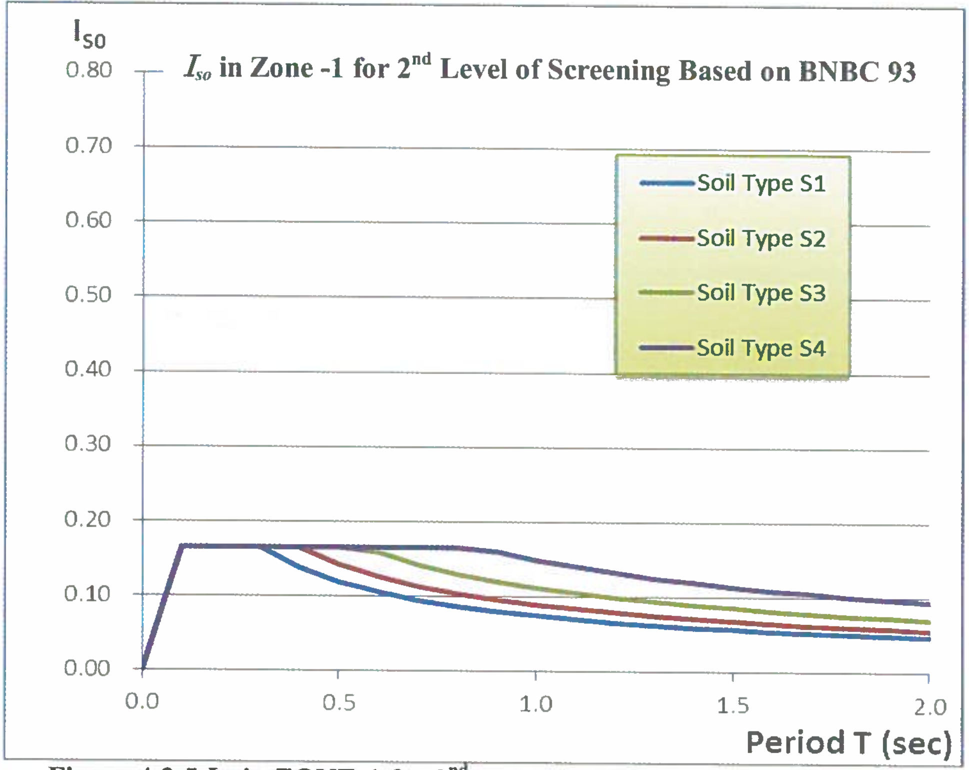


Figure 4.3.5 I_{S0} in ZONE-1 for 2^{nd} Level of Screening Based on BNBC93

Table 4.3.5 Quick-Reference of I_{S0} in ZONE-1 Based on BNBC93 Relationship with Building Height

Building Height(m)	Soil Type			
	S1	S2	S3	S4
30.0	0.08	0.09	0.12	0.16
27.0	0.08	0.10	0.12	0.17
24.0	0.09	0.11	0.13	0.17
21.0	0.09	0.11	0.14	0.17
18.0	0.10	0.12	0.15	0.17
15.0	0.11	0.13	0.17	0.17
12.0	0.12	0.15	0.17	0.17
9.0	0.14	0.17	0.17	0.17
6.0	0.17	0.17	0.17	0.17
3.0	0.17	0.17	0.17	0.17

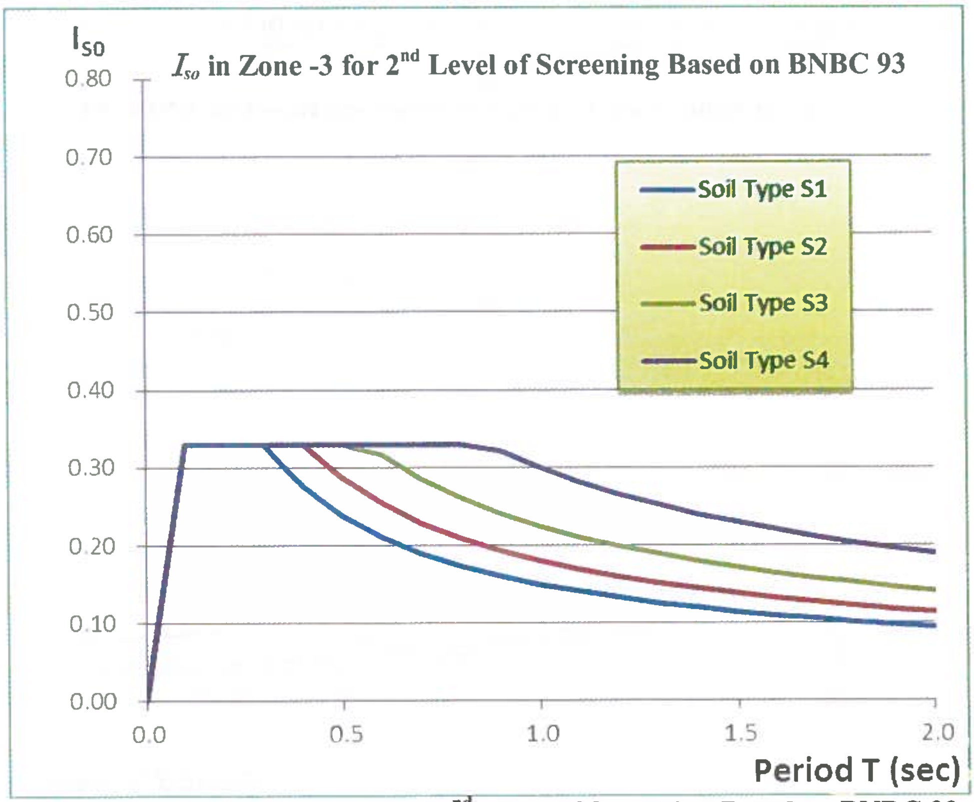


Figure 4.3.6 I_{SO} in ZONE-2 for 2nd Level of Screening Based on BNBC 93

Table 4.3.6 Quick-Reference of I_{SO} in ZONE-2 Based on BNBC 93 Relationship with Building Height

Building Height(m)	Soil Type			
	S1	S2	S3	S4
30.0	0.16	0.19	0.24	0.31
27.0	0.17	0.20	0.25	0.33
24.0	0.18	0.21	0.26	0.33
21.0	0.19	0.22	0.28	0.33
18.0	0.20	0.24	0.30	0.33
15.0	0.22	0.27	0.33	0.33
12.0	0.25	0.30	0.33	0.33
9.0	0.29	0.33	0.33	0.33
6.0	0.33	0.33	0.33	0.33
3.0	0.33	0.33	0.33	0.33

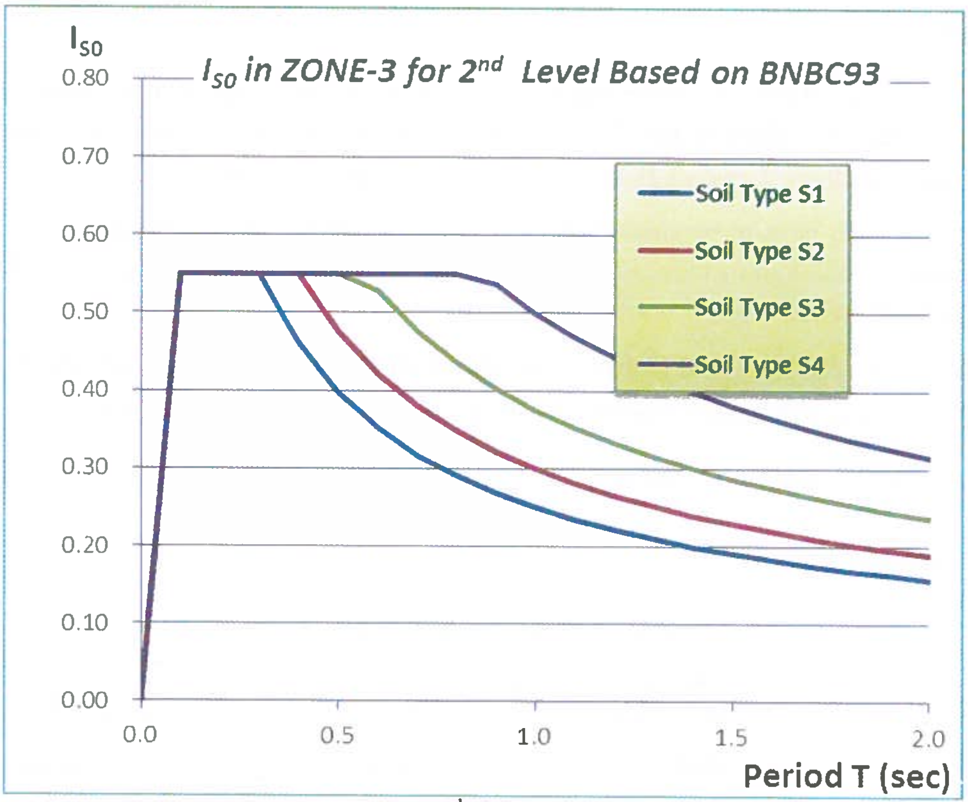


Figure 4.3.7 I_{S0} in ZONE-3 for 2^{nd} Level of Screening Based on BNBC 93

Table 4.3.7 Quick-Reference of I_{S0} in ZONE-3 Based on BNBC 93 Relationship with Building Height

Building Height(m)	Soil Type S1	S2	S3	S4
30.0	0.26	0.31	0.39	0.52
27.0	0.28	0.33	0.41	0.55
24.0	0.29	0.35	0.44	0.55
21.0	0.31	0.37	0.47	0.55
18.0	0.34	0.40	0.51	0.55
15.0	0.37	0.44	0.55	0.55
12.0	0.41	0.50	0.55	0.55
9.0	0.48	0.55	0.55	0.55
6.0	0.55	0.55	0.55	0.55
3.0	0.55	0.55	0.55	0.55

4.3.2 Example

Calculate to obtain a combination, which makes E_0 its maximum, by using Ductility-dominant equation and calculate to obtain E_0, which makes E_0 and E_0 the maximum when F value is its minimum, by using Strength-dominant equation. I_s of each floor obtained by calculation is shown below.

Basic seismic index, E_o have to be calculated based on both ductility dominant equation and Strength dominant equation. Maximum E_0 thus obtained out of two equations is the basic seismic index. This E_o have to be calculated for each storey and each direction.

Sample calculation of E_o , $C_T S_D$ and I_{so} have been shown in Table 4.3.8. In this example E_o have been calculated following Eq. 4 (blue cells) and Eq. 5 (white cells). For strength dominant equation cumulative $C\ (C_i + \sum_j \alpha_j C_j)$ is provided for the specific F value. But for ductility dominant equation individual C is provided for corresponding F value.

Table 4.3.8 Example of I_s Result

Results of the secondLevel screening												
Name of build		**Garrage Building**				Construction year			1	**1885**		2
Address												
Evaluated Engineer		**PWD**				Data ofevaluation				**23-Jun-12**		
Seismic demand index							Iso=	0.30		$C_{TU}\cdot S_D$=	0.15	
Direction	Story	C	F	Failure Mode	Eo	T	S_D	Is	$C_{TU}\cdot S_D$	Result	Adoption	Eq
X	4					0.700	1.000					5
		0.750	1.50		0.67			0.47	0.45	OK	●	
	3					0.700	1.000					5
		0.320	1.50		0.33			0.23	0.22	NG	●	
												4
	2					0.700	1.000					5
		0.203	1.18		0.18			0.13	0.15	NG	●	
		0.180	1.50		0.20			0.14	0.14	NG		
		0.040	1.18	3	0.21			0.15	0.14	NG	4	4
		0.180	1.50									
	1			4		0.700	1.000					5
		0.139	1.00		0.12			0.08	0.12	NG	●	
		0.110	1.50		0.14			0.10	0.09	NG		
		0.061	1.00		0.15			0.10	0.09	NG		4
		0.110	1.50									
	G	0.070	1.00	5	0.07	0.700	0.900	0.04	0.06		●	5
		0.048	1.13		0.06			0.04	0.03	NG		
		0.034	1.40		0.05			0.03	0.03	NG		
		0.030	1.00		0.06			0.04	0.03	NG		4
		0.020	1.13									
		0.034	1.40									

$\boxed{1}$	$I_{S0} = 0.8 \cdot {}^{2}/_{3} . Z \cdot I \cdot C_s$ $Z = 0.20, I = 1.0$ Soil $SC, h_n = 15.24m$ $T = 0.0446 \cdot (h_n)^0.9 = 0.546s$ $C_s = 2.88$ $I_{S0} = 0.8 \cdot \frac{2}{3} \cdot 0.2 \cdot 1.0 \cdot 2.88 = 0.30$
$\boxed{2}$	$C_{TU} \cdot S_D \geq 0.4 \cdot {}^{2}/_{3} . Z \cdot I \cdot C_s$ $C_{TU} \cdot S_D \geq 0.4 \cdot {}^{2}/_{3} . 0.2 \cdot 1.0 \cdot 2.88 = 0.15$
$\boxed{3}$	$E_0 = \frac{n+1}{n+i}\left(C_i + \sum_j \alpha_j C_j\right) \cdot F_1$ $E_0 = \frac{5+1}{5+2}(0.06 + 0.72 \cdot 0.11) \cdot 1.00 = 0.857 \cdot 0.139 \cdot 1.00 = 0.119$ $I_S = E_0 \cdot S_D \cdot T = 0.119 \cdot 1.0 \cdot 0.70 = 0.08$ $C_{TU} \cdot S_D = C \cdot \frac{n+1}{n+i} \cdot S_D = 0.139 \cdot 0.857 \cdot 1.00 = 0.12$
$\boxed{4}$	$E_0 = \frac{n+1}{n+i}\left(C_i + \sum_j \alpha_j C_j\right) \cdot F_1$ $E_0 = \frac{5+1}{5+2}(0.11 + 0) \cdot 1.50 = 0.857 \cdot 0.110 \cdot 1.50 = 0.14$ $I_S = E_0 \cdot S_D \cdot T = 0.14 \cdot 1.0 \cdot 0.70 = 0.10$ $C_{TU} \cdot S_D = C \cdot \frac{n+1}{n+i} \cdot S_D = 0.11 \cdot 0.857 \cdot 1.0 = 0.09$
$\boxed{5}$	$E_0 = \frac{n+1}{n+i}\sqrt{E_1{}^2 + E_2{}^2 + E_3{}^2}$ $E_1 = C_1 \times F_1 = 0.061 \times 1.00 = 0.061$ $E_2 = C_2 \times F_2 = 0.110 \times 1.50 = 0.165$ $E_0 = \frac{5+1}{5+2}\sqrt{0.061^2 + 0.165^2} = 0.857 \times 0.175 = 0.153$ $I_S = E_0 \cdot S_D \cdot T = 0.153 \cdot 1.00 \cdot 0.70 = 0.10$ $C_{TU} \cdot S_D = C \cdot \frac{n+1}{n+i} \cdot S_D = 0.110 \cdot 0.857 \cdot 1.00 = 0.09$

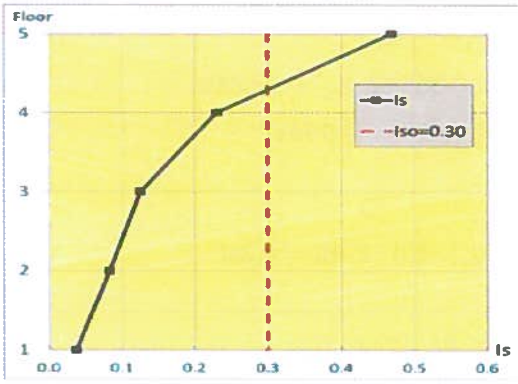


Figure 4.3.8 Example of I_s Result Graph

4.3.3 Seismic Zone Coefficient *Z*

Seismic zone coefficient *Z* shall be evaluated according to BNBC 2015 (Refer to Section 2.5.4.2 BNBC 2015). Seismic Zoning Map of Bangladesh is shown below.

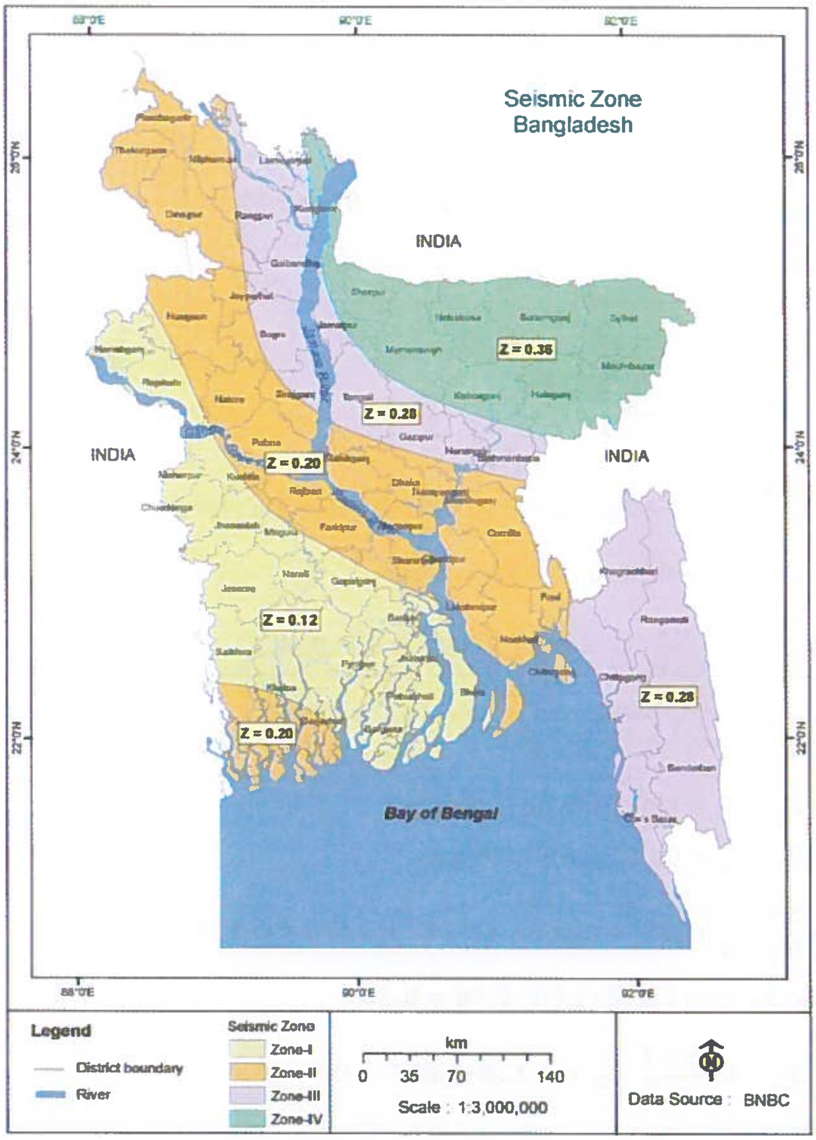


Figure 4.3.9 Seismic Zoning Map of Bangladesh
(Refer to Section 2.5.4.2, Part 6, BNBC 2015)

4.3.4 Structure Importance Factor *I*

Structure importance factor *I* shall be evaluated according to BNBC 2015 (Refer to Section 2.5.7.1of BNBC 2015). Occupancy category of buildings and other structures for Flood, Surge, Wind and Earthquake Loads are shown below.

Table 4.3.9 Importance Factors for Buildings and Structures for Earthquake Design (Table 6.2.17 of BNBC 2015)

Occupancy Category	Importance factor I
I , II	1
III	1.25
IV	1.5

Table 4.3.10 Occupancy Category of Buildings and Other Structures for Flood, Surge, Wind and Earthquake Loads (Table 6.1.1 of BNBC 2015)

Nature of Occupancy	Occupancy Category
Buildings and other structures that represent a low hazard to human life in the event of failure, including, but not limited to: ・ Agricultural facilities ・ Certain temporary facilities ・ Minor storage facilities	I
All buildings and other structures except those listed in Occupancy Categories I, III, and IV II	II
Buildings and other structures that represent a substantial hazard to human life in the event of failure, including, but not limited to: ・ Buildings and other structures where more than 300 people congregate in one area ・ Buildings and other structures with daycare facilities with a capacity greater than 150 ・ Buildings and other structures with elementary school or secondary school facilities with a capacity greater than 250 ・ Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities ・ Health care facilities with a capacity of 50 or more resident patients, but not having surgery or emergency treatment facilities ・ Jails and detention facilities Buildings and other structures, not included in Occupancy Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure, including, but not limited to: ・ Power generating stationsa ・ Water treatment facilities ・ Sewage treatment facilities ・ Telecommunication centers Buildings and other structures not included in Occupancy Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released.	III
Buildings and other structures designated as essential facilities, including, but not limited to: ・ Hospitals and other health care facilities having surgery or emergency treatment facilities ・ Fire, rescue, ambulance, and police stations and emergency vehicle garages ・ Designated earthquake, hurricane, or other emergency shelters ・ Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response ・ Power generating stations and other public utility facilities required in an emergency ・ Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, cooling towers, electrical substation structures, fire water storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for operation of Occupancy Category IV structures during an emergency ・ Aviation control towers, air traffic control centers, and emergency aircraft hangars ・ Water storage facilities and pump structures required to maintain water pressure for fire suppression ・ Buildings and other structures having critical national defense functions Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing highly toxic substances where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction.	IV

4.3.5 Normalized Acceleration Response Spectrum C_S

Normalized acceleration response spectrum C_S shall be evaluated according to BNBC 2015 (Refer to Section 2.5.6.3of BNBC 2015). C_S depends on S and values of T_B, T_C and T_D, (Figure 4.2.4) which are all functions of the site class.

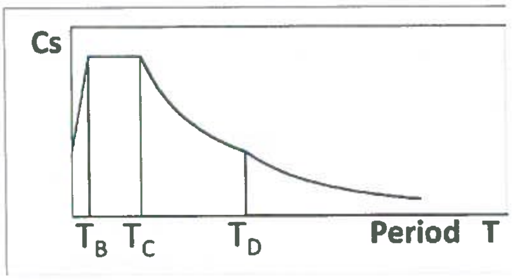


Figure 4.3.10 Typical Shape of the Elastic Response Spectrum Coefficient C_S

$$C_s = S\cdot\left(1+\frac{T}{T_B}(2.5\cdot\eta-1)\right) \quad for\ 0 \le T \le T_B$$

$$C_s = 2.5\cdot S\cdot\eta \quad \text{for } T_b \le T \le T_C$$

$$C_s = 2.5\cdot S\cdot\eta\cdot\left(\frac{T_C}{T}\right) \quad \text{for } T_C \le T \le T_D \qquad (4.2.4)$$

$$C_s = 2.5\cdot S\cdot\eta\cdot\left(\frac{T_C\cdot T_D}{T^2}\right) \quad \text{for } T_D \le T \le 4sec$$

Where:

S : Soil factor which depends on site class

T : Structure (building) period(s)

$T = 0.0466\cdot(h_n)^{0.9}$

h_n : Hight of building(m)

T_B : Lower limit of the period of the constant spectral acceleration branch given in Table 4.3.11 as a function of site class

T_C : Upper limit of the period of the constant spectral acceleration branch given in Table 4.3.11 as a function of site class

T_D : Lower limit of the period of the constant spectral displacement branch given in Table 4.3.11 as a function of site class

η : Damping correction factor as a function of damping with a reference value of η=1 for 5% viscous damping. It is given by the following equation:

$\eta = \sqrt{10/(5+\xi)} \ge 0.55$

Where, ξ is the viscous damping ratio of the structure, expressed as a percentage of critical damping. The value of η cannot be smaller than 0.55.

Table 4.3.11 Site Dependent Soil Factor and other Parameters Defining Elastic Response Spectrum from Table 6.2.16 of BNBC 2015

Soil type	T_B(s)	T_C(s)	T_D(s)
SA	0.15	0.40	2.00
SB	0.15	0.50	2.00
SC	0.20	0.60	2.00
SD	0.20	0.80	2.00
SE	0.15	0.50	2.00

4.3.6 Site Classification

Site Classification will be done in accordance with Table 4.3.12 based on the soil properties of upper 30 meters of the site profile.

Table4.3.12 Site Classification Based on Soil Properties
(From Table 6.2.13 of BNBC 2015)

Site classification based on soil properties				
Site Class	Description of soil profile up to 30 meters depth	Average Soil Properties in top 30 meters		
		Shear wave velocity Vs (m/s)	Standard Penetration Value N (blows/30cm)	Undrained shear strength Su (kPa)
SA	Rock or other rock - like geological formation, including at most 5 m of weaker material at the surface.	> 800	--	--
SB	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360-800	>50	>250
SC	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180-360	15-50	70-250
SD	Deposits of loose - to - medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft - to - firm cohesive soil.	<180	<15	<70
SE	A soil profile consisting of a surface alluvium layer with Vs values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with Vs > 800 m/s.	--	--	--

After selection of site class, Soil factor shall be determined according to following table.

Table4.3.13 Soil Factor S (From Table 6.2.16 of BNBC 2015)

Site class	Soil factor
SA	1.00
SB	1.20
SC	1.15
SD	1.35

4.3.7 Cumulative Strength Index at The Ultimate Deformation of Structure C_{TU}

To ensure seismic safety of a building, it is required to secure a certain minimum strength at ultimate deformation in addition to I_s index. Especially building with low strength capacity, as is the case in Bangladesh, may show unstable response and cause large deformation, even collapse. To resolve the problem, $C_{TU} \cdot S_D$ has been set to ensure minimum ultimate strength, even though the building may possess high ductile capacity. The upper limit of deformation is not fixed yet for Bangladesh because of lack of data and experiment on local design and construction conditions. Presently, the upper limit may be set as F=2.0 (about 1/80 as upper limit) to ensure the minimum strength after satisfying the required I_{so}.

It is equivalent to the ultimate strength that is required for a low rise building designed by R =5 of S_D soil in BNBC 2015.

$$F = C_s \times \Omega / R = 3.88 \times 3/5 \cong 2.00 \times C_s \times \Omega$$

Following figure shows the case that the building has three ductility indices. C_{TU} shown below is for the case when I_S is determined by strength-dominant equation and Ductility-dominant equation at the points 1, 2, and 3.

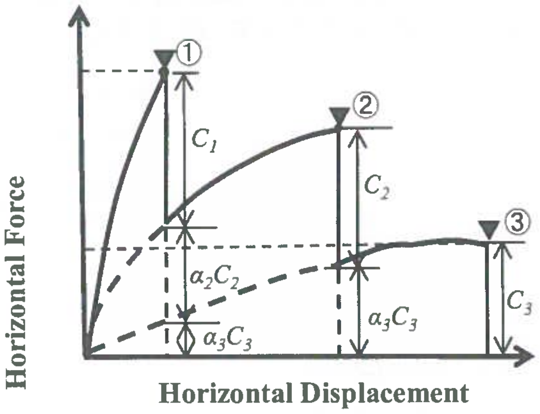


Figure 4.3.11 Calculation of C_{TU}

Ductility-dominant basic seismic index of structure

$$E_o = \frac{n+1}{n+i} \times \sqrt{E_1^2 + E_2^2 + E_3^2} = E_o = \frac{n+1}{n+i} \times \sqrt{(C_1 \times F_1)^2 + (C_2 \times F_2)^2 + (C_3 \times F_3)^2}$$

... Eq (4) of J. Standard

Strength-dominant basic seismic index of structure

$$E_o = \frac{n+1}{n+i}(C_1 + \Sigma_j \alpha_j C_j) \cdot F_1$$

...Eq (5) of J. Standard

At point 1 Eq (4) $C_{TU} = \frac{n+1}{n+i} \times C_3$

and Eq (5) $C_{TU} = \frac{n+1}{n+i}\left(C_i + \sum_j \alpha_j C_j\right) = \frac{n+1}{n+i}\left(C_1 + \alpha_2 \times C_2 + \alpha_3 \times C_3\right)$

At point 2 Eq (4) $C_{TU} = \frac{n+1}{n+i} \times C_3$

and Eq (5) $C_{TU} = \frac{n+1}{n+i}\left(C_i + \sum_j \alpha_j C_j\right) = \frac{n+1}{n+i}\left(C_2 + \alpha_3 \times C_3\right)$

At point 3 Eq (5) [Eq (4)] $C_{TU} = \frac{n+1}{n+i}\left(C_i + \sum_j \alpha_j C_j\right) = \frac{n+1}{n+i} \times C_3$

SUPPLEMENT

SUPPLEMENT 1. 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 S 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 S 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 $\propto$=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.

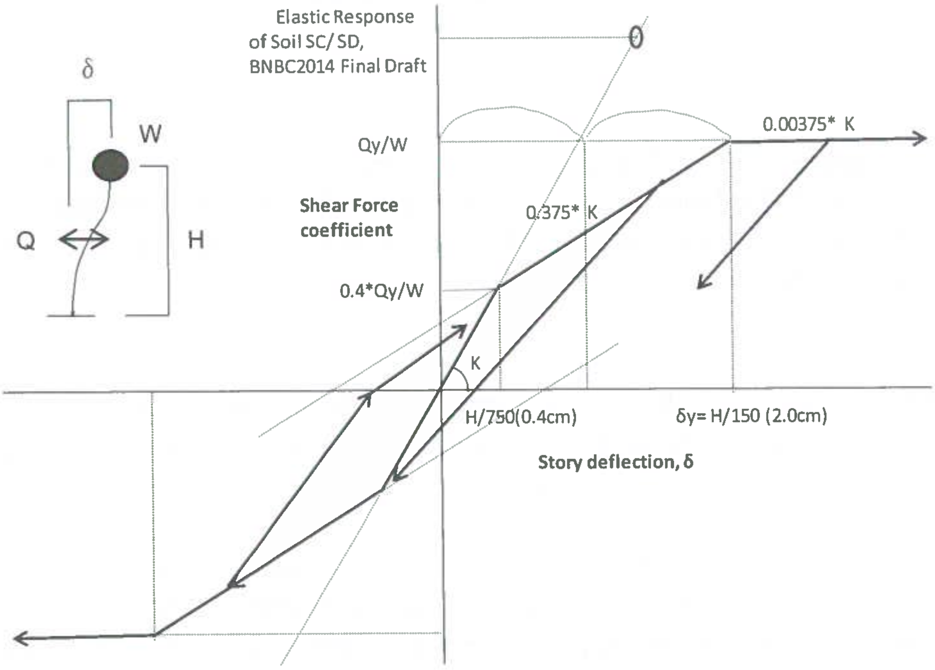


Figure S1.1 Restoring Force Characteristics (1)

A model supposing storey deflection angle 1/150 when the frame yield.

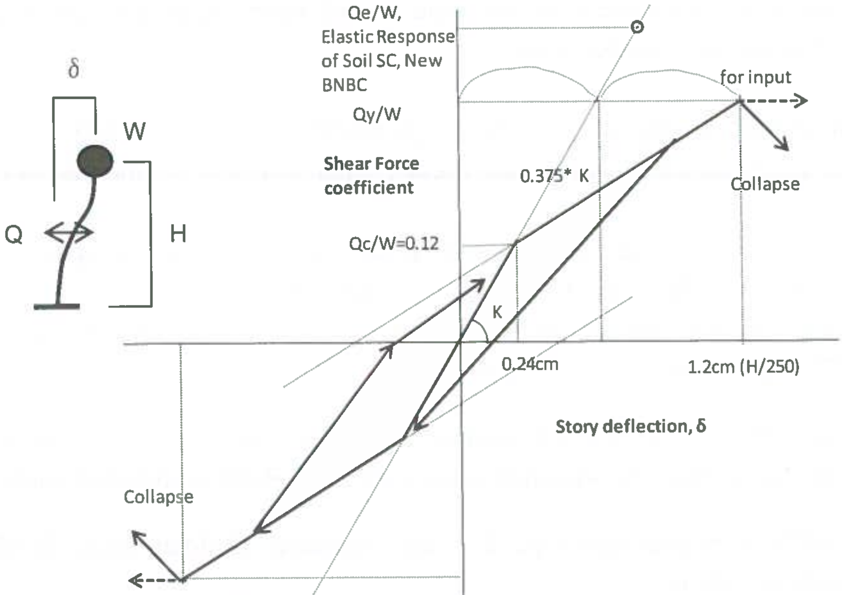


Figure S1.2 Restoring Force Characteristics (2)

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 S1.2 shows the results of case 7 and case 8.

Table S 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, μ (δ/δ_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 (δ/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 S 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(=δ/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=δ/ 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, μ (δ/δ_y)、μ	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. 3.2.3-2of 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*.

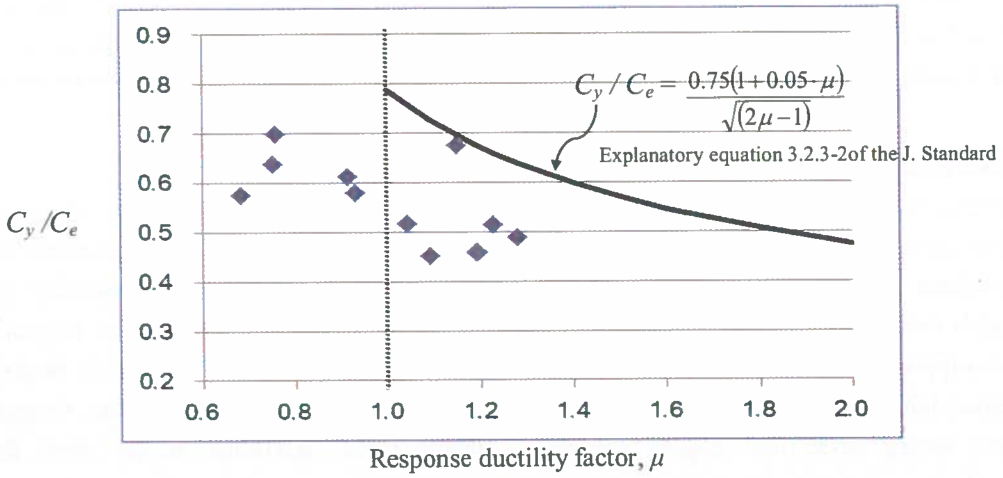


Figure S 1.3 Yield Shear Force Coefficient and Response Ductility Ratio ($C_y/C_e-\mu$), Soil Type SC

Strength Index and Ductility Index ($C-F$) Relation

Conversion from response ductility factor μ to ductility index F is as follows.

$$F=\frac{\sqrt{(2\mu-1)}}{0.75(1+0.05\mu)}$$ Commentary eq. 3.2.3-3of the J. Standard

Equation (15) of the Standard is applied in case that μ is less than 1.0.

Case 7, μ=1(=1/250) and μ 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.

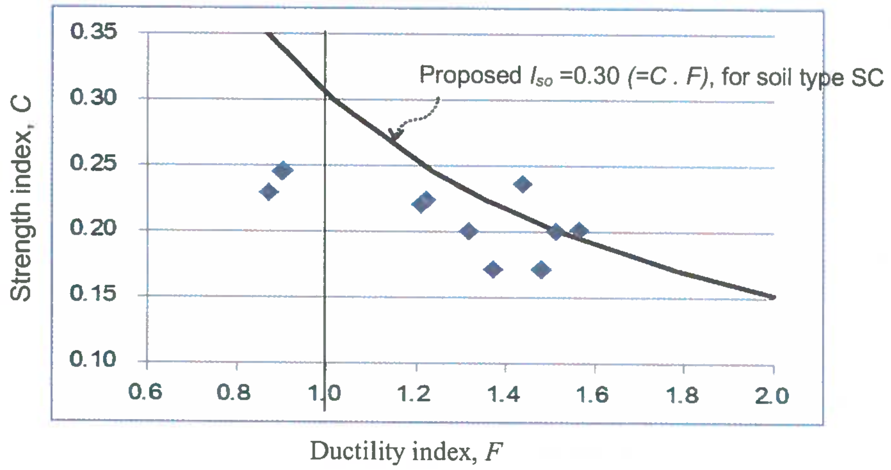


Figure S1.4 Strength Index and Ductility Index *(C–F)* Relation, Soil Type SC

(4) Strength Index and Ductility Index (C–F), Soil Type SD

Yield shear force coefficient and response ductility ratio (C_y/C_e–μ) . 3 waves × 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. 3.2.3-2of 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.

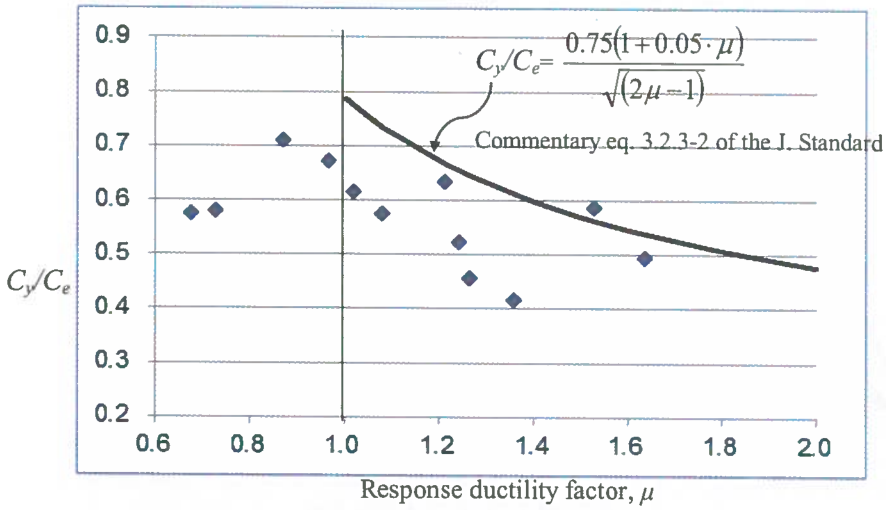


Figure S1.5 Yield Shear Force Coefficient and Response Ductility Ratio (C_y/C_e –μ), Soil Type SD

Strength Index and Ductility Index (*C–F*) Relation

Conversion from response ductility factor μ to ductility index *F* is as follows.

$$F=\frac{\sqrt{(2\mu-1)}}{0.75(1+0.05\mu)}$$ Commentary eq. 3.2.3-3of the J. Standard

Equation (15) of the Standard is applied in case that μ is less than 1.0.

Case 8, μ=1(=1/250) and μ is less than 1.0, $F=0.8+\frac{0.2(\mu 0.004-0.002)}{0.002}$ 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.

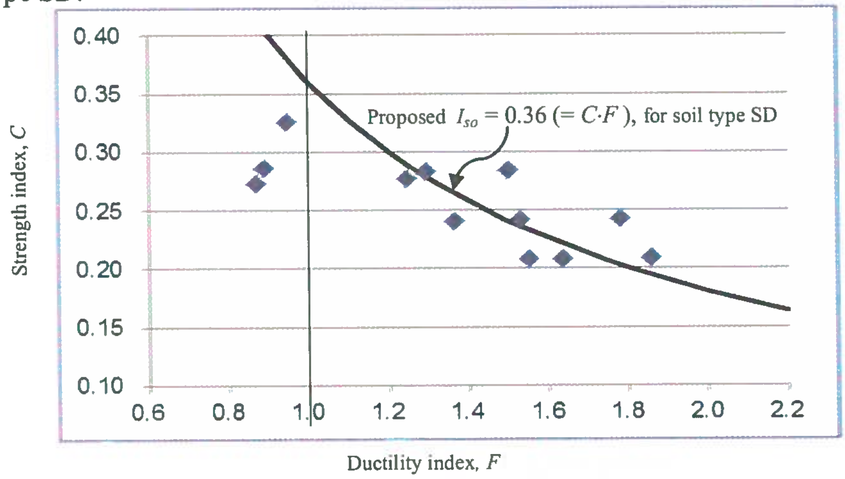


Figure S1.6 Strength Index and Ductility Index *(C–F)*, Soil Type SD

(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.

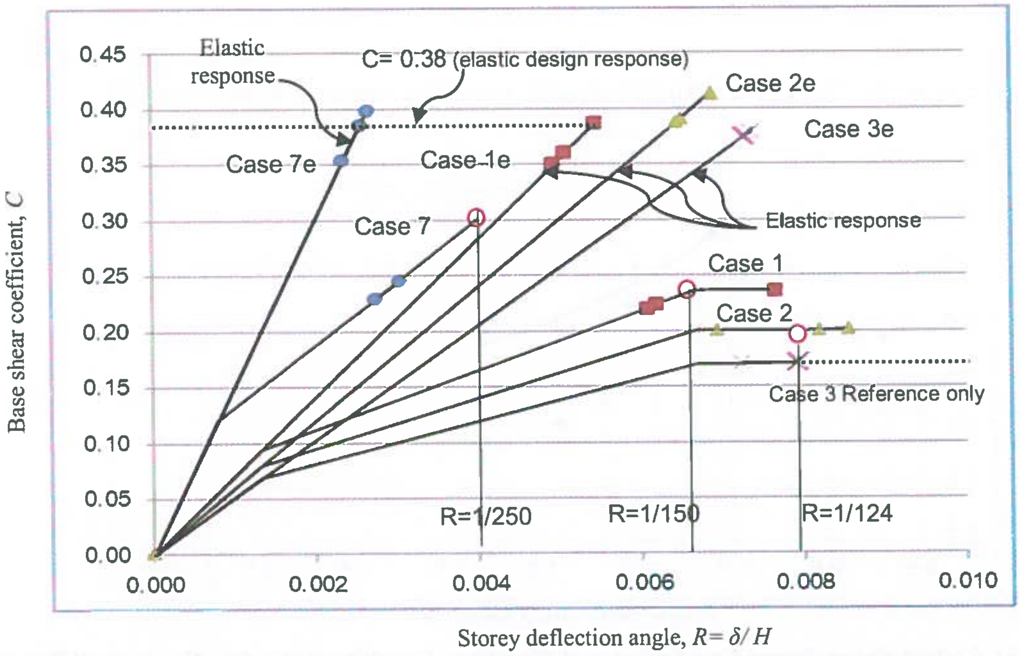


Figure S1.7 Response Shear Force Coefficient and Storey Deflection Angle, Soil SC

Soil type SD (soft soil, suffix e of each case shows elastic response, red color circle (○) shows target allowable response)

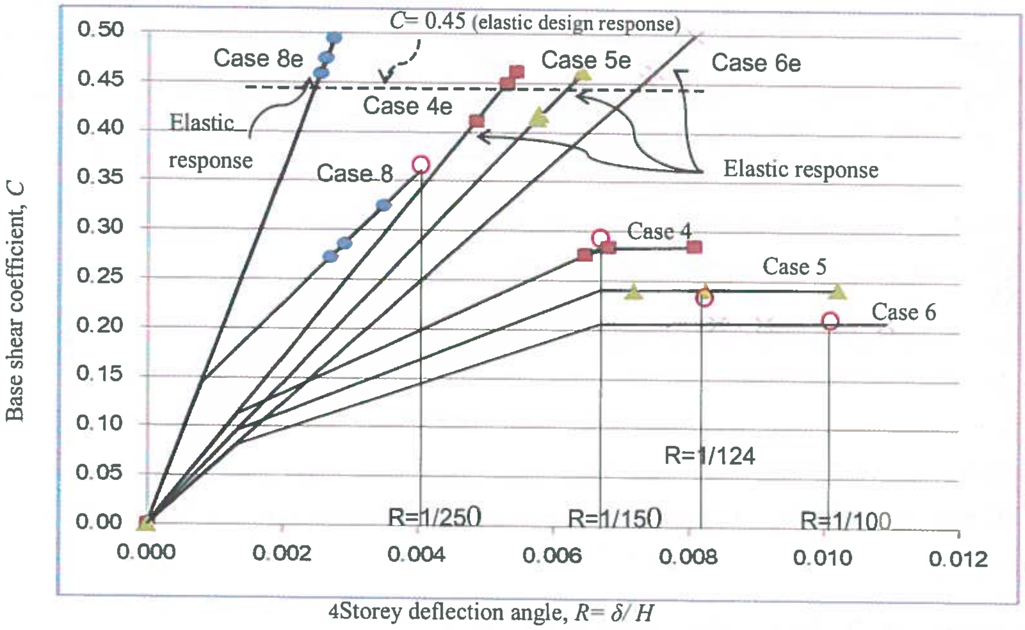


Figure S1.8 Response Shear Force Coefficient and Storey Deflection Angle, Soil SD

(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 S1.3 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 (Qy, 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), T1= 0.426sec. T2=0.176sec. T3=0.117sec.

(1/250), T1= 0.381sec. T2=0.157sec. T3=0.104sec.

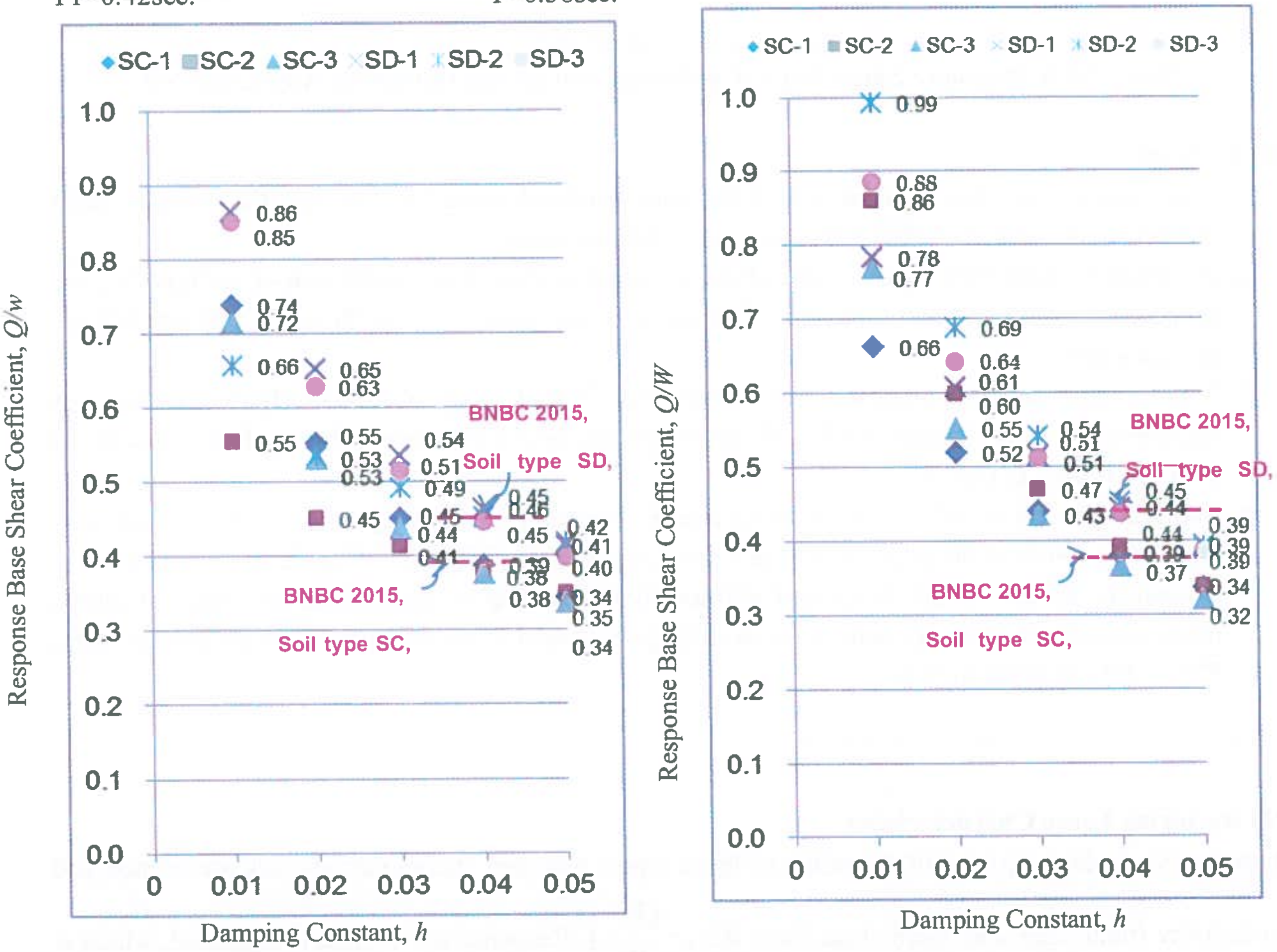
BNBC 2015, Fundamental building period, T

$T = C_t(h_n)^m$(6.2.38) BNBC 2015

Concrete moment-resisting frame, C_t=0.0466, m=0.9, 3 storey (h_n=9m height), T=0.337sec.

(2) Elastic Response and Damping Constant,

Soil type SC and SD of BNBC 2015, example; 3 storey RC building with 2 different stiffness



T1=0.42sec.

T=0.38sec.

Figure S 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; $S_a = \dfrac{2}{3}\dfrac{ZIC_S}{R} = \dfrac{2}{3} \times 0.2 \times 1 \times \dfrac{2.875}{1.0} = 0.383$ (R=1.0, Elastic response, in case of damping constant 5%)

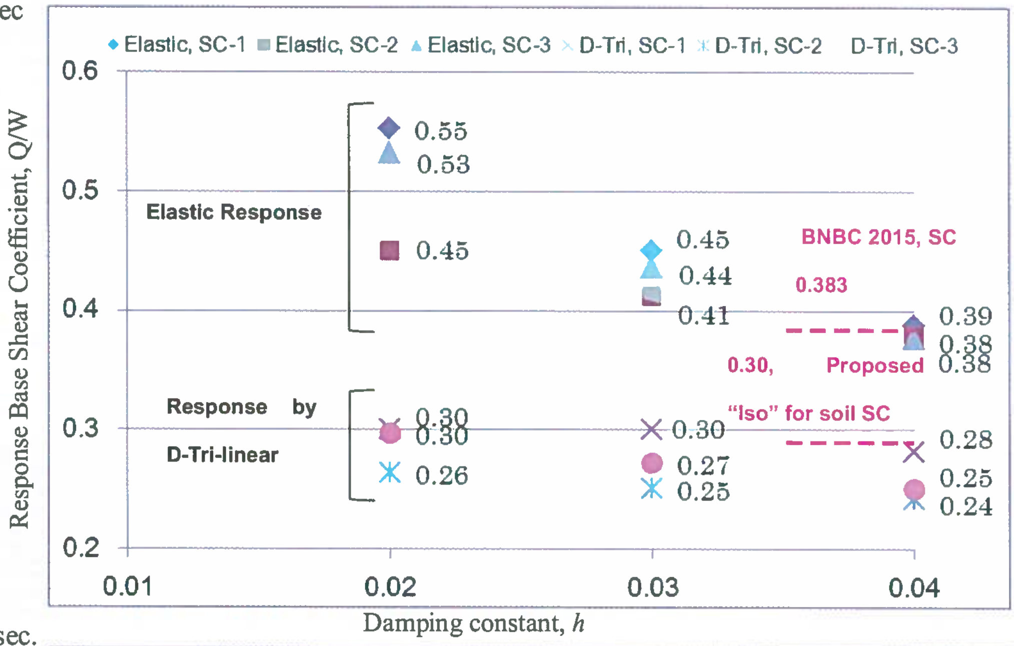
Soil type SD; $S_a = \dfrac{2}{3}\dfrac{ZIC_S}{R} = \dfrac{2}{3} \times 0.2 \times 1 \times \dfrac{3.375}{1.0} = 0.45$ (R =1.0, Elastic response, in case of damping constant 5%)

(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 S1.10. In case of model T_1= 0.42sec. and T_1= 0.38sec. Response shear force coefficient is lower than 0.30, which is proposed "I_{so}".

T_1=0.42sec



T1=0.38sec.

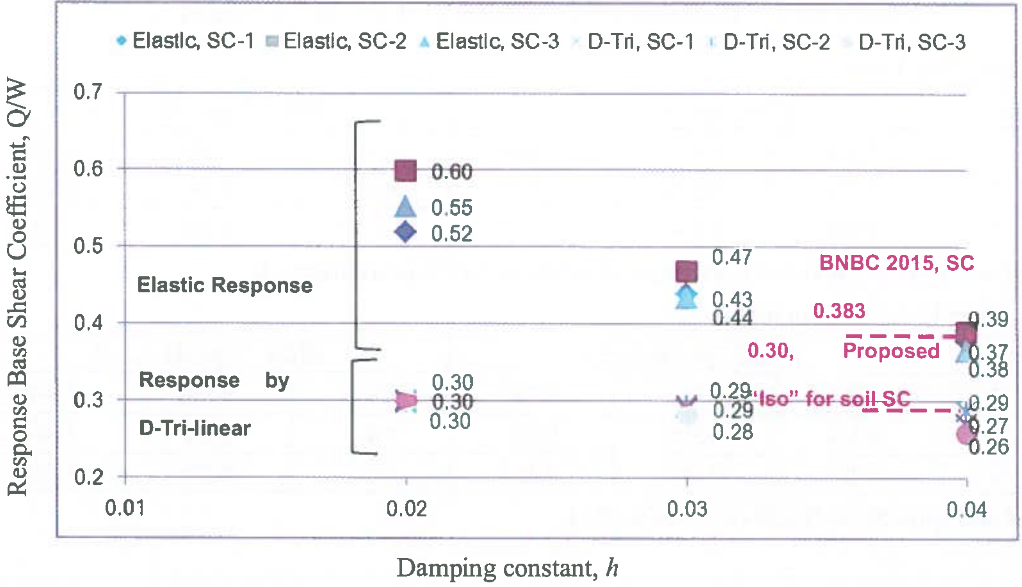


Figure S 1.10 Comparison between Elastic Response and D-Tri-Linear Response

(4) Result of Response of case 9 and 10, 3 storey, δ/h = 1/200, T1=0.426sec. T2 = 0.176sec. T3 =0.117sec. Damping C= Tangential stiffness 4%, BNBC 2015 Soil type SC (3 waves) and SD (3 waves)

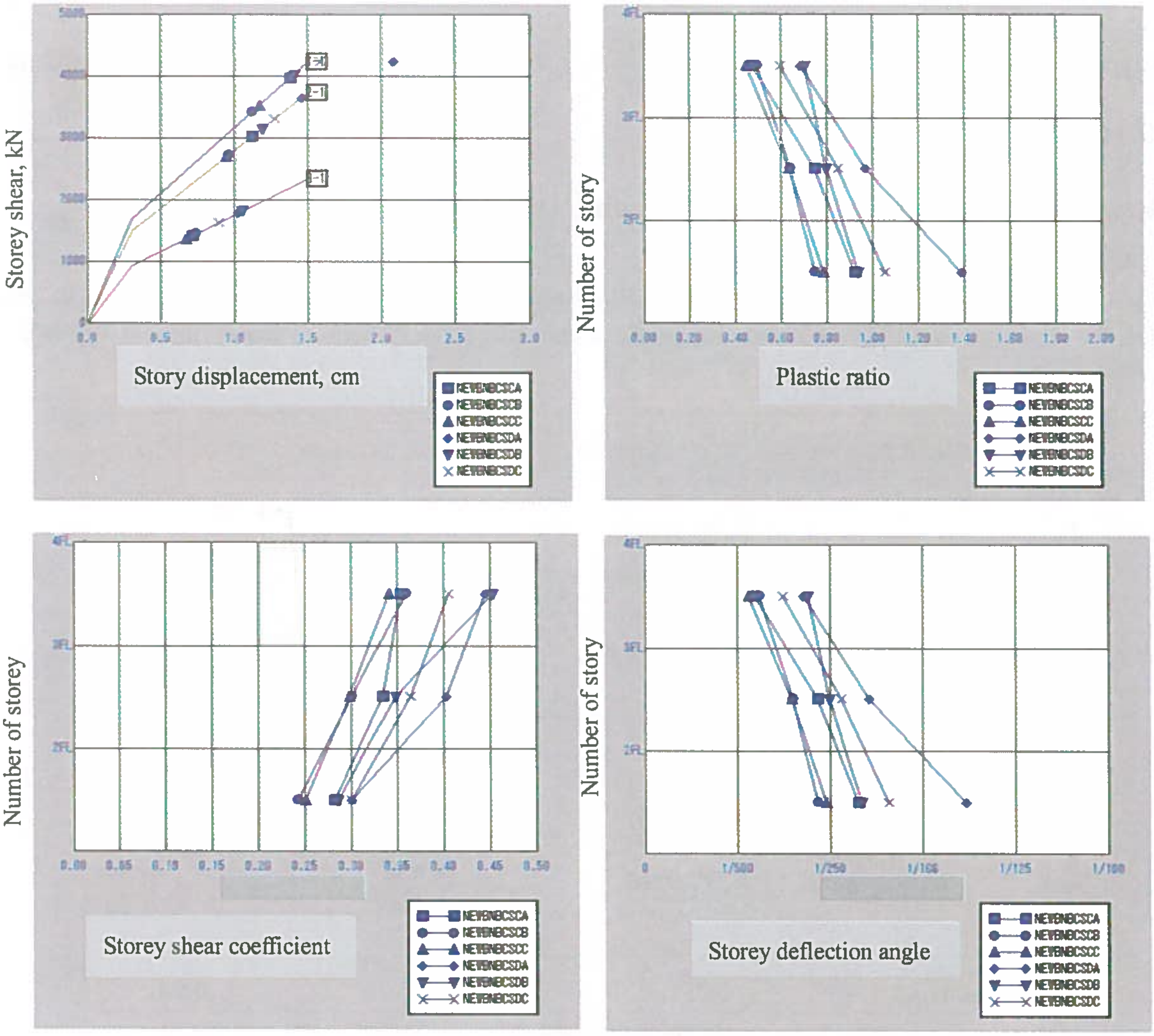


Figure S1.11 Response of Each Storey (Wave SC-A Means S-1.)

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 0.8139, average of soil type SD is more than 1.0.

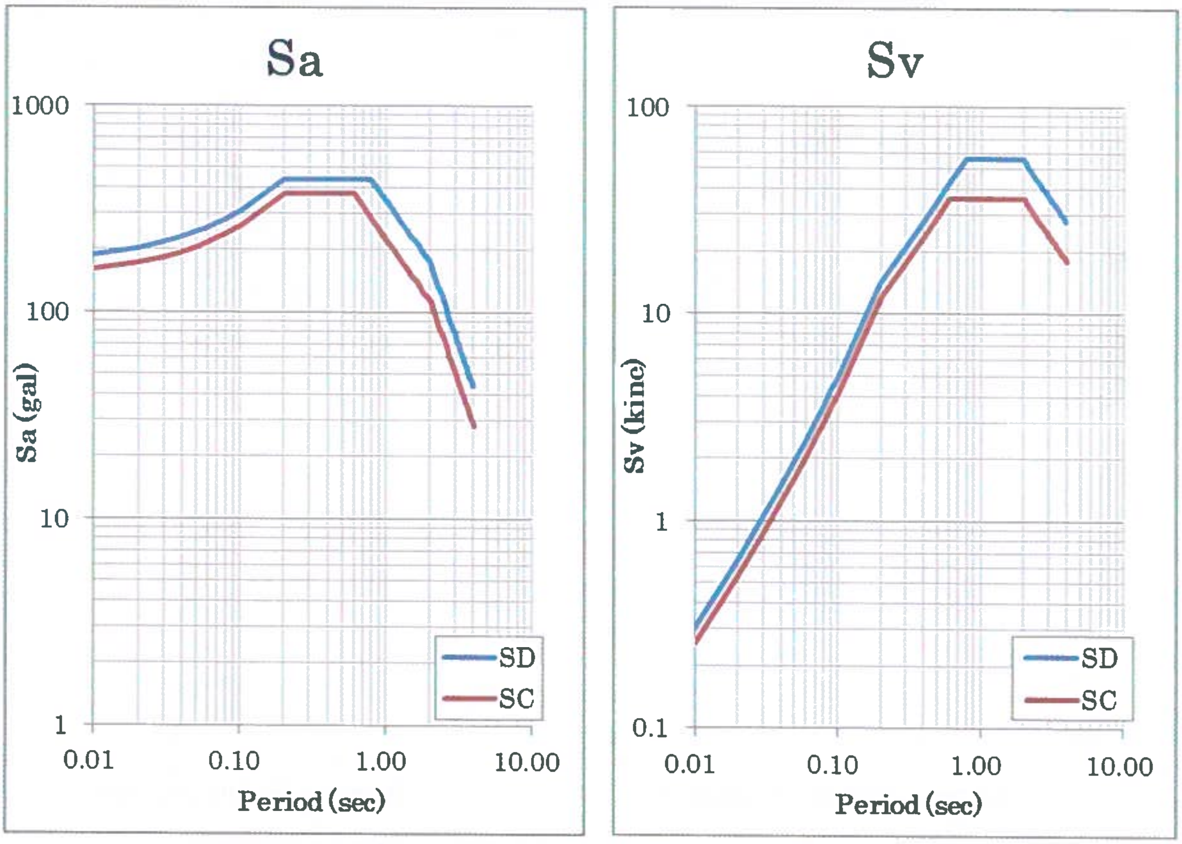
Response Shear Force Coefficient

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.



Response Acceleration Spectrum

Response Velocity Spectrum

Figure S1.12 Specification of Response Spectrum by BNBC 2015

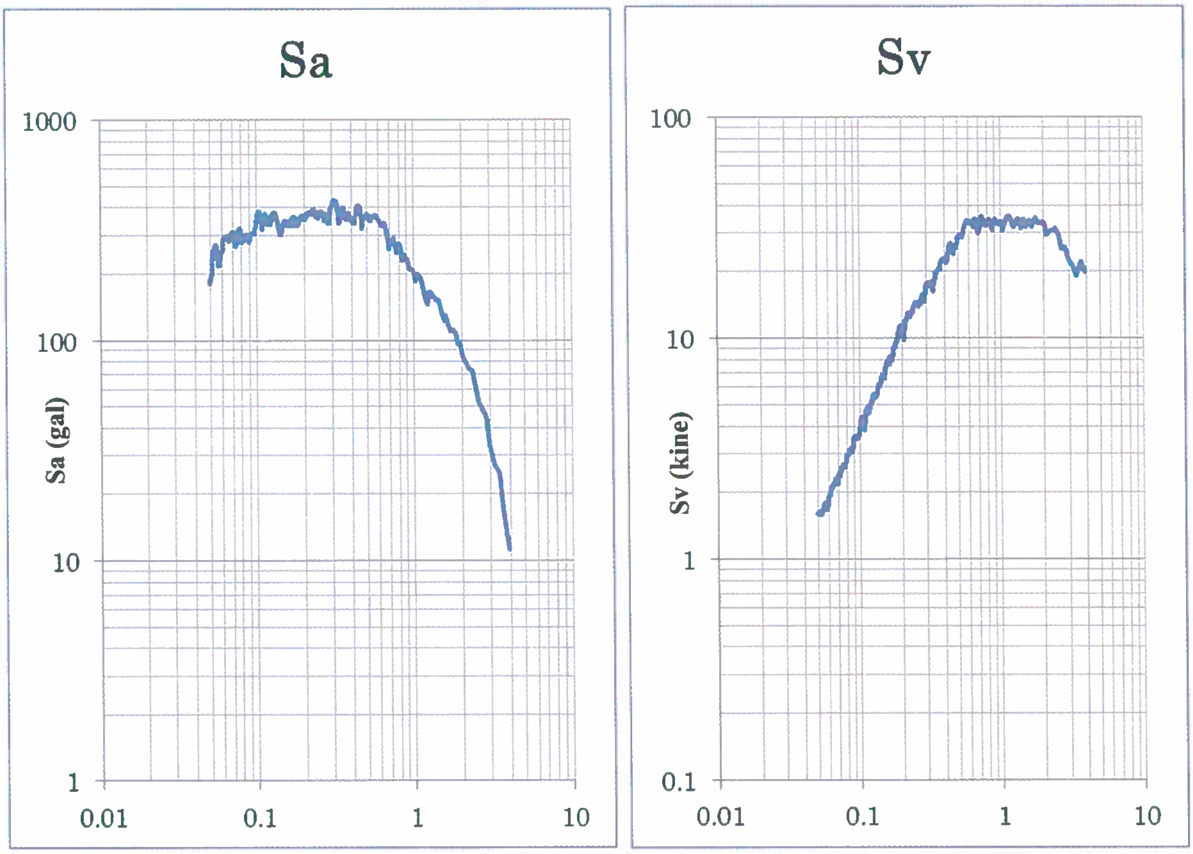
Table S1.4 Peak Ground Acceleration and Velocity of Each Wave

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.

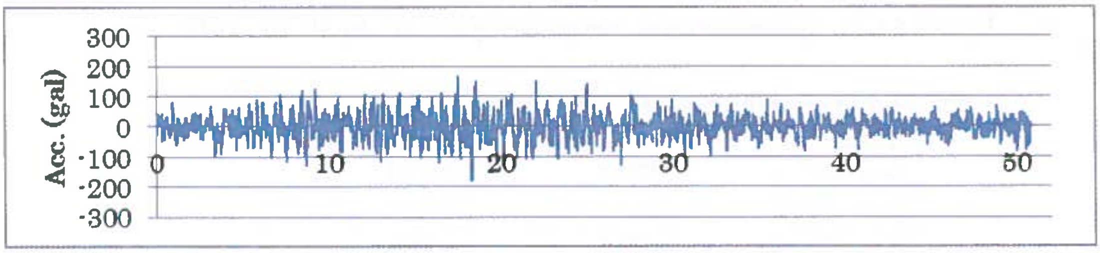
BNBC 2015, Response spectrum of Soil type SC-1



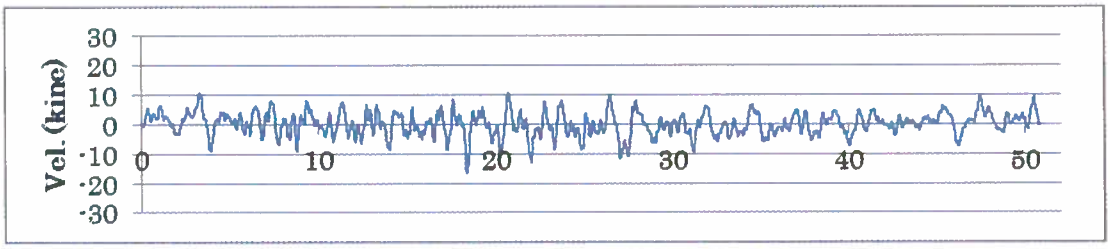
Response Acceleration Spectrum

Response Velocity Spectrum

Figure S 1.13 Response Spectrum of Soil Type SC by BNBC 2015



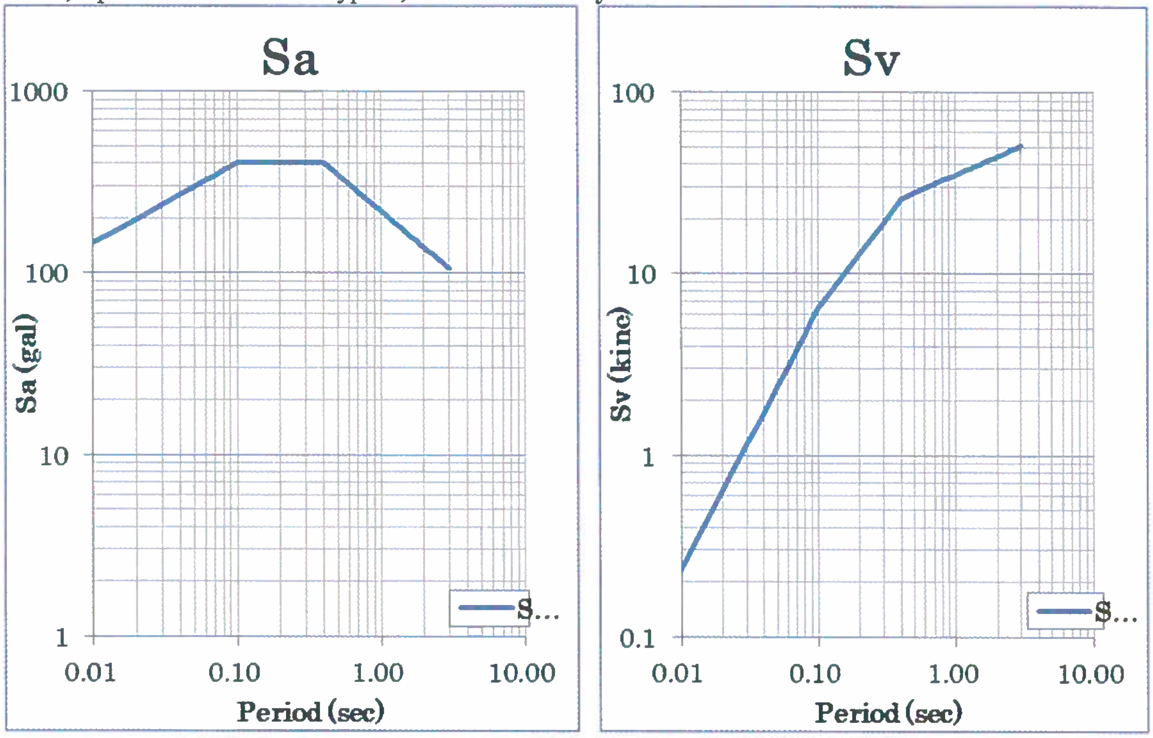
Acceleration at ground level



Velocity at ground level

Figure S1.14 Acceleration and Velocity Waves of Soil Type SC by BNBC 2015

BNBC93, Specification of Soil type 2, **Reference Only**

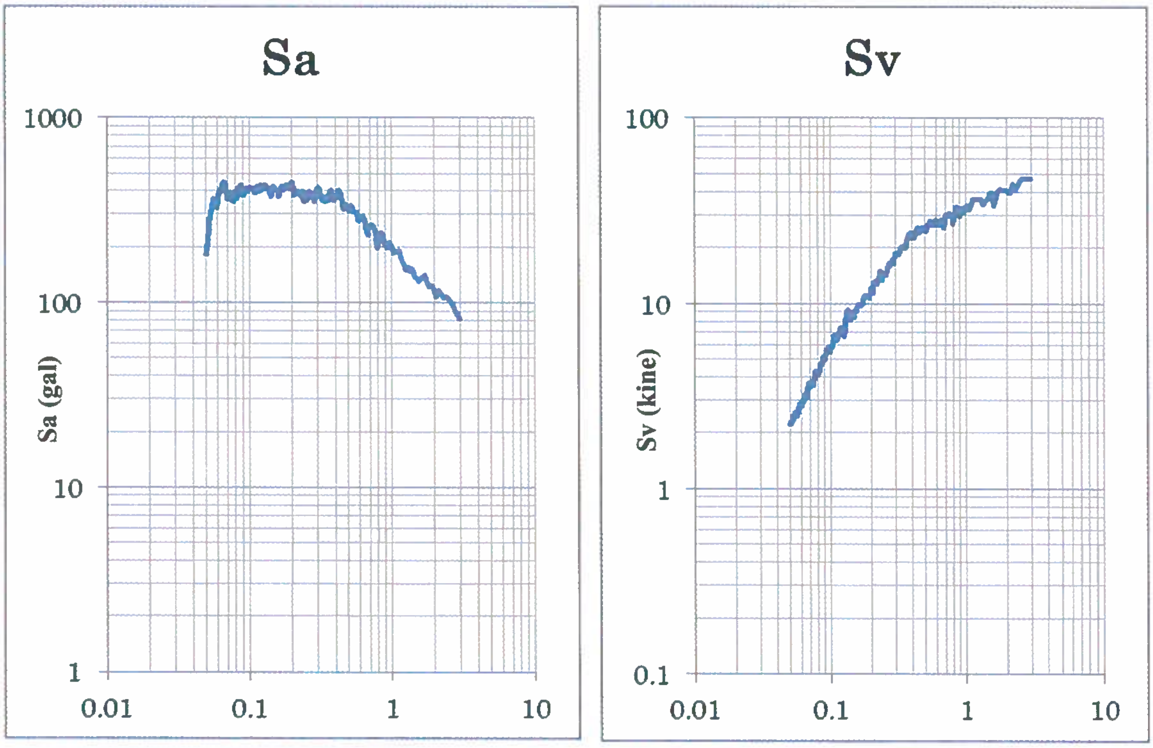


Response Acceleration Spectrum

Response Velocity Spectrum

Figure S1.15 Specification of Response Spectrum of Soil Type 2 by BNBC93

BNBC 93, Response spectrum of Soil Type 2-1, **Reference Only**



Response Acceleration Spectrum

Response Velocity Spectrum

Figure S1. 16 Response Spectrum of Soil Type 2 by BNBC93

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 BNBC93. Seismic evaluation of a 6 storey residential building and a 6 storey hospital building in Zone 2 and Zone 3 (Sylhet) designed by BNBC93 was done, and the result is shown in Figure S1.17. *C-F* relation at ground floor of residential building is shown in Figure S 1.18.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.

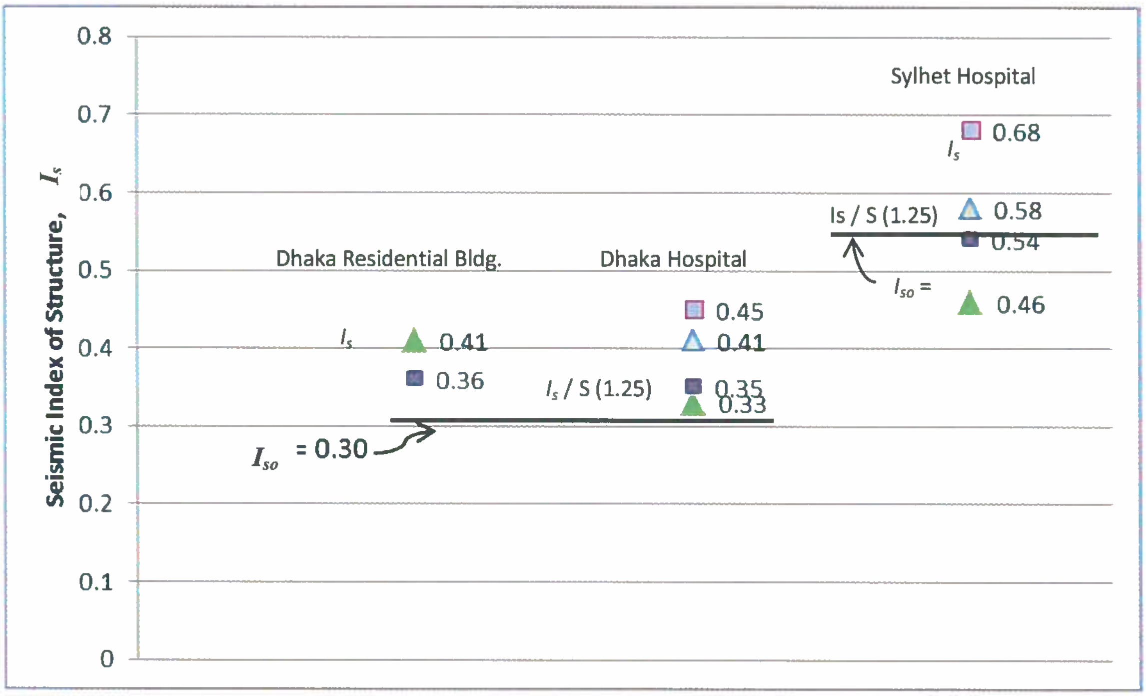


Figure S1.17 Seismic Index of Structure of Buildings Designed by BNBC93

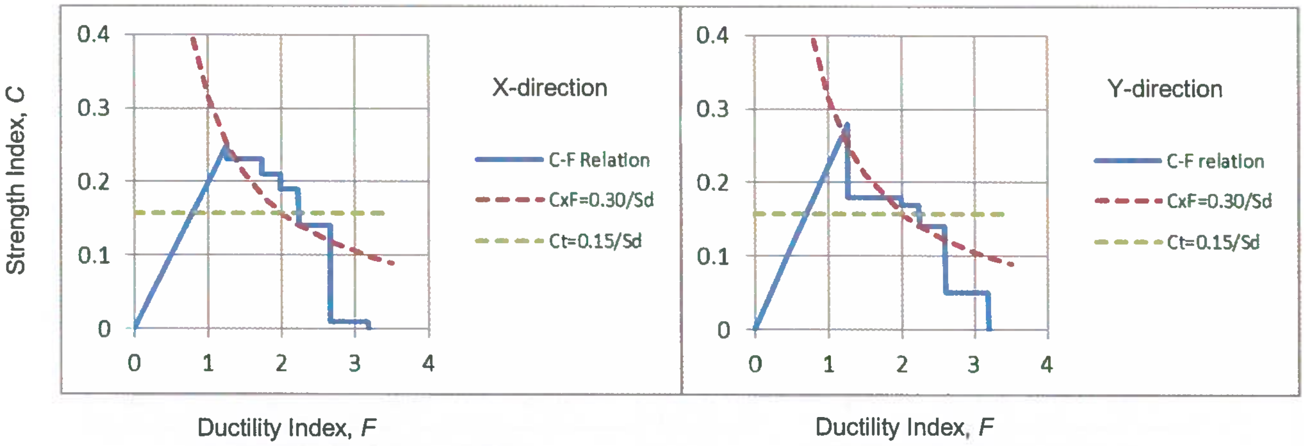


Figure S1.18 *C-F* Relation at Ground Floor, Dhaka Residential Building

Ductility index and shear force at Ground story is shown in Figure S1.19 for a residential building designed based on BNBC93.

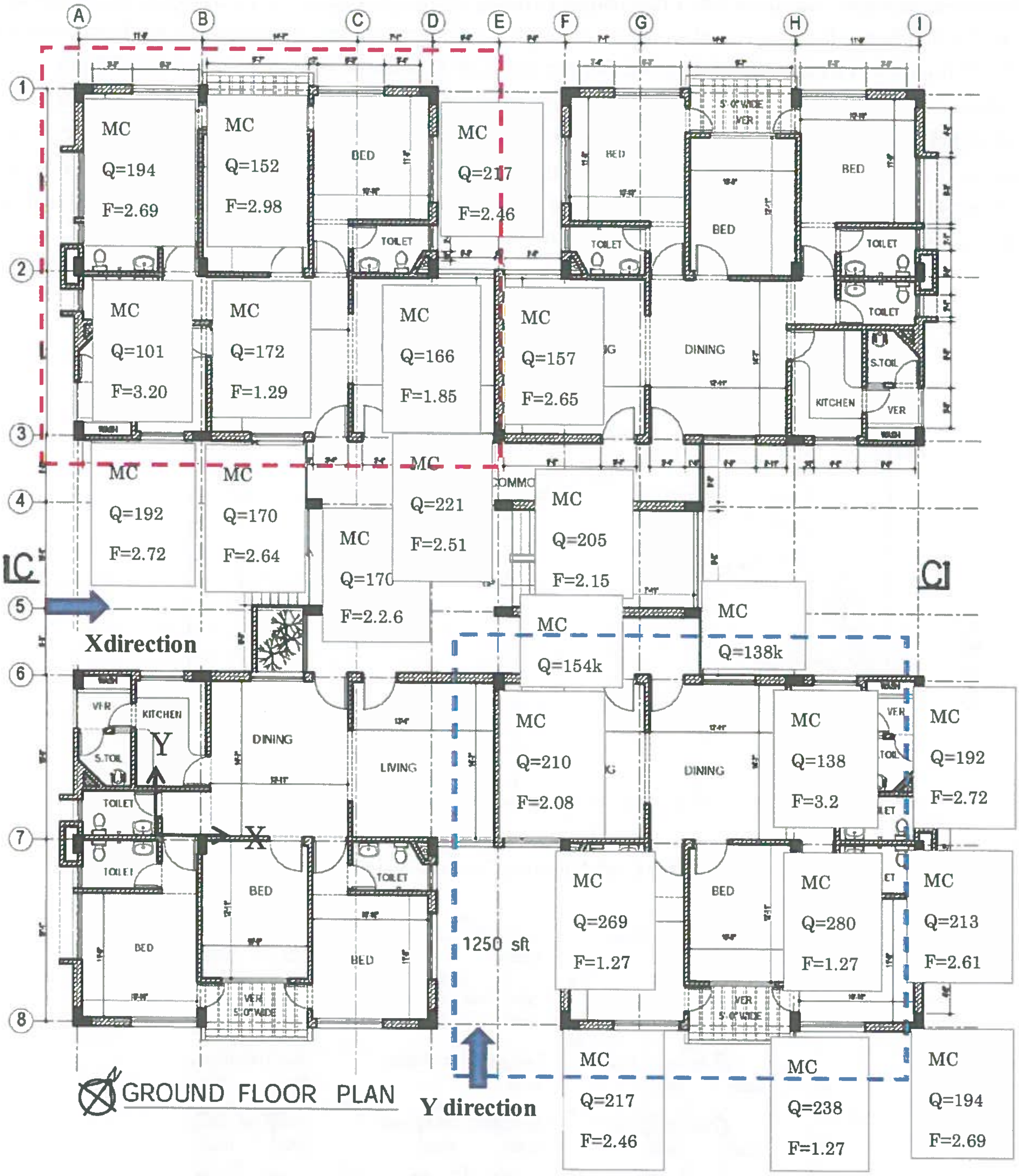
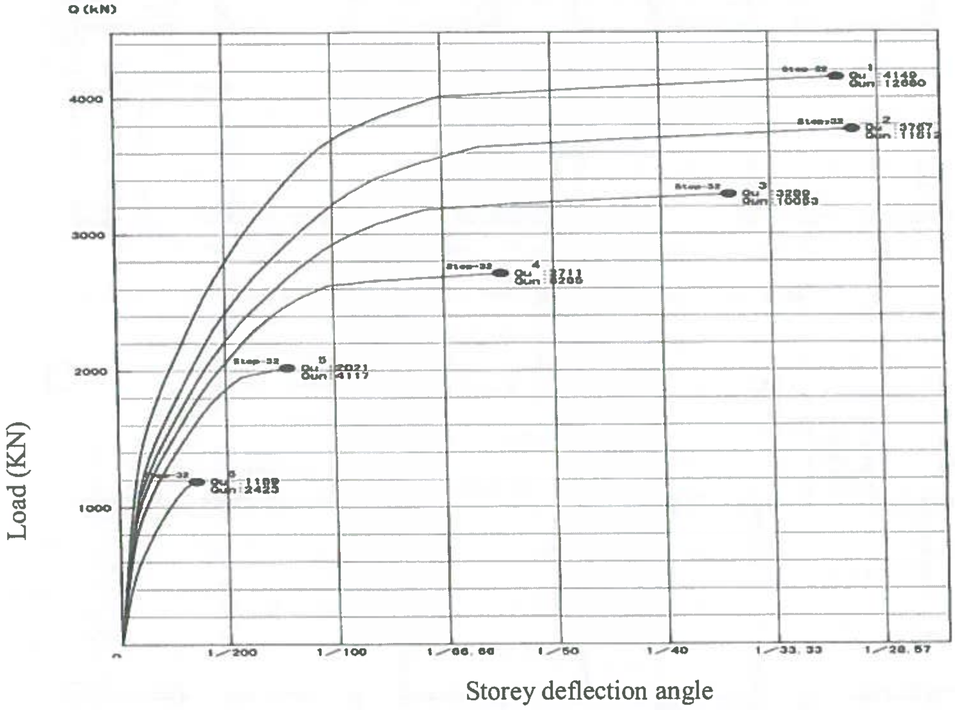


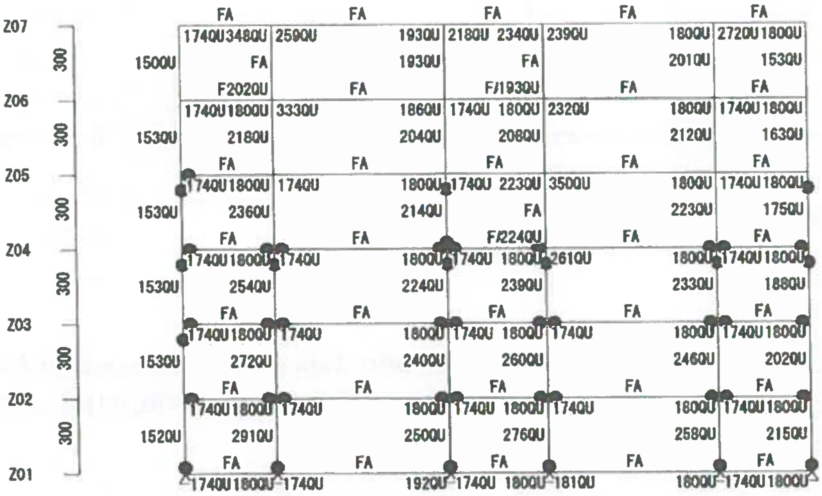
Figure S 1.19 A Sample of Strength *Q* (kN) and Ductility Index (*F*) of Columns at Ground Floor, Residential Building in Zone 2 (Dhaka), Designed by BNBC931/4 of Floor Area is Shown for X and Y Direction Respectively

1.5 PUSHOVER ANALYSIS

Pushover analysis was done for a residential building shown in Figure S1.19 designed by of BNBC93. (Reduction factor R= 8 was used, concrete strength F_c= 25N/mm^2). Incremental horizontal load with fixed distribution ratio was provided. Load-deflection curve of X direction frames is shown (a) of Figure S 1.20. Lower 4 storied had yielded, while upper 2 storied had not yielded yet. This shows that horizontal strength at higher storey is higher than that of lower storey, and building damage will be concentrated at lower storey. Formation of plastic hinges by flexural moment is shown (b) of the Figure. Column hinges occurred at the bottom of ground storey and others. Beam hinges also occurred at upper storied. It is noted that more investigation will be required related to the brittle failures such as shear failure of members.



(a) Load deflection Curve of X-direction



(b) Plastic hinges formation in a frame

Figure S 1.20 An Example of Result of Pushover Analysis

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 $T = 0.80$se., and $C = 1.2$ in case of ground type 2. This is 63% of max. value of *C* as shown in Figure S1.21. In case of Japanese code, natural period is estimated as $T = 0.49$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*.

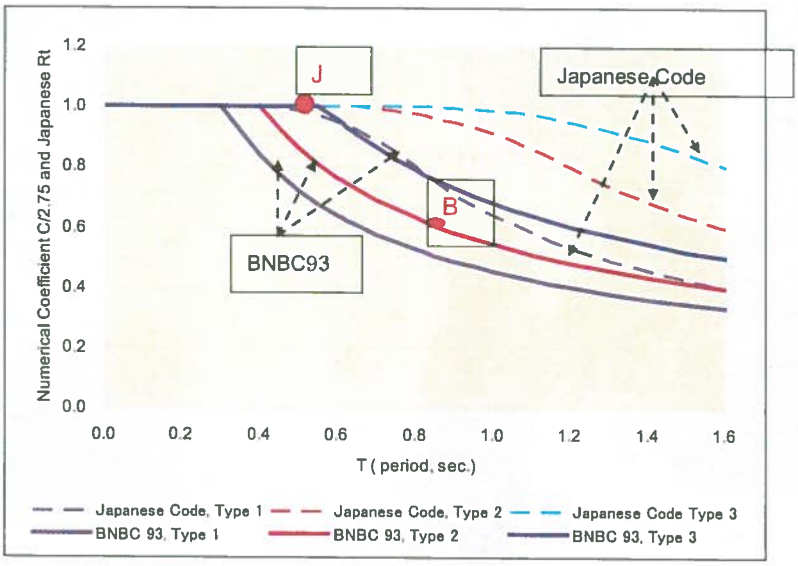


Figure S1.21 Comparison of Numerical Coefficient C/2.75 of BNBC93 and Japanese CodeR_t

Example: 6 storey new RC building with 24.3m height, located on ground type 2
Natural period by BNBC93,
$T = C_t(h_n)^{3/4} = 0.80$ sec.
RC moment frame, $C_t = 0.073$, h_n = building height by meter,
Reference only, by BNBC 2015: $T = C_t(h_n)^m = 0.82$ 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 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

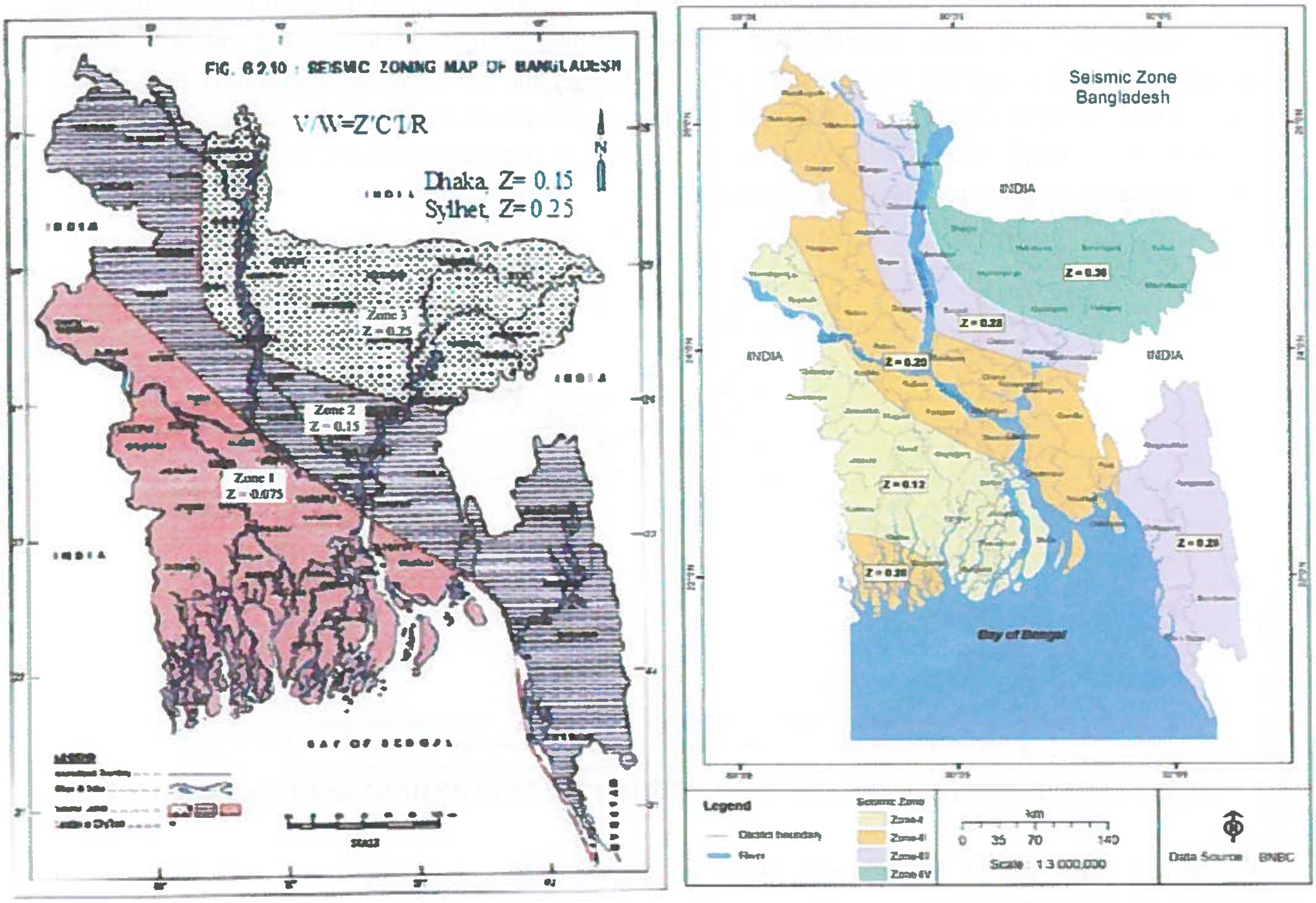


Figure S1.22 Seismic Zoning Map of BNBC 93 and BNBC 2015 (Figure 6.2.24)

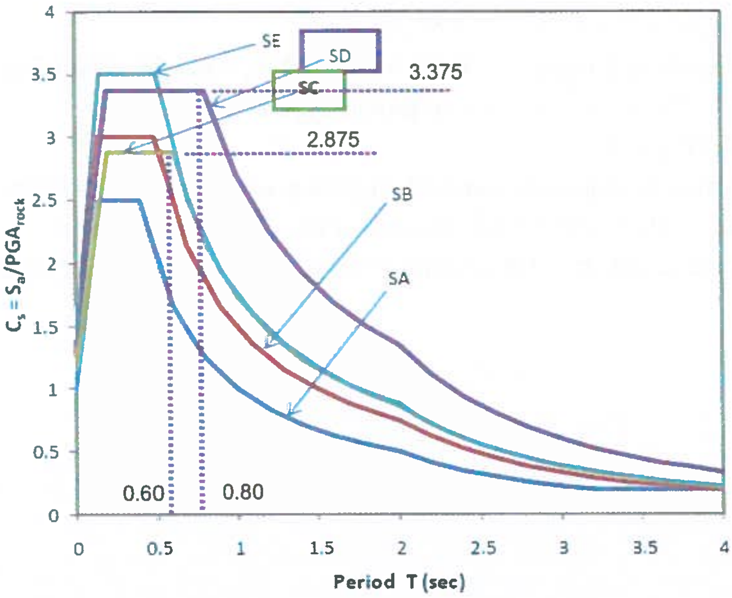


Figure S1.23 Design Acceleration Response Spectrum, BNBC2015

SUPPLEMENT 2. 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, BNBC93 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

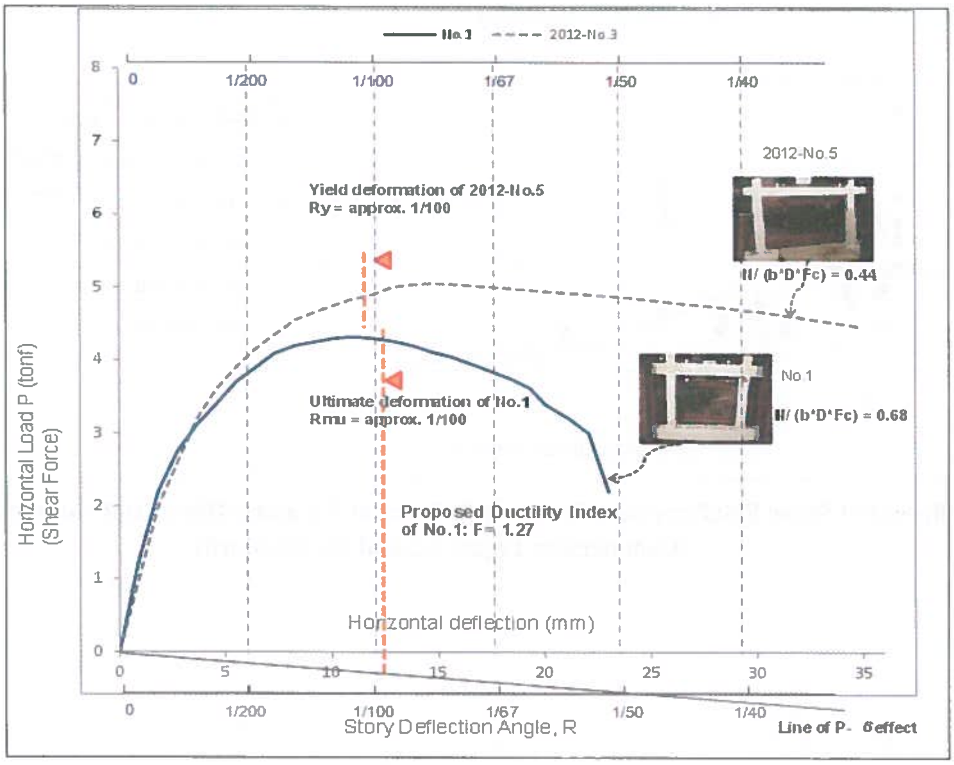
Simplified monotonic load-deflection curve of two frame specimens are shown in Figure S 2.1. Specimen 1 is low strength concrete and axial force ratio of column is 0.68. Storey deflection angle of this specimen at ultimate capacity R_{mu} is estimated as approximately 1/100. Specimen 2012-No.5 is ordinary 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 $R_{mu} \geq R_y$

$$F = \frac{\sqrt{2R_{mu}/R_y - 1}}{0.75 \cdot (1 + 0.05R_{mu}/R_y)} \qquad F \leqq 3.2 \qquad \text{The J. Standard (16)}$$

F= 1.27 is calculated (R_y= approx.1/100, specimen 2012-No.5, R_{mu}=approx.1/100, specimen No.1)



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

Figure S 2.1 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 S 2.2 and S2.3.

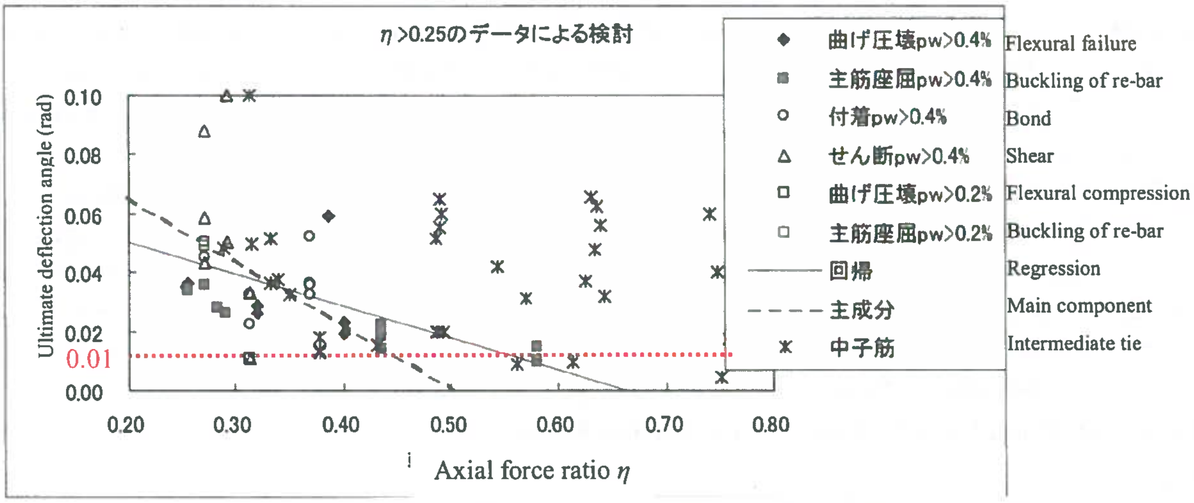


Figure S 2.2 Influence of Axial Force Ratio (by the Data $\eta > 0.25$) (Commentary Figure 1.2-3 of the Standard)

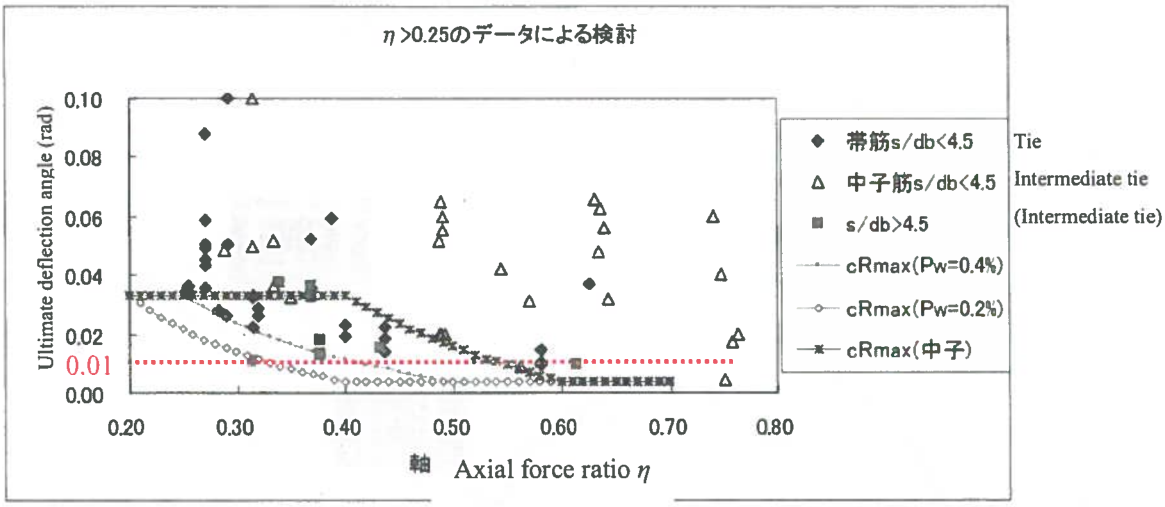
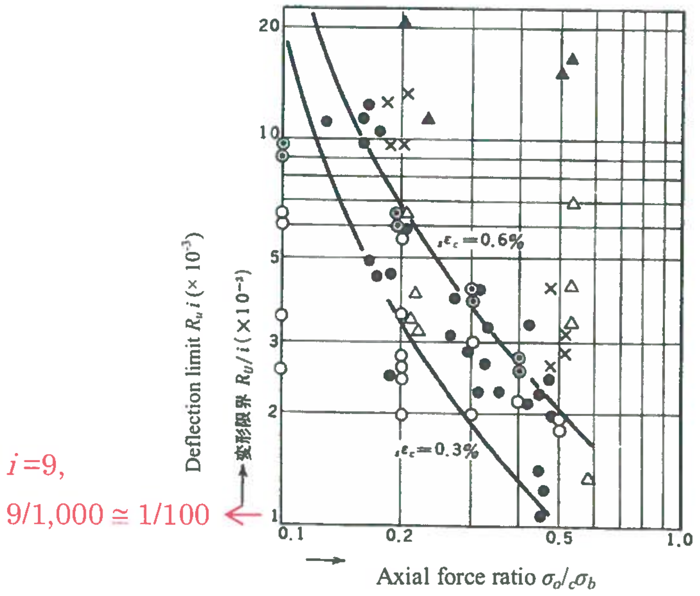


Figure S 2.3 Influence of Shear Reinforcement Ratio and Interval of Tie under High Axial Force of Column (Commentary Figure 1.2-4 of the Standard)

3) Relation of Deflection Limit and Axial Force Ratio

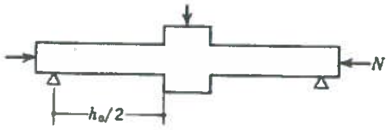
Deflection member angle limit and ductility factor related to axial force ratio is introduced by RC structural design code 2010, Japan

(Source: Architectural Institute of Japan, "Structural Calculation Code of Reinforced Concrete Structure 2010")



○池田　昭男　　i=5
（◎はフープ補強の多いもの p_w=0.57 %）
△遠藤利根穂　　i=4, 6
（▲はフープ補強の多いもの p_w=0.57 %）
●岡田　恒男　　i=9
（×はスパイラル補強 p_w=0.53 %）

$$\frac{R_u}{i}=\frac{1}{6}\times\frac{0.8\,{}_s\varepsilon_c}{\frac{\sigma_0}{{}_c\sigma_B}-0.08}$$



Where,
R_u: Deflection limit member angle
i : Clear span of column h/ depth D
σ_o: axial force/ section area
${}_c\sigma_b$: Compressive strength of concrete
${}_s\varepsilon_c$:strain of compressive re-bar

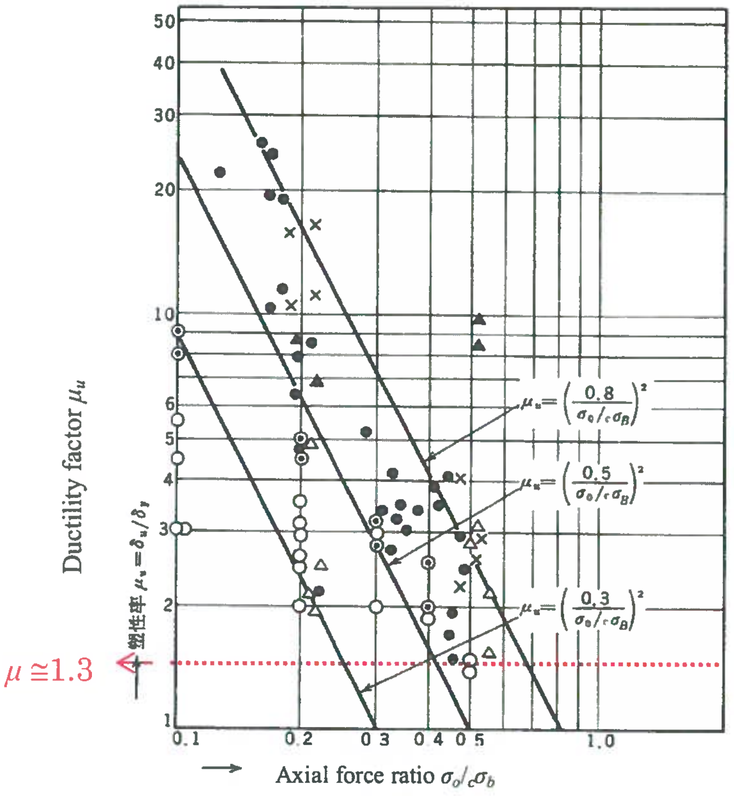


Figure S 2.4 Ductility Factor and Axial Force Ratio Relation (Commentary Figure 14.8 of above AIJ)

SUPPLEMENT 3. FLEXURAL STRENGTH OF COLUMNS BY SIMPLIFIED CALCULATION

Flexural strength of column after retrofit is calculated by Equation (3.3.4-2) of the Guidelines. This is the development of the Equation (A1.1-1) of the J. Standard. Following is the basis of the Equation J. (A1.1-1).

1) Flexural Strength of Column

In case neutral axis is within the section against axial force,

Assuming compressive and tensile re-bars are yield, and following unit/ dimension are supposed.

$\gamma = a_c / a_t = 1.0,\ d_1 = d / D = 0.9,\ d_{cl} = d_1 / D = 0.1,\ d_{tl} = d_t / D = 0.1 \quad x_{nl} = x_n / D$

$k_1 = k_3 = 0.85$ in case $F_c < 28$ N/mm,

Equilibrium of axial force,

$$N = k_1 \cdot k_3 \cdot x_{n1} \cdot b \cdot D \cdot F_c \qquad (1)$$

Neutral axis ratio,

$$X_{n1} = \frac{N}{b \cdot D \cdot F_c \cdot k_1 \cdot k_3} \qquad (2)$$

Flexural strength is calculated from the equilibrium of flexural moment against center of section,

$$M_u = 0.8 a_t \cdot \sigma_y \cdot D + 0.5 N \cdot D \left[1 - \frac{1}{k_3} \left(\frac{N}{b \cdot D \cdot F_c} \right) \right] \qquad (3)$$

k_l = k3 =0.85,

$$M_u = 0.8 a_t \cdot \sigma_y \cdot D + 0.5 N \cdot D \left[1 - \frac{1}{0.85} \left(\frac{N}{b \cdot D \cdot F_c} \right) \right] \qquad (4)$$

More simplified formula,

$$M_u = 0.8 a_t \cdot \sigma_y \cdot D + 0.5 N \cdot D \left[1 - \left(\frac{N}{b \cdot D \cdot F_c} \right) \right] \qquad (5)$$

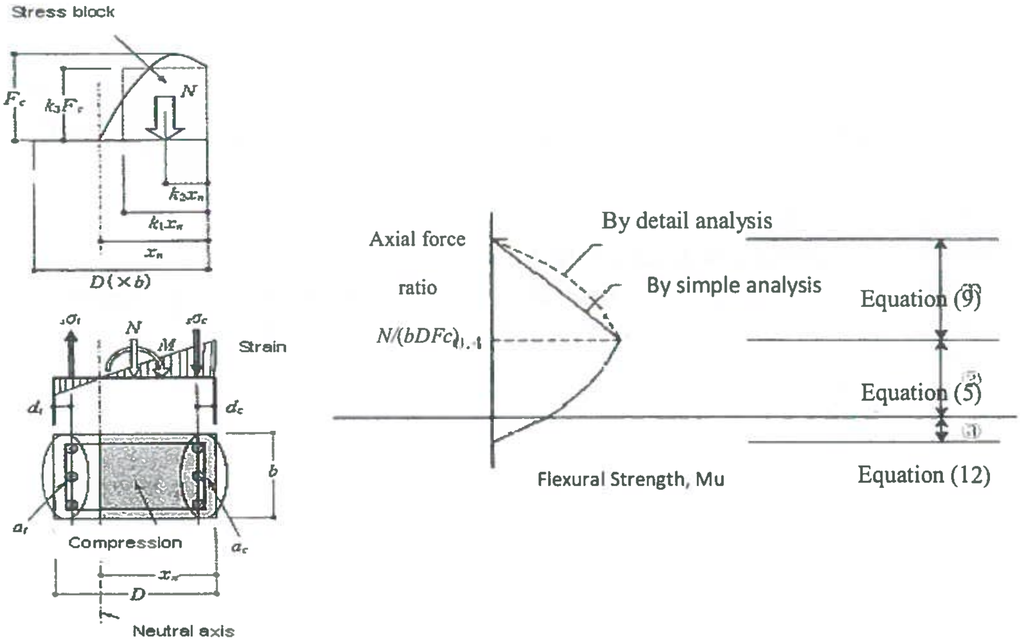


Figure S 3.1 Supposed Stress Distribution of Concrete and Re-bar (left) and Column Interaction (right)
Source; JSCA, Japan, "Seismic Design Guide for Buildings"

Equilibrium axial force, where maximum flexural strength, Differentiating N of eq. (4), M_u where inclination is zero,

$$M_u{}' = 0.5D - \frac{1}{0.85}\left(\frac{N}{b\cdot F_c}\right) = 0\,,\ \frac{N}{b\cdot D\cdot F_c} = 0.5\times 0.85 = 0.425 \qquad (6)$$

Differentiating N of eq. (5),

$$M_u{}' = 0.5D - \frac{N}{b\cdot F_c} = 0\,,\ \frac{N}{b\cdot D\cdot F_c} = 0.5 \qquad (7)$$

This axial stress, $\frac{N}{b\cdot D}$ is assumed as $0.4b\cdot D\cdot F_c$, and flexural strength in case axial stress is higher than this value,

$$M_u \times a_t 0.4b\cdot D\cdot F_c = 0.8a_t\cdot\sigma_y\cdot D + 0.5N\cdot D\times 1 - \frac{N}{b\cdot D\cdot F_c} = 0.8a_t\cdot\sigma_y\cdot D + 0.12b\cdot D^2\cdot F_c \quad (8)$$

Following formula is defined for $0.4bD\times F_c<N<N_{max}$,

$$M_u = \left(0.8a_t\cdot\sigma_y\cdot D + 0.12b\cdot D^2\cdot F_c\right)\times\left(\frac{N_{max} - N}{N_{max} - 0.4b\cdot D\cdot F_c}\right) \qquad (9)$$

In case N is tension, $0 < N < N_{min}$, where tension strength of concrete is ignored,
M_u (at N is zero) = $0.8\,a_t\cdot\sigma_y\cdot D(10)$
min N (at M_u is zero) = $2a_t\cdot\sigma_y(11)$
Inclination of the straight line is $0.4D$,
Flexural strength is shown,
$M_u = 0.8a_t\cdot\sigma_y\cdot D + 0.4N\cdot D(12)$

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 S 3.2.

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.

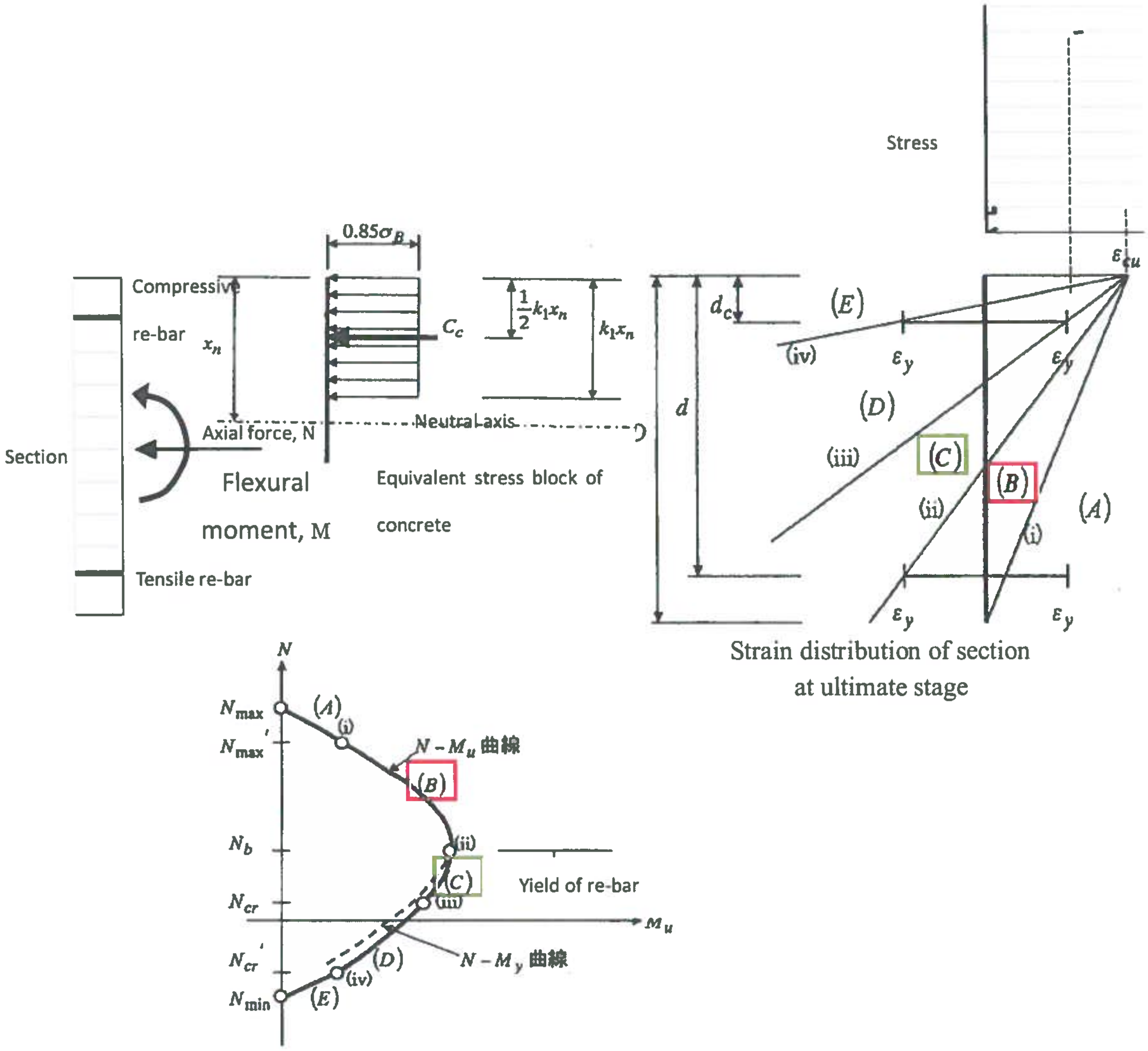


Figure S 3.2 Axial Force of Column and Deformability

Source: Architectural Institute of Japan, Kanto Branch, "Design of Earthquake Resistant Structures, Chapter 5 Reinforced Concrete Structure, (in Japanese)"

SUPPLEMENT 4. FORMULA FOR ULTIMATE SHEAR STRENGTH OF COLUMN USED FOR SEISMIC EVALUATION IN JAPAN

Comparison between Japanese modified Arakawa minimum formula and BNBC formula

Ultimate shear strength of columns shall be calculated with equation

$$Q_{su} = k_r \cdot \left\{ \boxed{\begin{array}{c}\dfrac{0.053 p_t^{0.23}(18+\sigma_B)}{M/(Q\cdot d)+0.12} \\ \text{borne by concrete}\end{array}} + \boxed{\begin{array}{c}0.85\sqrt{p_w \cdot {}_s\sigma_{wy}} \\ \text{borne by shear reinforcement}\end{array}} + \underset{\boxed{\text{increase in ultimate strength from effects of axial force}}}{\boxed{0.1\sigma_o}} \right\} \cdot b \cdot j \qquad \text{(A1.1-2)}$$

[Supplementary provisions 1 Column with modified to consider low strength concrete]

($k_r \leq 1.0$)

Where:

k_r : Reduction factor of low strength concrete ($\sigma_B < 13.5$ N/mm^2) $k_r = 0.244 + 0.056\ \sigma_B$

p_t : Tensile reinforcement ratio (%).

$\sigma_B = F_c$: Compressive strength.

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

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

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

M/Q : Shear span length. Default value is $h_o/2$.

h_o : Clear height of the column.

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

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 above equation and if the value of σ_o is greater than 8N/mm^2, the value of σ_o shall be 8N/mm^2 in using above equation.

The formula for ultimate shear strength of column used in the Japanese standard for seismic evaluation was proposed more than 50 years ago, using the evaluation formula for ultimate shear strength of beam as its original model. Since then, a number of modifications have been made and a term to consider the influence of axial force has been added to arrive at the current evaluation formula.

This evaluation formula is an empirical formula (evaluation of ultimate shear strength based on experiment of elements). In Japan, even though there is also a theoretical formula using the arch and truss theory under the lower bound theory of plasticity, this formula is still used widely for its high reliability in Japan today, as the standard formula.

This formula consists of three terms, the first term being those borne by concrete, the second term for the effects of shear reinforcements, and the third term for the influence of axial force, and by aggregating the three, evaluates the ultimate shear strength of the column. In Japan, concrete strength of 13.5 N/mm^2 or higher are subject to the scope, under the Japanese standard for seismic evaluation. However, in Bangladesh, in many cases, concrete strength of existing buildings falls short of 13.5 N/mm^2. Assuming such cases, by considering k_r, the reduction factor for low strength concrete proposed in Japan, the lower limit of applicable scope of concrete strength has been lowered from 13.5 N/mm^2 to 9 N/mm^2. The outline of Japanese evaluation formula for ultimate shear strength constitution is indicated below:

Q_{su} = Reduction factor for low strength concrete × (borne by concrete + borne by shear reinforcements + increase in ultimate strength from effects of axial force)

A paper summarizing experiments conducted in Japan (277 samples) and the influence of major parameters on this equation has been published and is indicated below:

The paper is a collection of experimental data concerning RC columns for experiments conducted between 1983 and 2008. Of the data collected, an outline of the 277 samples and their results, as well as the calculated results, are as indicated in Table S 4.1.

Table S 4.1 Summery of Collected Sample Data

Series of compressive concrete strength		Number of Data	Dimension B·D (mm)	Shear span ratio	Compressive strength of concrete σ_B	Tensile reinforcing		Shear reinforcing bar			Axial force	
Name	Range					Tensile reinforcement ratio	Yield Strength σ_{wy}	Shear reinforcement ratio P_w(%)	Yield Strength σ_{wy}	Unit of shear reinforcment $p_w \cdot \sigma_{wy}$	Axial stress σ_0 (N/mm²)	Axial force ratio η
L series	$\sigma_B < 15$	21	200～400 (296)	0.8～2.0 (1.43)	3.3～14.3 (10.7)	0.24～1.49 (0.79)	335～908 (441)	0.30～1.79 (0.30)	252～587 (345)	0.14～6.0 (1.08)	1.3～5.6 (3.1)	0.15～1.05 (0.33)
N series	$15 \leqq \sigma_B \leqq 36$	128	200～400 (283)	1.0～2.0 (1.48)	15.0～36.0 (28.0)	0.42～2.25 (1.00)	328～1098 (596)	0.06～1.76 (0.54)	209～1455 (600)	0.14～10.7 (3.16)	2.0～21.8 (8.2)	0.08～0.70 (0.28)
H series	$36 < \sigma_B \leqq 60$	60	200～400 (330)	1.0～2.0 (1.42)	37.0～60.0 (44.1)	0.60～1.45 (0.97)	378～946 (509)	0.10～1.60 (0.55)	327～1721 (1038)	0.58～11.2 (4.76)	4.0～26.8 (13.5)	0.10～0.60 (0.31)
SH series	$60 < \sigma_B$	68	250～400 (319)	1.0～2.0 (1.50)	61.0～116 (80.4)	0.61～1.33 (0.93)	349～998 (646)	0.15～1.56 (0.74)	365～1373 (893)	1.51～15.7 (6.58)	9.8～37.6 (23.0)	0.15～0.50 (0.29)

Reference: KITANI, E etc. Discussion of adaptability investigation a case for the modified ARAKAWA MINIMUM FORMULA, 2011

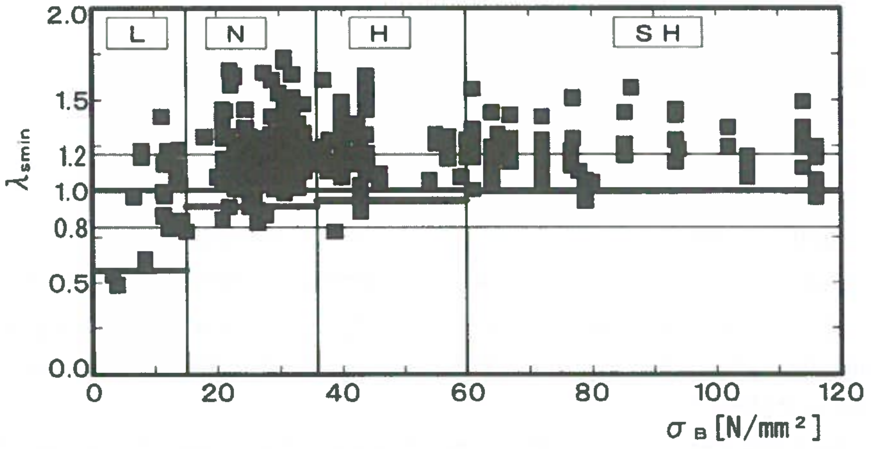


Figure S 4.1 Relationship between λ_{smin}(= Q_{max}/Q_{smin}) and σ_B(compressive strength of concrete)

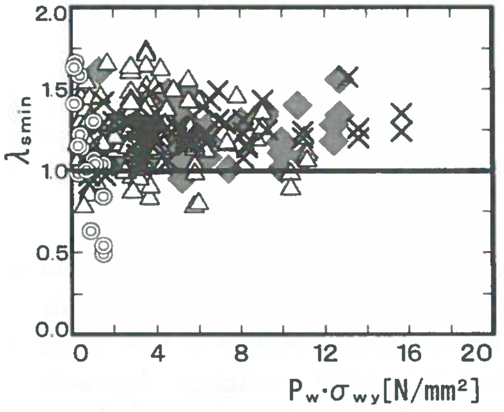


Figure S 4.2 Relationship between λ_{smin} and $p_w \cdot \sigma_{wy}$(Unit of shear reinforcement)

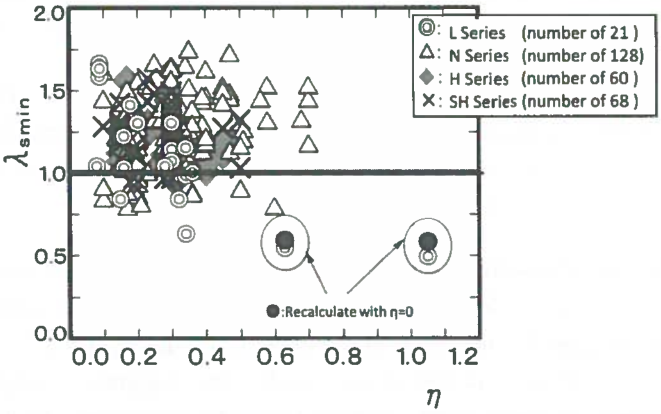


Figure S 4.3 Relationship between λ_{smin} and η (Axial force ratio)

From the comparison of the maximum value from this experiment with the calculated value, in general, with the exception of the range of low strength concrete, equation (A1.1-2) evaluates the lower bound of the experiment results. In this comparison no consideration has been given for the reduction factor for low strength concrete.

Here, in order to grasp the outline of the relationship between the shear evaluation formula for columns adopted by BNBC 93 and those of Japan, a comparison of calculation results using major parameters shall be conducted. Shear evaluation formula by BNBC 93 is as indicated below:

Design for shear shall be based on

$V_u \leq \emptyset \cdot V_n$

where $V_n = V_c + V_s$ and $\emptyset = 0.85$ 【BNBC-93 6.2.7,Part 6】

For members subject to axial compression,in additon to flexure and shear

$$V_c = 0.17 \cdot (1 + 0.073 \cdot N_u/A_g) \cdot \sqrt{f'_c} \cdot b_w \cdot d$$

$$V_s = A_v \cdot f_y \cdot d/s$$

N_u:*Axial load*

A_g:*Gross area of section*

b_w:web width

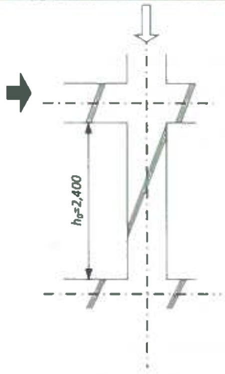
d :distance from extreme compression fibere to the centroid of tensile reinforcement

A_v:Area of shear reinforcement within a distance s

s :Spaceing of shear reinforcement

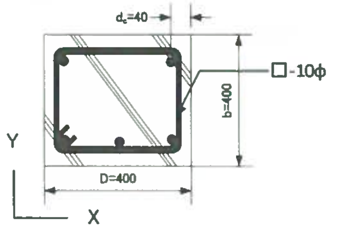
Calculation has been conducted on the member indicated below, using the same parameters used for the comparison between the experimental values and the calculated values indicated above.

Shear strength comper between JNBC and BNBC



Inner height: h_0= 2,400 *(mm)*

M/Q= 3.33⇒ 3.00



bXD= 400 X 400 *(mm)*

d_c= 40 *(mm)*

<reinforcing bar>

Tensile reinforcing bars	3- 22ϕ	a_t=	*1140.4* (mm^2)
		p_t=	*0.713* (%)
Total reinforcing bars	8- 22ϕ	a_g=	*3041.1* (mm^2)
Compressive strength of existing concrete (F_{c1}):		N/mm^2	13.5
Tensile strength of existing rebar(σ_y):		N/mm^2	400
Tensile strength of Shera reinforcement (σ_{wy}):		N/mm^2	280

The solid lines in the graphs are the evaluated results of the formula indicated in this literature. The coarse broken lines indicate the results of the original equation for each (Reduction factor for Bangladesh and low strength and upper bound of axial force $\sigma_0 \leq 8.0$ N/mm^2 for Japan's seismic evaluation are not considered). The thin broken lines and dashed lines indicate the proof stress by concrete + axial force and the proof stress by shear reinforcement bars for each equation.

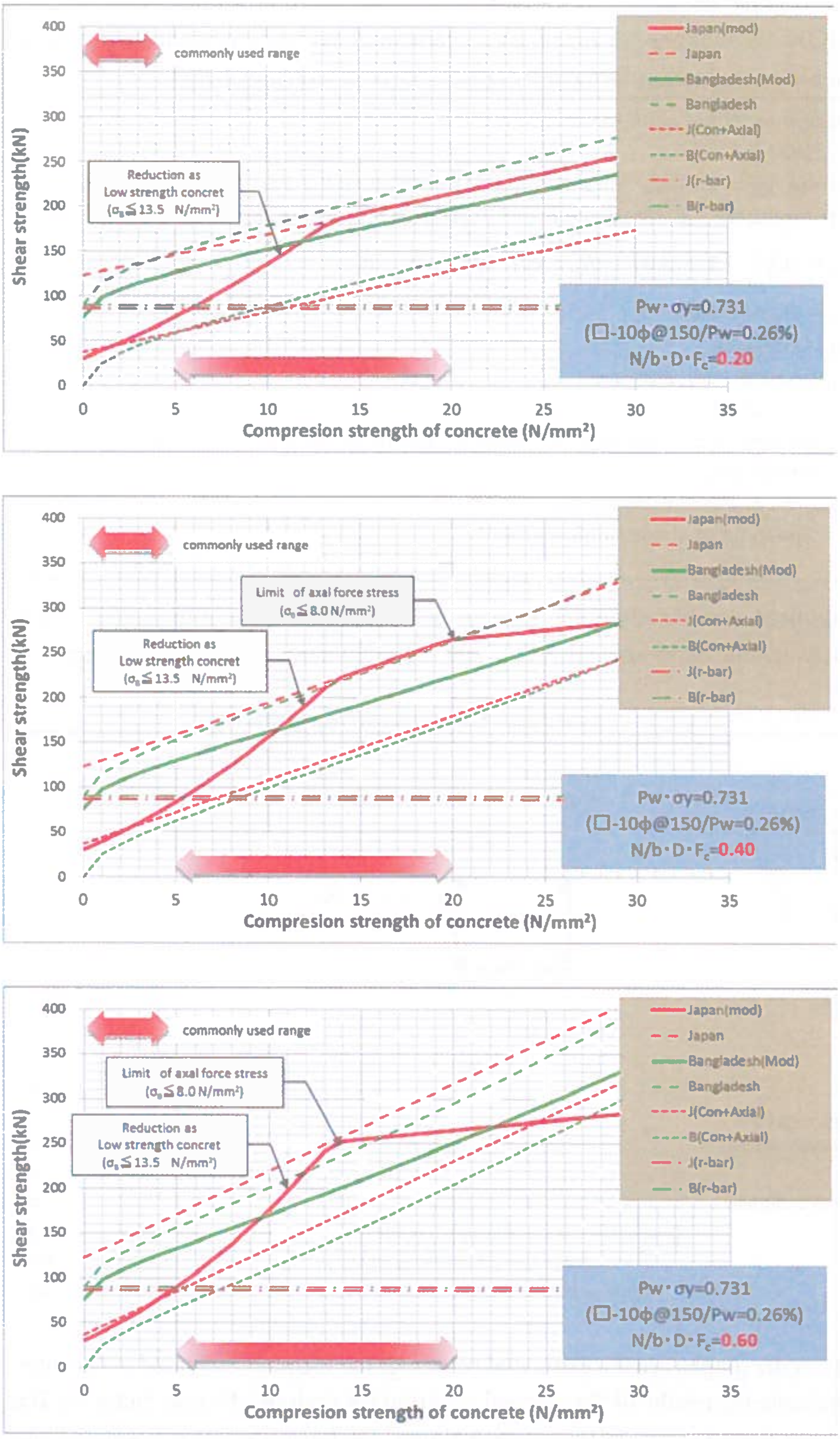


Figure S 4.4 Influence of difference in the strength of concrete on shear strength evaluation between Bangladesh and Japan(Axial force ratio is different for each graph)

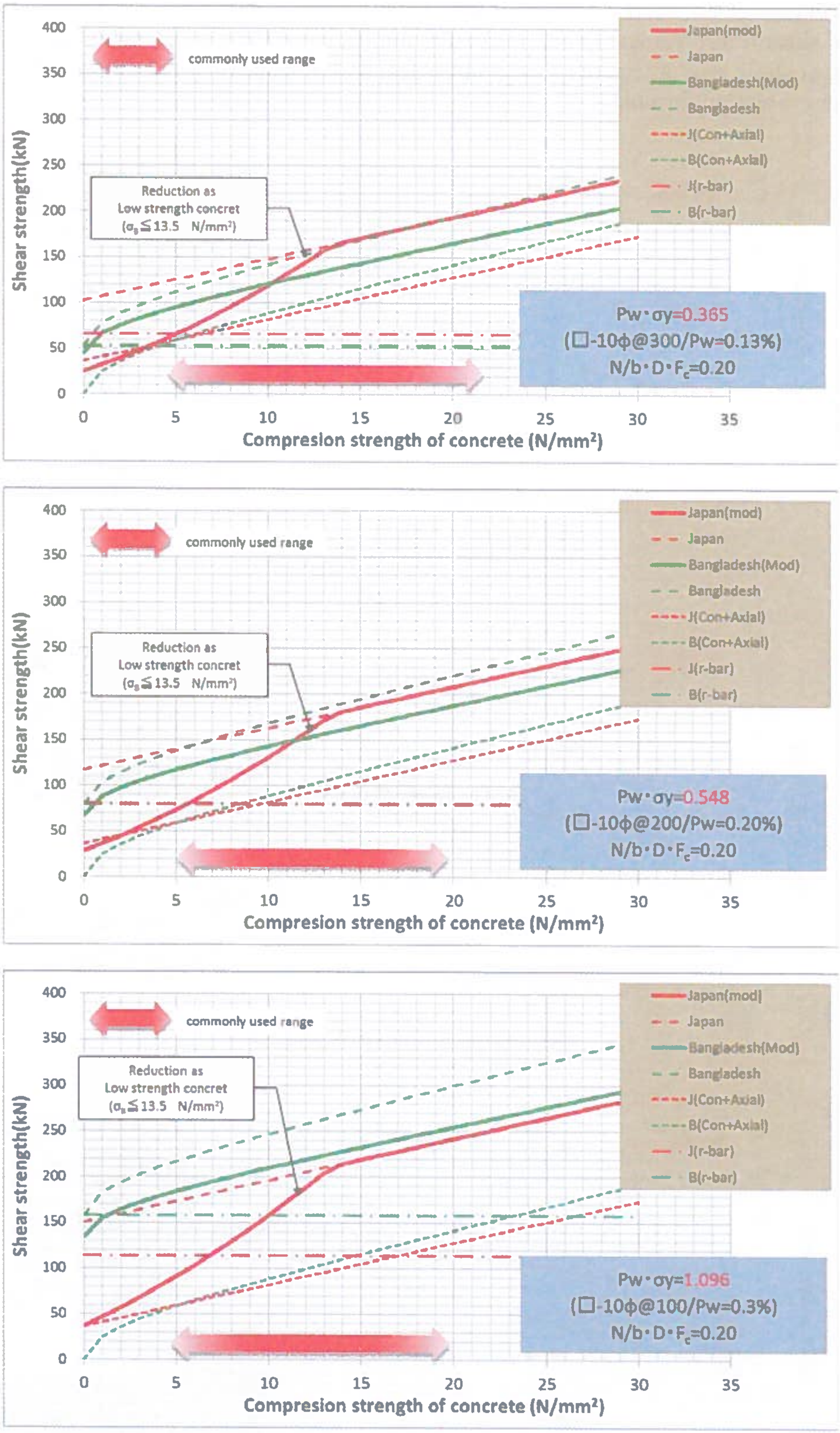


Figure S 4.5 Influence of difference in the strength of concrete on shear strength evaluation between Bangladesh and Japan (Unit of shear reinforcement is different for each graph)

- For concrete + axial force, the evaluated portion is almost parallel indicating that there is no difference in influence of the variance of concrete strength on proof stress of both evaluation formulas.
- As the axial force ratio increases, its influence on proof stress will increase in the Japanese evaluation formula. However, in the evaluation formula actually used in Japan, there is a limit on axial force and the values are modified for this part.

- With the reduction factor for low strength, the difference in proof stress is larger for 13.5 N/mm^2 and less.
- The unit of shear reinforcement ($p_w \cdot \sigma_{wy}$) becomes almost equivalent in evaluation around 0.8.
- As the unit of shear reinforcement ($p_w \cdot \sigma_{wy}$) increases, its influence on proof stress will increase in the Bangladesh evaluation formula.

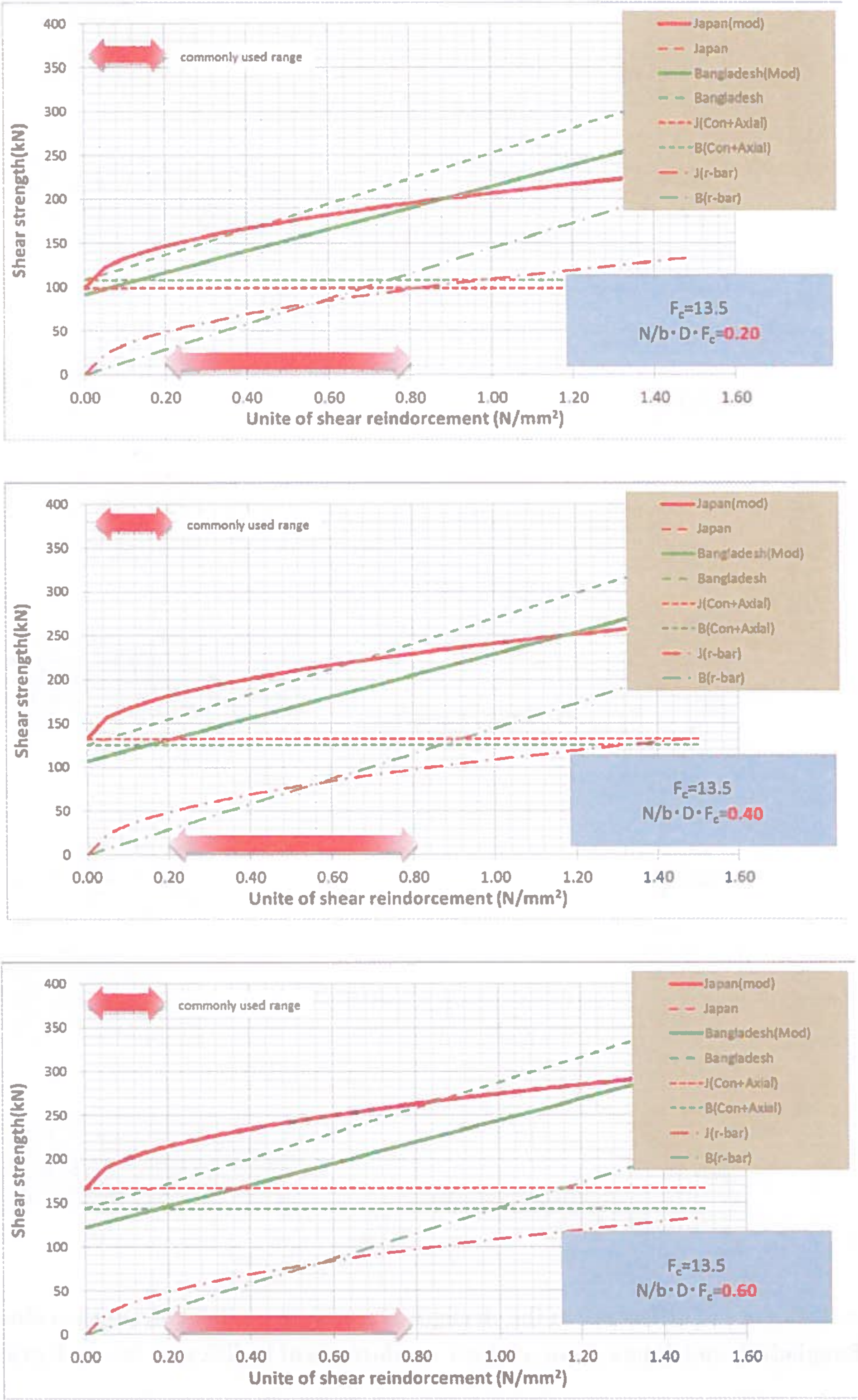


Figure S 4.6 Influence of difference in the unit of shear reinforcement on shear strength evaluation between Bangladesh and Japan (Axial force ratio is different for each graph)

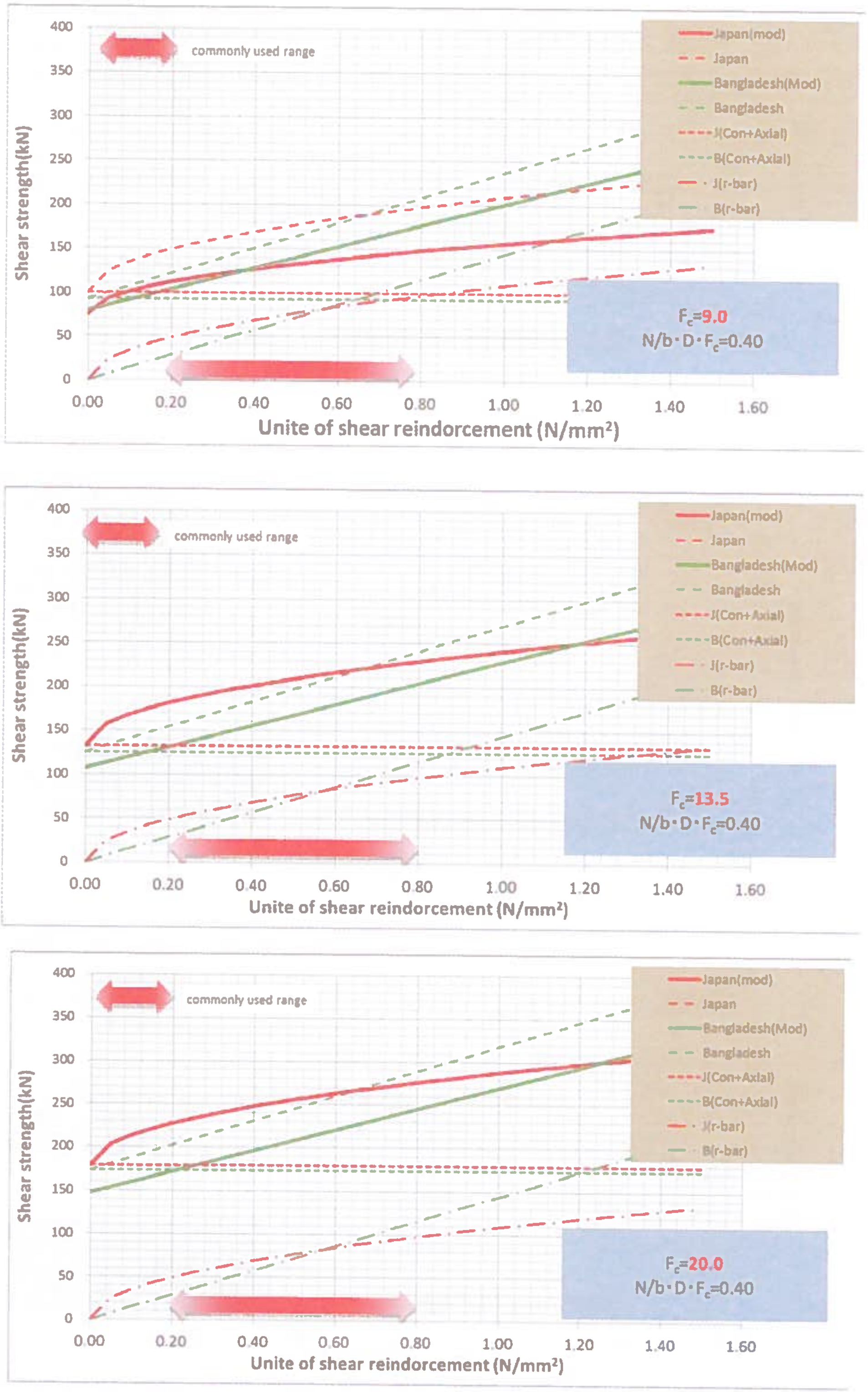


Figure S 4.7 Influence of difference in the unit of shear reinforcement on shear strength evaluation between Bangladesh and Japan (Axial force ratio is different for each graph)

- It can be clearly seen that the influence of unit of shear reinforcement ($p_w \cdot \sigma_{wy}$) on proof stress evaluation is the largest among the major factors.
  (Difference in evaluation for the two equations is large)

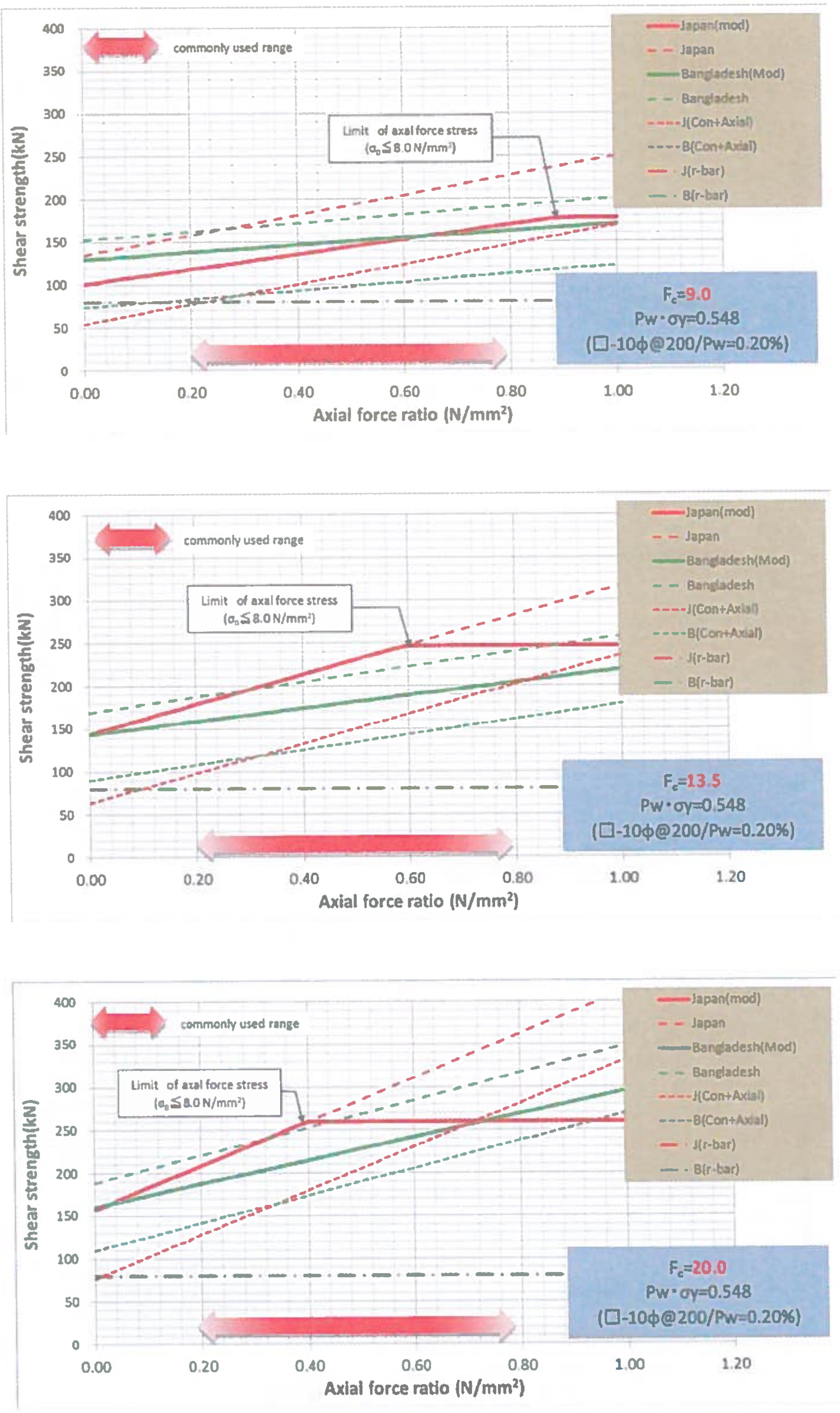


Figure S 4.8 Influence of difference in the axial force ratio on shear strength evaluation between Bangladesh and Japan (Concrete strength is different for each graph)

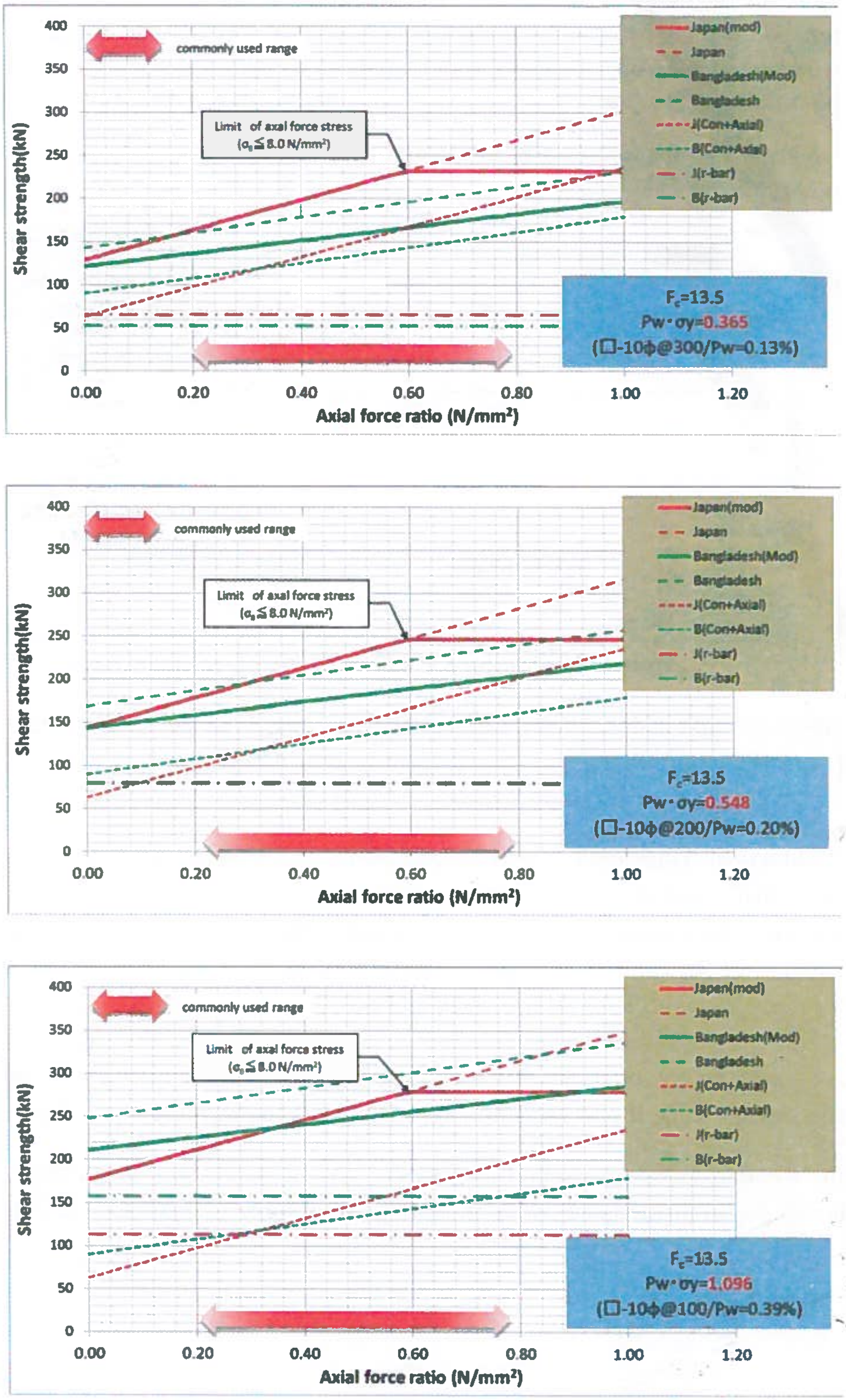


Figure S 4.9 Influence of Difference in the Axial Force Ratio on Shear Strength Evaluation Between Bangladesh and Japan (Unit of Shear Reinforcement is Different for Each Graph)

- There is also a large difference in evaluation between the two equations for axial force ratio. When axial force ratio is high, proof stress for Japanese evaluation formula will increase. However in the actual formula used in Japan, an upper bound is given to the axial force evaluation and the values are modified for this part.

A study was conducted on the influence of p_t, the tensile reinforcement ratio found only in the Japanese evaluation formula.

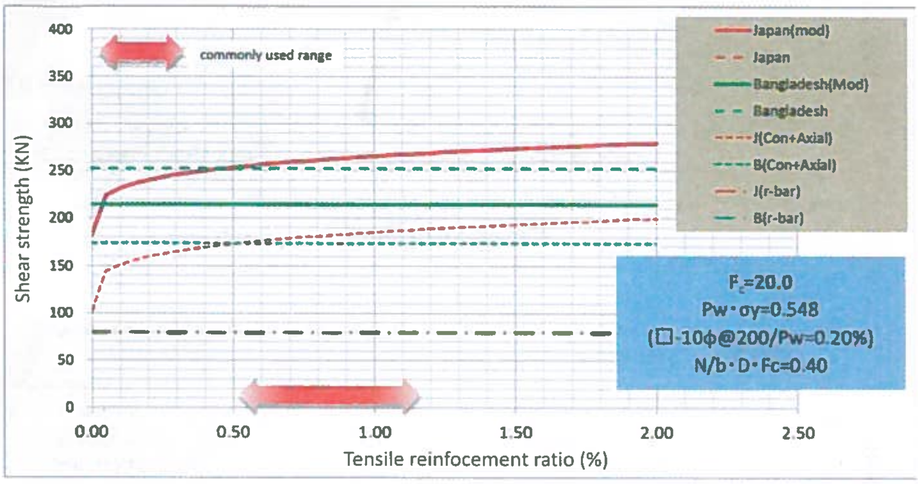


Figure S 4.10 Influence of Tensile Reinforcement Ratio

In the commonly used range in this country, variance of proof stress from tensile reinforcement is small and the influence of tensile reinforcement ratio (p_t) to the evaluation is small.

Summary

- For the buildings in Bangladesh, within the commonly used range of each parameter, the Japanese evaluation formula produced larger results.
- Presuming from the comparison between the experimental results and the Japanese evaluation formula results, the Bangladesh evaluation formula is on the safer side.
- However in case the axial force ratio is small and the concrete strength are low, and then it will fall slightly on the dangerous side.
- In the case where the unit of shear reinforcement ($p_w \cdot \sigma_{wy}$) is 0.80 or larger, it will fall slightly on the dangerous side. However, in real life, this is out of range of common use.

When adopting the Bangladesh evaluation formula for seismic evaluation, it is necessary to consider whether to modify the evaluation and use it, in case the axial force is small and the concrete is of low strength.

SUPPLEMENT 5. 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,

$$V_{ju} = K \cdot \phi \cdot F_j \cdot b_j \cdot D_j \qquad (8.3.1) \quad \text{(Design Guidelines)}$$

Where, κ: Coefficient by the configuration of beam column joint

$\kappa = 1.0$ + type

$\kappa = 0.7$ ⊦ type

$\kappa = 0.5$ L type

φ: Adjustment coefficient with/ without orthogonal beam

$\varphi = 1.0$ with orthogonal beam at both side

$\varphi = 0.85$ other cases

F_j: Standard strength of shear at beam column joint

$F_j = 1.6 \times \sigma_B{}^{0.7}$ (kg/mm^2)

$F_j = 0.8 \times \sigma_B{}^{0.7}$ (N/mm^2)

σ_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,

$b_j = b_b + b_{a1} + b_{a2}$(8.3.2)

Where,

b_b is beam width, b_{ai} is smaller value of $b_i/2$ and D/4, and bi 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

$$Vj = T + C_S{}' + C_C{}' - V_C = T + T' - V_C \qquad \text{(Exp8.3.1)}$$

Column shear force V_c

Beam failure,

$$V_C = 2 \cdot \frac{M_b \cdot L_b / L + M_b{}' \cdot L_b{}' / L}{L_C + L_C{}'} = \frac{\left(M_b + M_b{}'\right) \cdot L_b / L}{L_C} \quad \text{(Exp.8.3.2)}$$

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, $V_c = \dfrac{2 \cdot M_C}{L}$

Where,

L = clear length of column

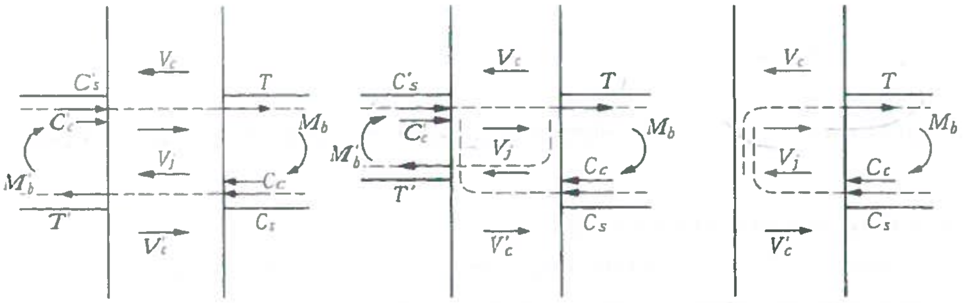


Figure S 5.1 Shear Force at Joint (Figure 8.3.1 of Design Guideline*)

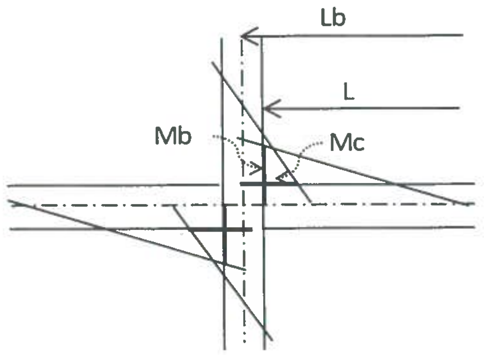


Figure S 5.2 Flexural Moment of Beam and Column

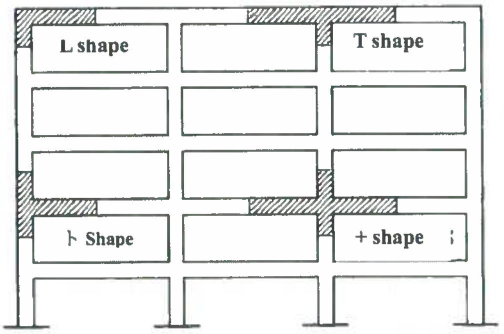


Figure S5.3 Classification of Beam Column Joint (Figure 8.3.2 of Design Guidelines*)

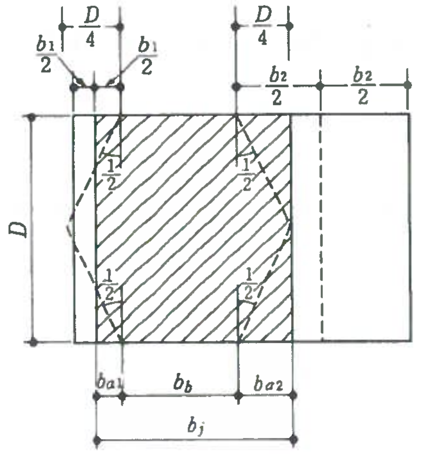
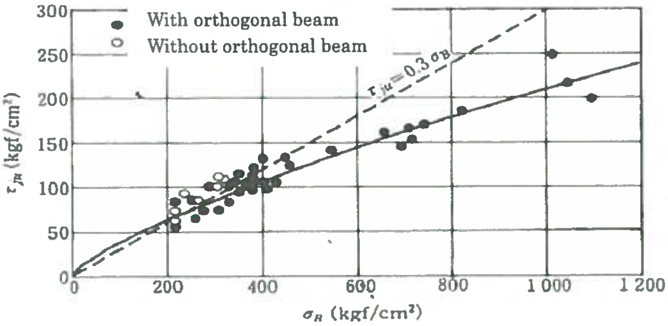
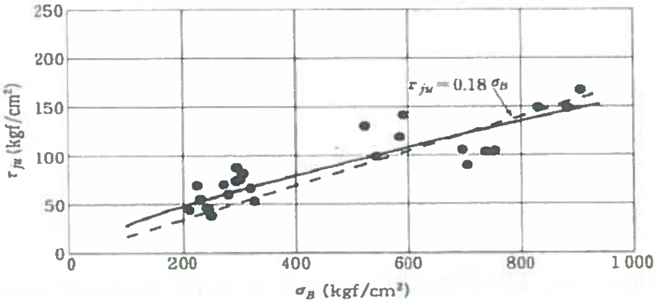


Figure S 5.4 Effective Width of Joint (Figure 8.3.3 of Design Guidelines*)



(a) τ_{ju}-σ_B relation of + type joint (8.3.4)

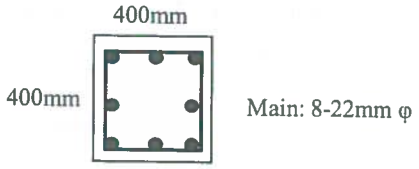


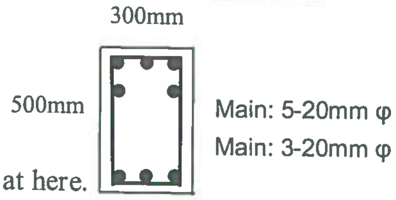
(b) τ_{ju} - σ_B relation of ⊢ type joint

Figure S5.5 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 8-22mmφ,
Beam section, 300mm × 500mm, top main bar 5-20mm, bottom 3-20mm,
Yield strength 400N/mm^2
Concrete strength: F_c = 14kN/mm^2, F_c= 9kN/mm^2
Storey height: 3m
Column span: 5m





Example calculation, one way direction only has been evaluated at here.
κ= 1.0 (+ type), 0.7 (⊦ type)
φ= 1.0, 0.85
$F_j = 0.8 \times \sigma B^{0.7}$ (N/mm^2)
$= 0.8\times 14^{0.7} = 0.8 \times 6.34 =$ 5.07kN/mm^2 (F_c 14N/mm^2)
$b_j = b_b + b_{a1} + b_{a2}$
= 300 + 50/2 +50/2 = 350mm
D_j = 400mm
F_c=14N/mm^2, F_j=5.07N/mm^2

$V_{ju}=\kappa\cdot\varphi\cdot Fj\cdot bj\cdot Dj$	Vju (kN)	$V_j=T+T'-V_c$	Vj (kN)	V_{ju}/V_j
κ=1.0, φ=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
κ=1.0, φ=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
κ=0.7, φ=1.0 (⊦type, orthogonal beam)	496	Beam hinge	537	0.924< 1.0
κ=0.7, φ=0.85 (⊦type, no orthogonal beam)	422	Beam hinge	537	0.786< 1.0

V_{ju}: Shear strength of beam column joint
κ: Coefficient by the configuration of beam column joint
φ: 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=9.0N/mm^2, F_j=3.72N/mm^2, assume equation V_{ju} can be applied for low strength concrete.

$V_{ju}=\kappa\cdot\varphi\cdot Fj\cdot bj\cdot Dj$	V_{ju} (kN)	$V_j = T + T' - V_c$	V_j (kN)	V_{ju}/V_j
κ=1.0, φ=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
κ=1.0, φ=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 < .0 0.607< 1.0
κ =0.7, φ=1.0 (⊦type, orthogonal beam)	365	Beam hinge	537	0.680 < 1.0
κ=0.7, φ=0.85 (⊦type, no orthogonal beam)	310	Beam hinge	537	0.577< 1.0

F_c=18N/mm^2, F_j=6.05N/mm^2

$V_{ju}=\kappa\cdot\varphi\cdot F_j\cdot b_j\cdot D_j$	V_{ju}(kN)	V_j= T+T'-Vc	V_j(kN)	V_{ju}/V_j
κ=1.0, φ=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
κ=1.0, φ=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
κ=0.7, φ=1.0 (⊦type, orthogonal beam)	593	Beam hinge	537	1.10 >1.0
κ=0.7, φ=0.85 (⊦type, no orthogonal beam)	504	Beam hinge	537	0.939>1.0

Column flexural strength, M_c= 284kN.m ($\frac{N}{b\cdot D\cdot F_c}$=0.4), 228.4kN.m (=0.6), 172.9kN.m (=0.8)

b) Shear Strength of Beam Column Joint

F_c 14kN/mm^2

$F_j = 0.8\times\sigma B^{0.7}$ (N/mm^2)

$= 0.8\times14^{0.7} = 0.8\times6.34 = 5.07$kN/mm^2 ($F_c$ 14N/mm^2)

$V_{ju}=\kappa\cdot\varphi\cdot F_j\cdot b_j\cdot D_j$

= 1.0 ×0.85×5.07×350×400 =603.3kN (422.3kN, in case κ=0.7)

F_c 9kN/mm^2, assume same equation is can be applied for low strength concrete.

$F_j = 0.8 \times \sigma B^{0.7}$ (N/mm^2)

$= 0.8 \times 9^{0.7} = 0.8\times6.34 = 3.72$kN/mm^2

$V_{ju} = \kappa\times\varphi\times F_j\times b_j\times D_j$

= 1.0 ×0.85×3.72×350×400 =443kN (310kN, in case κ=0.7)

c) Shear Force at Beam/Column Yield (Design Shear Force) V_j

$V_j=T+C_s'+C_c'-V_c=T+T'-V_c$ (Explanation 8.3.1)

Beam failure, Column shear force V_c

$$V_c=\frac{2\left(M_b\cdot L_b/L+M_b'\cdot L_b'/L'\right)}{L_c+L_c'}=\frac{\left(M_b+M_b'\right)\cdot L_b/L}{L_C}\qquad\text{(Explanation 8.3.2)}$$

V_c= (251+151) × 5.0/4.6/3.0

= 146kN (251×5.0/4.6/3=91kN, ⊦type)

Column failure,

V_c= 2×M_c/ L (clear length of column)

Flexural strength of beam

$M_b = 0.8\times a_t\times\sigma_y\times D$

= 251kN·m (5-20mmφ)

=151 kN·m (3-20mmφ)

Flexural strength of column, based on previous calculation of Supplement 6.

Equilibrium at the center of joint

F_c=14.0N/mm^2

(M_b+M_b')×L_b/L= 437kN·m < 2·M_c·L_c/L= 2×253×3.0/2.5=607kN·m (N=0.4×bDF_c) Beam hinge

< 209 =502 0.6 Beam hinge

> 165 =396 0.8 Column hinge

(396/437=0.91)

Shear force when beam/column yield

F_c 14N/mm^2

$V_j=T+T'-V_c$

= (628 (5-φ20) + 377 (3-φ20)) – 146 M_c is calculated as N=0.4×b·D·F_c

(ditto) -146 0.6

(ditto)×0.91 - 132 0.8

=1005- 146

1005 -146

915- 132

= 859kN > 603kN (V_{ju}, shear strength of joint of + type) 859/603= 1.42, shear failure,

859 >

783 >

In case of ⊦type (Beam hinge)

V_j= 628 – 91

= 537 > 422kN (V_{ju}, shear strength of joint of ⊦type) 537/422=1.27

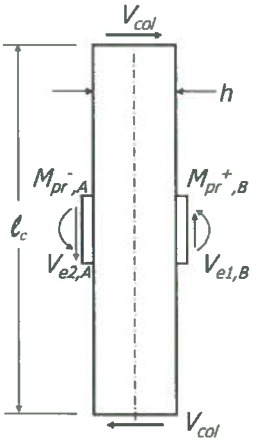
537 >

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



$$V_{col} = [(M_{pr}^+{}_{,B} + M_{pr}^-{}_{,A}) + (V_{e2,A} + V_{e1,B})\frac{h}{2}]/\ell_c$$

Figure S 5.6 Free Body Diagram of Column to Use Column Shear V_{col}

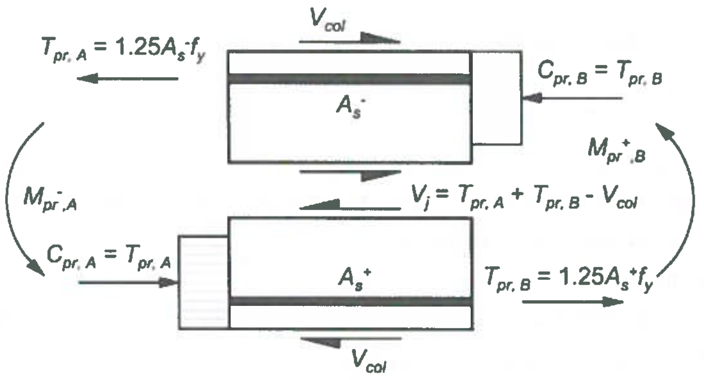


Figure S 5.7 Joint Shear Free Body Diagram

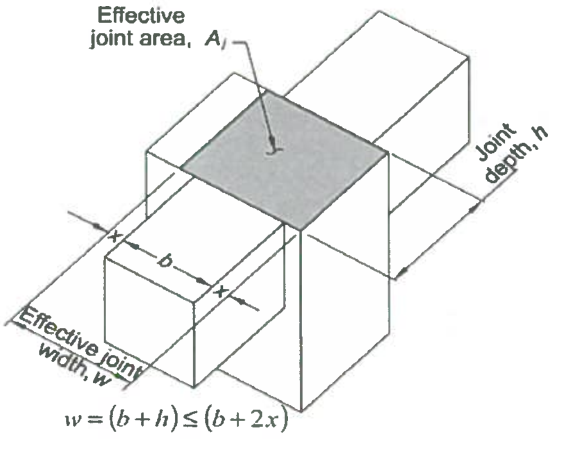


Figure S 5.8 Definition of Beam-Column Dimension

Tensile Force of Beam Rebar:

$T_{pr,A}$= 5×314 × 1.25 × 400 = 785 kN (628kN)

$T_{pr,A}$= 3 × 314×1.25 × 400 = 471 kN (376.8kN)

Joint Shear for f'_c =14 MPa

For top bar, a = 5×314×1.25×400 /(0.85×14×300) = 220mm (176mm)

So, $M_{pr,\,A}$= 5 × 314×1.25×400 (400 – 220/2) = 227650 kN-mm (196000 kN-mm)

For bottom bar, a'= 3×314×1.25×400/(0.85×14×300) = 132mm (105.5mm)

So, $M_{pr,\,B}$= 3×314×1.25×400 (400 – 132/2) = 176150 kN-mm (145900 kN-mm)

Beam end shear = (227650 + 176150)/ (5000 – 400) = 877.8 kN (743.3 kN)

V_{col} = (227650 + 176150 + 877.8×400)/ 3000 = 146.1 kN (123.8 kN)

So, Joint force if beam hinge form

V_j = 785 + 471–146.1 = 1109.9 kN (881 kN)

Joint Shear for f'_c =9 MPa

For top bar, a = 5 ×314×1.25×400/(0.85×9×300) = 342mm (273.6mm)

So, $M_{pr,\,A}$ = 5×314×1.25×400 (400 – 342/2) = 179760 kN-mm (165300 kN-mm)

For bottom bar, a’ = 3×314×1.25×400/(0.85×9×300) = 205mm (164.2mm)

So, $M_{pr,\,B}$ = 3×314×1.25×400 (400 – 205/2) = 158960 kN-mm (134900 kN-mm)

Beam end shear = (179760+ 158960)/ (5000 – 400) = 736.4 kN (652.6 kN)

V_{col} = (179760 + 158960 + 736.4 × 400)/ 3000 = 122.7 kN (108.8 kN)

So, Joint force if beam hinge form

V_j = 785 + 471 – 122.7 = 1133.3 kN (896 kN)

Shear Strength of Joint

Shear strength capacity of the joint shall not be greater than as specified below –

$V_{ju} = 1.66\,\phi\sqrt{f_c'}\cdot A_j$ for joint confined on all four sides

$= 1.24\,\phi\sqrt{f_c'}\cdot A_j$ for joint confined on three faces or two opposite faces

$= 1.0\,\phi\sqrt{f_c'}\cdot A_j$ for others

Since 0.75 × 400 (column width) ≤ 300 (beam width), so the joint is confined.

Since the beam passes through the centre of the column

b_j = 400mm

d_j = 400mm

A_j = 400 × 400 = 160000mm^2

For f'_c = 14 MPa:

V_{ju} = 844.7 kN (993.8 kN) for joint confined on all four sides

= 631.0 kN (742.4 kN) for joint confined on three faces or two opposite faces

= 508.8 kN (598.6 kN) for others

For $f'_c = 9$ MPa:

V_{ju} = 677.3 kN (796.8 kN) for joint confined on all four sides

= 505.9 kN (595.2 kN) for joint confined on three faces or two opposite faces

= 408.0 kN (480.0 kN) for others

For $f'_c = 14$ 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 $f'_c = 9$ 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 × 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< 13.5N/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 2nd 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~ (F=1.5~). It is recommended to control storey deflection angle within 1/124 (F=1.5) or less in case of low strength concrete.

SUPPLEMENT 6. UPPER LIMIT OF DRIFT ANGLE OF THE FLEXURAL COLUMN, ${}_cR_{max(t)}$ BY TENSILE REINFORCEMENT RATIO p_t

1. BOND SPLIT FAILURE

In case there is a possibility of a bond split failure of the main reinforcement, the upper limit of drift angle of the flexural column ${}_cR_{max(t)}$, will restrict the upper limit of the drift angle of the flexural column as its deformability will decrease.

From the results of past experimentation in Japan, it has been confirmed that bond split failures tend to increase when tensile reinforcement bar quantity exceeds 1%. Therefore, in the seismic evaluation standard in Japan, for columns with tensile reinforcement bar quantity of 1.0%or more, distortion performance is limited to F=1.0 (member distortion angle 1/250).

However, in the evaluation, columns with a shear-span depth of column ratio ($M/(Q{\cdot}D)$ of 3 or more or with round steel bar main reinforcement are excluded from the subject of study, since the possibility of a bond split failure is low. Another seismic evaluation system in Japan[3)] indicates that a bond split failure is easy to occur especially in members where the shear-span depth of column ratio is around 1.5 with deformed bars as its main reinforcement.

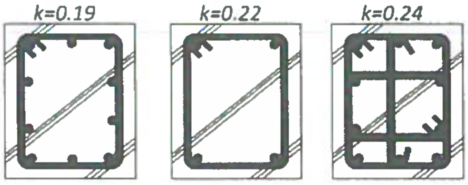
Shear-span depth of column ratios of almost all the buildings in Bangladesh are 3.0 or more (ignoring the effects of brick walls) leading to believe the possibility of destruction from a bond split failure to be low. However, it is difficult to predict that there are no possibilities, when the conditions are different from the members in Japan, such as concrete strength, the combination of rebar strengths, rebar quantity and their arrangements.

Therefore, a study has been made to include this regulation in the seismic evaluation standard for Bangladesh.

There are no experimental or building damage data from earthquakes covering a bond split failure in Bangladesh. Therefore, using the bond failure strength formula used in Japan, a study for a bond split failure was conducted on the members of the seven buildings where seismic evaluation were conducted. The study method of Japan's bond split failure is shown below:

Bond failure strength

$$\underbrace{n\cdot\varphi^2\cdot\sigma_y/4}_{\text{Design stress}} \le \underbrace{k\cdot\sqrt{F_c}\cdot Y\cdot(a-l_h)}_{\text{Strength}}$$



where:

φ :Diameter of the vertical reinforcing bar (mm)

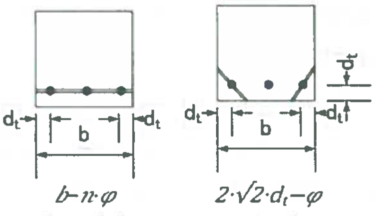
σ_y:Yeild strength of reinforcing bar (N/mm²)

$$k:\begin{cases}0.19\cdots\cdots\text{No supplemental tie}\\0.22\cdots\cdots\text{Vertical reinforcing bar at only four coners}\\0.24\cdots\cdots\text{All virtical reinforcing bars are hooked}\end{cases}$$

F_c:Compressive concrete strength (N/mm²)

Y : Length of bond splitting (mm)

$$Y=\begin{cases}b-n\cdot\varphi & : b/n\le 2\cdot\sqrt{2}\cdot d_t\ (\text{Side splitting})\\2\cdot(2\cdot\sqrt{2}\cdot d_t-\varphi) & : b/n> 2\cdot\sqrt{2}\cdot d_t\ (\text{Coner splitting})\end{cases}$$



(Side splitting) (Coner splitting)

n :Total number of the tensil reinforcing-bar (When $b/n>2\cdot\sqrt{2}\cdot d_t$ $n=2$)

b :width(mm)

d_t:Distance from the centroid of the tensil reinforcing-bar to the surface of tensil side of concrete

a : shear span M/Q(mm)

l_h:Length of lost bond strength of vertical reincorcing-bar(mm) $l_h=0.5\cdot(M/(Q\cdot D))\cdot d$
for $1.5\le M/(Q\cdot D)\le 3.0$

D :Depth of olumn.(mm)

d : Effective depth of column. D-50mm may be applied.(mm)

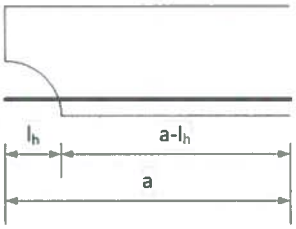


Table S6.1Overviewof the Seven Buildings where Seismic Evaluation was Conducted

Building Name	A	B	C	D
Usage	OFFICE	HOSPITAL	HOSPITAL	HOSPITAL
Structural Type	Reinforced concrete	Reinforced concrete	Reinforced concrete	Reinforced concrete
Frame Type	Moment Resisting Frame	Moment Resisting Frame	Moment Resisting Frame	Moment Resisting Frame
Above Ground story	5	5	5	6
Eaves High(m)	15	24.7	18	-
Year of design	1985	1984	1999	In planning
Aggregate	Brick	Brick	Brick	-

Building Name	E	F	G	
Usage	Garment factory	Garment factory	HOSPITAL	
Structural Type	Reinforced concrete	Reinforced concrete	Reinforced concrete	
Frame Type	Moment Resisting Frame	Moment Resisting Frame	Moment Resisting Frame	
Above Ground story	4	6	5	
Eaves High(m)	15.2	19.3	22.5	
Year of design	2002	2003	1964	
Aggregate	Brick	Brick	Stone	

Table S 6.2 Structural Specifications of Columns Where Calculation Was Conducted (Minimum, Maximum and Average)

	Seismic zone	Complete year	Story	b (orD) width(depth)	ratio of side length	$\sigma_0(N/mm^2)$ Non limite	$h_0(m)$	h0/D	M/Q·d (Non limit)	$F_c(N/mm^2)$ Include(design)	pt(%)	ag/b·D(%)	Hoop spaing (mm)	pw(%)	pw·wσy	s/db	τ (N/mm^2)	η (N/mm^2)	c τ u/Fc
Minmum	0.15	1964	6	250	0.33	–	2.53	3.60	1.93	6	0.161	0.643	75	0.104	0.290	3	0.00	0.0	0.000
Maximum	0.20	2013	4	750	1.00	17.50	4.01	15.80	9.84	25	1.683	4.714	300	1.040	4.180	15	2.10	1.9	1.640
Average	–	1999	–	–	–	3.80	3.30	8.40	4.90	–	0.689	1.907	229	0.329	1.207	9	0.70	0.4	0.087

2. STUDY USING JAPAN'S BOND SPLIT FAILURE FORMULA

A study was conducted using Japan's bond split failure formula of the possibility of a bond split failure occurring on each member. The results are shown below. The following shows low strength concrete (F_c<13.5 N/mm^2) and for normal concrete, separately.

Results indicate that members with the possibility of a bond split failure are all low strength concrete members.

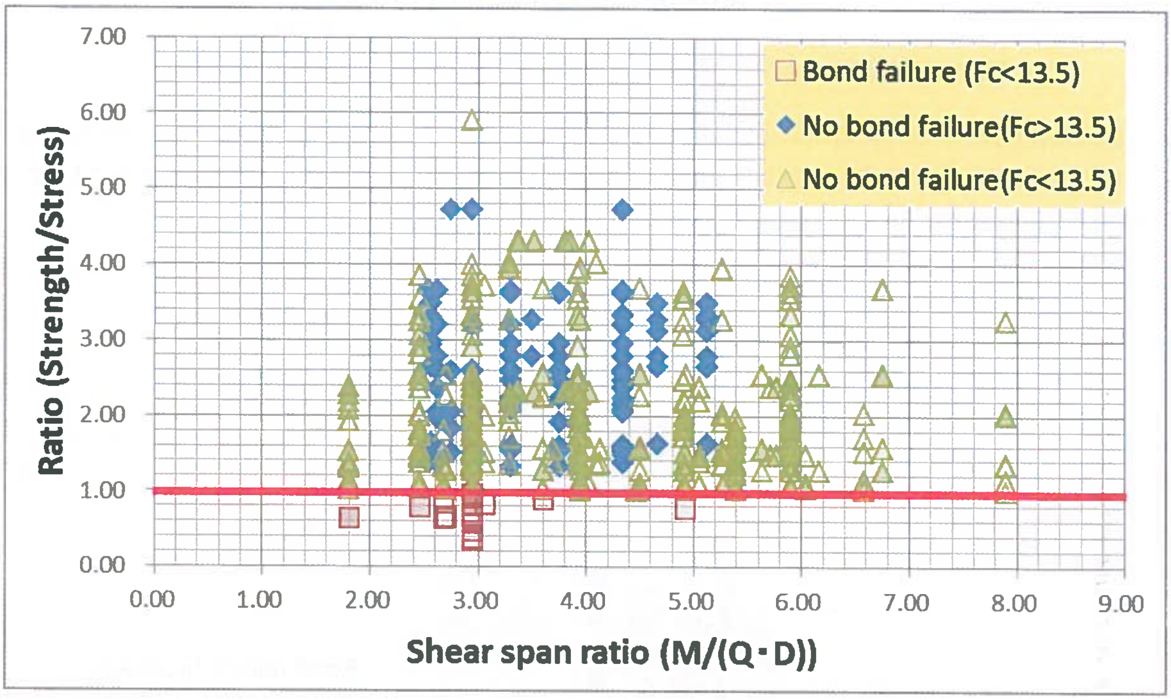


Figure S6.1 Tensile Reinforcement Ratio and Judgment Value

The relationship between the tensile reinforcement ratios and judgment values indicates that there are possibilities of a bond split failure even in members of which tensile reinforcement ratio is lower than 1.00%.

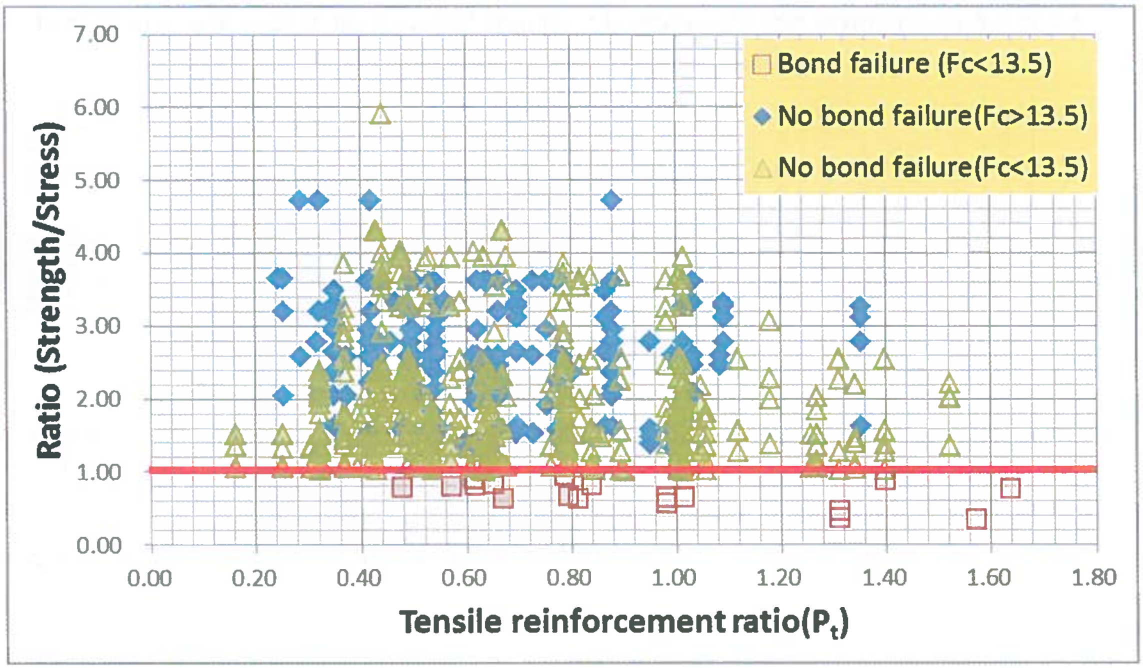


Figure S6.2 Shear-Span Depth of Column Ratio and Judgment Value

The relationship between the shear-span depth of column ratios ($M/(Q{\cdot}D)$) and judgment values indicates that many of the members with possibility of a bond split failure fall the range 3.00 or lower, however, there are some members with the ratio greater than 3.00 with possibility.

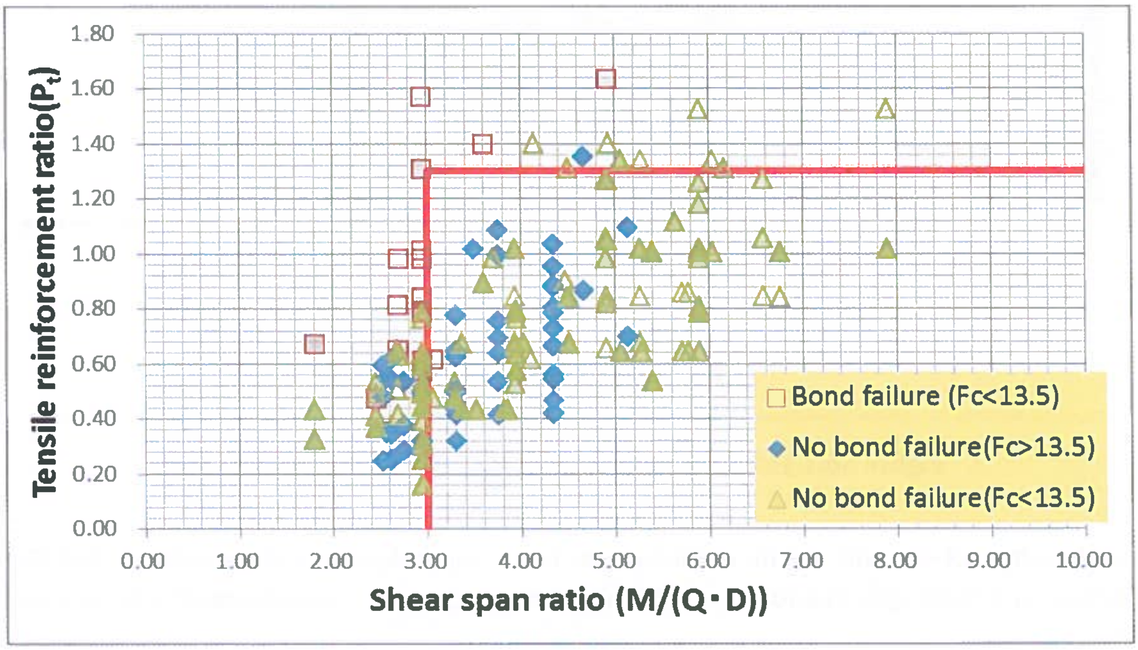


Figure S6.3 Tensile Reinforcement Ratio and Shear-Span Depth of Column Ratio

The relationship between the tensile reinforcement ratios and shear-span depth of column ratios was investigated. Members with possibility of a bond split failure with shear-span depth of column ratio of 3.00 or greater had tensile reinforcement ratio of 1.3% or greater.

3. STUDY OF BOND-SPLIT-FAILURE OF THE COLUMNS, WHICH SHEAR-SPAN DEPTH OF COLUMN RATION IS 3.00 OR LESS (LOW-STRENGTH-CONCRETE ONLY)

Studied the bond-split-failure of the columns, which are low-strength-concrete and have the shear-span depth of column ratio of 3.0 or less.

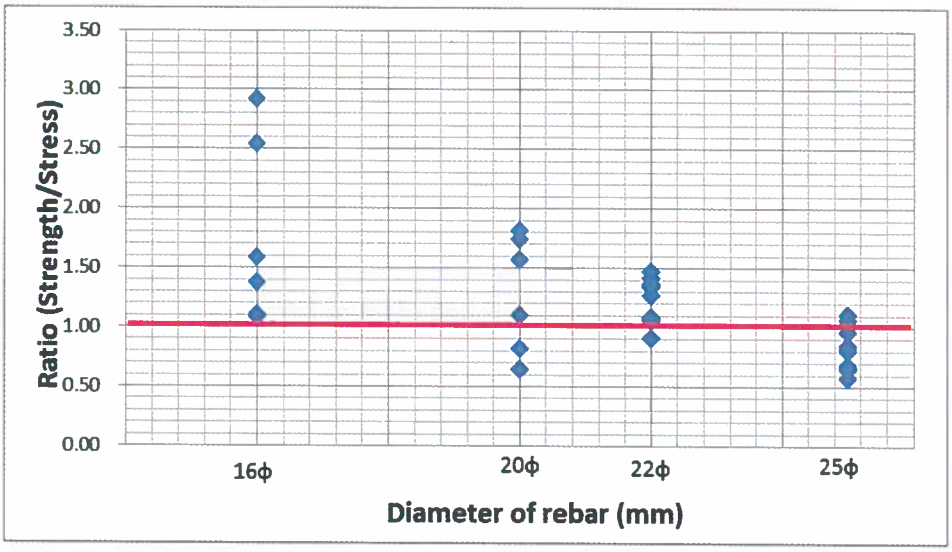


Figure S 6.4 Diameter and Judgment Value

Here is the relation between the diameter of reinforcing-bar and the verdict of bond-split-failure. It shows the possibility of bond-split-failure when the diameter of reinforcing-bar is 20mm or more.

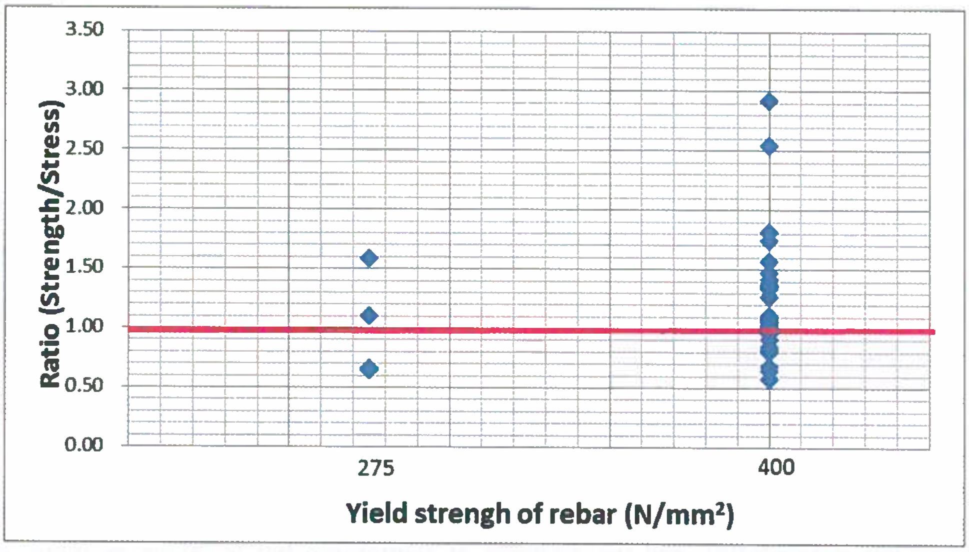


Figure S 6.5 Yield Strength and Judgment Value

Here is the relation between the strength of reinforcing-bar and the verdict of bond-split-failure. It shows that the possibility of bond-split-failure becomes higher when grade400 is used. Possibility of bond-split-failure is not completely erased even the strength of reinforcing bar is grade300.

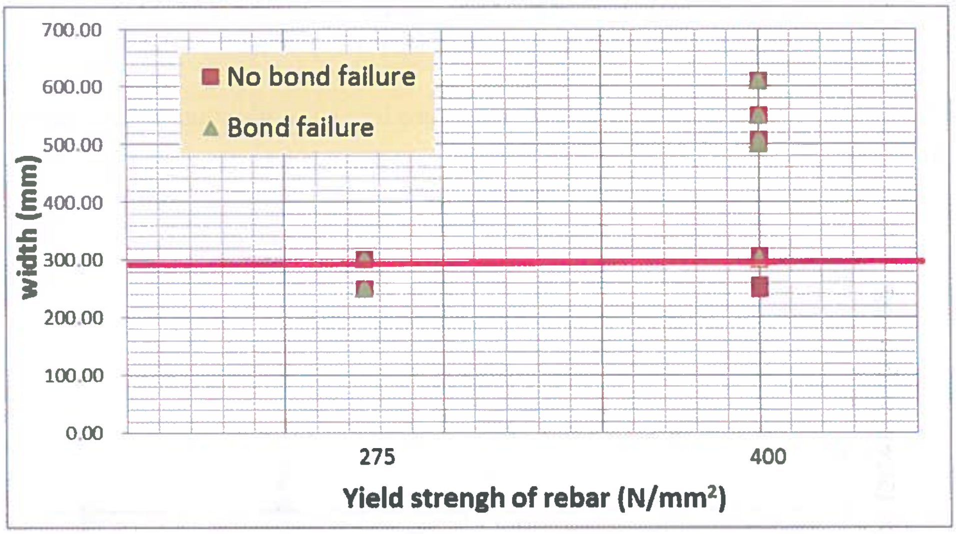


Figure S 6.6 Width and Yield strength

Here is the relation between the strength of reinforcing-bar and the column width. The drawing shows that when the strength of reinforcing bar is Grade300, the conditions of column width of 300mm or less and the diameter of reinforcing-bar of 200mm or more increases the possibility of bond-split-failure.

4. SUMMARY

Bond split failures of members were investigated using the bond split failure formula used in Japan. The results of the investigation conducted for the buildings this time shows that the most members likely to cause bond split failures are the members made of low strength concrete (F_c<13.5N/mm^2) with a shear-span depth of column ratio of 3.0 or smaller or with a tensile reinforcement ratio of 1.3% or greater. Judging from test results in Japan, the possibility of a bond split failure is small because of the large shear-span depth of column ratio.

However, based on the consideration that there are no test results using locally available members or earthquake damage data, this regulation should be retained for the time being.

The following shall be specified based on the study results. The specified range contains members unlikely to cause a bond split failure, however, the regulation shall apply for safety.

Distortion limit of members by tensile reinforcement ratio(P_t)

The condition is limited to the buildings with low-strength concrete(F_c<13.5N/mm^2).
$F = 1.0$ for $P_t \geqq 1.3\%$.
Further study is recommended when the shear-span-depth-of-column-ratio is 3.0 or less, reinforcing bar is Grade400, and the diameter of reinforcing bar is 20mm or more, or when reinforcing bar is Grade300, column width is 300mm, and the diameter of reinforcing bar is 20mm or more.

Reference materials:
1) Seismic evaluation standards and commentary for existing reinforced concrete buildings, revised edition
2) Seismic evaluation standards and commentary for existing reinforced concrete buildings, 2001 revised edition. Both published by the Japan Building Disaster Prevention Association
3) Seismic evaluation and seismic reinforcement design manual for existing buildings, 2012 edition. Published by Japan Association for Building Research Promotion.

SUPPLEMENT 7. REPORT ON SEISMIC EVALUATION OF A SAMPLE BUILDING

Contents

1. General Remarks

1.1 Objective

To find out Seismic capacity of the Sample building by detail seismic evaluation and compare that with required seismic capacity as per BNBC 2015.

1.2 Flow Chart of Seismic Evaluation

The Seismic Evaluation flow diagram is shown in FigureS8-1. The Evaluation method is based on "Manual for Seismic Evaluation of Existing Reinforced Concrete Buildings" published by PWD.

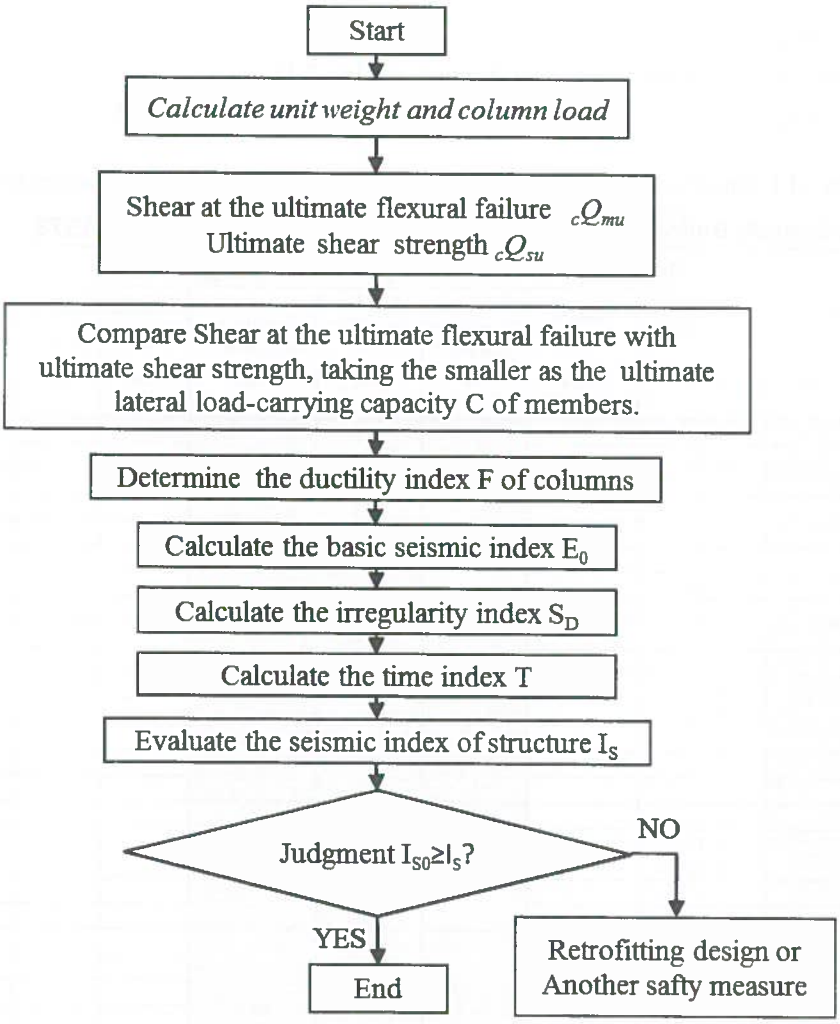


Figure S7.1 Flow Diagram for Seismic Evaluation

2. Site Investigation

The main object of the work site investigation is to prepare as built structural drawings which is essential to do the seismic evaluation.

2.1 Material Testing

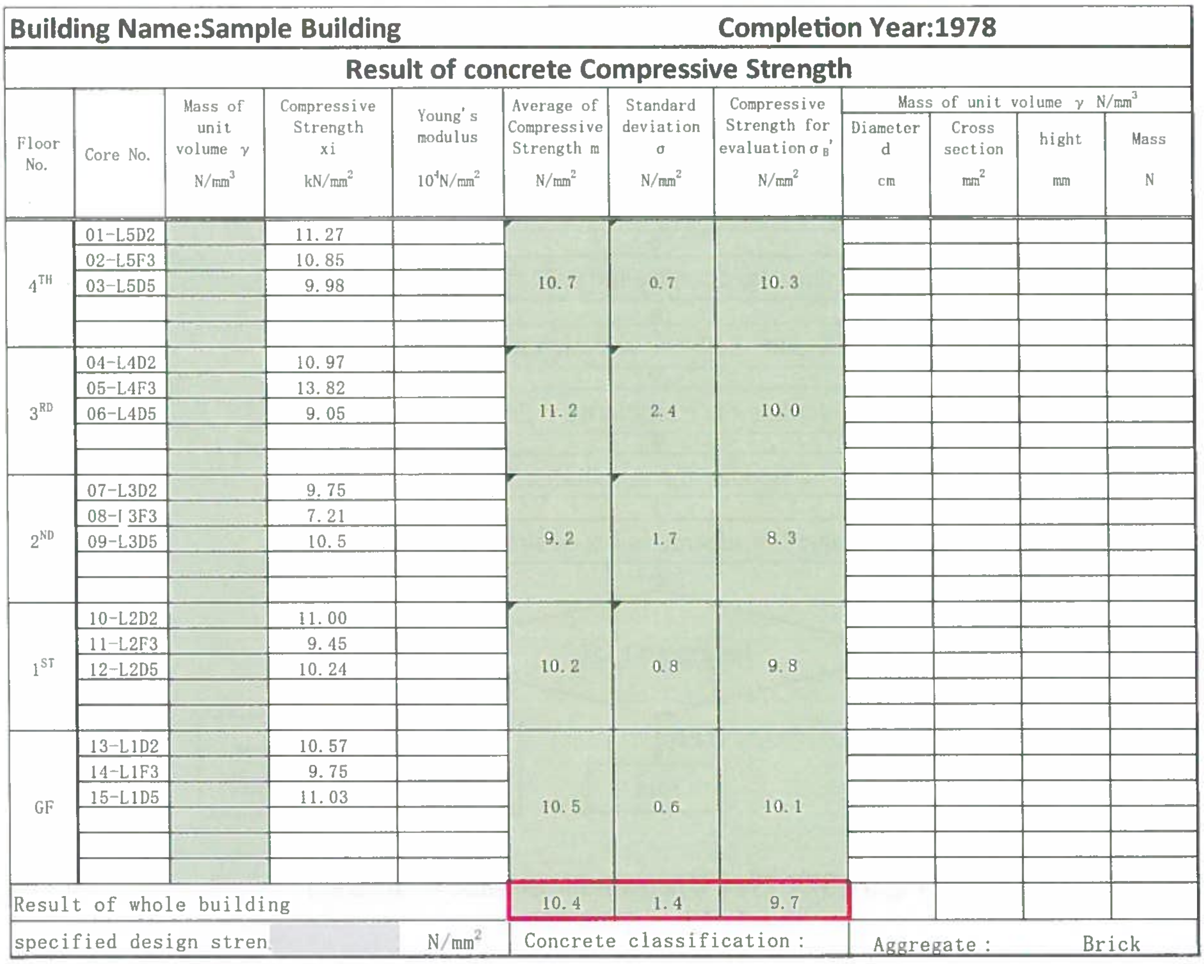
1) Tests (both non-destructive and destructive) are carried out to determine the following properties of materials:

- Concrete Strength
- Carbonation of concrete
- Rebar strength
- Rebound hammer test on RC surface

2) Test result of materials.

- Concrete compressive strength :f_c =9.7 N/mm^2 (Table 2.1)
- Rebar yield strength :f_y=400 N/mm^2(Table 2.2)(Grade 400W)

Table S7.1Results of Concrete Compressive Strength Tests Carried out on Concrete Core Samples

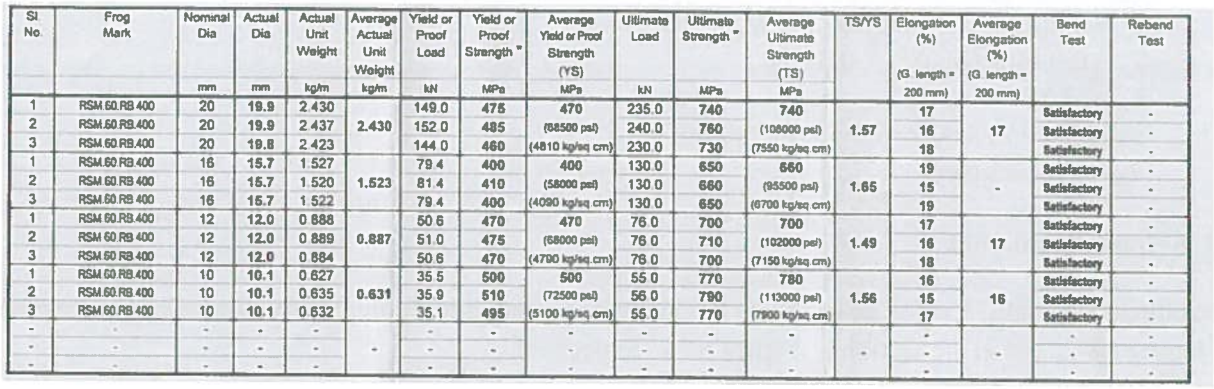


Building Name:Sample Building **Completion Year:1978**

Result of concrete Compressive Strength

Floor No.	Core No.	Mass of unit volume γ N/mm^3	Compressive Strength xi kN/mm^2	Young's modulus 10^4N/mm^2	Average of Compressive Strength m N/mm^2	Standard deviation σ N/mm^2	Compressive Strength for evaluation σ_B' N/mm^2	Mass of unit volume γ N/mm^3			
								Diameter d cm	Cross section mm^2	hight mm	Mass N
4TH	01-L5D2		11.27		10.7	0.7	10.3				
	02-L5F3		10.85								
	03-L5D5		9.98								
3RD	04-L4D2		10.97		11.2	2.4	10.0				
	05-L4F3		13.82								
	06-L4D5		9.05								
2ND	07-L3D2		9.75		9.2	1.7	8.3				
	08-I 3F3		7.21								
	09-L3D5		10.5								
1ST	10-L2D2		11.00		10.2	0.8	9.8				
	11-L2F3		9.45								
	12-L2D5		10.24								
GF	13-L1D2		10.57		10.5	0.6	10.1				
	14-L1F3		9.75								
	15-L1D5		11.03								
Result of whole building					10.4	1.4	9.7				
specified design stren				N/mm^2	Concrete classification :			Aggregate :		Brick	

Table S7.2 Result of Rebar Tensile Strength Test



Sl No.	Frog Mark	Nominal Dia mm	Actual Dia mm	Actual Unit Weight kg/m	Average Actual Unit Weight kg/m	Yield or Proof Load kN	Yield or Proof Strength * MPa	Average Yield or Proof Strength (YS) MPa	Ultimate Load kN	Ultimate Strength * MPa	Average Ultimate Strength (TS) MPa	TS/YS	Elongation (%) (G. length = 200 mm)	Average Elongation (%) (G. length = 200 mm)	Bend Test	Rebend Test
1	RSM.60.RB.400	20	19.9	2.430	2.430	149.0	475	470 (68500 psi) (4810 kg/sq cm)	235.0	740	740 (108000 psi) (7550 kg/sq.cm)	1.57	17	17	Satisfactory	-
2	RSM.60.RB.400	20	19.9	2.437		152.0	485		240.0	760			16		Satisfactory	-
3	RSM.60.RB.400	20	19.8	2.423		144.0	460		230.0	730			18		Satisfactory	-
1	RSM.60.RB.400	16	15.7	1.527	1.523	79.4	400	400 (58000 psi) (4090 kg/sq.cm)	130.0	650	660 (95500 psi) (6700 kg/sq.cm)	1.65	19	-	Satisfactory	-
2	RSM.60.RB.400	16	15.7	1.520		81.4	410		130.0	660			15		Satisfactory	-
3	RSM.60.RB.400	16	15.7	1.522		79.4	400		130.0	650			19		Satisfactory	-
1	RSM.60.RB.400	12	12.0	0.888	0.887	50.6	470	470 (68000 psi) (4790 kg/sq.cm)	76.0	700	700 (102000 psi) (7150 kg/sq.cm)	1.49	17	17	Satisfactory	-
2	RSM.60.RB.400	12	12.0	0.889		51.0	475		76.0	710			16		Satisfactory	-
3	RSM.60.RB.400	12	12.0	0.884		50.6	470		76.0	700			18		Satisfactory	-
1	RSM.60.RB.400	10	10.1	0.627	0.631	35.5	500	500 (72500 psi) (5100 kg/sq cm)	55.0	770	780 (113000 psi) (7900 kg/sq.cm)	1.56	16	16	Satisfactory	-
2	RSM.60.RB.400	10	10.1	0.635		35.9	510		56.0	790			15		Satisfactory	-
3	RSM.60.RB.400	10	10.1	0.632		35.1	495		55.0	770			17		Satisfactory	-
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-		-	-	-	-	-	-		-		-	-
-	-	-	-	-		-	-	-	-	-	-		-		-	-

2.2 As Built Structural Drawings

1) Following actions are taken to prepare the as built architectural and structural drawings:

- Field visit and measurement for number of stories and storey height, column section size, internal column length, wall section size, spandrel wall and hanging wall dimensions, wall opening size
- Chipping, stripping and rebar scanning; column bar arrangement
- Digital survey and measurement; beam (spandrel wall and hanging wall) section size, beam span
- Digging and measurement of foundation size and thickness.

2) As built structural drawings are prepared based on above measurements.

3) The prepared as built structural and architectural drawings are listed in Appendix.

3. Seismic Evaluation

3.1 Principle of Seismic Evaluation

The principle of evaluation is as follows:

- Seismic Index I_S is calculated as per the Manual. Second level screening methods is followed as per the manual.
- Seismic Evaluation is carried out based on the as built structural drawings and material test data.
- Seismic Index I_S is calculated by the following formula based on the Manual.

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

Where:

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

S_D : Irregularity index (defined in 3.3).

T : Time index (defined in 3.4)

- Seismic Demand Index:

Seismic Demand Index I_{S0} is determined by as per requirement of 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 2.5.6.2 of BNBC 2015

I : Structure importance factor, as defined in Section 2.5.7.1 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 2.5.5a-d of BNBC 2015

Z = 0.2

I = 1.0 (Importance factor)

$$I_{SO} = 0.8 \cdot 2/3 \cdot Z \cdot I \cdot C_s$$
$$Z = 0.20, I = 1.0\ Soil\ SD(S3), h_n = 15.24m$$
$$T = 0.0446 \cdot (h_n)^{0.9} = 0.546s$$
$$C_s = 3.38$$
$$I_{SO} = 0.8 \cdot 2/3 \cdot 0.2 \cdot 1.0 \cdot 3.38 = 0.36$$

I_{SO}=*0.36*isselected.

3.2 Seismic Evaluation

The ultimate strength, *F* index and ultimate failure mode for individual column are summarized in the plan of **Figure S8-2**, and in the section of **Figure S8-2**, respectively.

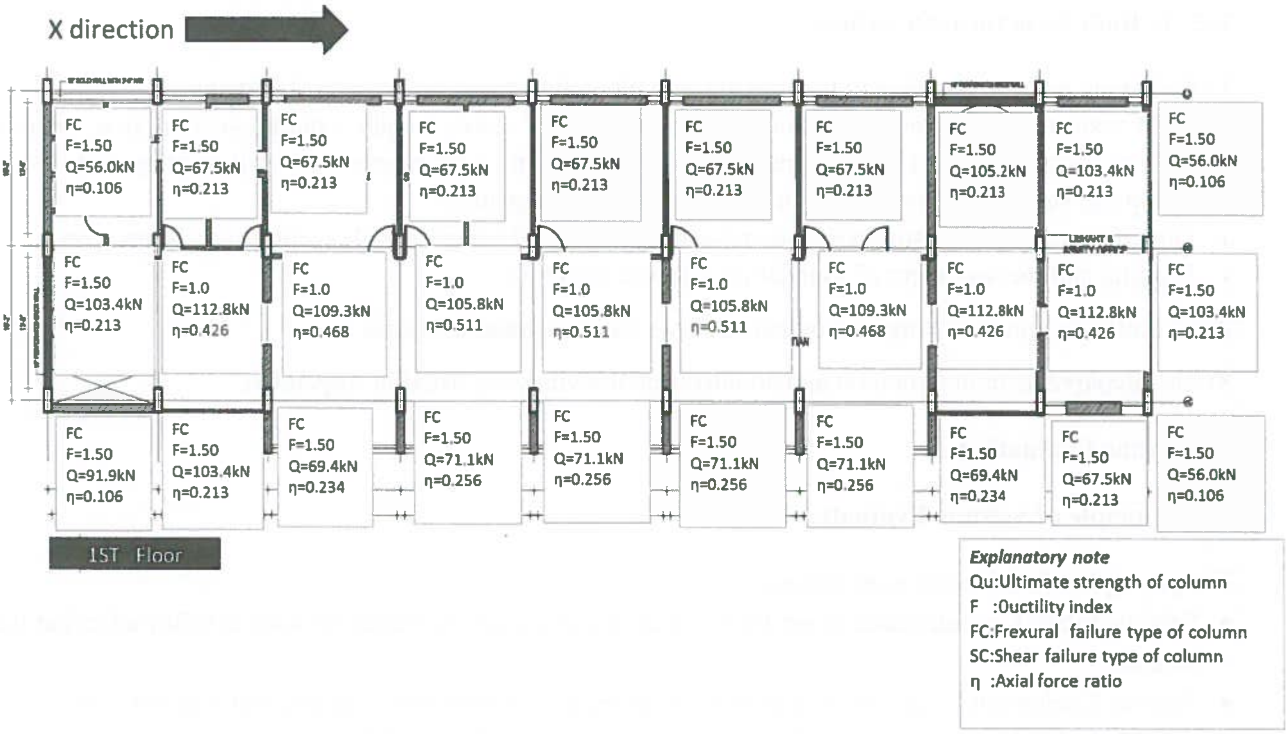


Figure S 7.2 The Ultimate Strength, *F* Index and Ultimate Failure Mode for Individual Column (Plan)

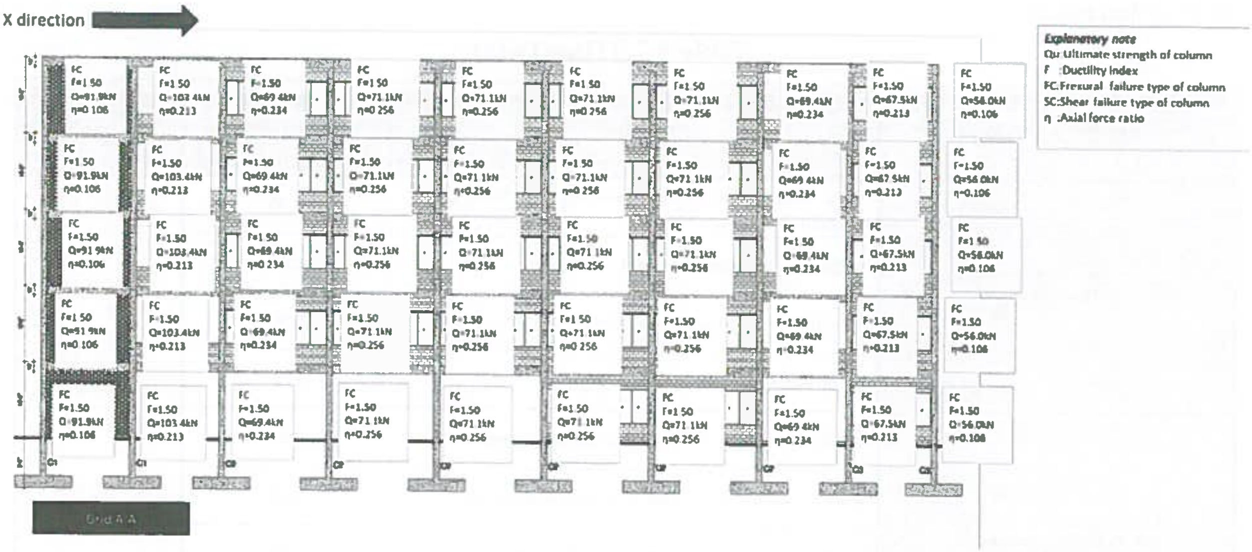


Figure S 7.3 The Ultimate Strength, *F* Index and Ultimate Failure Mode for Individual Column (Elevation)

3.2.2 Seismic Index (I_S)

The following indexes are calculated;

1) Strength Index (*C*), Ductility Index (*F*)

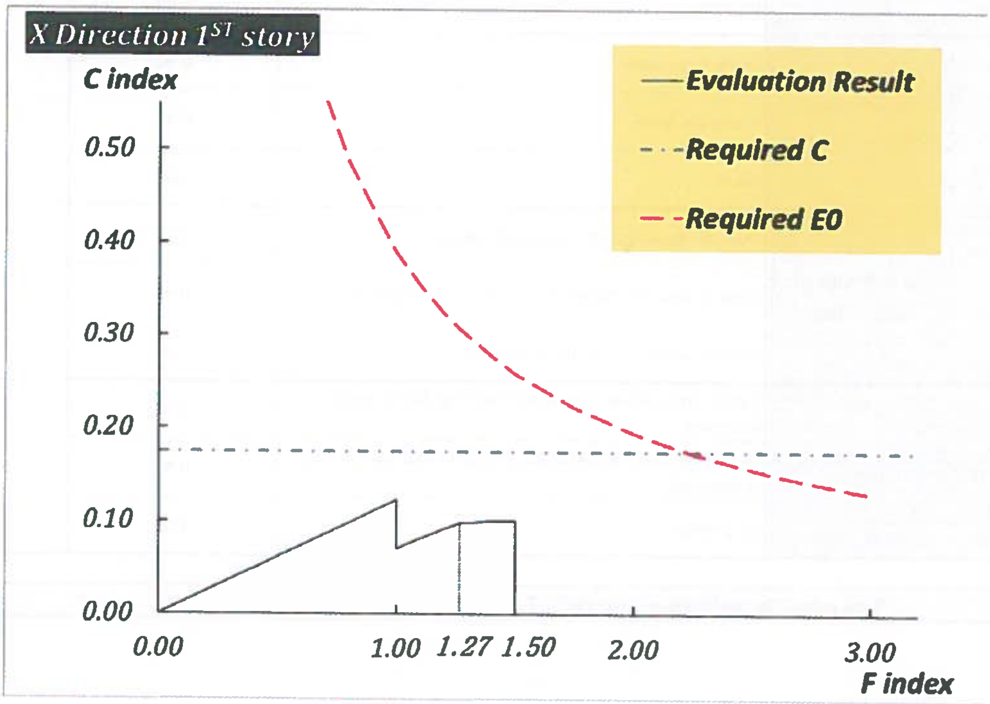


Figure S 7.4 *C-F* Curve

2) Time Index (*T*)

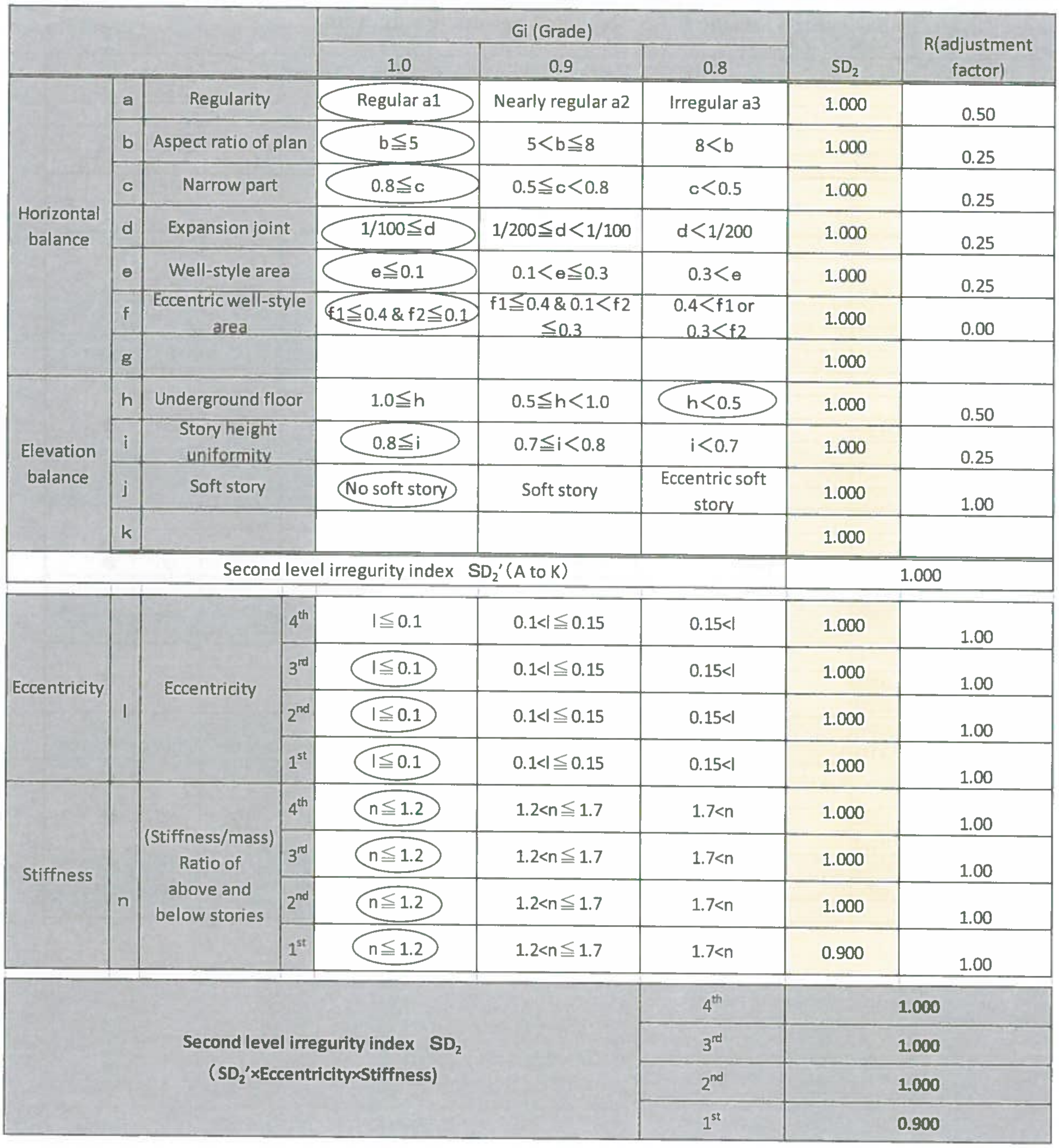
Table S 7.3Time Index

Item to be checked		Degree	T value (Check circle at relevant degree)	check ○ at relevant degree
I.Deflection		Tilting of a building or obvious uneven settlement is observed	0.70	○
		Landfill site of former rice field	0.90	
		Deflection of beam or column is observed visually	0.90	
		No correspondence to the foregoing	1.00	
II.Cracking in walls and columns		Rain leak with rust of reinforcing bar is observed	0.80	
		Inclined cracking in columns is obviously observed	0.90	
		Countless cracking is observed in external wall	0.90	
		Rain leak without rust of reinforcing bar is observed	0.90	
		No correspondence to the foregoing	1.00	○
III.Fire experience		Trace	0.70	
		Experience but traceless	0.80	
		No experience	1.00	○
IV.Deterioration	IV-i.Rust of reinforceing bar	Grade iii or more	0.90	
		Grade iior more	0.95	
		Grade i	1.00	○
	IV-ii.Depth of carbonation	Over the thickness of concrete(Grade ii)	0.90	
		Same or less the thickness of cover concrete(Grade i)	0.95	
		No correspondence to the foregoing	1.00	○
V.Finishing condition		Significant spalling of external finishing due to aging is observed	0.90	
		Significant spalling and deterioration of internal finishing is observed	0.90	
		No problem	1.00	○

Time index T by the first level inspection T_1	0.7

3) Irregularity Index (S_D)

Table S7.4 Irregularity Index

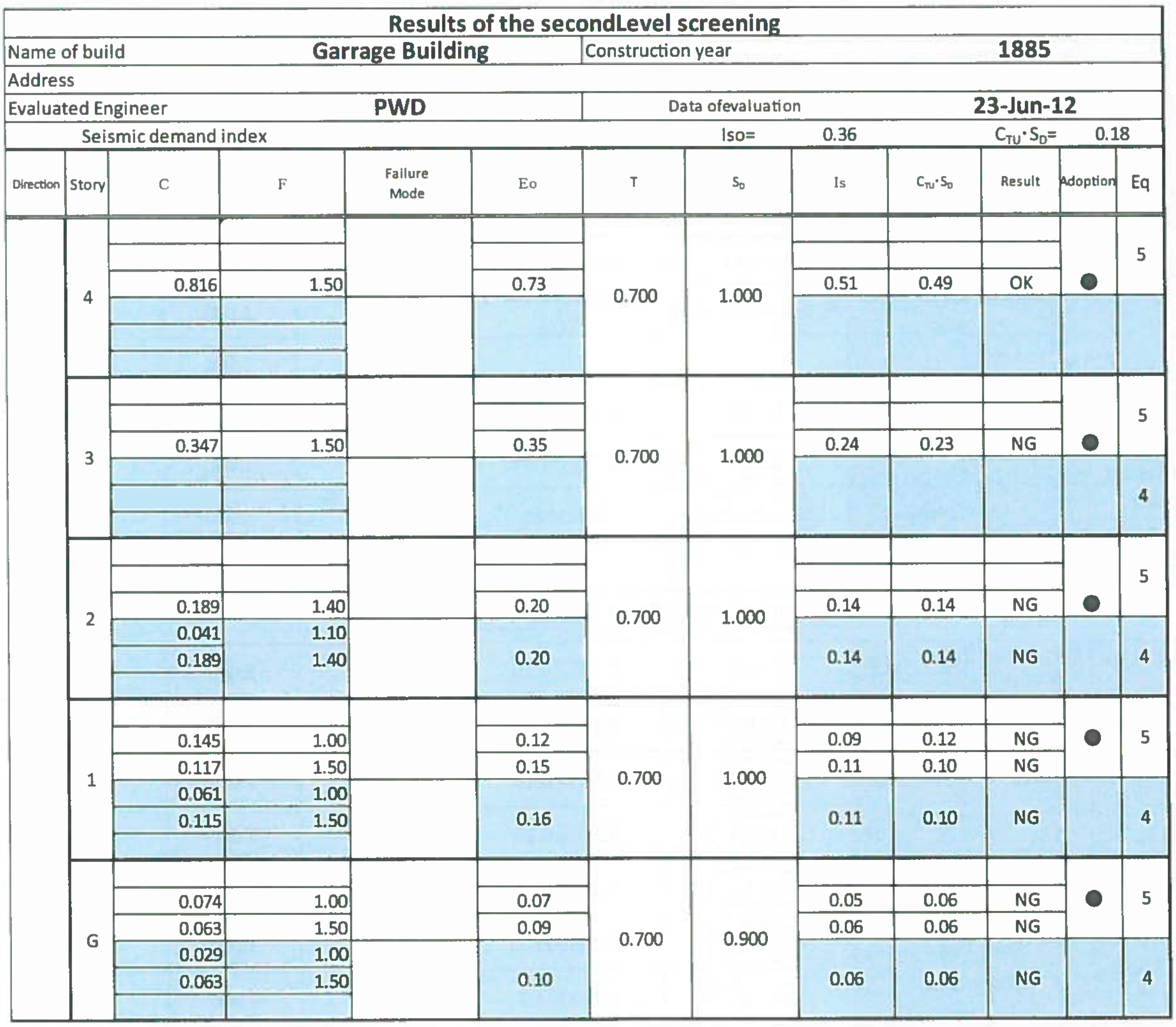


				Gi (Grade)			SD_2	R(adjustment factor)
				1.0	0.9	0.8		
Horizontal balance	a	Regularity		[✓ Regular a1]	Nearly regular a2	Irregular a3	1.000	0.50
	b	Aspect ratio of plan		[✓ b≦5]	5<b≦8	8<b	1.000	0.25
	c	Narrow part		[✓ 0.8≦c]	0.5≦c<0.8	c<0.5	1.000	0.25
	d	Expansion joint		[✓ 1/100≦d]	1/200≦d<1/100	d<1/200	1.000	0.25
	e	Well-style area		[✓ e≦0.1]	0.1<e≦0.3	0.3<e	1.000	0.25
	f	Eccentric well-style area		[✓ f1≦0.4 & f2≦0.1]	f1≦0.4 & 0.1<f2 ≦0.3	0.4<f1 or 0.3<f2	1.000	0.00
	g						1.000	
Elevation balance	h	Underground floor		1.0≦h	0.5≦h<1.0	[✓ h<0.5]	1.000	0.50
	i	Story height uniformity		[✓ 0.8≦i]	0.7≦i<0.8	i<0.7	1.000	0.25
	j	Soft story		[✓ No soft story]	Soft story	Eccentric soft story	1.000	1.00
	k						1.000	
Second level irregurity index SD_2' (A to K)							1.000	
Eccentricity	l	Eccentricity	4th	l≦0.1	0.1<l≦0.15	0.15<l	1.000	1.00
			3rd	[✓ l≦0.1]	0.1<l≦0.15	0.15<l	1.000	1.00
			2nd	[✓ l≦0.1]	0.1<l≦0.15	0.15<l	1.000	1.00
			1st	[✓ l≦0.1]	0.1<l≦0.15	0.15<l	1.000	1.00
Stiffness	n	(Stiffness/mass) Ratio of above and below stories	4th	[✓ n≦1.2]	1.2<n≦1.7	1.7<n	1.000	1.00
			3rd	[✓ n≦1.2]	1.2<n≦1.7	1.7<n	1.000	1.00
			2nd	[✓ n≦1.2]	1.2<n≦1.7	1.7<n	1.000	1.00
			1st	[✓ n≦1.2]	1.2<n≦1.7	1.7<n	0.900	1.00
Second level irregurity index SD_2 (SD_2'×Eccentricity×Stiffness)						4th	**1.000**	
						3rd	**1.000**	
						2nd	**1.000**	
						1st	**0.900**	

4) Seismic Index (I_S)

Calculated seismic Index I_S is shown in Table 3.3.

Table S 7.5 Result of Seismic Evaluation



Results of the secondLevel screening

Name of build **Garrage Building** Construction year **1885**

Address

Evaluated Engineer **PWD** Data ofevaluation **23-Jun-12**

Seismic demand index Iso= 0.36 $C_{TU} \cdot S_D$= 0.18

Direction	Story	C	F	Failure Mode	Eo	T	S_D	Is	$C_{TU} \cdot S_D$	Result	Adoption	Eq
	4	0.816	1.50		0.73	0.700	1.000	0.51	0.49	OK	●	5
	3	0.347	1.50		0.35	0.700	1.000	0.24	0.23	NG	●	5
												4
	2	0.189	1.40		0.20	0.700	1.000	0.14	0.14	NG	●	5
		0.041	1.10									
		0.189	1.40		0.20			0.14	0.14	NG		4
	1	0.145	1.00		0.12	0.700	1.000	0.09	0.12	NG	●	5
		0.117	1.50		0.15			0.11	0.10	NG		
		0.061	1.00									
		0.115	1.50		0.16			0.11	0.10	NG		4
	G	0.074	1.00		0.07	0.700	0.900	0.05	0.06	NG	●	5
		0.063	1.50		0.09			0.06	0.06	NG		
		0.029	1.00									
		0.063	1.50		0.10			0.06	0.06	NG		4

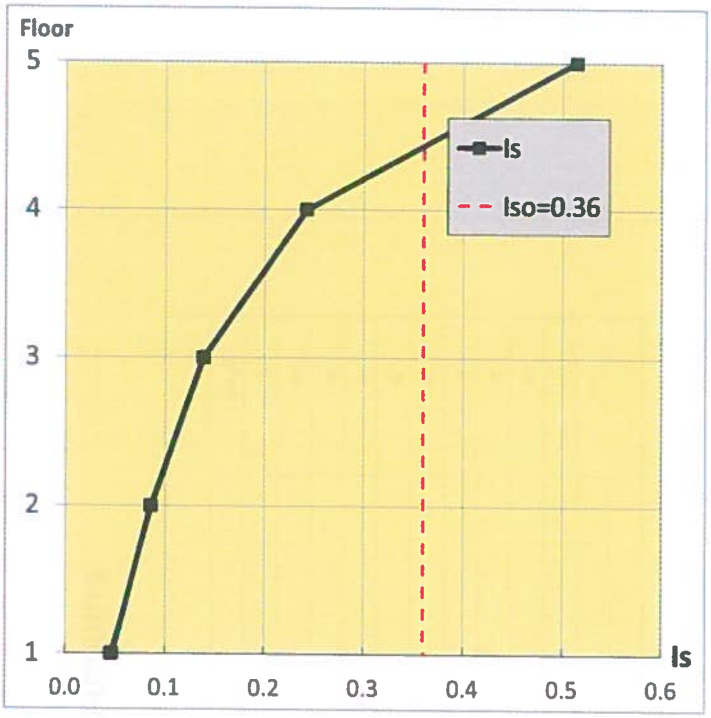


Figure S7.5 I_S **Distribution**

3.3 Judgment

I_S at all stories without 4th storey for both directions is drastically smaller than I_{so}, and therefore retrofitting design should be carried out.

APPENDIX
AS BUILT DRAWINGS

STRUCTURAL AS BUILT DRAWINGS

SL NO.	DRAWING TITLE	SHEET NO
01	COLUMN LAYOUT AND COLUMN SCHEDULE	01
02	MAT REINFORCEMENT DETAILS (BOTTOM REBAR)	02
03	MAT REINFORCEMENT DETAILS (TOP REBAR)	03
04	LONG SECTION OF FOUNDATION BEAM	04
05	SECTIONS OF FOUNDATION BEAM-1	05
06	SECTIONS OF FOUNDATION BEAM-2	06
07	TYPICAL FLAT SLAB REINFORCEMENT DETAILS (BOTTOM REBAR)	07
08	TYPICAL FLAT SLAB REINFORCEMENT DETAILS (TOP REBAR)	08
09	ROOF SLAB REINFORCEMENT DETAILS (BOTTOM REBAR)	09
10	ROOF SLAB REINFORCEMENT DETAILS (TOP REBAR)	10
11	ROOF BEAM DETAILS	11

PROJECT NAME:

CNCRP

SUB HEAD:

PROJECT LOCATION:

CLIENT:
CNCRP & PWD

CONSULTANT:

DRAWING TITLE:-
INDEX

SUPPLEMENT 8. REPORT ON PREPARATION OF AS-BUILT DRAWINGS (A SAMPLE REPORT)

Prepared By: ____

Table of Contents:

1. General

i. Objective:

The main objective of the work is to prepare as built architectural and structural drawings, which is essential for seismic assessment and retrofitting design of an existing building.

ii. Work period:

23/07/2013 to 07/10/2013

iii. Target building: Building Overview

Name	Sample building
Usage	Hospital Building
Storey	5 (Five)
Structure	RC framed structure (Flat Slab with drop panel)
Construction year Extension	1964
Total floor area	3865Sqm
Floor Height	3.5m
Total Height	18.3m from Plinth Level(PL)

Front view of Sample building

Structural Characteristics

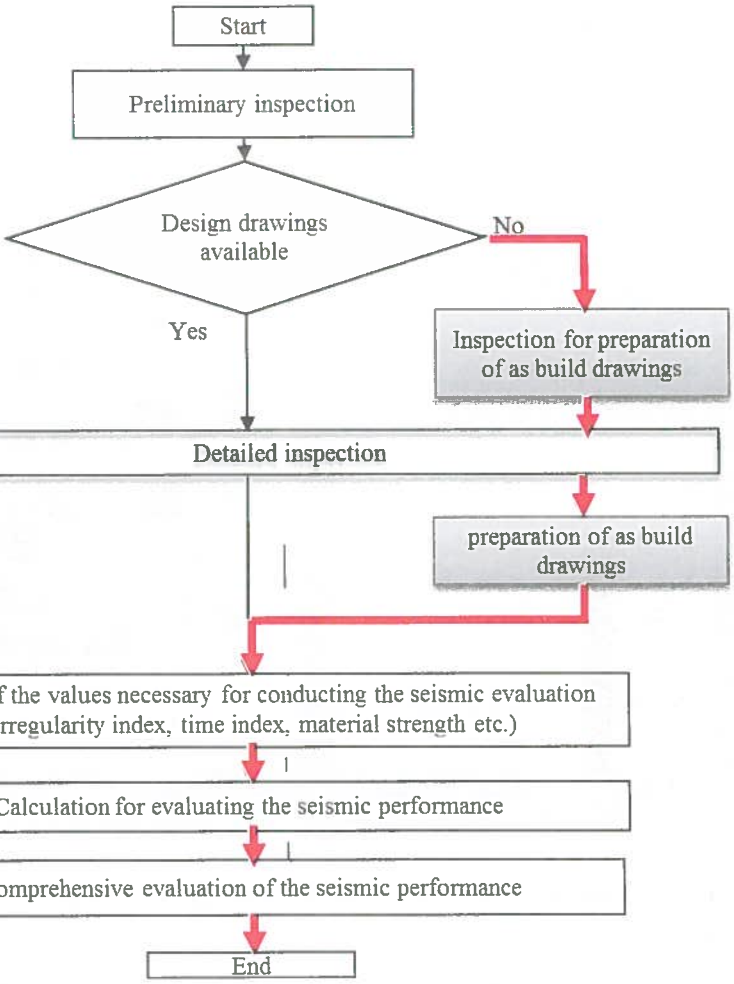
- The building is separated into ten blocks, from block-A to block-J. The interest of this report is block-J.

- It is a RC framed structure. Slab is flat slab with drop panel. There is peripheral floor beams only in roof slab.
- Building is on Mat Foundation as shown in the drawings.

iv. Investigation

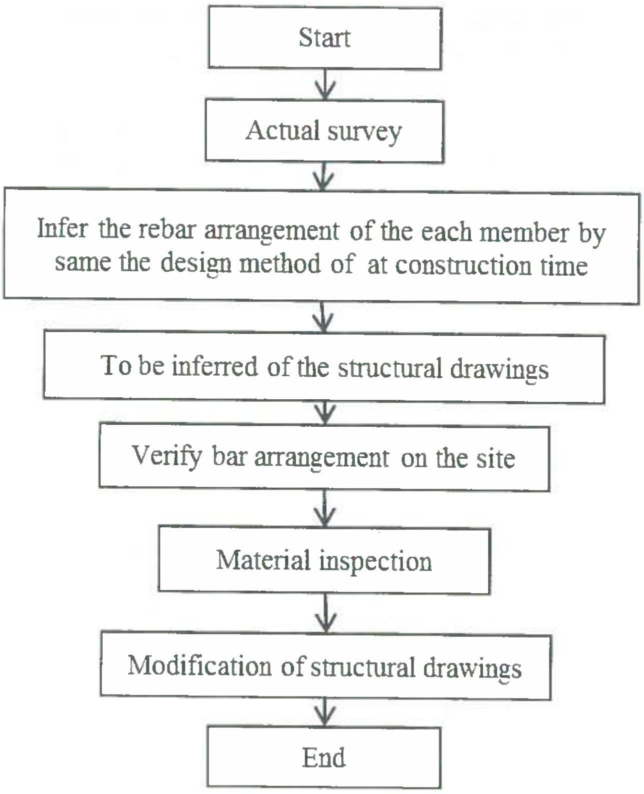
Seismic evaluation of the building is conducted as per the following flow diagram.

This work was conducted for the work of preparing as-built drawings shown in the flow diagram, when design drawings are missing.



Supplement No.8

The investigation flow chart and the field investigation items list for preparation of as built drawings.

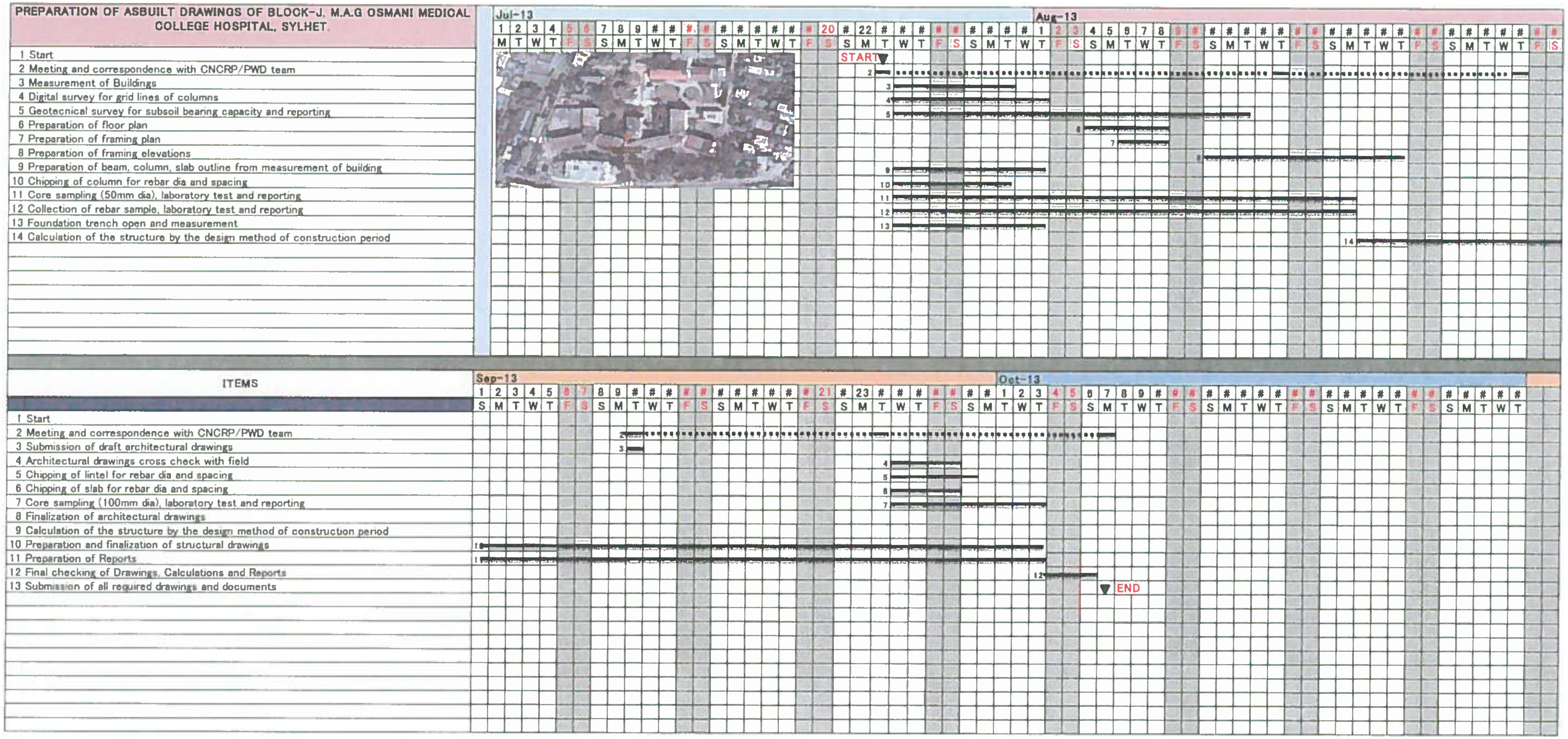


Items	Comparision of the design drawings on site	Necessity for the issuance of survey maps	Application methods
Floor space	No	Yes	Based on the digital survey and measurement
Number of stories and story height	No	Yes	Same as above
Column section size	No	Yes	Same as above
Internal column length	No	Yes	Same as above
Wall section size	No	Yes	Same as above
Spandrel wall and hanging wall dimensions	No	Yes	Same as above
Wall openeing size	No	Yes	Same as above
Column bar arrangement	No	Yes	Chipping, Stripping and Rebar Scanning
Concrete strength	No	Yes	Core cutting, Smidth Hammer Sampling and Analysis
Concrete specific gravity (Light and normal)	No	Yes	
Column and wall reinforcement yield point strength	No	Yes	Sample collection from building and laboratory test.
Beam (spandrel wall and hanging wall) Section size	No	Yes	Based on the digital survey and measurement
Beam span	No	Yes	Same as above
Beam (spandrel wall and hanging wall) reinforcement yield point strength	No	Yes	Sample collection from building and laboratory test.

Show below the investigation item was conducted.

- Foundation
- Column
- Beam
- Slab
- Material Testing (Concrete strength, Carbonation of concrete, Rebar strength)

2. Schedule of works



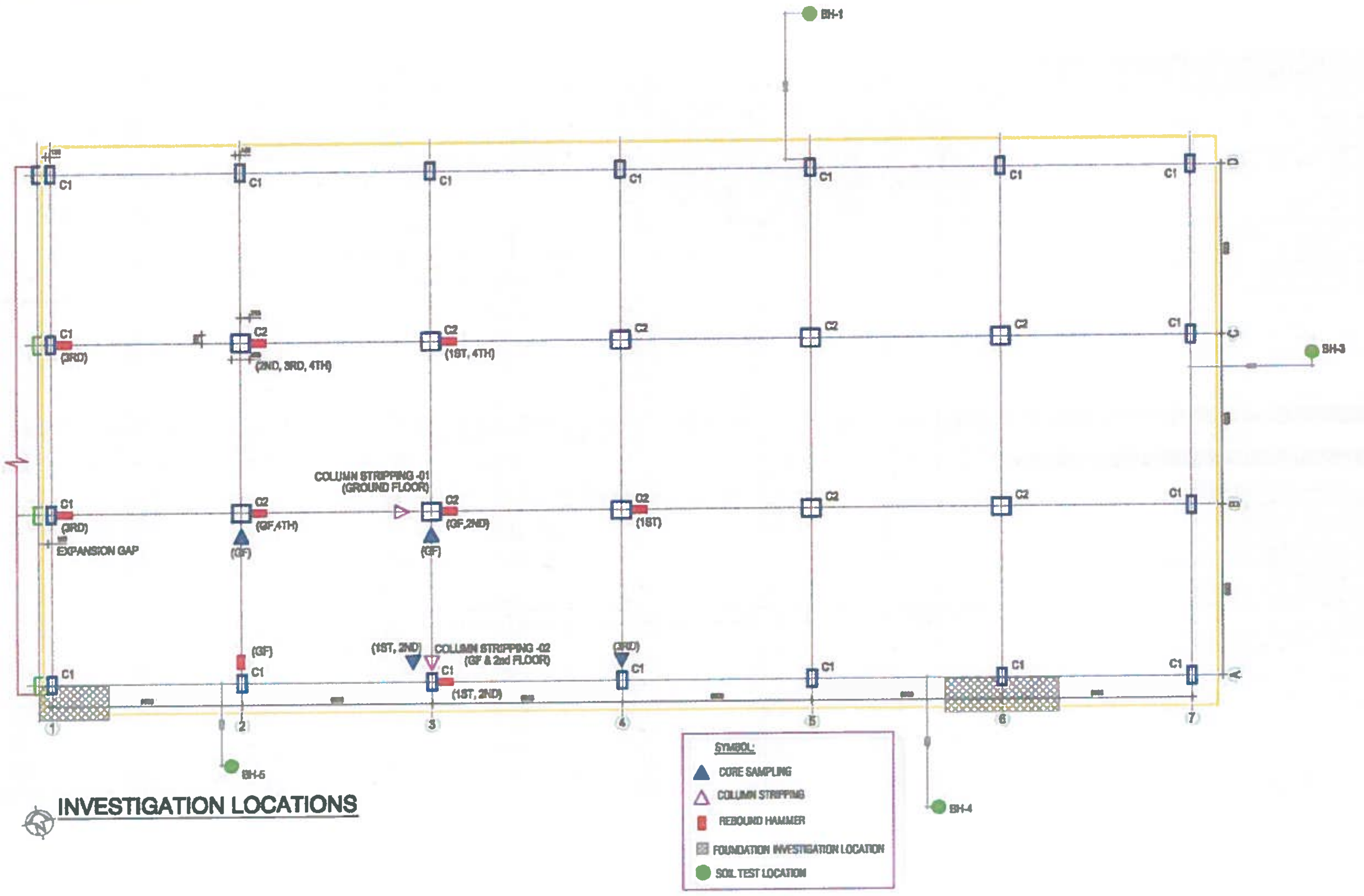
PREPARATION OF ASBUILT DRAWINGS OF BLOCK-J, M.A.G OSMANI MEDICAL COLLEGE HOSPITAL, SYLHET.

1 Start
2 Meeting and correspondence with CNCRP/PWD team
3 Measurement of Buildings
4 Digital survey for grid lines of columns
5 Geotecnical survey for subsoil bearing capacity and reporting
6 Preparation of floor plan
7 Preparation of framing plan
8 Preparation of framing elevations
9 Preparation of beam, column, slab outline from measurement of building
10 Chipping of column for rebar dia and spacing
11 Core sampling (50mm dia), laboratory test and reporting
12 Collection of rebar sample, laboratory test and reporting
13 Foundation trench open and measurement
14 Calculation of the structure by the design method of construction period

ITEMS

1 Start
2 Meeting and correspondence with CNCRP/PWD team
3 Submission of draft architectural drawings
4 Architectural drawings cross check with field
5 Chipping of lintel for rebar dia and spacing
6 Chipping of slab for rebar dia and spacing
7 Core sampling (100mm dia), laboratory test and reporting
8 Finalization of architectural drawings
9 Calculation of the structure by the design method of construction period
10 Preparation and finalization of structural drawings
11 Preparation of Reports
12 Final checking of Drawings, Calculations and Reports
13 Submission of all required drawings and documents

3. Investigation Locations

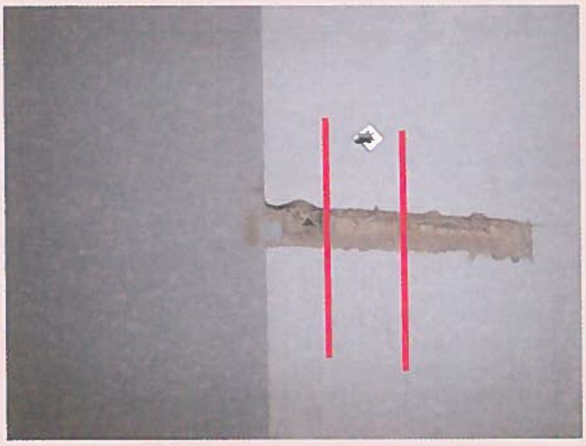


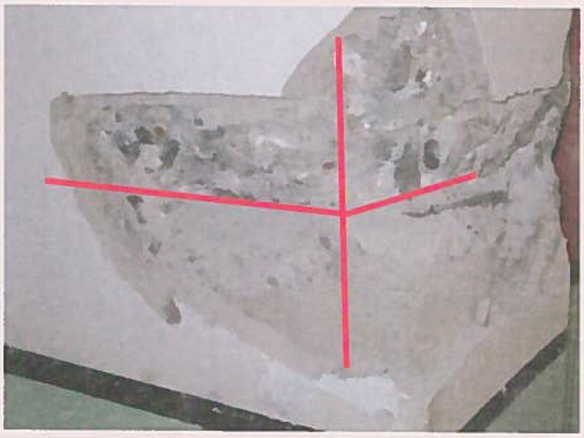
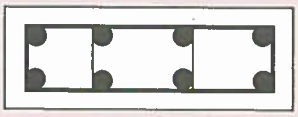
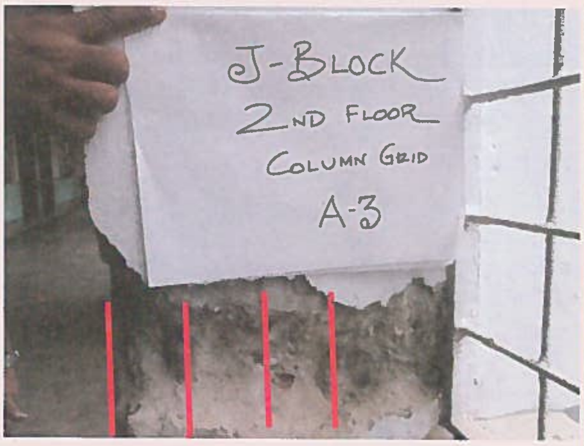
4. Investigation of Building

Measurement of Building	**Picture**
Measured Items: ▪ Structural elements ▪ Brick wall length, thickness and height ▪ Doors and windows sizes ▪ Stair	
Concrete Core Sampling-Step 1	**Picture**
▪ Fixation of core cutter	
Concrete Core Sampling-Step 2	**Picture**
▪ Location- Column at ground floor level ▪ Diameter- 100 mm ▪ Length- 200 mm	

Concrete Core Sampling-Step 3	Picture
Concrete Compressive Strength ▪ 50mm dia core = (Report Attached) ▪ 100mm dia core = (Report Attached)	

Neutralization	Picture
▪ Concrete is Neutralized in a depth from 50mm to 100 mm	

Column Investigation-1 (Interior Column)	Picture
▪ Method- Stripping of Column for Main/vertical rebar ▪ Location- Column at Grid point3-B at ground floor ▪ Column Dimension- 625mm × 625mm ▪ Main rebar diameter 12-d25mm at ground floor ▪ Shear Rebar size-10mm, ▪ Spacing-200mm	

Column Investigation-2 (Exterior Column)	**Picture**
- Method- Stripping of Column for shear rebar - Location- Column at Grid point3-A at ground floor - Rebar size-10mm, Spacing-200mm	
Column Investigation-3 (Exterior Column)	**Picture**
- Main rebar diameter 8-d25mm - Shear Rebar size-10mm, - Spacing-200mm 	
Beam Investigation	**Picture**
- Location- The building frame is mainly a flat slab system from 1st to Roof floor. There are peripheral beams at the roof level but investigation could not carried out because the space has been used as "Operation Theatre" - Dimension- 250 × 500 (Short Direction) and 250 × 600 (Long Direction)	

Foundation Investigation	**Picture**
- Location= Grid line -6A - Type = Raft Foundation - Slab thickness = 400mm - Foundation Beam = 600mm×1225mm - F. Depth = 1825mm from FGL - Soil type = Shown in subsoil investigation report. - No sign of differential settlement observed in the building.	

Slab Investigation-1 (Bottom Rebar)	**Picture**
Investigation at the bottom of slab. - Location = Room at ground floor - 200mm thick flat slab with 100mm thick (2250mm × 2250mm-Interior) and (1125mm × 2250mm-exterior) drop panels. - Clear cover = 40mm	

Slab Investigation-2 (Top Rebar)

Picture

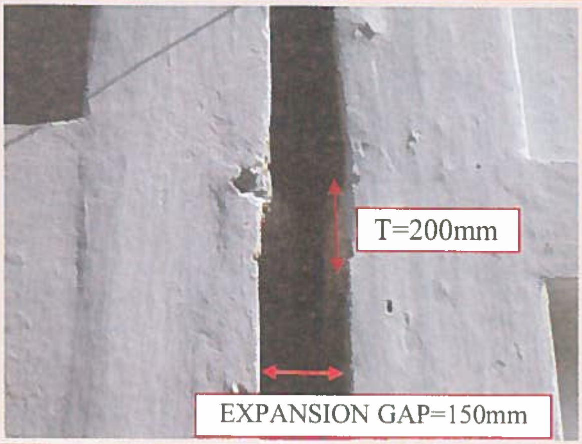
Investigation at the top of slab.

- Location- In between Grid line-C3 and C4 at 1st floor level
- Top Clear Cover- 40mm
- Col. strip top - d16mm@150mm c/c
- Col. strip bot. - d16mm@225mm c/c
- Mid. strip bot. - d16mm@300mm c/c

Slab Investigation-3 (Slab Thickness)

Picture

- Location- Grid line-A1 at 1st floor
- Thickness = 200mm



Lintel Investigation-1 (For Depth)

Picture

- Dimension – 125mm × 300mm
- Clear Cover- 50mm

Lintel Investigation-2 (For Rebar)

Picture

- Top rebar - 1 - 20mm
- Bottom rebar- 1 - 22mm
- Shear Rebar = 10mm @ 200mm c/c

Digital Survey to find the column grids

Picture

- The Building is horizontally regular
- Digital survey performed to find out the actual column grids

Subsoil Investigation

Picture

- Location- Around the building
- Bearing capacity as per report attached.

After Investigation **Picture**

- During the investigation the building was under schedule repair and renovation.
- All the core cutting and stripping locations have been filled by non-shrink grout and furnished under the renovation work as shown below.

Ground Floor Corridor and Room
(Snap Date 25/09/2013)

5. Architectural Drawings

See Appendix-‘A’

6. Structural Drawings

See Appendix 'B'

7. Re-bar Test Report

Test Report

Serial No.	Location	Sample ID	Length of Sample	Diameter of Sample	Average Cross-Sectional Area	Ultimate Load	Crushing Strength	Mode of Failure
			(inch)	(inch)	(square inch)	(lb.)		
1	Ground Floor, Grid Line D7	1-E	3.7	1.94	2.96	4,647	1570 psi (10.8 MPa)	Combined*
2	1st Floor, Grid Line B9	2-E	3.7	1.95	2.99	6,882	2300 psi (15.9 MPa)	Combined*
3	2nd Floor, Grid Line B9	3-E	3.7	1.95	2.99	6,323	2110 psi (14.6 MPa)	Combined*
4	3rd Floor, Grid Line B9	4-E	3.7	1.95	2.99	2,860	960 psi (6.6 MPa)	Combined*
5	4th Floor, Grid Line D6	5-E	3.7	1.95	2.99	5,765	1930 psi (13.3 MPa)	Combined*

*Combined = Mortar & Aggregate Failure

8. Result of Concrete Compressive Strength

Test Report

Serial No.	Location	Sample ID	Length of Sample	Diameter of Sample	Average Cross-Sectional Area	Ultimate Load	Crushing Strength	Mode of Failure
			(inch)	(inch)	(square inch)	(lb.)		
1	Ground Floor Column, Grid Line B2 of J-Block	6-J	6.0	3.90	11.95	22,427	1810 psi (12.5 MPa)	Combined*
2	Ground Floor Column, Grid Line B3 of J-Block	7-J	5.7	3.89	11.88	16,113	1300 psi (9 MPa)	Combined*

*Combined = Mortar & Aggregate Failure

9. Comments

1. Construction Year of this building was 1964. At that time WSD Method (Working Stress Design Method) was popular. So, for back calculations (structural elements investigation) WSD method have been used.
2. In case of reinforced concrete elements design the book "Simplified Design of Reinforced Concrete by Harry Parker" has been used. This book was very much popular in that time. The book reflects "Building Code Requirements for Reinforced Concrete structure (ACI-1963)".
3. One of the Design Engineer of PWD worked at that time period said in a conversation that the ratio of concrete casting was (Cement: Sand: Shingles) is 1:2:4. Depending upon the information, concrete test result and other issues connected with the construction, the concrete strength considered during design is f'_c=2500 Psi (17.25 MPa). Information about the reinforcement is 40,000 Psi (275 MPa)-plain rebar and we have got similar test result. But for the 4th floor level which constructed in 1995 is with 60,000 Psi (415 MPa) deformed rebar and test shows the same.
4. Shingles has been used as aggregate, because this material is locally available. One of the main reasons of low concrete strength may be due to shingles.
5. With the above discussed concrete strength and rebar strength column capacity has been calculated and cross checked with the column load. Column load calculated considering only the gravity loads both dead loads and live loads.
6. Capacity of columns calculated with observed section, reinforcement and concrete strength are lower than the calculated column loads (calculations Attached).
7. Flat slab design performed and rebar found by calculation matched with the observed (Calculations Attached).
8. Mat depth and foundation beam size are observed by excavation. Reinforcement provided is from back calculations (Calculations Attached).
9. Several concrete cores have been taken from columns in different floors. The core dia were both 50mm and 100mm. Test result attached will provide a clear picture regarding the strength and carbonation of existing column concrete.
10. Rebound Hammer test also performed in several columns. The results are as attached table-7. Concrete strength in rebound is higher than core test values.
11. No corrosion observed in the existing rebars.
12. No serious cracks were observed in concrete elements or in brick walls of the building..
13. The concrete and brick wall surface is in dry condition and no dampness observed in visual inspection.
14. Observation does not show any settlement of the existing foundation.
15. Digital survey performed and attached to get the actual column grids.
16. Subsoil investigation carried out to get the soil strata, properties and standard penetration test values.

10. Appendix (Calculation Sheets)

DESIGN CALCULATIONS

UNIT WEIGHT AND COLUMN LOAD

1.0 Determination of Unit Weight:

Considerations:

*First floor slab consider for calculation
*Slab Sixe 133'-0"x 65'-0"
*Dimension of structural member as per structural drawings
*Live load considered 60 Psf (Typ Flr.) & 30 Psf (Roof)
*Unit weight of concrete 150 pcf
*Unit weight of wall 120 pcf

First Floor Size

Length	40538	mm	133	ft
Width	19837	mm	65	ft

SL	ITEMS	TYP. FLR	UNIT	ROOF FLR	UNIT
1	SW of slab	864.50	kips	864.50	kips
2	Drops	50.63	kips	0.00	kips
3	FF	216.13	kips	432.25	kips
4	Column	129.16	kips	0.00	kips
5	Walls	679.06	kips	0.00	kips
6	Live Load	518.70	kips	259.35	kips
TOTAL FLOOR WEIGHT		2458.17	kips	1556.10	kips

UNIT WEIGHT (Ksf)	0.284	Ksf	0.180	Ksf
UNIT WEIGHT (Psf)	284	Psf	180	Psf

2.0 Determination of Column Load @ PL Level(Service):

Interior Column Size (Typ.)	24x24	inxin
Tributory area = (22x21) Sft	462	Sft
Column Load =	609	Kips

Exterior Column Size (Typ.)	12x24	inxin
Tributory area = (22x10.5) Sft	231	Sft
Column Load =	304	Kips

Corner Column Size (Typ.)	12x24	inxin
Tributory area = (11x10.5) Sft	115.5	Sft
Column Load =	152	Kips

3.0 Determination of Column Load @ Foundation Level(Service):

Load calculated by StaadPro software and attached in next page.

Column Capacity of a Existing Building

INTERIOR COLUMN

General Design Specifications:

Unit wt.of concrete = g_c = 150 cfs

Unit wt.of steel = g_s = 490 cfs

Yield strength of steel = f_y = 40000 psi

Compressive strength of concrete = f'_c = 2500 psi

Allowable tensile strength of steel = $f_{s=0.45fy}$ = 18000 psi

Allowable compressive strength of concrete = f_c $(0.45f'_c)$ = 1125 psi

Allowable shear strength of concrete = v $2(f'_c)^{1/2}$ = 100 psi

Modulus of elasticity of steel = E_s = 29000000 psi

Modulus of elasticity of concrete = E_c = 3031244 psi

Modular ratio = n=(Es/Ec) = 10

Stress ratio = r=(fs/fc) = 16.00

Coefficient = k=[n/(n+r)] = 0.37

Lever arm coeff = j=(1-k/3) = 0.88

Resisting moment Coefficient = R=(1/2*fckj) = 184.23

Size of column :

B = = 24 in

D = = 24 in

No/Size of Bars = = 8 #

Bar per member = = 12 Nos.

Dia of Main Bars = = 1.00 in.

Area of Bars = = 0.79 sq. in.

fc = = 1125 psi

fs = = 18000 psi

Gross Concrete Area,Ag = = 576.00 sq. in

Steel Area, As = = 9.42 sq. in

Net Concrete area, Ac = = 566.58 sq. in

Capacity of Column, P = = **475.56 kip**

(P = 0.85x0.25f'cxAg + fsxAs

Design of Ties

Diameter (1/4 of Main Bar) = = 3 #

Provide = = 3 #

Spacing of Stirrups

(a) Least Lateral Dimension of Column = = 24 in

(b) 16 times the diameter of main bar = = 16 in

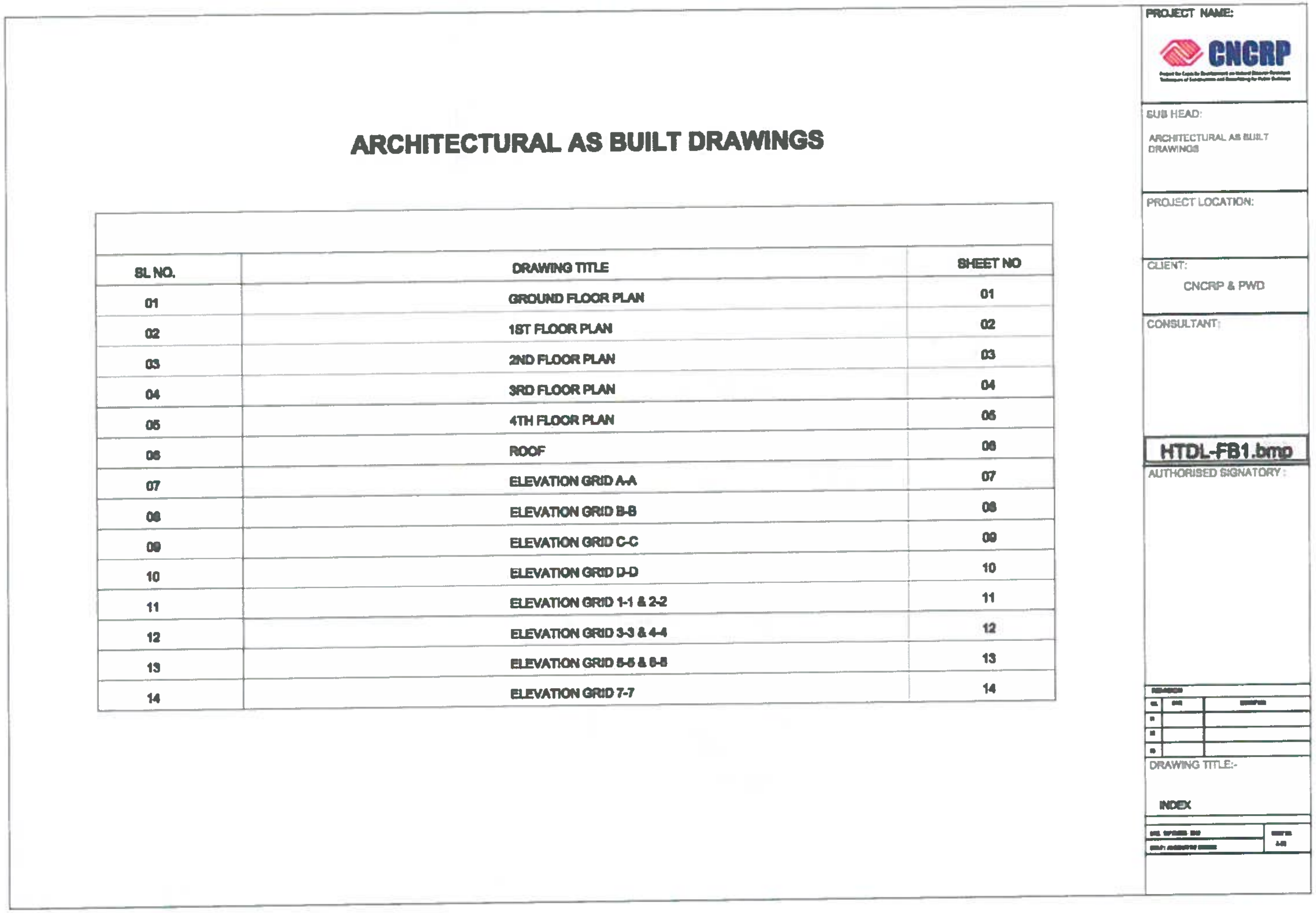
(c) 48 times the diameter of tie bar = = 18 in

Provide = = 16 in

Pecentage of Reinforcement = = 1.64 %

(This should be between 1% to 4%)

11. Appendix ‘A’ (Architectural Drawings)



ARCHITECTURAL AS BUILT DRAWINGS

SL NO.	DRAWING TITLE	SHEET NO
01	GROUND FLOOR PLAN	01
02	1ST FLOOR PLAN	02
03	2ND FLOOR PLAN	03
04	3RD FLOOR PLAN	04
05	4TH FLOOR PLAN	05
06	ROOF	06
07	ELEVATION GRID A-A	07
08	ELEVATION GRID B-B	08
09	ELEVATION GRID C-C	09
10	ELEVATION GRID D-D	10
11	ELEVATION GRID 1-1 & 2-2	11
12	ELEVATION GRID 3-3 & 4-4	12
13	ELEVATION GRID 5-5 & 6-6	13
14	ELEVATION GRID 7-7	14

PROJECT NAME:

CNCRP

SUB HEAD:

ARCHITECTURAL AS BUILT DRAWINGS

PROJECT LOCATION:

CLIENT:

CNCRP & PWD

CONSULTANT:

HTDL-FB1.bmp

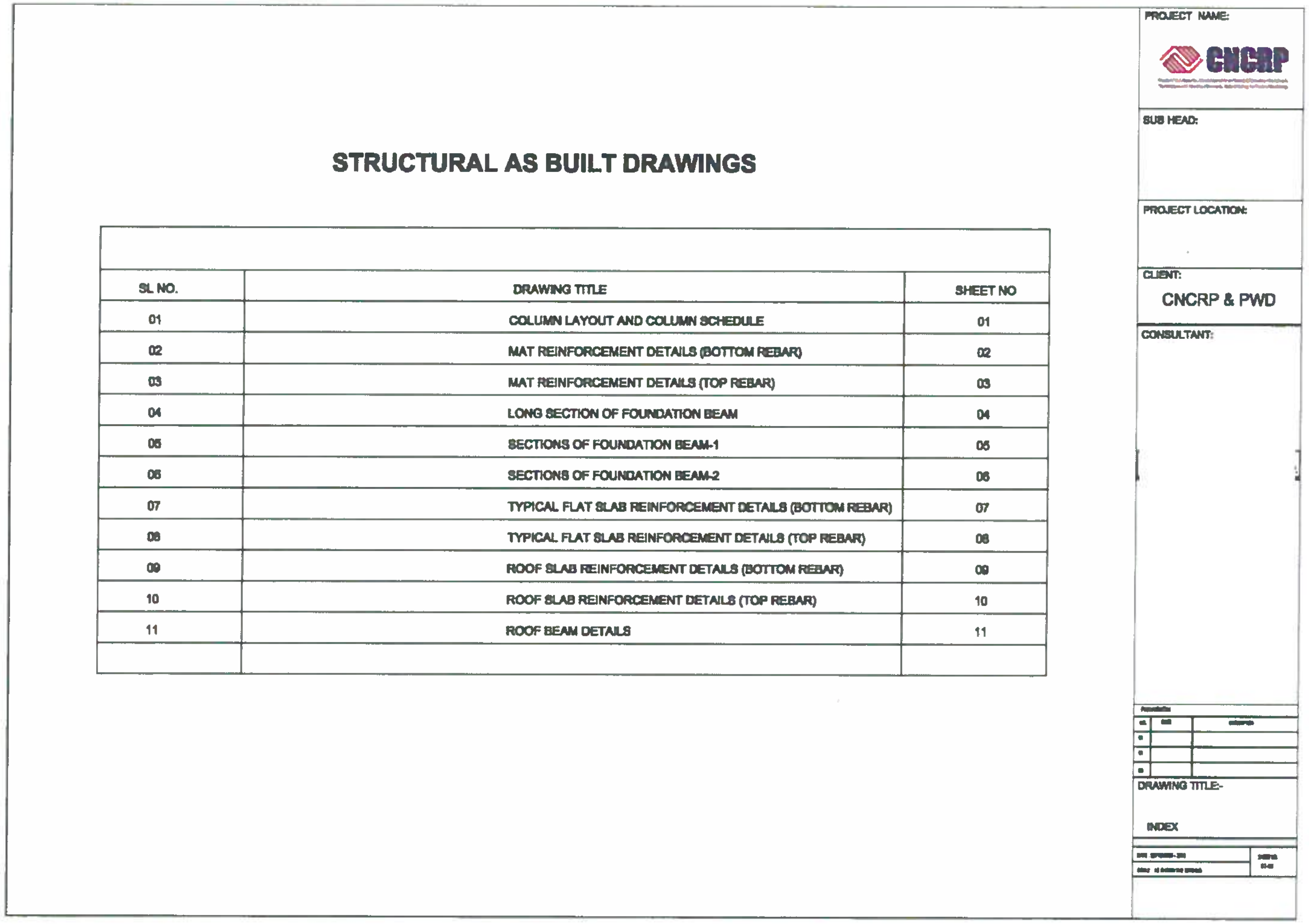
AUTHORISED SIGNATORY :

DRAWING TITLE:-

INDEX

12. Appendix 'B' (Structural Drawings)

Drawing was prepared based on the calculation result. Therefore, it is different from the result of actual measurement.



STRUCTURAL AS BUILT DRAWINGS

SL NO.	DRAWING TITLE	SHEET NO
01	COLUMN LAYOUT AND COLUMN SCHEDULE	01
02	MAT REINFORCEMENT DETAILS (BOTTOM REBAR)	02
03	MAT REINFORCEMENT DETAILS (TOP REBAR)	03
04	LONG SECTION OF FOUNDATION BEAM	04
05	SECTIONS OF FOUNDATION BEAM-1	05
06	SECTIONS OF FOUNDATION BEAM-2	06
07	TYPICAL FLAT SLAB REINFORCEMENT DETAILS (BOTTOM REBAR)	07
08	TYPICAL FLAT SLAB REINFORCEMENT DETAILS (TOP REBAR)	08
09	ROOF SLAB REINFORCEMENT DETAILS (BOTTOM REBAR)	09
10	ROOF SLAB REINFORCEMENT DETAILS (TOP REBAR)	10
11	ROOF BEAM DETAILS	11

PROJECT NAME:

CNCRP

SUB HEAD:

PROJECT LOCATION:

CLIENT:
CNCRP & PWD

CONSULTANT:

DRAWING TITLE:-

INDEX

SUPPLEMENT 9. EVALUATION EXAMPLE

The procedure of the seismic evaluation is shown in this example with a moment resisting structure which is outlined in the section 1. Since the main purpose of the example is to show how to calculate the Basic Seismic Index of structure (E_0) based on the Japanese standard of seismic evaluation.

Note: RC core wall is ignored to avoid complicacy in calculation.

1 Outline of the Structure

Outline of the example structure is described in this section. The building has 6-storey.

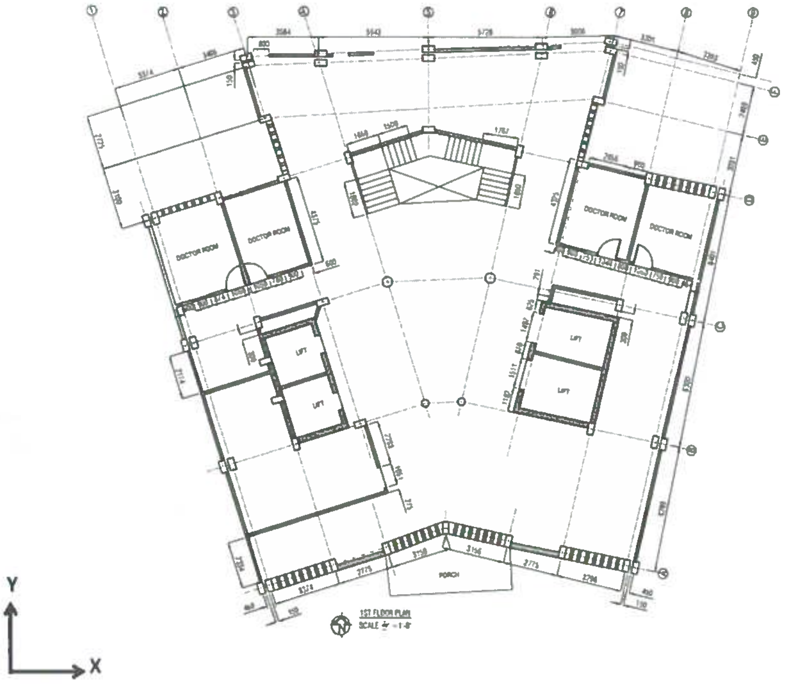
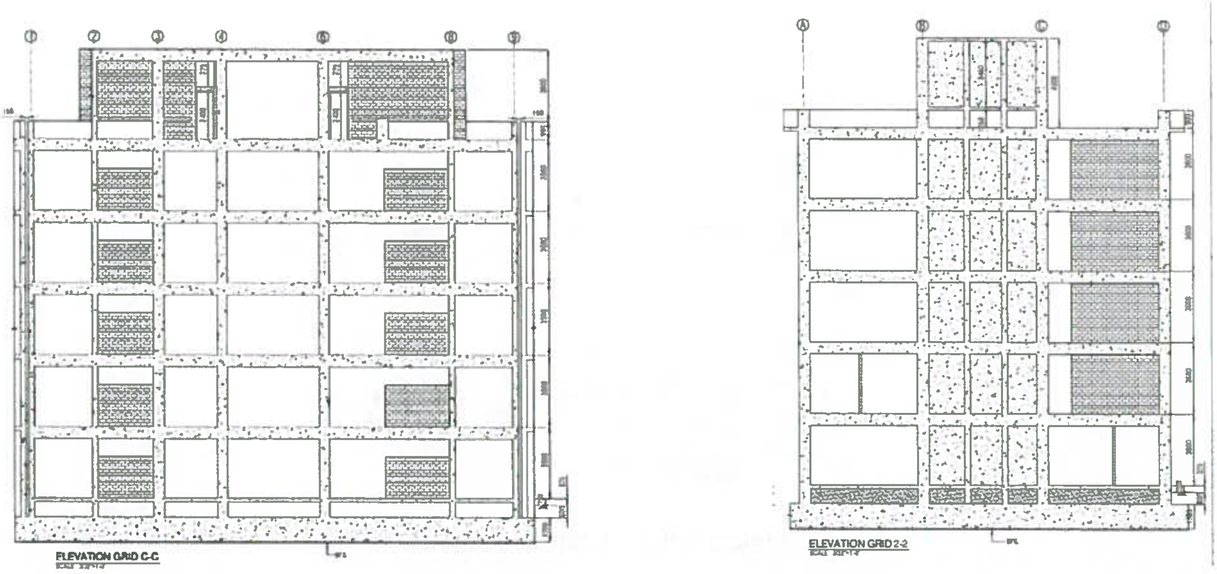


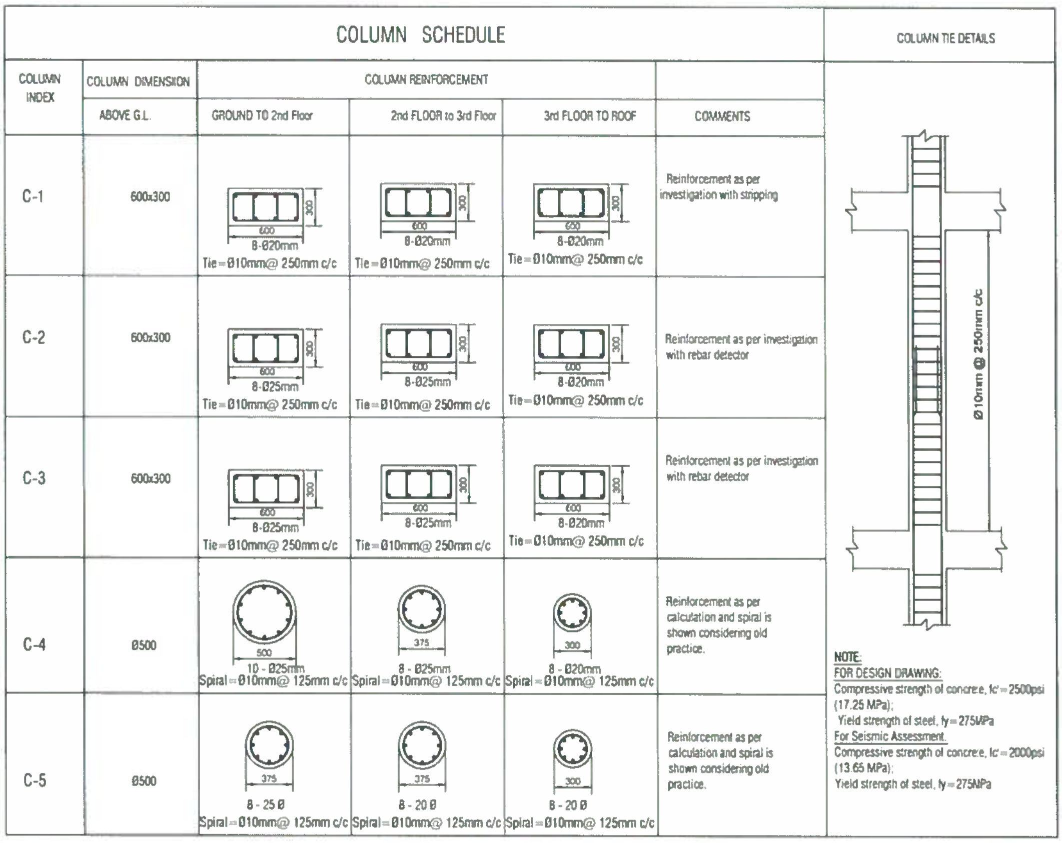
Figure S 9.1 Typical Floor Plan



C-C Grid

2-2 Grid

Figure S 9.2 Grid Elevation



COLUMN SCHEDULE

COLUMN INDEX	COLUMN DIMENSION	COLUMN REINFORCEMENT			
	ABOVE G.L.	GROUND TO 2nd Floor	2nd FLOOR to 3rd Floor	3rd FLOOR TO ROOF	COMMENTS
C-1	600x300	600 300 8-Ø20mm Tie=Ø10mm@ 250mm c/c	600 300 8-Ø20mm Tie=Ø10mm@ 250mm c/c	600 300 8-Ø20mm Tie=Ø10mm@ 250mm c/c	Reinforcement as per investigation with stripping
C-2	600x300	600 300 8-Ø25mm Tie=Ø10mm@ 250mm c/c	600 300 8-Ø25mm Tie=Ø10mm@ 250mm c/c	600 300 8-Ø20mm Tie=Ø10mm@ 250mm c/c	Reinforcement as per investigation with rebar detector
C-3	600x300	600 300 8-Ø25mm Tie=Ø10mm@ 250mm c/c	600 300 8-Ø25mm Tie=Ø10mm@ 250mm c/c	600 300 8-Ø20mm Tie=Ø10mm@ 250mm c/c	Reinforcement as per investigation with rebar detector
C-4	Ø500	500 10 - Ø25mm Spiral=Ø10mm@ 125mm c/c	375 8 - Ø25mm Spiral=Ø10mm@ 125mm c/c	300 8 - Ø20mm Spiral=Ø10mm@ 125mm c/c	Reinforcement as per calculation and spiral is shown considering old practice.
C-5	Ø500	375 8 - 25 Ø Spiral=Ø10mm@ 125mm c/c	375 8 - 20 Ø Spiral=Ø10mm@ 125mm c/c	300 8 - 20 Ø Spiral=Ø10mm@ 125mm c/c	Reinforcement as per calculation and spiral is shown considering old practice.

COLUMN TIE DETAILS

NOTE:

FOR DESIGN DRAWING:

Compressive strength of concrete, fc'=2500psi (17.25 MPa);

Yield strength of steel, fy=275MPa

For Seismic Assessment.

Compressive strength of concrete, fc'=2000psi (13.65 MPa);

Yield strength of steel, fy=275MPa

Figure S 9.3 Column Schedule

Tie spacing @250
90degrees hook

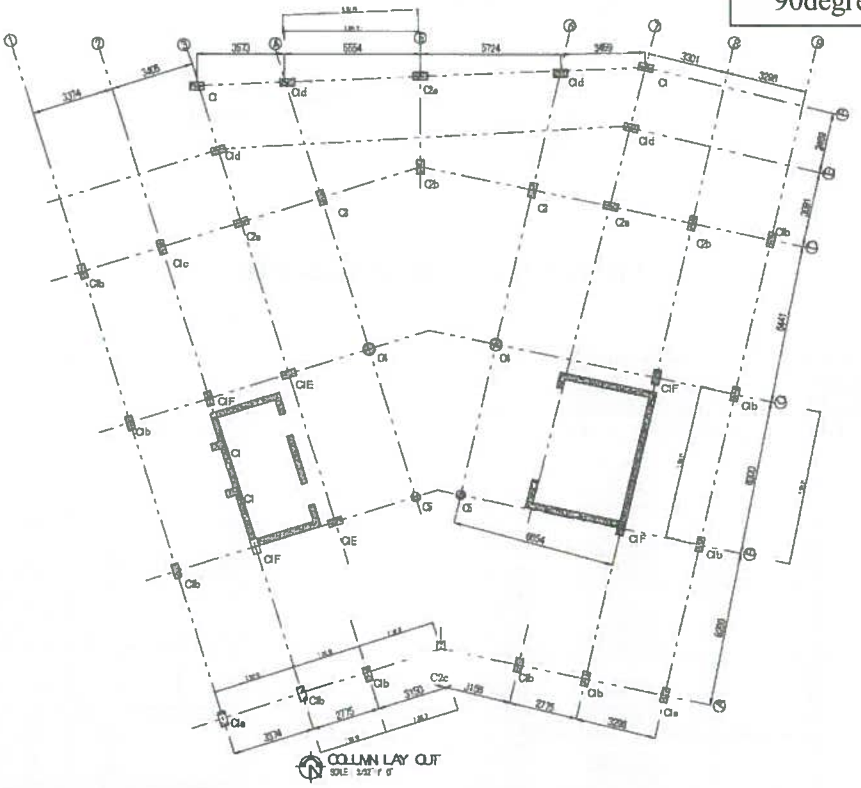


Figure S 9.4 Column Layout

[Material Properties]

Concrete: $\sigma_B = 13.7$ kN/mm^2 (Ordinary concrete)

Rebar : $\sigma_y = 275$ kN/mm^2 ($\sigma_{wy} = 275$ kN/mm^2)

2. Preliminary Calculation

2.1 Structural weight and axial force of column

Table S 9.1 Building weight

Story	N+1/N+i	Σ Wi(kN)
6	0.583	3210.0
5	0.636	9949.0
4	0.700	15918.0
3	0.778	21879.0
2	0.875	27842.0
1	1.000	33801.0

Table S 9.2 Floor Wise Axial Force on Column

COLUMN ID	FLOOR	Nos.	N (KN)	b (mm)	D (mm)
C1	6	2	75	300	600
C1	5	4	150	300	600
C1	4	4	225	300	600
C1	3	4	300	300	600
C1	2	4	375	300	600
C1	1	4	450	300	600
C1-A	5	2	75	600	300
C1-A	4	2	150	600	300
C1-A	3	2	225	600	300
C1-A	2	2	300	600	300
C1-A	1	2	375	600	300
C1-B	5	10	129	600	300
C1-B	4	10	258	600	300
C1-B	3	10	387	600	300
C1-B	2	10	516	600	300
C1-B	1	10	645	600	300
C1-C	5	1	155	600	300
C1-C	4	1	310	600	300
C1-C	3	1	465	600	300
C1-C	2	1	620	600	300
C1-C	1	1	775	600	300
C1-D	6	2	129	300	600
C1-D	5	4	258	300	600
C1-D	4	4	387	300	600
C1-D	3	4	516	300	600
C1-D	2	4	645	300	600
C1-D	1	4	775	300	600
C1-E	6	2	171	300	600
C1-E	5	2	341	300	600
C1-E	4	2	513	300	600
C1-E	3	2	684	300	600
C1-E	2	2	855	300	600
C1-E	1	2	1025	300	600
C1-F	6	4	171	600	300
C1-F	5	4	341	600	300
C1-F	4	4	513	600	300
C1-F	3	4	684	600	300
C1-F	2	4	855	600	300
C1-F	1	4	1025	600	300

COLUMN ID	FLOOR	Nos.	N (KN)	b (mm)	D (mm)
C2-A	6	1	188	300	600
C2-A	5	3	375	300	600
C2-A	4	3	563	300	600
C2-A	3	3	750	300	600
C2-A	2	3	938	300	600
C2-A	1	3	1125	300	600
C2-B	6	1	188	300	600
C2-B	5	1	375	300	600
C2-B	4	1	563	300	600
C2-B	3	1	750	300	600
C2-B	2	1	938	300	600
C2-B	1	1	1125	300	600
C2-C	5	2	188	600	300
C2-C	4	2	375	600	300
C2-C	3	2	563	600	300
C2-C	2	2	750	600	300
C2-C	1	2	938	600	300
C3	6	2	250	600	300
C3	5	2	500	600	300
C3	4	2	750	600	300
C3	3	2	1000	600	300
C3	2	2	1250	600	300
C3	1	2	1500	600	300
C4	6	2	300	330	330
C4	5	2	600	330	330
C4	4	2	900	330	330
C4	3	2	1200	330	330
C4	2	2	1500	440	440
C4	1	2	1800	440	440
C5	6	2	150	330	330
C5	5	2	300	330	330
C5	4	2	450	330	330
C5	3	2	600	330	330
C5	2	2	750	330	330
C5	1	2	900	330	330

3. The Second Level Screening Method

According to the second level seismic capacity evaluation, the seismic capacity of a structure is evaluated based on the performance of the vertical elements on the assumption that beams are strong enough not to fail.

3.1 The Ultimate Flexural Strength Of Column

(1) The ultimate flexural strength of column
The ultimate flexural strength of column is calculated with the Eq. (A1.1-1 in Standard)

The ultimate flexural strength of colum[Supplementary Provisions 1.1 Ultimate stength A1.1-1]

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

$$M_u = \{0.8a_t \cdot \sigma_y \cdot D + 0.12b \cdot D^2 \cdot F_c\} \cdot \left(\frac{N_{max} - N}{N_{max} - 0.4b \cdot D \cdot F_c}\right) (N \cdot mm) \cdots\cdots 1$$

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

$$M_u = 0.8a_t \cdot \sigma_y \cdot D + 0.5N \cdot D \cdot \left(1 - \frac{N}{b \cdot D \cdot F_c}\right) (N \cdot mm) \cdots\cdots 2$$

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

$$M_u = 0.8a_t \cdot \sigma_y \cdot D + 0.4N \cdot D (N \cdot mm) \cdots\cdots 3$$

Where:

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

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

N *:Axial force (N)*

a_t *:Total cross sectional area of tensile reinforcing bars in existing part of column (mm)*

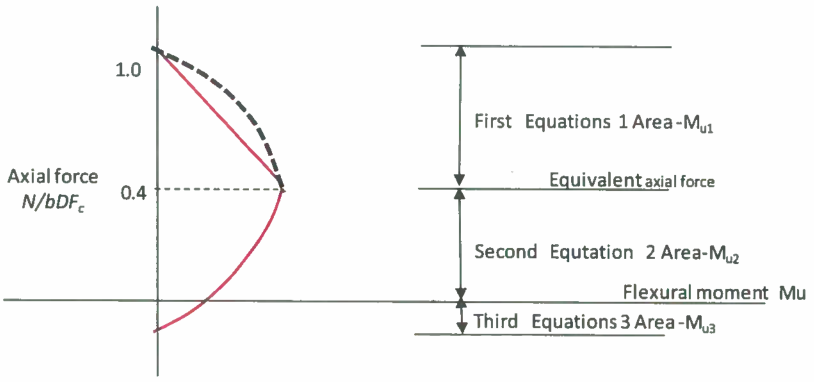
a_g *:Total cross sectional area of reinforcing bars in existing part of column (mm)*

b *:Width in existing part of column (mm)*

D *:Depth in existing part of column (mm)*

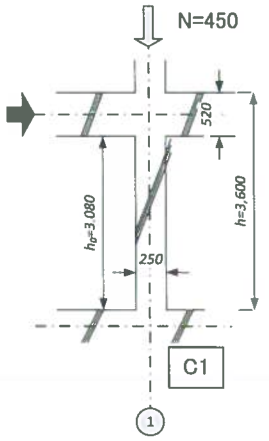
σ_y *:Yield strength of reinforcing bars in existing part of column (N/mm^2)*

F_c *:Compressive strength of concrete for existing structures (N/mm^2)*



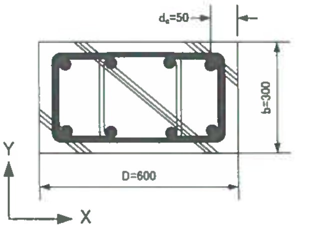
Direction	*X*	Location	GRID	*F*	*3*	*G*	Floor	Column	C1

Existing column



Story height: *h=* *3,600* *(mm)*

Inner height: h_0= *3,080* *(mm)*



8-20ϕ
2-10ϕ@250
(Grade 40)
σ_B=13.7N/mm^2

C1

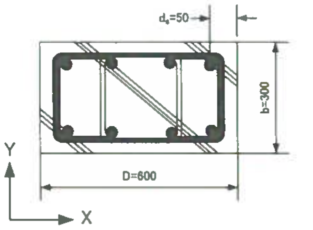
Material Properties

Compressive strength of existing concrete (F_{c1}) 13.7 N/mm^2

Tensile strength of existing rebar(σ_y): 275 N/mm^2 *(Grade 40)*

Strength of existing column

Colum:	C1				
axial force:			*N=*	450	*(kN)*
bXD=	*300*	X		600	*(mm)*
d_c=	50	*(mm)*			



8-20ϕ
2-10ϕ@250
(Grade 40)
σ_B=13.7N/mm^2

C1

<reinforcing bar>

Tensile reinforcing bars	2-	20ϕ		a_t=	*628.3*	(mm^2)
Total reinforcing bars	8-	20ϕ		a_g=	*2513.3*	(mm^2)
Tie	2-	10ϕ	*@250*	a_w=	*157.1*	(mm^2)
	Hook	*90*	*Degrees*			

column(C1)

$bXDXF_c$= 300X600X13.7= 2,466,000 (N)

$N/bXDXF_c$= 450X10^3/2466000= 0.182

$$N_{max} = a_g \cdot \sigma_y + b \cdot D \cdot F_C \ (N)$$
$$N_{min} = -a_g \cdot \sigma_y \ (N)$$

N_{max}= 2513.27X275+300X600X13.7= 3,157,150 (N)

N_{min}= -2513.27X275= -691,150 (N)

$0.4XbXDXF_c$= 0.4X2466000= 986,400 (N)

M_{u1}= (0.8X628.32X275X600+0.12X300X600^2X13.7)X(3157150-450000)/(3157150-986400)

= 324,858,046 (N·mm)= 324.9 (kN·m)

M_{u2}= 0.8X628.32X275X600+0.5X450X10^3X600X(1-450X10^3/2466000)

= 193,303,010 (N·mm)= 193.3 (kN·m)

M_{u3}= 0.8X628.32X275X600+0.4X450X10^3X600

= 190,938,046 (N·mm)= 190.9 (kN·m)

Use Mu2

The shear force at the ultimate flexural strength Q_{mu} can be calculated as follow s on the assumption that the M_u at the top and the bottom are the same.

$Q_{mu}=2XM_u/h_0$= 2X193.3/3080X10^-3= 125.52 (kN)

The ultimate flexural strength of each column can be calculated in the same procedure.

(2) The ultimate shear strength of column

The ultimate shear strength of column is calculated with the Eq.(A1.1-2 in Standard)

Shear strength of column[Modified supplementary Provisons 1.1 Ultimate stength A1.1-2]

$$Q_{su} = k_r \cdot \left\{ \frac{0.053 P_t^{0.23}(18+F_C)}{M/(Q \cdot d) + 0.12} + 0.85\sqrt{p_w \cdot \sigma_{wy}} + 0.1 \cdot \sigma_0 \right\} \cdot b \cdot j \ (N)$$

$M/Q \cdot d$ *shall be in the range of 1.0 to 3.0*

Where:

k_r : *Reduction factor of low strength concrete*$(\sigma_B < 13.5N/mm^2)$ $k_r = 0.244 + 0.056\sigma_B$
P_t :Tensile reinforcement ratio (%)
p_w :Shear reinforcement ratio $p_w = 0.012$ *for* $p_w \geq 0.012$
σ_{wy} :Yield strength of shear reinforcement (N/mm²)
σ_0 :Axial stress in column (N/mm²) $\sigma_0 \leq 8.0$
d :Effective depth of the retrofitted column(mm):d-dc
d_c :Distance between centroid of compressive reinforcement bar and extreme compression fiber
M/Q :Shear span length. Default value is $h_0/2$
h_0 :Clear height of column(mm)

k_r= $0.244+0.056X\sigma_B$= 0.244+0.056X13.7= 1.000

$P_t = a_t/(b \cdot D) \times 100 =$ 628.3/(300X600)X100= 0.349 (%)

M/Q= $h_0/2$= 3080/2= 1540 (mm) $M/Q \cdot d$= 1540/(600-50)= 2.8 2.80

p_w= $a_w/b \cdot$@(spacing)= 157.08/(300X250)= 0.00209

σ_0= $N/b \cdot D$= 450X10^3/(300X600)= 2.50 (N/mm²) <8 (N/mm²)

Q_{su}= 1X{0.053X0.349^0.23X(13.7+18)/(2.8+0.12)+0.85X√(0.002X275)+0.1X2.5}X300X0.8X600

= 193932.17 (N)= 193.9 (kN)

The strength of column

Q_{mu} < Q_{su} **Flexural failure type**

Q_u= 125.51 (kN)

The ultimate shear strength of each column can be calculated in the same procedure.
The calculated strength of each member is listed as follows.

Table S 9.3 The Strength of Members

ID	Name	Floor	Mu (kN·m)	Qmu (kN)	angle correction	Qsu (kN)	Qu	cQsu/cQmu	Mode	Extremely brittle	Failure Mode
1	C1	6	104.7	68.0	1.00	163.8	68.0	2.41	flexural	5.1	Flexural column
5	C1-D	6	119.6	77.6	1.00	168.1	77.6	2.17	flexural	5.1	Flexural column
6	C1-E	6	130.6	84.8	0.91	171.5	77.6	2.02	flexural	5.1	Flexural column
7	C1-F	6	106.8	69.3	0.91	131.4	63.4	1.90	flexural	10.3	Flexural column
8	C2-A	6	135.0	87.7	1.00	172.8	87.7	1.97	flexural	5.1	Flexural column
9	C2-B	6	135.0	87.7	0.91	172.8	80.2	1.97	flexural	5.1	Flexural column
11	C3	6	116.6	75.7	0.91	137.8	69.2	1.82	flexural	10.3	Flexural column
12	C4	6	107.9	70.1	0.91	122.9	64.1	1.75	flexural	9.3	Flexural column
13	C5	6	90.7	58.9	0.91	110.9	53.8	1.88	flexural	9.3	Flexural column
1	C1	5	125.2	81.3	1.00	169.8	81.3	2.09	flexural	5.1	Flexural column
2	C1-A	5	93.8	60.9	0.91	123.8	55.7	2.03	flexural	10.3	Flexural column
3	C1-B	5	101.2	65.7	0.91	128.1	60.1	1.95	flexural	10.3	Flexural column
4	C1-C	5	104.7	68.0	0.91	130.2	62.2	1.91	flexural	10.3	Flexural column
5	C1-D	5	152.2	98.8	1.00	178.4	98.8	1.81	flexural	5.1	Flexural column
6	C1-E	5	171.0	111.1	0.91	185.1	101.6	1.67	flexural	5.1	Flexural column
7	C1-F	5	127.0	82.4	0.91	145.0	75.4	1.76	flexural	10.3	Flexural column
8	C2-A	5	178.3	115.8	1.00	187.8	115.8	1.62	flexural	5.1	Flexural column
9	C2-B	5	178.3	115.8	0.91	187.8	105.9	1.62	flexural	5.1	Flexural column
10	C2-C	5	108.9	70.7	0.91	132.8	64.7	1.88	flexural	10.3	Flexural column
11	C3	5	142.7	92.7	0.91	157.8	84.7	1.70	flexural	10.3	Flexural column
12	C4	5	127.2	82.6	0.91	146.9	75.5	1.78	flexural	9.3	Flexural column
13	C5	5	107.9	70.1	0.91	122.9	64.1	1.75	flexural	9.3	Flexural column
1	C1	4	144.2	93.7	1.00	175.8	93.7	1.88	flexural	5.1	Flexural column
2	C1-A	4	104.0	67.6	0.91	129.8	61.8	1.92	flexural	10.3	Flexural column
3	C1-B	4	117.5	76.3	0.91	138.4	69.8	1.81	flexural	10.3	Flexural column
4	C1-C	4	123.6	80.2	0.91	142.6	73.4	1.78	flexural	10.3	Flexural column
5	C1-D	4	180.8	117.4	1.00	188.7	117.4	1.61	flexural	5.1	Flexural column
6	C1-E	4	204.8	133.0	0.91	198.8	121.6	1.50	flexural	5.1	Flexural column
7	C1-F	4	143.8	93.4	0.91	158.8	85.4	1.70	flexural	10.3	Flexural column
8	C2-A	4	260.0	168.8	1.00	209.9	168.8	1.24	flexural	5.1	Flexural column
9	C2-B	4	260.0	168.8	0.91	209.9	154.4	1.24	flexural	5.1	Flexural column
10	C2-C	4	177.3	115.1	0.91	155.5	105.3	1.35	flexural	10.3	Flexural column
11	C3	4	207.9	135.0	0.91	185.5	123.5	1.37	flexural	10.3	Flexural column
12	C4	4	140.5	91.3	0.91	173.5	83.5	1.90	flexural	9.3	Flexural column
13	C5	4	120.2	78.1	0.91	134.9	71.4	1.73	flexural	9.3	Flexural column
1	C1	3	161.9	105.2	1.00	181.8	105.2	1.73	flexural	5.1	Flexural column
2	C1-A	3	113.6	73.7	0.91	135.8	67.4	1.84	flexural	10.3	Flexural column
3	C1-B	3	131.8	85.6	0.91	148.7	78.3	1.74	flexural	10.3	Flexural column
4	C1-C	3	139.5	90.6	0.91	155.0	82.8	1.71	flexural	10.3	Flexural column
5	C1-D	3	205.3	133.3	1.00	199.1	133.3	1.49	flexural	5.1	Flexural column
6	C1-E	3	231.2	150.1	0.91	212.5	137.3	1.42	flexural	5.1	Flexural column
7	C1-F	3	157.0	102.0	0.91	172.5	93.3	1.69	flexural	10.3	Flexural column
8	C2-A	3	286.2	185.8	1.00	224.8	185.8	1.21	flexural	5.1	Flexural column
9	C2-B	3	286.2	185.8	0.91	224.8	170.0	1.21	flexural	5.1	Flexural column
10	C2-C	3	194.8	126.5	0.91	170.5	115.7	1.35	flexural	10.3	Flexural column
11	C3	3	217.2	141.1	0.91	205.5	129.0	1.46	flexural	10.3	Flexural column
12	C4	3	115.3	74.9	0.91	173.5	68.5	2.32	flexural	9.3	Flexural column
13	C5	3	127.2	82.6	0.91	146.9	75.5	1.78	flexural	9.3	Flexural column
1	C1	2	178.3	115.8	1.00	187.8	115.8	1.62	flexural	5.1	Flexural column
2	C1-A	2	122.4	79.5	0.91	141.8	72.7	1.78	flexural	10.3	Flexural column
3	C1-B	2	144.1	93.6	0.91	159.0	85.6	1.70	flexural	10.3	Flexural column
4	C1-C	2	152.5	99.0	0.91	167.4	90.6	1.69	flexural	10.3	Flexural column
5	C1-D	2	225.8	146.6	1.00	209.4	146.6	1.43	flexural	5.1	Flexural column
6	C1-E	2	250.5	162.6	0.91	226.2	148.7	1.39	flexural	5.1	Flexural column
7	C1-F	2	166.7	108.2	0.91	186.2	99.0	1.72	flexural	10.3	Flexural column
8	C2-A	2	304.0	197.4	1.00	239.9	197.4	1.22	flexural	5.1	Flexural column
9	C2-B	2	304.0	197.4	0.91	239.9	180.5	1.22	flexural	5.1	Flexural column
10	C2-C	2	207.9	135.0	0.91	185.5	123.5	1.37	flexural	10.3	Flexural column
11	C3	2	195.9	127.2	0.91	225.5	116.3	1.77	flexural	10.3	Flexural column
12	C4	2	280.9	182.4	0.91	286.0	166.8	1.57	flexural	7.0	Flexural column
13	C5	2	153.1	99.4	0.91	163.8	90.9	1.65	flexural	9.3	Flexural column
1	C1	1	193.3	125.5	1.00	193.8	125.5	1.54	flexural	5.1	Flexural column
2	C1-A	1	130.6	84.8	0.91	147.8	77.6	1.74	flexural	10.3	Flexural column
3	C1-B	1	154.3	100.2	0.91	169.4	91.7	1.69	flexural	10.3	Flexural column
4	C1-C	1	162.6	105.6	0.91	179.8	96.6	1.70	flexural	10.3	Flexural column
5	C1-D	1	242.3	157.4	1.00	219.8	157.4	1.40	flexural	5.1	Flexural column
6	C1-E	1	255.8	166.1	0.91	239.8	151.9	1.44	flexural	5.1	Flexural column
7	C1-F	1	168.6	109.5	0.91	199.8	100.1	1.82	flexural	10.3	Flexural column
8	C2-A	1	290.5	188.7	1.00	254.8	188.7	1.35	flexural	5.1	Flexural column
9	C2-B	1	290.5	188.7	0.91	254.8	172.5	1.35	flexural	5.1	Flexural column
10	C2-C	1	216.8	140.8	0.91	200.5	128.7	1.42	flexural	10.3	Flexural column
11	C3	1	174.6	113.4	0.91	240.7	103.7	2.12	flexural	10.3	Flexural column
12	C4	1	247.2	160.5	0.91	290.0	146.8	1.81	flexural	7.0	Flexural column
13	C5	1	140.5	91.3	0.91	173.5	83.5	1.90	flexural	9.3	Flexural column

Since the frame against calculating direction is inclined, the ultimate strength of column multiply the

angle correction.

3.2 The Ductility Index *F*

The *F* index for the independent column is calculated according to its failure mode, considering the strength margin for shear failure (ultimate shear strength/shear force at ultimate flexural strength) and deflection angle.

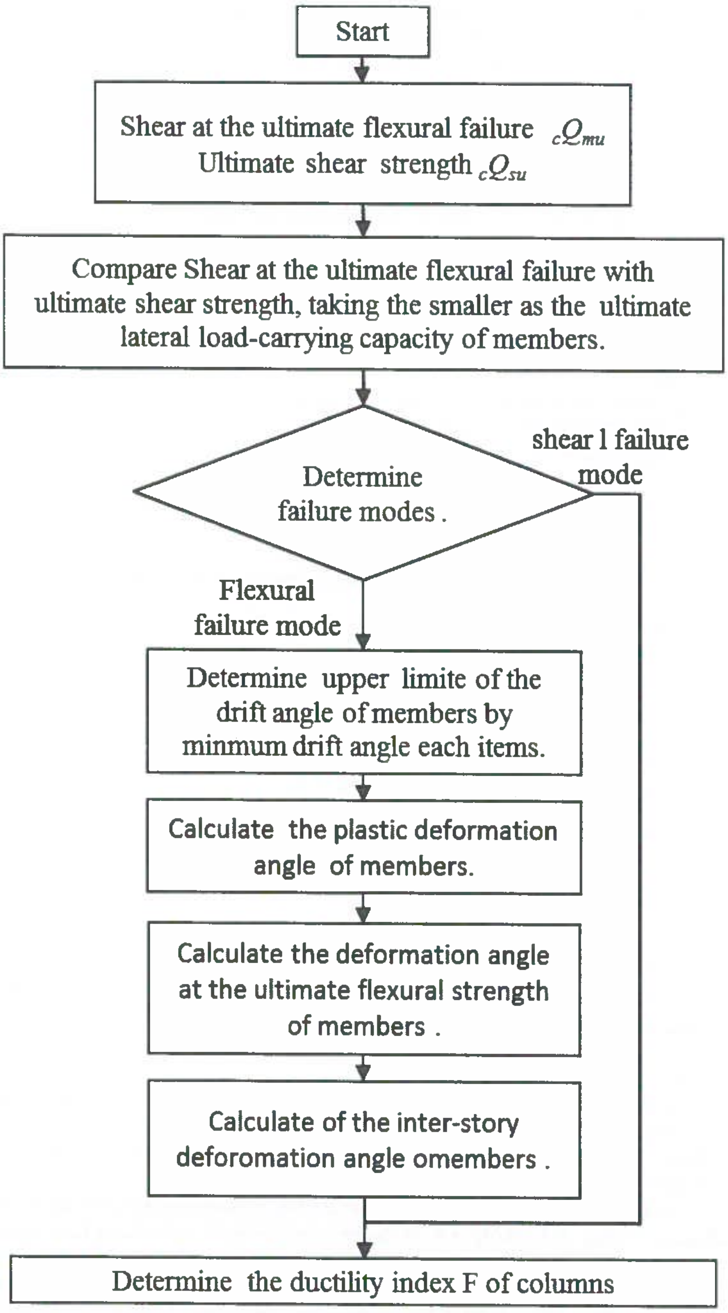


Figure S 9.5 Flowchart to Calculate the Ductility Index of Column

The ductility index of column is calculated with bellow.

<Flexural column>

In case $R_{mu} < R_y$

$$F = 1.0 + 0.27\frac{R_{mu} - R_{250}}{R_y - R_{250}}$$ *[Standard (15)]*

In case $R_{mu} \geq R_y$

$$F = \frac{\sqrt{2\,R_{mu}/R_y - 1}}{0.75 \cdot \left(1 + 0.05\,R_{mu}/R_y\right)} \leq 3.2$$ *[Standard (16)]*

where:

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

R_{250} *:Standard inter-story deformation angle, R =1/250.*

R_{mu} *:Inter-story deformation angle at the ultimate deformation capacity in flexural failure of the column member.*

$R_{mu} = (h_0/H_0) \cdot {}_cR_{mu} \geq R_{250}$ *[Supplementary Provisons 1.3 Yield Deformation Flexural COlumns A1.3–1]*

where: $h_0/H_0 \leq 1.0$

${}_cR_{mu} = {}_cR_{my} + {}_cR_{mp} \leq {}_cR_{30}$ *[Supplementary Provisons 1.2 Ultimate Deformation A1.2–2]*

where:

h_0 *:Clear height of column*

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

${}_cR_{my}$ *:Yield deformation angle of column (measured in clear height of column)*

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

${}_cR_{mp}$ *:Plastic deformation angle of the column (measured in the clear height of column)*

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

${}_cR_{my} = {}_cR_{150}\ \ for\ \ h_0/D \geq 3.0$

${}_cR_{my} = {}_cR_{250}\ \ for\ \ h_0/D \leq 2.0$

[Supplementary Provisons 1.3 Yield Deformation Flexural COlumns A1.3–2]

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

${}_cR_{mp} = 10({}_cQ_{su}/{}_cQ_{mu} - q) \cdot {}_cR_{my} \geq 0$ *[Supplementary Provisons 1.2 Ultimate Deformation A1.2–3]*

$q = 1.0\ \ for\ \ s \leq 100mm$

$q = 1.1\ \ for\ \ s > 100mm$ *[Supplementary Provisons 1.2 Ultimate Deformation A1.2–4]*

where:

${}_cQ_{su}$ *:Ultimate shear strength of the column*

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

s *:Spacing of hoops*

The value of ${}_cR_{my}$ *shall not be greater than that of* ${}_cR_{max}$

$${}_cR_{max} = min\{{}_cR_{max(n)}, {}_cR_{max(s)}, {}_cR_{max(t)}, {}_cR_{max(b)}, {}_cR_{max(h)}\}$$

[Supplementary Provisons 1.2 Ultimate Deformation A1.2–5]

${}_cR_{max(n)}$: *upper limit of the deformation angle of the flexural column determined by the axial force*

${}_cR_{\mathrm{max}(n)} = {}_cR_{250}\qquad for\ \ \eta > 0.6$

${}_cR_{\mathrm{max}(n)} = {}_cR_{150}\qquad for\ \ 0.6 \geq \eta \geq \eta_H$

${}_cR_{\mathrm{max}(n)} = {}_cR_{30} \cdot \left(\frac{{}_cR_{150}}{{}_cR_{30}}\right)^{n'} \leq {}_cR_{30}\ \ for\ other\ case$

[Modified Supplementary Provisons 1.2 Ultimate Deformation A1.2–6]

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

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

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

$\eta_L = 0.20\ \ and\ \ \eta_H = 0.4\ for\ s > 100mm.$

${}_cR_{max(s)}$: *upper limit of the deformation angle of the flexural column determined by the shear force*

${}_cR_{\mathrm{max}(s)} = {}_cR_{250}\ \ for\ \ {}_c\tau_u/F_c > 0.2$

[Supplementary Provisons 1.2 Ultimate Deformation A1.2–7]

${}_cR_{\mathrm{max}(s)} = {}_cR_{30}\ \ for\ other\ case$

${}_cR_{max(t)}$: *upper limit of the deformation angle of the flexural column determined by the tensile reinforcement ratio*

${}_cR_{\mathrm{max}(t)} = {}_cR_{250}\ \ for\ \ P_t > 1.0\%$ *[Supplementary Provisons 1.2 Ultimate Deformation A1.2–8]*

${}_cR_{\mathrm{max}(t)} = {}_cR_{30}\ \ for\ other\ case$

${}_cR_{max(b)}$: *upper limit of the deformation angle of the flexural column determined by the spacing of hoops*

${}_cR_{max(b)} = {}_cR_{50}\ \ for\ \ s/d_b > 8$ *[Supplementary Provisons 1.2 Ultimate Deformation A1.2–9]*

${}_cR_{max(b)} = {}_cR_{30}\ \ for\ other\ case$

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

${}_cR_{\mathrm{max}(h)} = {}_cR_{250}\ \ for\ \ h_0/D \leq 2$ *[Supplementary Provisons 1.2 Ultimate Deformation A1.2–10]*

${}_cR_{\mathrm{max}(h)} = {}_cR_{30}\ \ for\ other\ case$

where:

N_s *:Additional axial force of column due to earthquakes*

${}_c\tau_u$ *: Shear stress at the column strength*

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

s:Spacing of hoops

d_b*:Diameter of the flexural reinforcing bar of the column*

<Shear column>

$$F = 1.0 + 0.27\frac{R_{su} - R_{250}}{R_y - R_{250}}$$ *[Standard (14)]*

where:

R_{su} *:Inter-story deformation angle at the ultimate deformation capacity in shear failure of the column member.*

$R_{su} = \frac{{}_cQ_{su}/{}_cQ_{mu}-0.3}{0.7} \cdot R_{my} \geq R_{250}$ $for\ {}_c\alpha \cdot {}_cQ_{mu} < {}_cQ_{su}$

[Supplementary Provisons 1.2 Ultimate Deformation A1.2–11]

$R_{su} = R_{250}$ $for\ {}_c\alpha \cdot {}_cQ_{mu} \geq {}_cQ_{su}$

${}_c\alpha = 0.3 + 0.7(R_{250}/R_{my})$ *[Supplementary Provisons 1.2 Ultimate Deformation A1.2–12]*

where:

${}_cQ_{su}$ *:Ultimate shear strength of the column*

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

${}_c\alpha$ *:Effective strength factor of the colum*

R_{my} *:Yield inter-story deformation angle*

$R_{my} = (h_0/H_0) \cdot {}_cR_{my} \geq R_{250}$

[Supplementary Provisons 1.3 Yield Deformation Flexural COlumns A1.3–1]

${}_cR_{my}$ *:Yield deformation angle of column (measured in clear height of column)*

Upper limit of the deflection angle of flexural colum ${}_cR_{max}$

$${}_cR_{max} = min\{{}_cR_{max(n)}, {}_cR_{max(s)}, {}_cR_{max(t)}, {}_cR_{max(b)}, {}_cR_{max(h)}, {}_cR_{max(p)}, {}_cR_{max(a)}, {}_cR_{max(B)}\}$$

[${}_cR_{max(n)}$] Upper limit of the deformation angle of the flexural column determined by the axial force

η= *450X10^3/(300X600X13.7)=* 0.182

Concrete : *Ordinary strength concrete*

tie spacing: *@250* with 90 *Degrees hook*

η_L= 0.20 η_H= 0.40

n'= *-0.088*

${}_cR_{max(n)}$= ${}_cR_{30}$ X(${}_cR_{250}$/${}_cR_{30}$)$^{n'}$= (1/30X((1/250)/(1/30)))^(-0.09)

= 1/25 > 1/30

⇒ 1/30

[${}_cR_{max(s)}$] upper limit of the deformation angle of the flexural column determined by the shear force

${}_c\tau_u$= *min(125.51,193.93)X10^3/(300X0.8X600)=* 0.87

${}_c\tau_u/F_c$= *0.88/13.7=* 0.064 < 0.2

${}_cR_{max(s)}$= 1/30

[${}_cR_{max(t)}$] upper limit of the deformation angle of the flexural column determined by the tensile

p_t= 0.349 < 1.3

${}_cR_{max(t)}$= 1/30

[${}_cR_{max(b)}$] upper limit of the deformation angle of the flexural column determined by the spacing of hoop

s= *@250* d_b= 20ϕ

s/d_b= *250/20=* 12.5 > 8.0

${}_cR_{max(b)}$= 1/50

[${}_cR_{max(h)}$] upper limit of the deformation angle of the flexural column determined by the clear height

h_0/D= *3080/600=* 5.13 > 2.0

${}_cR_{max(h)}$= 1/30

[${}_cR_{max(p)}$] upper limit of the deformation angle of the flexural column determined by the beam-column joint

Concrete : *Ordinary strength concrete*

${}_cR_{max(p)}$= 1/100

[${}_cR_{max(a)}$] upper limit of the deformation angle of the flexural column determined by the anchor length

Concrete : *Ordinary strength concrete*

${}_cR_{max(a)}$= 1/100

[${}_cR_{max(B)}$] upper limit of the deformation angle of the flexural column determined by the clear height

$\hbar_0 \leq 2$ *(consider brick walls)*

h_0/D= *3080/600=* 5.13 > 2.0

${}_cR_{max(B)}$= 1/30

[${}_cR_{max}$] 1/100

The yielding deflection angle of the colum ${}_{c}R_{my}$

$h_0/D=$ 3080/600= 5.1

${}_{c}R_{my}=$ 1/150

The plastic deflection angle of the column ${}_{c}R_{mp}$

s= @250 ≤ 100 *(mm)*

q= 1.0

The strength margin for shear failure

${}_{c}Q_{su}/{}_{c}Q_{mu}=$ *193.93/125.51=* *1.55*

The plastic deflection angle of the column

${}_{c}R_{mp}=$ (1.55-1)X1/150= 1/28

The deformation angle at ultimate flexural strength of column

${}_{c}R_{mu}=$ 1/150+1/28= 1/24 > 1/100

${}_{c}R_{max}$ ⇒ 1/100

The inter-story deflection angle at the deformation capacity

$h_0/H_0=$ 3080/3080= 1.00

$R_{mu}=$ 1X1/100= 1/100 ≥ 1/250 OK

$R_{mu}=$ 1/100 ≥ 1/150 =R_y

F= *√2X1/100/1/150-1/0.75*(1+0.05X1/100/1/150)*

= *1.75* ≤ *3.2* *OK*

F= 1.75 *Flexural failure type*

The ductility indices for each column are calculated in the same procedure. The result is listed in the table below

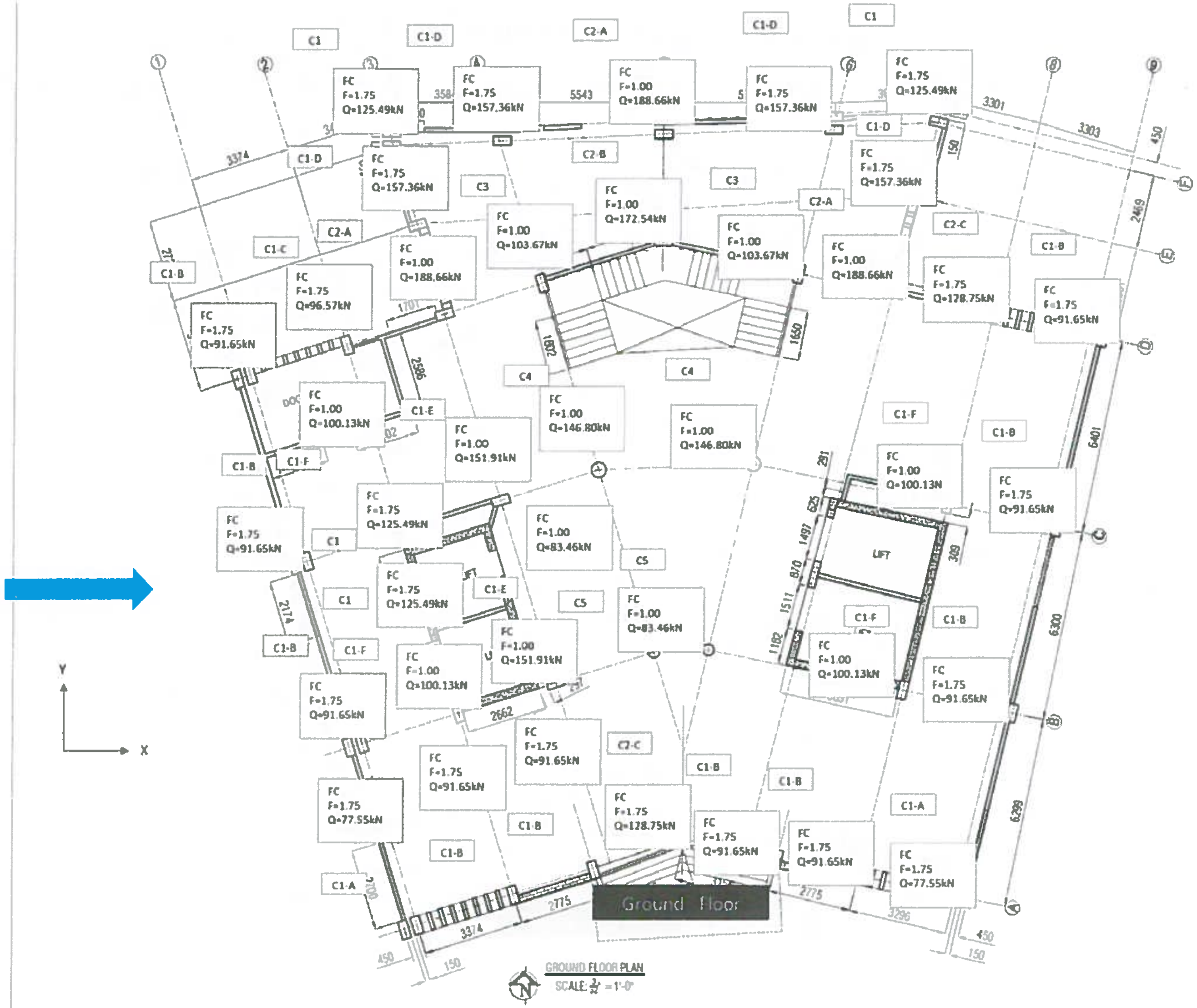
Table S 9.4 The Ductility Indices (4th to 6th Level)

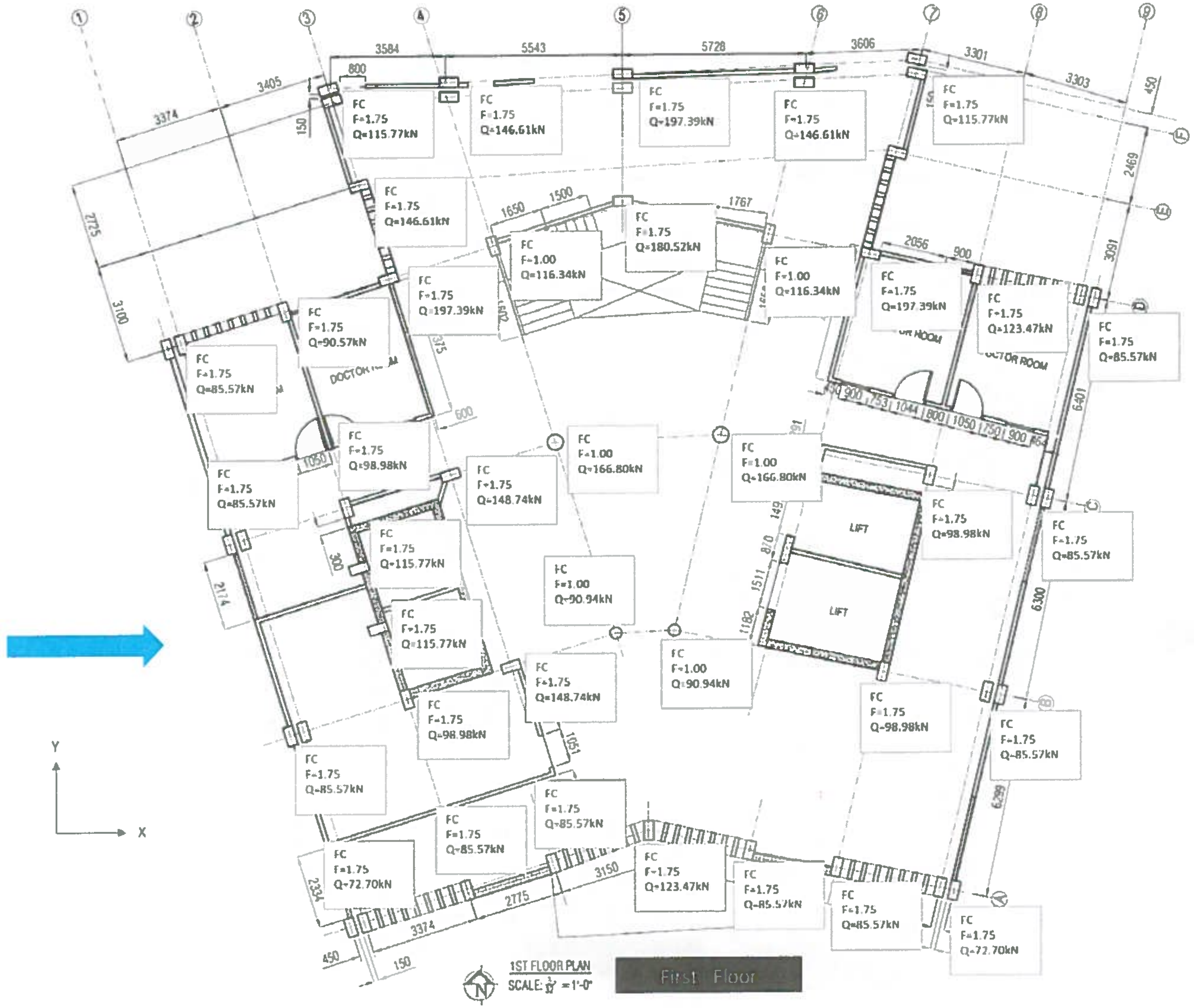
Story	Floor	No	Name	equivalent column Nomber	Qmu(kN)	Qsu(kN)	cQsu/cQmu	Failure Mode	cRmax	cRmy	cRmp	cRmu	Rsu	Rmy	Rmu	F	Qu
6	X	1	C1	2	68.0	163.8	2.41	Flexural column	1/100	1/150	1/11	1/100	—	1/150	1/100	1.75	67.99
		5	C1-D	2	77.6	168.1	2.17	Flexural column	1/100	1/150	1/14	1/100	—	1/150	1/100	1.75	77.64
		6	C1-E	2	84.8	171.5	2.02	Flexural column	1/100	1/150	1/16	1/100	—	1/150	1/100	1.75	77.58
		7	C1-F	4	69.3	131.4	1.90	Flexural column	1/100	1/150	1/19	1/100	—	1/150	1/100	1.75	63.40
		8	C2-A	1	87.7	172.8	1.97	Flexural column	1/100	1/150	1/17	1/100	—	1/150	1/100	1.75	87.66
		9	C2-B	1	87.7	172.8	1.97	Flexural column	1/100	1/150	1/17	1/100	—	1/150	1/100	1.75	80.17
		11	C3	2	75.7	137.8	1.82	Flexural column	1/100	1/150	1/21	1/100	—	1/150	1/100	1.75	69.24
		12	C4	2	70.1	122.9	1.75	Flexural column	1/100	1/150	1/23	1/100	—	1/150	1/100	1.75	64.10
		13	C5	2	58.9	110.9	1.88	Flexural column	1/100	1/150	1/19	1/100	—	1/150	1/100	1.75	53.83
5	X	1	C1	2	81.3	169.8	2.09	Flexural column	1/100	1/150	1/15	1/100	—	1/150	1/100	1.75	81.27
		2	C1-A	2	60.9	123.8	2.03	Flexural column	1/100	1/150	1/16	1/100	—	1/150	1/100	1.75	55.70
		3	C1-B	10	65.7	128.1	1.95	Flexural column	1/100	1/150	1/18	1/100	—	1/150	1/100	1.75	60.12
		4	C1-C	1	68.0	130.2	1.91	Flexural column	1/100	1/150	1/18	1/100	—	1/150	1/100	1.75	62.17
		5	C1-D	4	98.8	178.4	1.81	Flexural column	1/100	1/150	1/21	1/100	—	1/150	1/100	1.75	98.83
		6	C1-E	2	111.1	185.1	1.67	Flexural column	1/100	1/150	1/26	1/100	—	1/150	1/100	1.75	101.58
		7	C1-F	4	82.4	145.0	1.76	Flexural column	1/100	1/150	1/23	1/100	—	1/150	1/100	1.75	75.40
		8	C2-A	3	115.8	187.8	1.62	Flexural column	1/100	1/150	1/29	1/100	—	1/150	1/100	1.75	115.77
		9	C2-B	1	115.8	187.8	1.62	Flexural column	1/100	1/150	1/29	1/100	—	1/150	1/100	1.75	105.88
		10	C2-C	2	70.7	132.8	1.88	Flexural column	1/100	1/150	1/19	1/100	—	1/150	1/100	1.75	64.70
		11	C3	2	92.7	157.8	1.70	Flexural column	1/100	1/150	1/25	1/100	—	1/150	1/100	1.75	84.74
		12	C4	2	82.6	146.9	1.78	Flexural column	1/250	1/250	1/37	1/250	—	1/250	1/250	1.00	75.54
		13	C5	2	70.1	122.9	1.75	Flexural column	1/100	1/150	1/23	1/100	—	1/150	1/100	1.75	64.10
4	X	1	C1	2	93.7	175.8	1.88	Flexural column	1/100	1/150	1/19	1/100	—	1/150	1/100	1.75	93.66
		2	C1-A	2	67.6	129.8	1.92	Flexural column	1/100	1/150	1/18	1/100	—	1/150	1/100	1.75	61.78
		3	C1-B	10	76.3	138.4	1.81	Flexural column	1/100	1/150	1/21	1/100	—	1/150	1/100	1.75	69.80
		4	C1-C	1	80.2	142.6	1.78	Flexural column	1/100	1/150	1/22	1/100	—	1/150	1/100	1.75	73.37
		5	C1-D	4	117.4	188.7	1.61	Flexural column	1/100	1/150	1/30	1/100	—	1/150	1/100	1.75	117.39
		6	C1-E	2	133.0	198.8	1.50	Flexural column	1/100	1/150	1/38	1/100	—	1/150	1/100	1.75	121.61
		7	C1-F	4	93.4	158.8	1.70	Flexural column	1/100	1/150	1/25	1/100	—	1/150	1/100	1.75	85.42
		8	C2-A	3	168.8	209.9	1.24	Flexural column	1/100	1/150	1/105	1/100	—	1/150	1/100	1.75	168.81
		9	C2-B	1	168.8	209.9	1.24	Flexural column	1/100	1/150	1/105	1/100	—	1/150	1/100	1.75	154.38
		10	C2-C	2	115.1	155.5	1.35	Flexural column	1/100	1/150	1/60	1/100	—	1/150	1/100	1.75	105.30
		11	C3	2	135.0	185.5	1.37	Flexural column	1/100	1/150	1/55	1/100	—	1/150	1/100	1.75	123.47
		12	C4	2	91.3	173.5	1.90	Flexural column	1/250	1/250	1/31	1/250	—	1/250	1/250	1.00	83.46
		13	C5	2	78.1	134.9	1.73	Flexural column	1/100	1/150	1/24	1/100	—	1/150	1/100	1.75	71.41

Table S 9.5 The Ductility Indices (1st to 3rd Level)

Story	Floor	No	Name	equivalent column Nomber	Qmu(kN)	Qsu(kN)	cQsu/cQmu	Failure Mode	cRmax	cRmy	cRmp	cRmu	Rsu	Rmy	Rmu	F	Qu
3	X	1	C1	2	105.2	181.8	1.73	Flexural column	1/100	1/150	1/24	1/100	−	1/150	1/100	1.75	105.16
		2	C1-A	2	73.7	135.8	1.84	Flexural column	1/100	1/150	1/20	1/100	−	1/150	1/100	1.75	67.44
		3	C1-B	10	85.6	148.7	1.74	Flexural column	1/100	1/150	1/24	1/100	−	1/150	1/100	1.75	78.29
		4	C1-C	1	90.6	155.0	1.71	Flexural column	1/100	1/150	1/25	1/100	−	1/150	1/100	1.75	82.84
		5	C1-D	4	133.3	199.1	1.49	Flexural column	1/100	1/150	1/38	1/100	−	1/150	1/100	1.75	133.31
		6	C1-E	2	150.1	212.5	1.42	Flexural column	1/100	1/150	1/48	1/100	−	1/150	1/100	1.75	137.28
		7	C1-F	4	102.0	172.5	1.69	Flexural column	1/100	1/150	1/25	1/100	−	1/150	1/100	1.75	93.26
		8	C2-A	3	185.8	224.8	1.21	Flexural column	1/100	1/150	1/137	1/100	−	1/150	1/100	1.75	185.84
		9	C2-B	1	185.8	224.8	1.21	Flexural column	1/100	1/150	1/137	1/100	−	1/150	1/100	1.75	169.95
		10	C2-C	2	126.5	170.5	1.35	Flexural column	1/100	1/150	1/60	1/100	−	1/150	1/100	1.75	115.68
		11	C3	2	141.1	205.5	1.46	Flexural column	1/250	1/250	1/70	1/250	−	1/250	1/250	1.00	129.01
		12	C4	2	74.9	173.5	2.32	Flexural column	1/250	1/250	1/21	1/250	−	1/250	1/250	1.00	68.48
		13	C5	2	82.6	146.9	1.78	Flexural column	1/250	1/250	1/37	1/250	−	1/250	1/250	1.00	75.54
2	X	1	C1	2	115.8	187.8	1.62	Flexural column	1/100	1/150	1/29	1/100	−	1/150	1/100	1.75	115.77
		2	C1-A	2	79.5	141.8	1.78	Flexural column	1/100	1/150	1/22	1/100	−	1/150	1/100	1.75	72.70
		3	C1-B	10	93.6	159.0	1.70	Flexural column	1/100	1/150	1/25	1/100	−	1/150	1/100	1.75	85.57
		4	C1-C	1	99.0	167.4	1.69	Flexural column	1/100	1/150	1/25	1/100	−	1/150	1/100	1.75	90.57
		5	C1-D	4	146.6	209.4	1.43	Flexural column	1/100	1/150	1/46	1/100	−	1/150	1/100	1.75	146.61
		6	C1-E	2	162.6	226.2	1.39	Flexural column	1/100	1/150	1/52	1/100	−	1/150	1/100	1.75	148.74
		7	C1-F	4	108.2	186.2	1.72	Flexural column	1/100	1/150	1/24	1/100	−	1/150	1/100	1.75	98.98
		8	C2-A	3	197.4	239.9	1.22	Flexural column	1/100	1/150	1/130	1/100	−	1/150	1/100	1.75	197.39
		9	C2-B	1	197.4	239.9	1.22	Flexural column	1/100	1/150	1/130	1/100	−	1/150	1/100	1.75	180.52
		10	C2-C	2	135.0	185.5	1.37	Flexural column	1/100	1/150	1/55	1/100	−	1/150	1/100	1.75	123.47
		11	C3	2	127.2	225.5	1.77	Flexural column	1/250	1/250	1/37	1/250	−	1/250	1/250	1.00	116.34
		12	C4	2	182.4	286.0	1.57	Flexural column	1/250	1/250	1/53	1/250	−	1/250	1/250	1.00	166.80
		13	C5	2	99.4	163.8	1.65	Flexural column	1/250	1/250	1/46	1/250	−	1/250	1/250	1.00	90.94
1	X	1	C1	2	125.5	193.8	1.54	Flexural column	1/100	1/150	1/34	1/100	−	1/150	1/100	1.75	125.49
		2	C1-A	2	84.8	147.8	1.74	Flexural column	1/100	1/150	1/23	1/100	−	1/150	1/100	1.75	77.55
		3	C1-B	10	100.2	169.4	1.69	Flexural column	1/100	1/150	1/25	1/100	−	1/150	1/100	1.75	91.65
		4	C1-C	1	105.6	179.8	1.70	Flexural column	1/100	1/150	1/25	1/100	−	1/150	1/100	1.75	96.57
		5	C1-D	4	157.4	219.8	1.40	Flexural column	1/100	1/150	1/51	1/100	−	1/150	1/100	1.75	157.36
		6	C1-E	2	166.1	239.8	1.44	Flexural column	1/250	1/250	1/73	1/250	−	1/250	1/250	1.00	151.91
		7	C1-F	4	109.5	199.8	1.82	Flexural column	1/250	1/250	1/35	1/250	−	1/250	1/250	1.00	100.13
		8	C2-A	3	188.7	254.8	1.35	Flexural column	1/250	1/250	1/100	1/250	−	1/250	1/250	1.00	188.66
		9	C2-B	1	188.7	254.8	1.35	Flexural column	1/250	1/250	1/100	1/250	−	1/250	1/250	1.00	172.54
		10	C2-C	2	140.8	200.5	1.42	Flexural column	1/100	1/150	1/46	1/100	−	1/150	1/100	1.75	128.75
		11	C3	2	113.4	240.7	2.12	Flexural column	1/250	1/250	1/24	1/250	−	1/250	1/250	1.00	103.67
		12	C4	2	160.5	290.0	1.81	Flexural column	1/250	1/250	1/35	1/250	−	1/250	1/250	1.00	146.80
		13	C5	2	91.3	173.5	1.90	Flexural column	1/250	1/250	1/31	1/250	−	1/250	1/250	1.00	83.46

Failure mode of each column is shown in plan and elevation.




1ST FLOOR PLAN
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First Floor
Y
X
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FC F=1.75 Q=146.61kN
FC F=1.75 Q=197.39kN
FC F=1.75 Q=146.61kN
FC F=1.75 Q=115.77kN
FC F=1.75 Q=146.61kN
FC F=1.75 Q=180.52kN
FC F=1.00 Q=116.34kN
FC F=1.00 Q=116.34kN
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FC F=1.75 Q=197.39kN
FC F=1.75 Q=123.47kN
FC F=1.75 Q=85.57kN
FC F=1.75 Q=90.57kN
FC F=1.75 Q=85.57kN
FC F=1.75 Q=98.98kN
FC F=1.00 Q=166.80kN
FC F=1.00 Q=166.80kN
FC F=1.75 Q=85.57kN
FC F=1.75 Q=148.74kN
FC F=1.75 Q=98.98kN
FC F=1.75 Q=85.57kN
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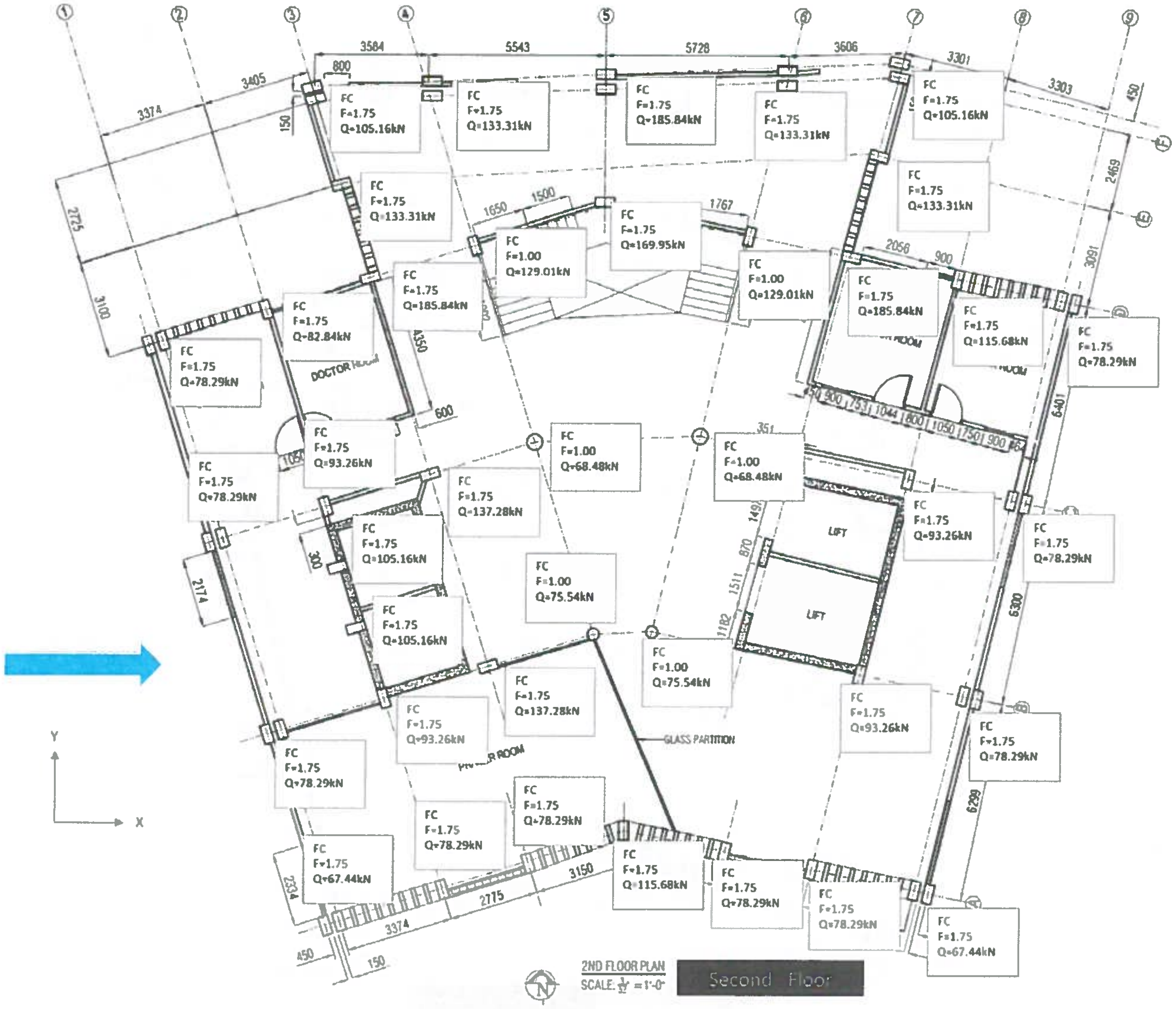
FC F=1.75 Q=72.70kN
FC F=1.75 Q=123.47kN
FC F=1.75 Q=85.57kN
FC F=1.75 Q=85.57kN
FC F=1.75 Q=72.70kN
LIFT
LIFT
DOCTOR ROOM
3584
5543
5728
3606
3301
3303
3405
3374
2725
3100
2469
3091
6401
6300
6299
2174
2334
3374
2775
3150
450
150
1650
1500
1767
2056
900
800
1050
300
600
375
1182
1511
870
149
1051


1
2
3
4
5
6
7
8
9
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5728
3606
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3303
450
800
3405
3374
150
2725
3100
2469
3091
6401
6300
6299
2174
300
4350
600
1650
1500
1767
2056
900
351
1491
870
1511
1182
2334
3374
2775
3150
450
150
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FC
F=1.75
Q=133.31kN
FC
F=1.75
Q=185.84kN
FC
F=1.75
Q=133.31kN
FC
F=1.75
Q=105.16kN
FC
F=1.75
Q=133.31kN
FC
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FC
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FC
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FC
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FC
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Q=78.29kN
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FC
F=1.00
Q=68.48kN
FC
F=1.00
Q=68.48kN
FC
F=1.75
Q=78.29kN
FC
F=1.75
Q=137.28kN
FC
F=1.75
Q=93.26kN
FC
F=1.75
Q=78.29kN
LIFT
LIFT
FC
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FC
F=1.00
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FC
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Q=93.26kN
FC
F=1.75
Q=93.26kN
FC
F=1.75
Q=78.29kN
GLASS PARTITION
FC
F=1.75
Q=78.29kN
FC
F=1.75
Q=78.29kN
FC
F=1.75
Q=78.29kN
FC
F=1.75
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FC
F=1.75
Q=78.29kN
FC
F=1.75
Q=78.29kN
FC
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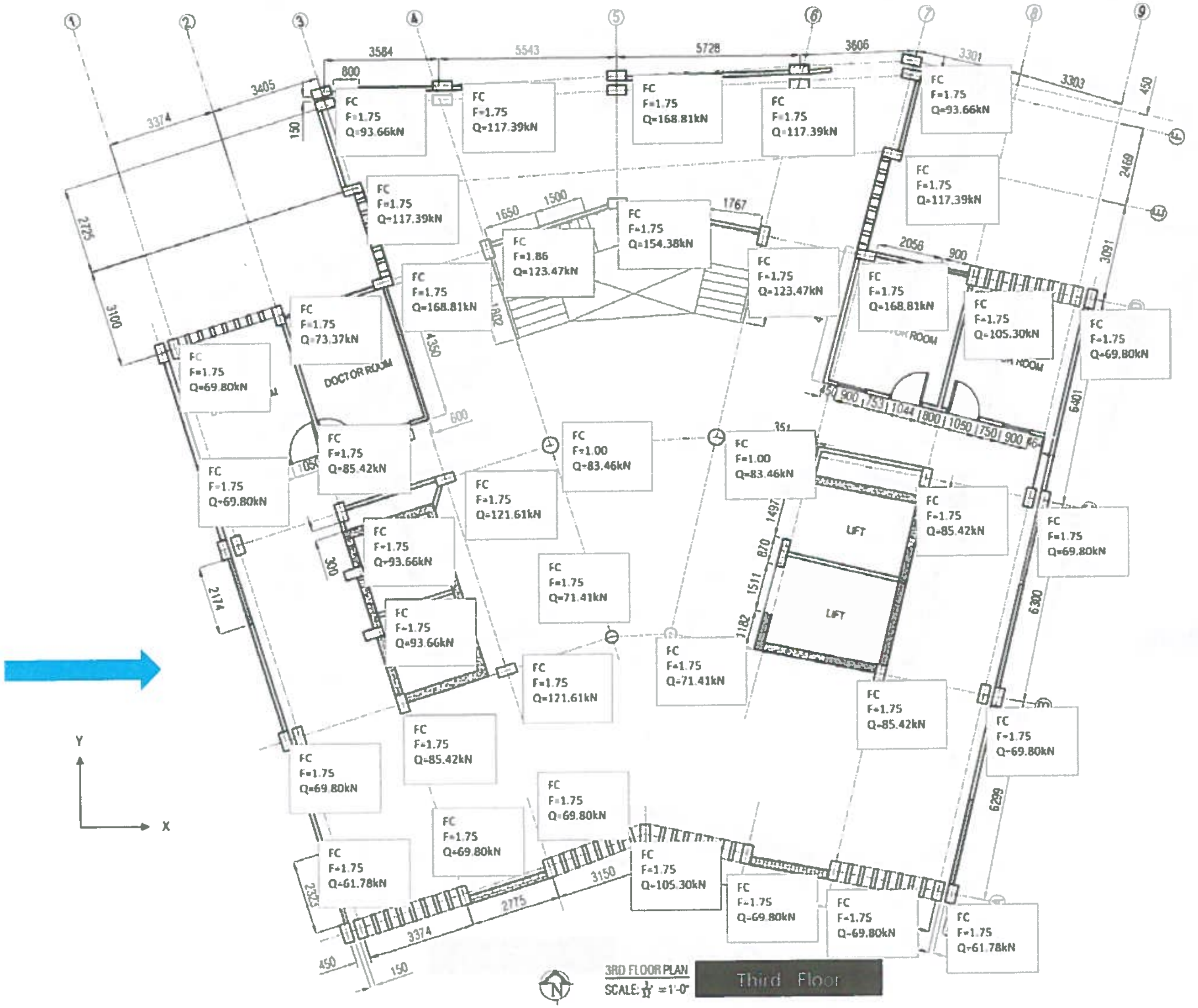
Q=67.44kN
Y
X
N
2ND FLOOR PLAN
SCALE: 1/32" = 1'-0"
Second Floor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Q=93.66kN
FC
F=1.75
Q=117.39kN
FC
F=1.75
Q=168.81kN
FC
F=1.75
Q=117.39kN
FC
F=1.75
Q=93.66kN
FC
F=1.75
Q=117.39kN
FC
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FC
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FC
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FC
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Q=168.81kN
FC
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FC
F=1.75
Q=69.80kN
FC
F=1.75
Q=73.37kN
FC
F=1.75
Q=69.80kN
DOCTOR ROOM
FC
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FC
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Q=83.46kN
FC
F=1.00
Q=83.46kN
FC
F=1.75
Q=69.80kN
FC
F=1.75
Q=121.61kN
FC
F=1.75
Q=93.66kN
FC
F=1.75
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FC
F=1.75
Q=69.80kN
FC
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LIFT
LIFT
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FC
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FC
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FC
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FC
F=1.75
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FC
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FC
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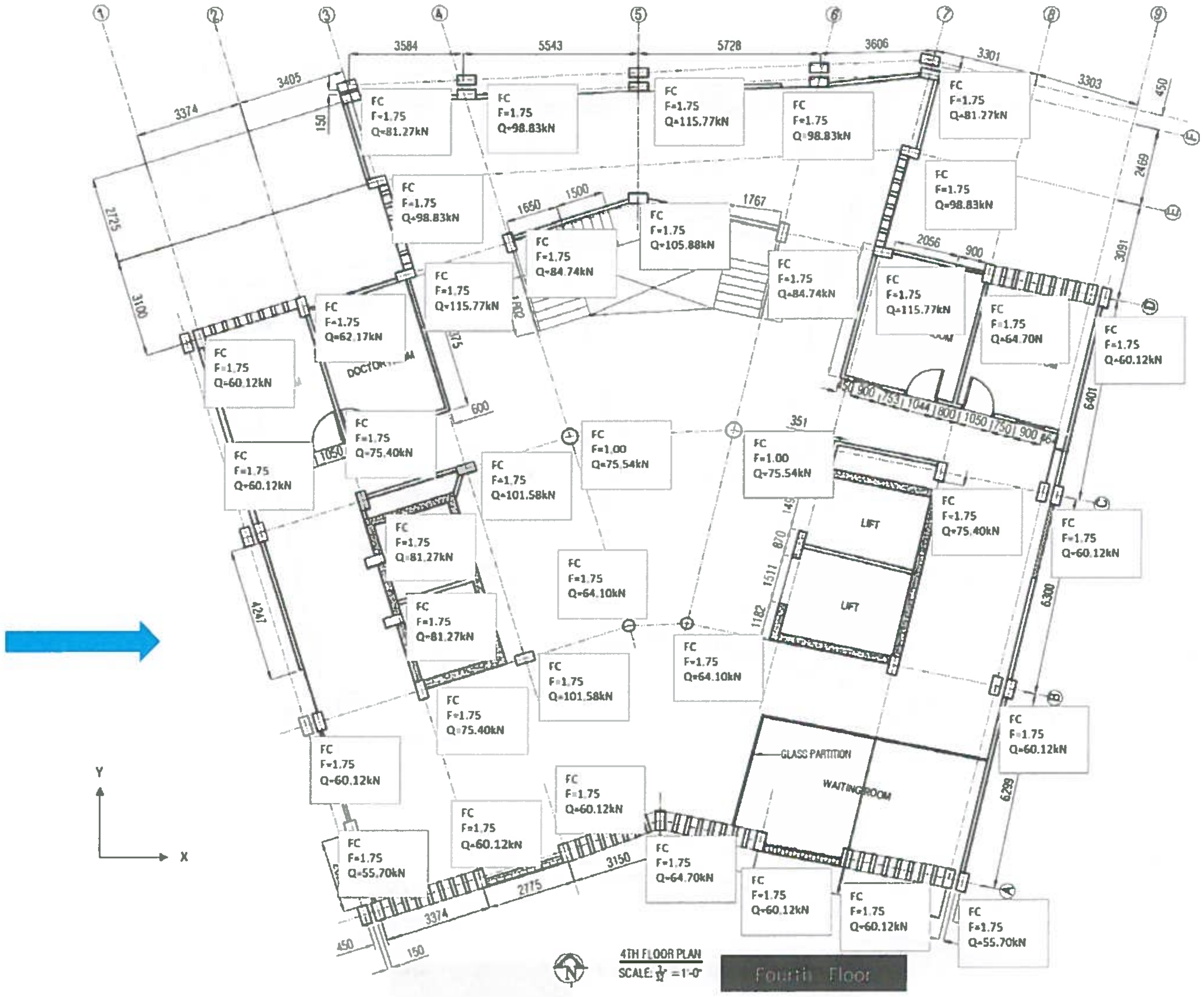
5543
5728
3606
3301
3303
450
800
3405
3374
150
2725
3100
1650
1500
1767
2056
900
2469
3091
1802
4350
600
300
2174
1497
870
1511
3182
6401
6300
6299
2323
3374
2775
3150
450
150
Y
X
3RD FLOOR PLAN
SCALE: 1/32" = 1'-0"
Third Floor


1
2
3
4
5
6
7
8
9
3584
5543
5728
3606
3301
3303
450
3405
3374
150
2725
3100
2469
3091
6401
6300
6299
F
E
D
C
B
A
1650
1500
1767
2056
900
1802
375
600
1050
351
149
870
1511
1182
900 1753 1044 1800 1050 1750 900
4247
3150
2775
3374
450
150
DOCTOR
LIFT
LIFT
GLASS PARTITION
WAITING ROOM
FC F=1.75 Q=81.27kN
FC F=1.75 Q=98.83kN
FC F=1.75 Q=115.77kN
FC F=1.75 Q=98.83kN
FC F=1.75 Q=81.27kN
FC F=1.75 Q=98.83kN
FC F=1.75 Q=98.83kN
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FC F=1.00 Q=75.54kN
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FC F=1.75 Q=101.58kN
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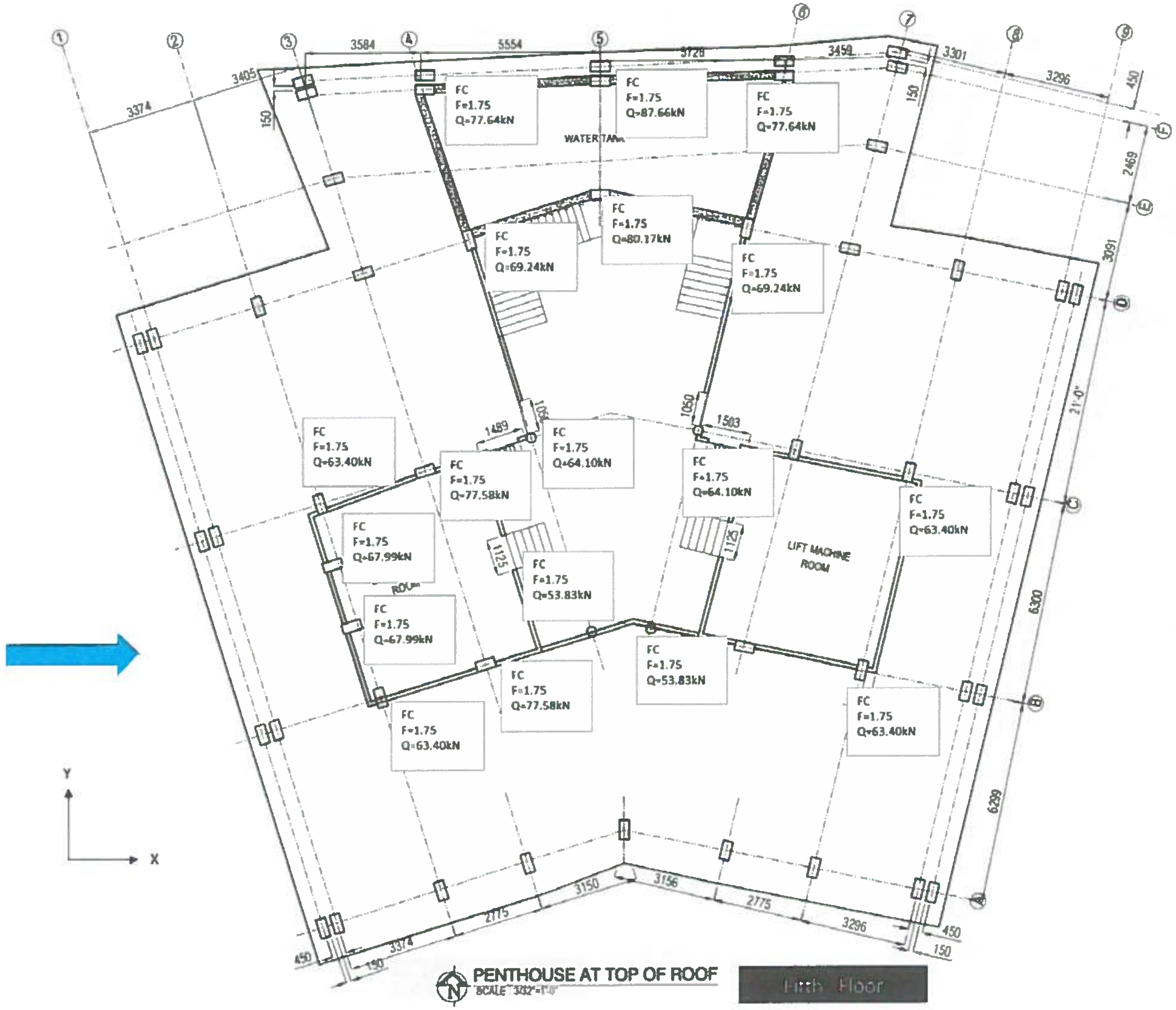
Y
X
N
4TH FLOOR PLAN
SCALE: 1/32" =1'-0"
Fourth Floor



FC
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FC
F=1.75
Q=87.66kN
FC
F=1.75
Q=77.64kN
WATER TANK
FC
F=1.75
Q=69.24kN
FC
F=1.75
Q=80.17kN
FC
F=1.75
Q=69.24kN
FC
F=1.75
Q=63.40kN
FC
F=1.75
Q=77.58kN
FC
F=1.75
Q=64.10kN
FC
F=1.75
Q=64.10kN
FC
F=1.75
Q=67.99kN
FC
F=1.75
Q=53.83kN
LIFT MACHINE ROOM
FC
F=1.75
Q=63.40kN
FC
F=1.75
Q=67.99kN
FC
F=1.75
Q=53.83kN
FC
F=1.75
Q=77.58kN
FC
F=1.75
Q=63.40kN
FC
F=1.75
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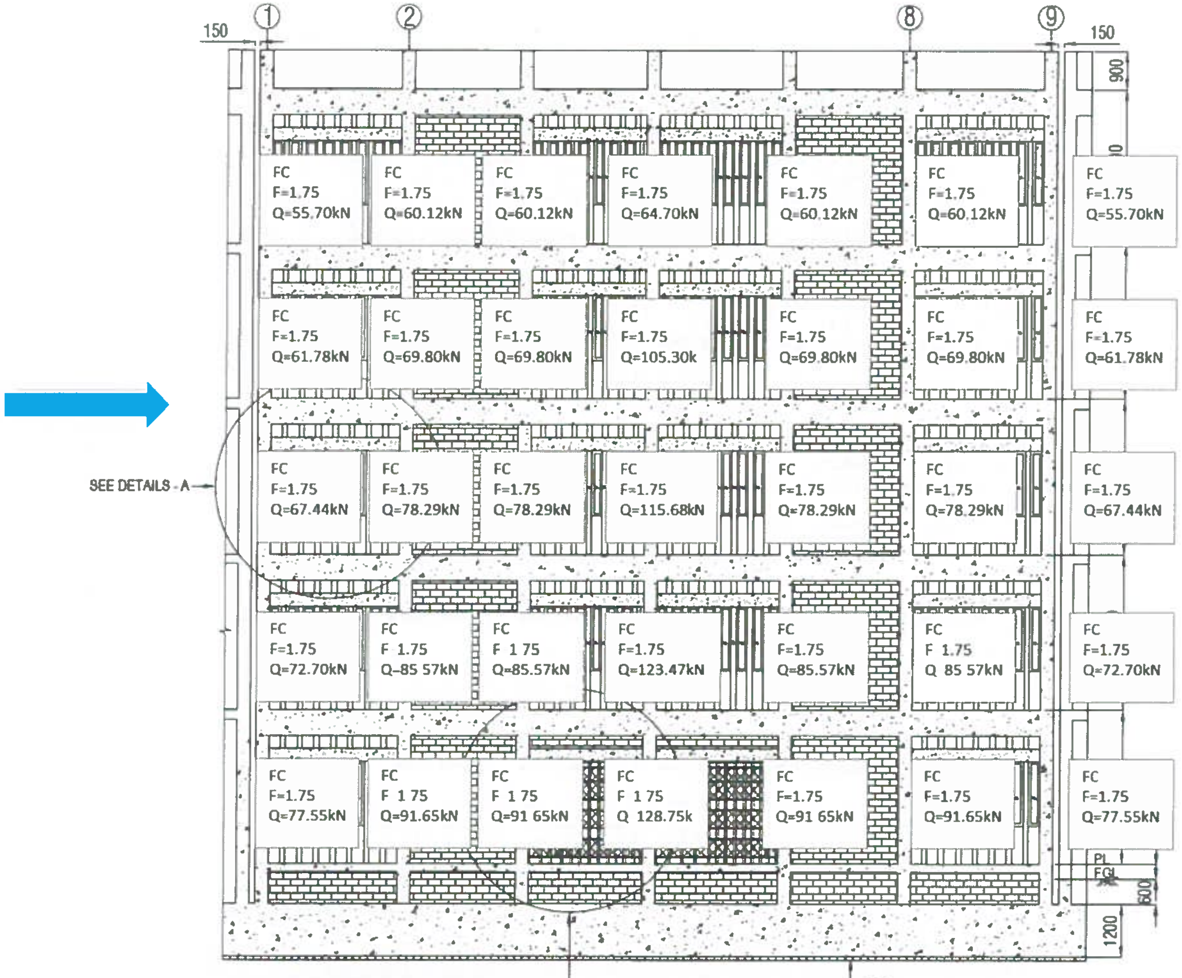
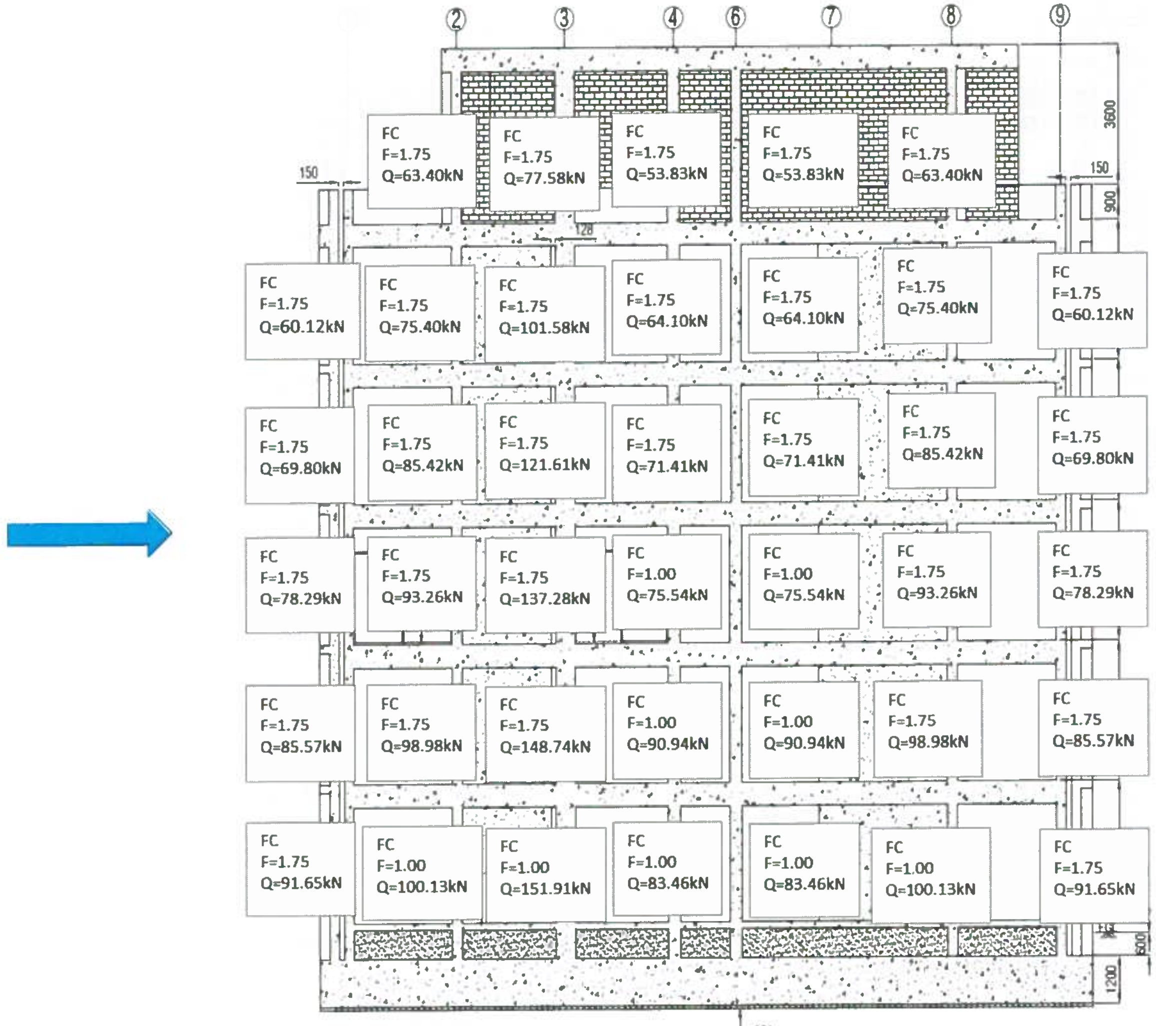
3584
5554
3728
3459
3301
3405
3374
3296
450
150
2469
3091
21'-0"
6300
6299
1489
1050
1503
1125
1125
3150
3156
2775
2775
3296
3374
450
150
Y
X
PENTHOUSE AT TOP OF ROOF
SCALE 3/32"=1'-0"
Fifth Floor

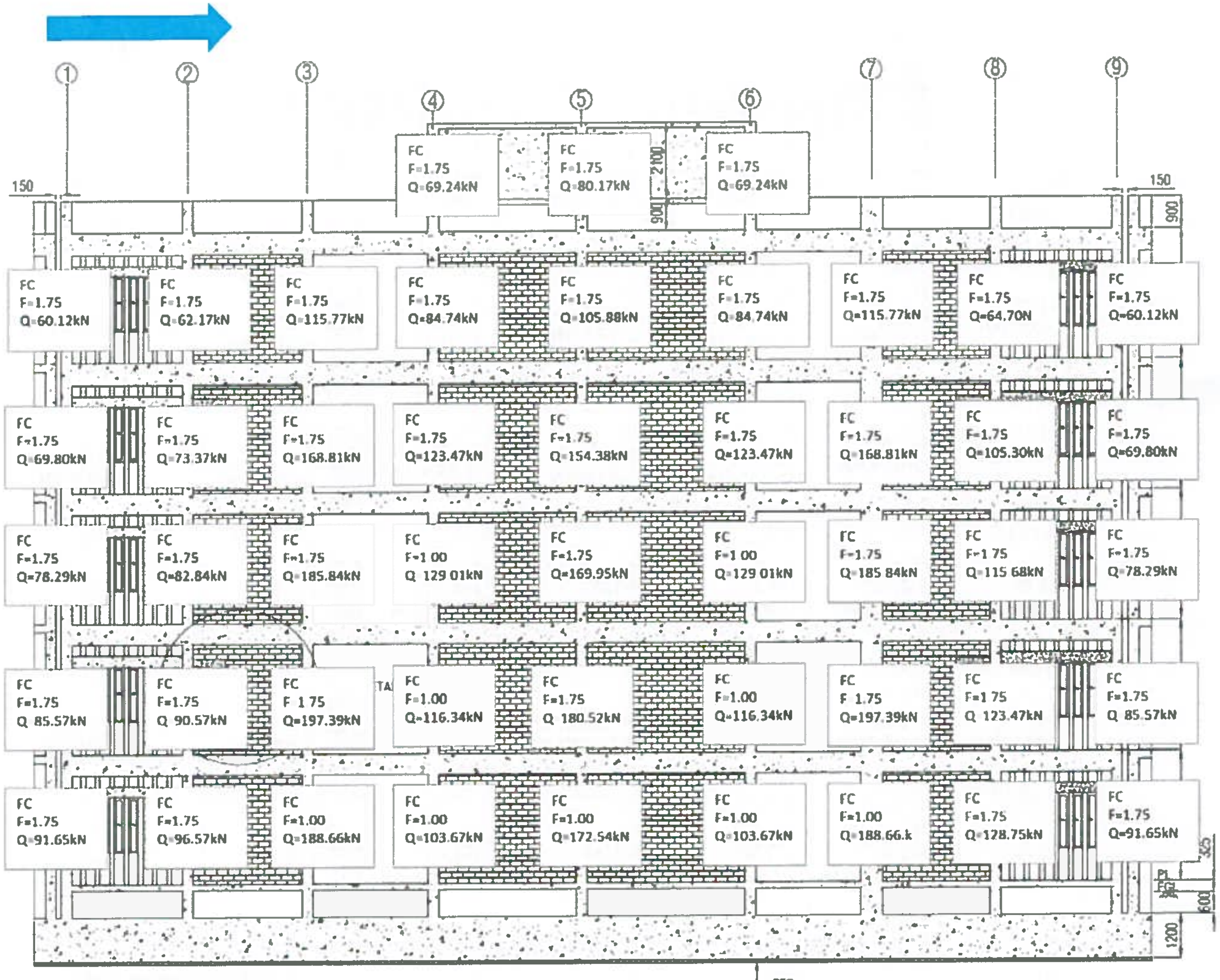


ELEVATION GRID A-A

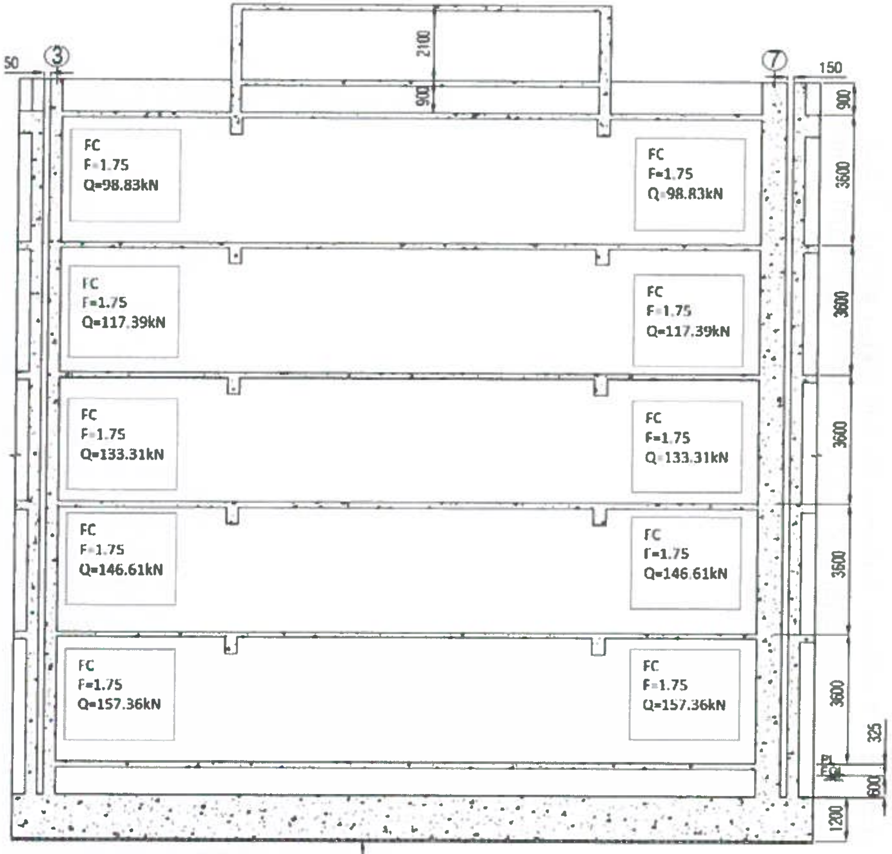


ELEVATION GRID B-B
SCALE 3/32"=1'-0"

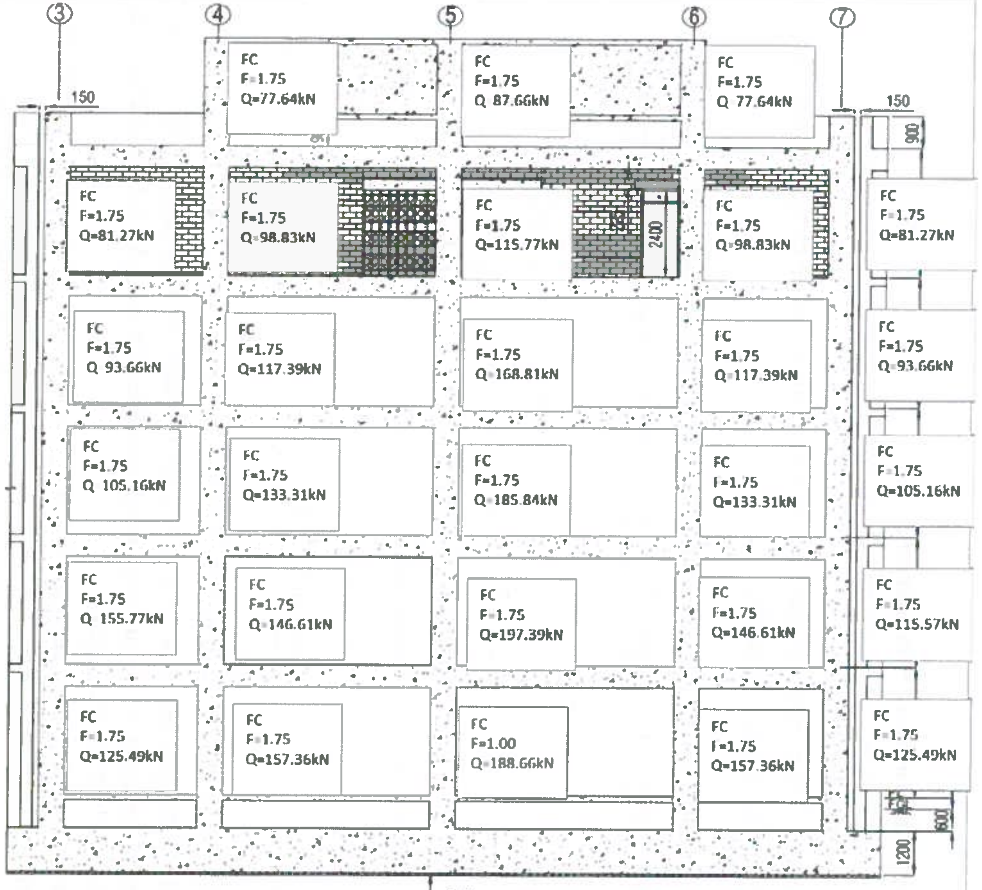
ELEVATION GRID C-C
SCALE 3/32"=1'-0"



ELEVATION GRID D-D
SCALE 3/32"=1'-0"



ELEVATION GRID E-E



ELEVATION GRID F-F

3.3 The Basic Seismic Index of Structure E_0

(1) The effective strength factor

The effective strength factor indicates the ratio of the restoring force at the ultimate deflection angle of the first group (R_1) to the ultimate strength.

Table S 9.6 Effective Strength Factor (In case the first group F_1)

Cumulative point of the first group $F1 > 1.0$ (Deformation angle $R1>R_{250}$=1/250)				
	F_1	$F_1=1.0$	$1.0<F_1<1.27$	$1.27<F_1$
	R_1	R_{250}	$R_{250}<R_1<R_{150}$	$R_{150}<R_1$
Second and higher groups	Shear ($R_{su}=R_{250}$)	1.0	0.0	0.0
	Shear ($R_1<R_{su}$)	α_S	α_S	0.0
	Flexural ($R_{my}<R_1$)	1.0	1.0	1.0
	Flexural ($R_1<R_{my}$)	α_m	α_m	1.0
	Flexural($R_{my}=R_{150}$)	0.72	α_m	1.0

(Note)

α_S : Effective strength factor of a shear column, calculated by $\alpha_s=Q(F_1)/Q_{su}=\alpha_m\ Q_{mu}/Q_{su} \leq 1.0$

α_m : Effective strength factor of a flexural column, calculated by $\alpha_m=(F_1)/Q_{mu}=0.3+0.7\times R_1/R_{my}$

R_{my} : Deformation angle at flexural yielding, calculated by Eq. (A1.3-1) in the Supplementary Provisions 1 of the standard.

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

$Q(F_1)$: Shear force at the deformation capacity R_1 of a column in the second and higher groups.

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

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

Table S 9.7 Effective Strength Factor (4th to 6th Level)

Story	Direction	No	Name	equivalent column Nomber	R1=1/500			R1=1/250			R1=1/200			R1=1/166			1/150<R1	
					α s(1/500)	α m(1/500)	α Qu	α s(1/250)	α m(1/250)	α Qu	α s(1/200)	α m(1/200)	α Qu	α s(1/166)	α m(1/166)	α Qu	α	α Qu
6	X	1	C1	2	–	0.51	69.35	–	0.72	97.91	–	0.83	112.19	–	0.93	126.81	1.00	135.99
		5	C1-D	2	–	0.51	79.20	–	0.72	111.81	–	0.83	128.11	–	0.93	144.81	1.00	155.29
		6	C1-E	2	–	0.51	79.13	–	0.72	111.71	–	0.83	128.01	–	0.93	144.69	1.00	155.16
		7	C1-F	4	–	0.51	129.34	–	0.72	182.60	–	0.83	209.23	–	0.93	236.50	1.00	253.61
		8	C2-A	1	–	0.51	44.71	–	0.72	63.12	–	0.83	72.32	–	0.93	81.75	1.00	87.66
		9	C2-B	1	–	0.51	40.88	–	0.72	57.72	–	0.83	66.14	–	0.93	74.76	1.00	80.17
		11	C3	2	–	0.51	70.62	–	0.72	99.70	–	0.83	114.24	–	0.93	129.13	1.00	138.48
		12	C4	2	–	0.51	65.38	–	0.72	92.30	–	0.83	105.76	–	0.93	119.54	1.00	128.19
		13	C5	2	–	0.51	54.91	–	0.72	77.52	–	0.83	88.82	–	0.93	100.40	1.00	107.66
5	X	1	C1	2	–	0.51	82.90	–	0.72	117.03	–	0.83	134.10	–	0.93	151.58	1.00	162.54
		2	C1-A	2	–	0.51	56.82	–	0.72	80.21	–	0.83	91.91	–	0.93	103.89	1.00	111.41
		3	C1-B	10	–	0.51	306.60	–	0.72	432.84	–	0.83	495.97	–	0.93	560.61	1.00	601.17
		4	C1-C	1	–	0.51	31.70	–	0.72	44.76	–	0.83	51.29	–	0.93	57.97	1.00	62.17
		5	C1-D	4	–	0.51	201.61	–	0.72	284.63	–	0.83	326.14	–	0.93	368.65	1.00	395.32
		6	C1-E	2	–	0.51	103.61	–	0.72	146.27	–	0.83	167.60	–	0.93	189.45	1.00	203.15
		7	C1-F	4	–	0.51	153.82	–	0.72	217.16	–	0.83	248.83	–	0.93	281.26	1.00	301.61
		8	C2-A	3	–	0.51	177.13	–	0.72	250.07	–	0.83	286.53	–	0.93	323.88	1.00	347.31
		9	C2-B	1	–	0.51	54.00	–	0.72	76.23	–	0.83	87.35	–	0.93	98.73	1.00	105.88
		10	C2-C	2	–	0.51	65.99	–	0.72	93.16	–	0.83	106.75	–	0.93	120.66	1.00	129.39
		11	C3	2	–	0.51	86.43	–	0.72	122.02	–	0.83	139.81	–	0.93	158.04	1.00	169.47
		12	C4	2	–	0.65	98.21	–	1.00	151.09	–	0.00	0.00	–	0.00	0.00	0.00	0.00
		13	C5	2	–	0.51	65.38	–	0.72	92.30	–	0.83	105.76	–	0.93	119.54	1.00	128.19
4	X	1	C1	2	–	0.51	95.53	–	0.72	134.87	–	0.83	154.54	–	0.93	174.68	1.00	187.32
		2	C1-A	2	–	0.51	63.01	–	0.72	88.96	–	0.83	101.93	–	0.93	115.22	1.00	123.55
		3	C1-B	10	–	0.51	356.00	–	0.72	502.59	–	0.83	575.89	–	0.93	650.95	1.00	698.05
		4	C1-C	1	–	0.51	37.42	–	0.72	52.83	–	0.83	60.53	–	0.93	68.42	1.00	73.37
		5	C1-D	4	–	0.51	239.47	–	0.72	338.07	–	0.83	387.38	–	0.93	437.87	1.00	469.55
		6	C1-E	2	–	0.51	124.04	–	0.72	175.11	–	0.83	200.65	–	0.93	226.81	1.00	243.21
		7	C1-F	4	–	0.51	174.25	–	0.72	246.00	–	0.83	281.88	–	0.93	318.62	1.00	341.67
		8	C2-A	3	–	0.51	258.28	–	0.72	364.62	–	0.83	417.80	–	0.93	472.25	1.00	506.42
		9	C2-B	1	–	0.51	78.73	–	0.72	111.15	–	0.83	127.36	–	0.93	143.96	1.00	154.38
		10	C2-C	2	–	0.51	107.41	–	0.72	151.63	–	0.83	173.75	–	0.93	196.39	1.00	210.60
		11	C3	2	–	0.51	125.93	–	0.72	177.79	–	0.83	203.72	–	0.93	230.27	1.00	246.93
		12	C4	2	–	0.65	108.49	–	1.00	166.91	–	0.00	0.00	–	0.00	0.00	0.00	0.00
		13	C5	2	–	0.51	72.83	–	0.72	102.82	–	0.83	117.82	–	0.93	133.18	1.00	142.81

Table S 9.8 Effective Strength Factor (1st to 3rd Level)

Story	Direction	No	Name	equivalent column Nomber	R1=1/500			R1=1/250			R1=1/200			R1=1/166			1/150<R1	
					α s(1/500)	α m(1/500)	α Qu	α s(1/250)	α m(1/250)	α Qu	α s(1/200)	α m(1/200)	α Qu	α s(1/166)	α m(1/166)	α Qu	α	α Qu
3	X	1	C1	2	-	0.51	107.26	-	0.72	151.43	-	0.83	173.51	-	0.93	196.13	1.00	210.32
		2	C1-A	2	-	0.51	68.79	-	0.72	97.11	-	0.83	111.28	-	0.93	125.78	1.00	134.88
		3	C1-B	10	-	0.51	399.28	-	0.72	563.69	-	0.83	645.89	-	0.93	730.08	1.00	782.90
		4	C1-C	1	-	0.51	42.25	-	0.72	59.64	-	0.83	68.34	-	0.93	77.25	1.00	82.84
		5	C1-D	4	-	0.51	271.96	-	0.72	383.95	-	0.83	439.94	-	0.93	497.28	1.00	533.26
		6	C1-E	2	-	0.51	140.03	-	0.72	197.69	-	0.83	228.52	-	0.93	256.04	1.00	274.57
		7	C1-F	4	-	0.51	190.24	-	0.72	268.58	-	0.83	307.74	-	0.93	347.86	1.00	373.02
		8	C2-A	3	-	0.51	284.33	-	0.72	401.41	-	0.83	459.95	-	0.93	519.90	1.00	557.52
		9	C2-B	1	-	0.51	86.68	-	0.72	122.37	-	0.83	140.21	-	0.93	158.49	1.00	169.95
		10	C2-C	2	-	0.51	117.99	-	0.72	166.57	-	0.83	190.87	-	0.93	215.74	1.00	231.35
		11	C3	2	-	0.65	167.71	-	1.00	258.01	-	0.00	0.00	-	0.00	0.00	0.00	0.00
		12	C4	2	-	0.65	89.03	-	1.00	136.97	-	0.00	0.00	-	0.00	0.00	0.00	0.00
		13	C5	2	-	0.65	98.21	-	1.00	151.09	-	0.00	0.00	-	0.00	0.00	0.00	0.00
2	X	1	C1	2	-	0.51	118.09	-	0.72	166.71	-	0.83	191.02	-	0.93	215.92	1.00	231.54
		2	C1-A	2	-	0.51	74.15	-	0.72	104.69	-	0.83	119.95	-	0.93	135.59	1.00	145.40
		3	C1-B	10	-	0.51	436.42	-	0.72	616.13	-	0.83	705.98	-	0.93	797.99	1.00	855.73
		4	C1-C	1	-	0.51	46.19	-	0.72	65.21	-	0.83	74.72	-	0.93	84.46	1.00	90.57
		5	C1-D	4	-	0.51	299.09	-	0.72	422.25	-	0.83	483.82	-	0.93	546.89	1.00	586.45
		6	C1-E	2	-	0.51	151.71	-	0.72	214.18	-	0.83	245.41	-	0.93	277.40	1.00	297.47
		7	C1-F	4	-	0.51	201.92	-	0.72	285.07	-	0.83	326.64	-	0.93	369.21	1.00	395.93
		8	C2-A	3	-	0.51	302.01	-	0.72	426.37	-	0.83	488.55	-	0.93	552.23	1.00	592.18
		9	C2-B	1	-	0.51	92.07	-	0.72	129.97	-	0.83	148.93	-	0.93	168.34	1.00	180.52
		10	C2-C	2	-	0.51	125.93	-	0.72	177.79	-	0.83	203.72	-	0.93	230.27	1.00	246.93
		11	C3	2	-	0.65	151.24	-	1.00	232.68	-	0.00	0.00	-	0.00	0.00	0.00	0.00
		12	C4	2	-	0.65	216.84	-	1.00	333.60	-	0.00	0.00	-	0.00	0.00	0.00	0.00
		13	C5	2	-	0.65	118.23	-	1.00	181.88	-	0.00	0.00	-	0.00	0.00	0.00	0.00
1	X	1	C1	2	-	0.51	128.00	-	0.72	180.71	-	0.83	207.07	-	0.93	234.05	1.00	250.99
		2	C1-A	2	-	0.51	79.10	-	0.72	111.67	-	0.83	127.96	-	0.93	144.64	1.00	155.10
		3	C1-B	10	-	0.51	467.44	-	0.72	659.91	-	0.83	756.15	-	0.93	854.70	1.00	916.54
		4	C1-C	1	-	0.51	49.25	-	0.72	69.53	-	0.83	79.67	-	0.93	90.05	1.00	96.57
		5	C1-D	4	-	0.51	321.00	-	0.72	453.18	-	0.83	519.27	-	0.93	586.95	1.00	629.42
		6	C1-E	2	-	0.65	197.49	-	1.00	303.83	-	0.00	0.00	-	0.00	0.00	0.00	0.00
		7	C1-F	4	-	0.65	260.35	-	1.00	400.53	-	0.00	0.00	-	0.00	0.00	0.00	0.00
		8	C2-A	3	-	0.65	367.90	-	1.00	565.99	-	0.00	0.00	-	0.00	0.00	0.00	0.00
		9	C2-B	1	-	0.65	112.15	-	1.00	172.54	-	0.00	0.00	-	0.00	0.00	0.00	0.00
		10	C2-C	2	-	0.51	131.32	-	0.72	185.40	-	0.83	212.44	-	0.93	240.12	1.00	257.50
		11	C3	2	-	0.65	134.78	-	1.00	207.35	-	0.00	0.00	-	0.00	0.00	0.00	0.00
		12	C4	2	-	0.65	190.84	-	1.00	293.60	-	0.00	0.00	-	0.00	0.00	0.00	0.00
		13	C5	2	-	0.65	108.49	-	1.00	166.91	-	0.00	0.00	-	0.00	0.00	0.00	0.00

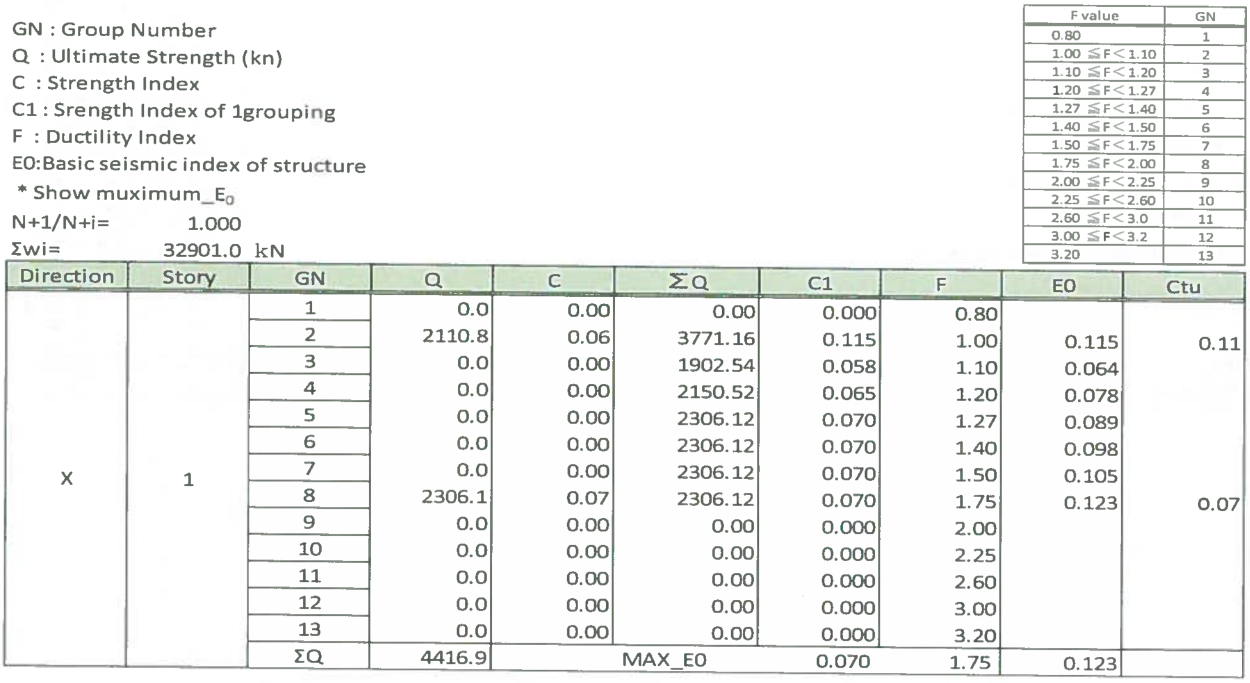
Basic seismic capacity index have to be calculated based on both strength dominant equation and ductility dominant equation. The basic seismic capacity index have to checked at different critical stage of failure forming some grouping among vertical structural member. The maximum value between two equations among various combinations will be the basic seismic capacity index of a particular storey for a specific direction.

(2) Basic seismic capacity index E_0 is calculated by strength dominant equation
The Basic seismic capacity index E_0 is calculated by strength dominant equation with bellow.
Strength-dominant basic seismic index of structure (Eq. (5))

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

Where:

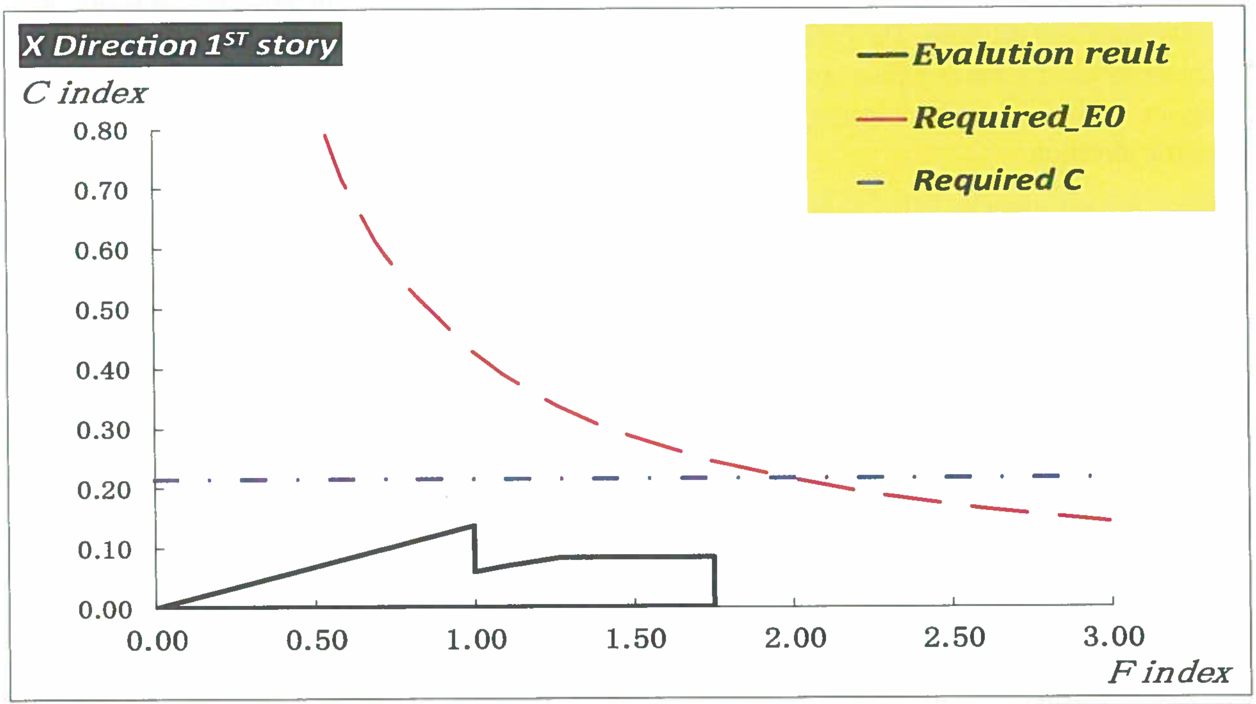
C_1 : Strength index of the first group
C_j : Strength index of the *j-th* group
F_1 : Ductility index of the first group
α_j : Effective strength factor in the *j-th* group at the ultimate deformation R_1 corresponding to the first group (ductility index of F_1), given in bellow Table 1.



GN : Group Number
Q : Ultimate Strength (kn)
C : Strength Index
C1 : Srength Index of 1grouping
F : Ductility Index
E0:Basic seismic index of structure
* Show muximum_E_0
N+1/N+i= 1.000
Σwi= 32901.0 kN

F value	GN
0.80	1
1.00 ≦F＜1.10	2
1.10 ≦F＜1.20	3
1.20 ≦F＜1.27	4
1.27 ≦F＜1.40	5
1.40 ≦F＜1.50	6
1.50 ≦F＜1.75	7
1.75 ≦F＜2.00	8
2.00 ≦F＜2.25	9
2.25 ≦F＜2.60	10
2.60 ≦F＜3.0	11
3.00 ≦F＜3.2	12
3.20	13

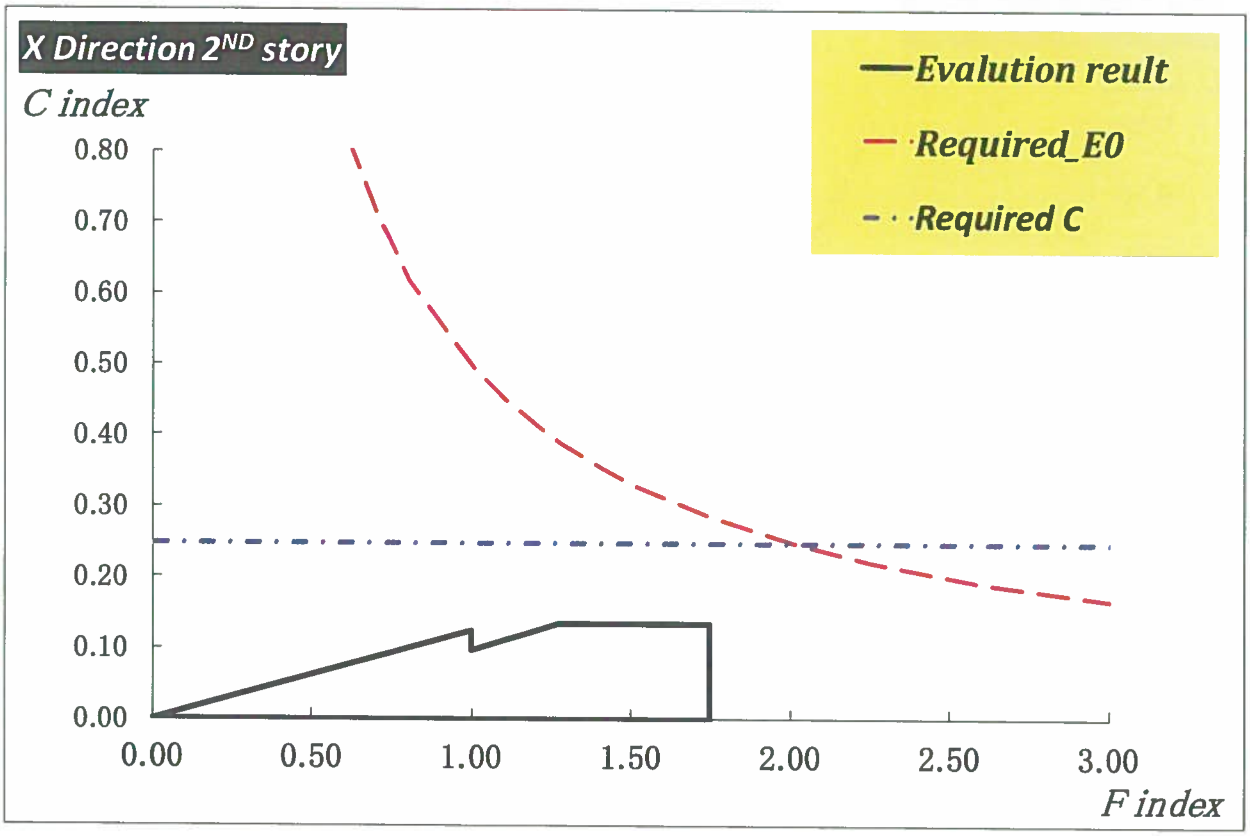
Direction	Story	GN	Q	C	ΣQ	C1	F	E0	Ctu
X	1	1	0.0	0.00	0.00	0.000	0.80		
		2	2110.8	0.06	3771.16	0.115	1.00	0.115	0.11
		3	0.0	0.00	1902.54	0.058	1.10	0.064	
		4	0.0	0.00	2150.52	0.065	1.20	0.078	
		5	0.0	0.00	2306.12	0.070	1.27	0.089	
		6	0.0	0.00	2306.12	0.070	1.40	0.098	
		7	0.0	0.00	2306.12	0.070	1.50	0.105	
		8	2306.1	0.07	2306.12	0.070	1.75	0.123	0.07
		9	0.0	0.00	0.00	0.000	2.00		
		10	0.0	0.00	0.00	0.000	2.25		
		11	0.0	0.00	0.00	0.000	2.60		
		12	0.0	0.00	0.00	0.000	3.00		
		13	0.0	0.00	0.00	0.000	3.20		
		ΣQ	4416.9	MAX_E0		0.070	1.75	0.123	



GN : Group Number
Q : Ultimate Strength (kn)
C : Strength Index
C1 : Srength Index of 1grouping
F : Ductility Index
E0:Basic seismic index of structure
* Show muximum_E_0
N+1/N+i= 0.875
Σwi= 27092.0 k N

F value	GN
0.80	1
1.00 ≦F＜1.10	2
1.10 ≦F＜1.20	3
1.20 ≦F＜1.27	4
1.27 ≦F＜1.40	5
1.40 ≦F＜1.50	6
1.50 ≦F＜1.75	7
1.75 ≦F＜2.00	8
2.00 ≦F＜2.25	9
2.25 ≦F＜2.60	10
2.60 ≦F＜3.0	11
3.00 ≦F＜3.2	12
3.20	13

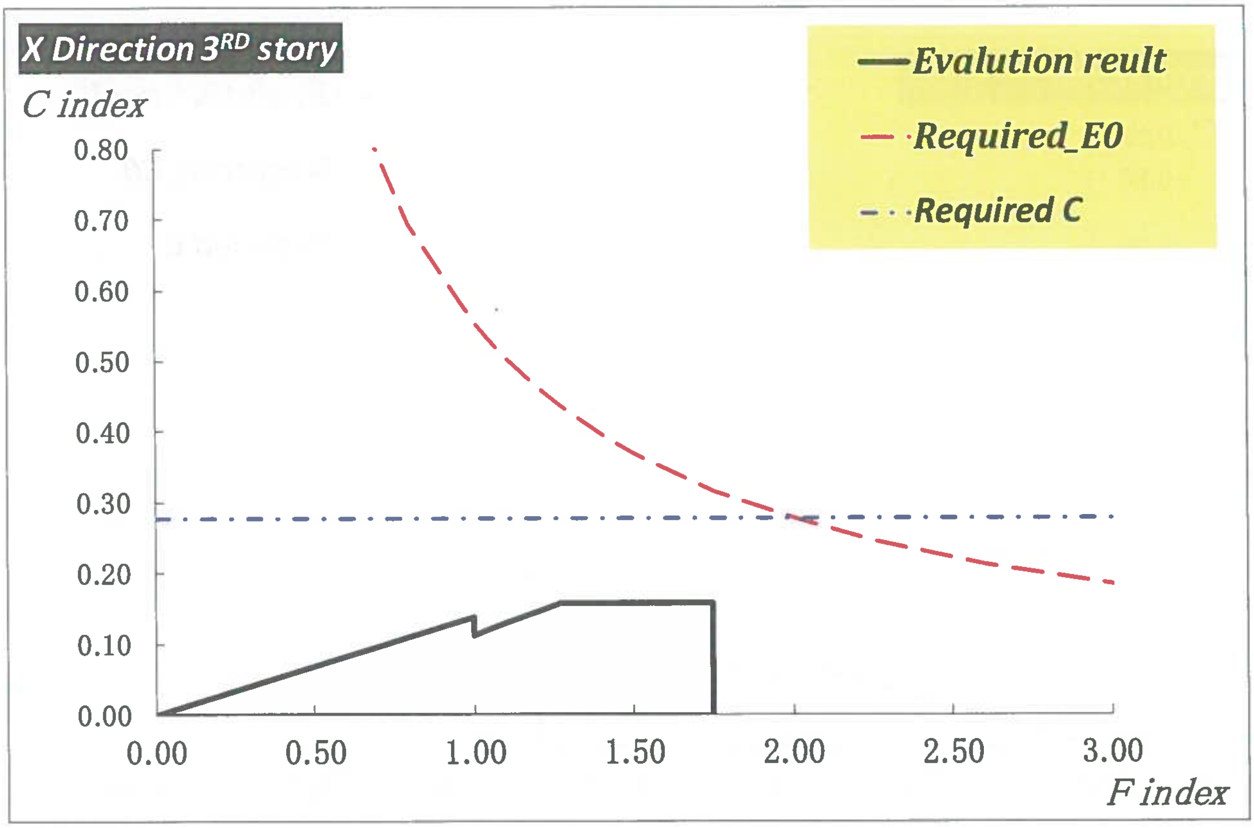
Direction	Story	GN	Q	C	ΣQ	C1	F	E0	Ctu
X	2	1	0.0	0.00	0.00	0.000	0.80		
		2	748.2	0.028	3356.52	0.124	1.00	0.108	0.11
		3	0.0	0.000	2988.75	0.110	1.10	0.106	
		4	0.0	0.000	3378.30	0.125	1.20	0.131	
		5	0.0	0.000	3622.73	0.134	1.27	0.149	
		6	0.0	0.000	3622.73	0.134	1.40	0.164	
		7	0.0	0.000	3622.73	0.134	1.50	0.176	
		8	3622.7	0.134	3622.73	0.134	1.75	0.205	0.12
		9	0.0	0.000	0.00	0.000	2.00		
		10	0.0	0.000	0.00	0.000	2.25		
		11	0.0	0.000	0.00	0.000	2.60		
		12	0.0	0.000	0.00	0.000	3.00		
		13	0.0	0.000	0.00	0.000	3.20		
		ΣQ	4370.9		MAX_E0	0.134	1.75	0.205	



GN : Group Number
Q : Ultimate Strength (kn)
C : Strength Index
C1 : Srength Index of 1grouping
F : Ductility Index
E0:Basic seismic index of structure
* Show muximum_E_0
N+1/N+i= 0.778
Σwi= 21279.0 k N

F value	GN
0.80	1
1.00 ≦F＜1.10	2
1.10 ≦F＜1.20	3
1.20 ≦F＜1.27	4
1.27 ≦F＜1.40	5
1.40 ≦F＜1.50	6
1.50 ≦F＜1.75	7
1.75 ≦F＜2.00	8
2.00 ≦F＜2.25	9
2.25 ≦F＜2.60	10
2.60 ≦F＜3.0	11
3.00 ≦F＜3.2	12
3.20	13

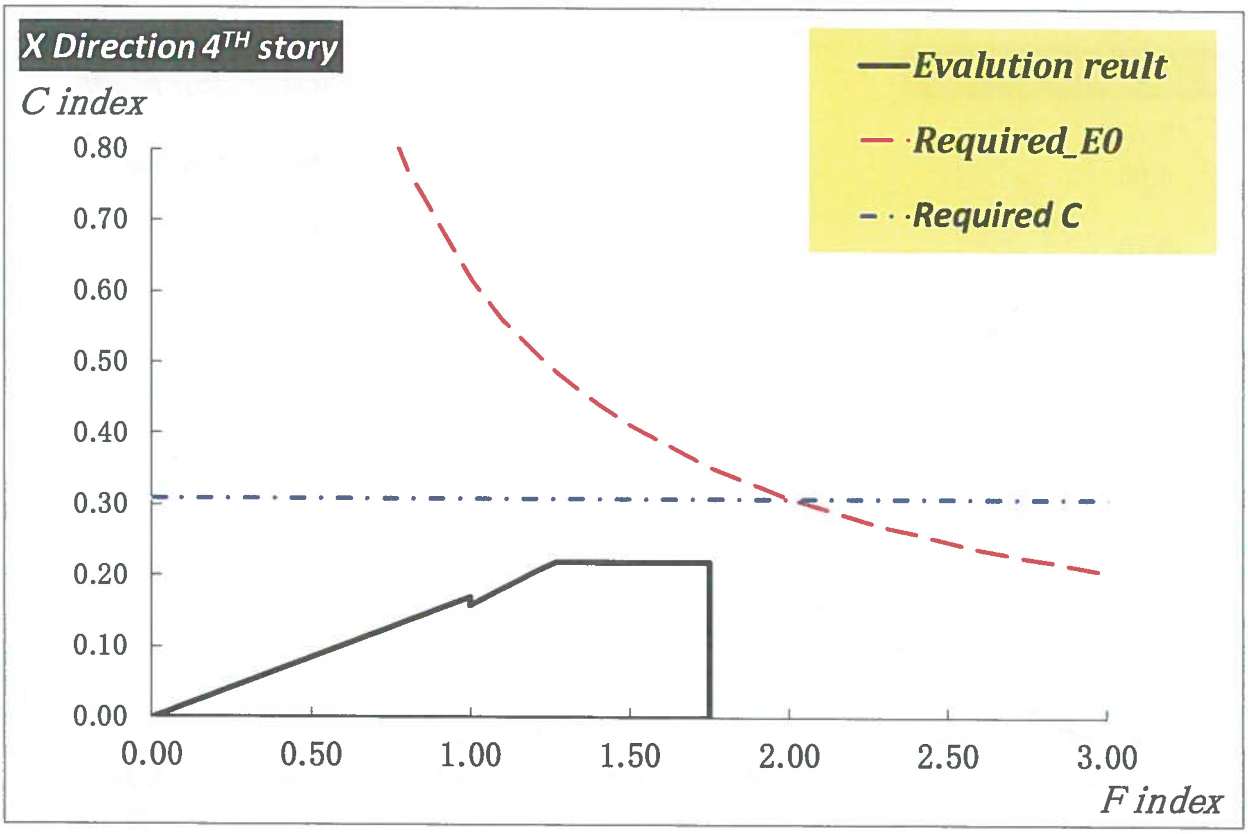
Direction	Story	GN	Q	C	ΣQ	C1	F	E0	Ctu
X	3	1	0.0	0.00	0.00	0.000	0.80		
		2	546.1	0.026	2958.51	0.139	1.00	0.108	0.11
		3	0.0	0.000	2764.26	0.130	1.10	0.111	
		4	0.0	0.000	3124.55	0.147	1.20	0.137	
		5	0.0	0.000	3350.62	0.157	1.27	0.156	
		6	0.0	0.000	3350.62	0.157	1.40	0.171	
		7	0.0	0.000	3350.62	0.157	1.50	0.184	
		8	3350.6	0.157	3350.62	0.157	1.75	0.214	0.12
		9	0.0	0.000	0.00	0.000	2.00		
		10	0.0	0.000	0.00	0.000	2.25		
		11	0.0	0.000	0.00	0.000	2.60		
		12	0.0	0.000	0.00	0.000	3.00		
		13	0.0	0.000	0.00	0.000	3.20		
		ΣQ	3896.7	MAX_E0		0.157	1.75	0.214	



F value	GN
0.80	1
1.00 ≦F＜1.10	2
1.10 ≦F＜1.20	3
1.20 ≦F＜1.27	4
1.27 ≦F＜1.40	5
1.40 ≦F＜1.50	6
1.50 ≦F＜1.75	7
1.75 ≦F＜2.00	8
2.00 ≦F＜2.25	9
2.25 ≦F＜2.60	10
2.60 ≦F＜3.0	11
3.00 ≦F＜3.2	12
3.20	13

GN : Group Number
Q : Ultimate Strength (kn)
C : Strength Index
C1 : Srength Index of 1grouping
F : Ductility Index
E0:Basic seismic index of structure
* Show muximum_E_0
N+1/N+i= 0.700
Σwi= 15468.0 k N

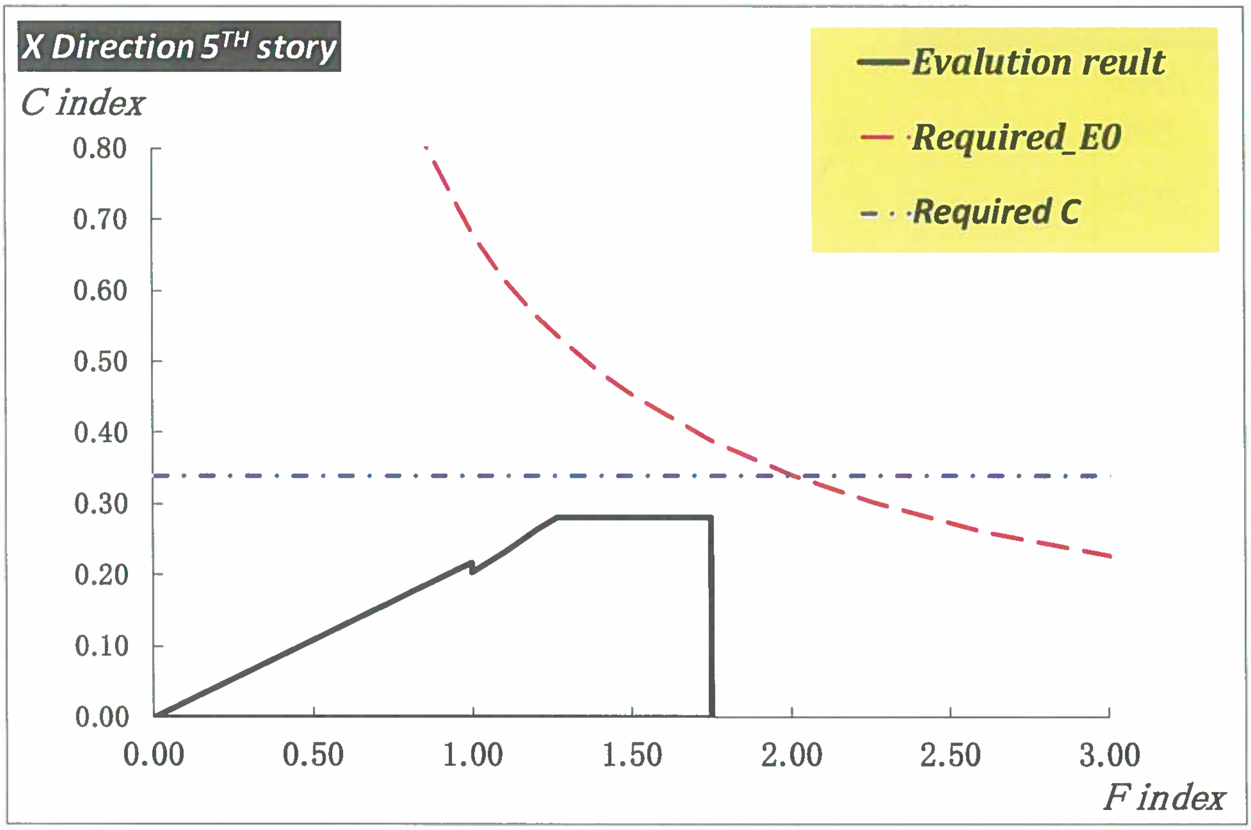
Direction	Story	GN	Q	C	ΣQ	C1	F	E0	Ctu
X	4	1	0.0	0.00	0.00	0.000	0.80		
		2	166.9	0.011	2613.37	0.169	1.00	0.118	0.12
		3	0.0	0.000	2803.24	0.181	1.10	0.140	
		4	0.0	0.000	3168.61	0.205	1.20	0.172	
		5	0.0	0.000	3397.86	0.220	1.27	0.195	
		6	0.0	0.000	3397.86	0.220	1.40	0.215	
		7	0.0	0.000	3397.86	0.220	1.50	0.231	
		8	3397.9	0.220	3397.86	0.220	1.75	0.269	0.15
		9	0.0	0.000	0.00	0.000	2.00		
		10	0.0	0.000	0.00	0.000	2.25		
		11	0.0	0.000	0.00	0.000	2.60		
		12	0.0	0.000	0.00	0.000	3.00		
		13	0.0	0.000	0.00	0.000	3.20		
		ΣQ	3564.8	MAX_E0		0.220	1.75	0.269	



F value	GN
0.80	1
1.00 ≦F＜1.10	2
1.10 ≦F＜1.20	3
1.20 ≦F＜1.27	4
1.27 ≦F＜1.40	5
1.40 ≦F＜1.50	6
1.50 ≦F＜1.75	7
1.75 ≦F＜2.00	8
2.00 ≦F＜2.25	9
2.25 ≦F＜2.60	10
2.60 ≦F＜3.0	11
3.00 ≦F＜3.2	12
3.20	13

GN : Group Number
Q : Ultimate Strength (kn)
C : Strength Index
C1 : Srength Index of 1grouping
F : Ductility Index
E0:Basic seismic index of structure
＊Show muximum_E_0
N+1/N+i= 0.636
Σwi= 9649.0 k N

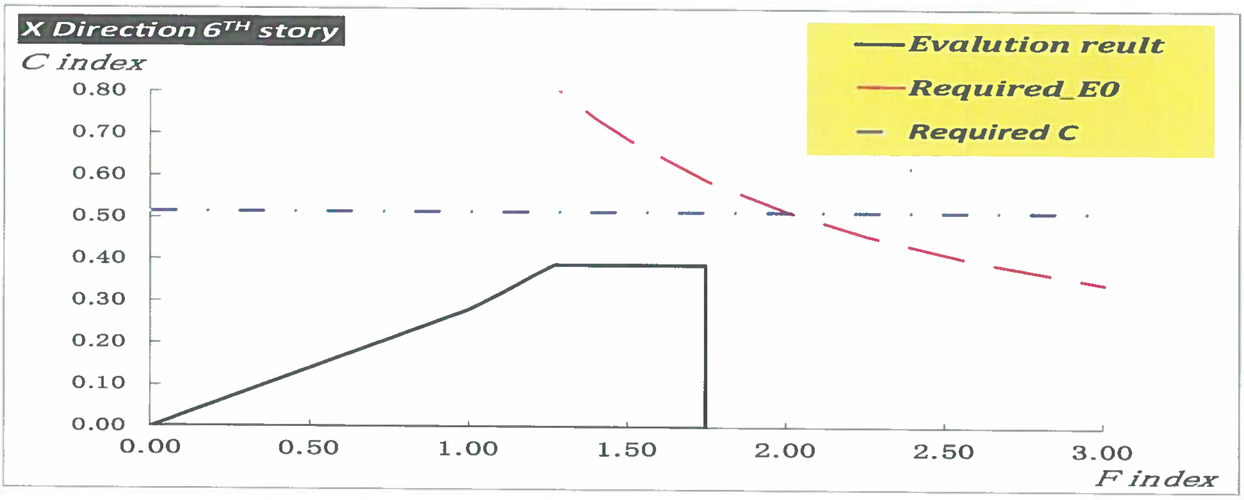
Direction	Story	GN	Q	C	ΣQ	C1	F	E0	Ctu
X	5	1	0.0	0.00	0.00	0.000	0.80		
		2	151.1	0.016	2107.77	0.218	1.00	0.139	0.14
		3	0.0	0.000	2242.04	0.232	1.10	0.163	
		4	0.0	0.000	2534.26	0.263	1.20	0.201	
		5	0.0	0.000	2717.62	0.282	1.27	0.228	
		6	0.0	0.000	2717.62	0.282	1.40	0.251	
		7	0.0	0.000	2717.62	0.282	1.50	0.269	
		8	2717.6	0.282	2717.62	0.282	1.75	0.314	0.18
		9	0.0	0.000	0.00	0.000	2.00		
		10	0.0	0.000	0.00	0.000	2.25		
		11	0.0	0.000	0.00	0.000	2.60		
		12	0.0	0.000	0.00	0.000	3.00		
		13	0.0	0.000	0.00	0.000	3.20		
		ΣQ	2868.7	MAX_E0		0.282	1.75	0.314	



GN : Group Number
Q : Ultimate Strength (kn)
C : Strength Index
C1 : Srength Index of 1grouping
F : Ductility Index
E0:Basic seismic index of structure
* Show muximum_E_0
N+1/N+i= 0.583
Σwi= 3210.0 k N

F value	GN
0.80	1
1.00 ≦F＜1.10	2
1.10 ≦F＜1.20	3
1.20 ≦F＜1.27	4
1.27 ≦F＜1.40	5
1.40 ≦F＜1.50	6
1.50 ≦F＜1.75	7
1.75 ≦F＜2.00	8
2.00 ≦F＜2.25	9
2.25 ≦F＜2.60	10
2.60 ≦F＜3.0	11
3.00 ≦F＜3.2	12
3.20	13

Direction	Story	GN	Q	C	ΣQ	C1	F	E0	Ctu
X	6	1	0.0	0.000	0.00	0.000	0.80		0.66
		2	0.0	0.000	894.39	0.279	1.00	0.163	
		3	0.0	0.000	1024.82	0.319	1.10	0.205	
		4	0.0	0.000	1158.40	0.361	1.20	0.253	
		5	0.0	0.000	1242.21	0.387	1.27	0.287	
		6	0.0	0.000	1242.21	0.387	1.40	0.316	
		7	0.0	0.000	1242.21	0.387	1.50	0.339	
		8	1242.2	0.387	1242.21	0.387	1.75	0.395	
		9	0.0	0.000	0.00	0.000	2.00		
		10	0.0	0.000	0.00	0.000	2.25		
		11	0.0	0.000	0.00	0.000	2.60		
		12	0.0	0.000	0.00	0.000	3.00		
		13	0.0	0.000	0.00	0.000	3.20		
		ΣQ	1242.2	MAX_E0		0.387	1.75	0.395	



(3) Basic seismic capacity index E_0 is calculated by ductility dominant equation
The Basic seismic capacity index E_0 is calculated by ductility dominant equation with bellow.
Ductility-dominant basic seismic index of structure (Eq.(4))

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

Where:

E_1 : $C_1 \cdot F_1$
E_2 : $C_2 \cdot F_2$
E_3 : $C_3 \cdot F_3$
C_1 : The strength index C of the first group (with small F index).
C_2 : The strength index C of the second group (with medium F index).
C_3 : The strength index C of the third group (with large F index).
F_1 : The ductility index F of the first group.
F_2 : The ductility index F of the second group.
F_3 : The ductility index F of the third group.

The maximum E_0 is calculated by ductility dominant equation is found following method.

F value	GN
0.80	1
1.00 ≦F＜1.10	2
1.10 ≦F＜1.20	3
1.20 ≦F＜1.27	4
1.27 ≦F＜1.40	5
1.40 ≦F＜1.50	6
1.50 ≦F＜1.75	7
1.75 ≦F＜2.00	8
2.00 ≦F＜2.25	9
2.25 ≦F＜2.60	10
2.60 ≦F＜3.0	11
3.00 ≦F＜3.2	12
3.20	13

GN : Group number

C : Strength index

F : F index

E0 :Basic seismic index of structure (Noconsist extremely short columes)

* Show muxim (Muximum 0.138)

Ctu	0.07

C1	F1	C2	F2	C3	F3
0.06	1.00	0.07	1.75		

X Direction 1 Story N+1/N+i= 1

	GN -1	GN -2	GN -3	C1	F1	C2	F2	C3	F3	E0	Ctu
	2 - 2	3 - 13		0.064	1.00	0.058	1.10			0.090	0.058
	2 - 3	4 - 13		0.064	1.00	0.065	1.20			0.101	0.065
	2 - 4	5 - 13		0.064	1.00	0.070	1.27			0.110	0.070
	2 - 5	6 - 13		0.064	1.00	0.070	1.40			0.117	0.070
	2 - 6	7 - 13		0.064	1.00	0.070	1.50			0.123	0.070
*	2 - 7	8 - 13		0.064	1.00	0.070	1.75			0.138	0.070
	2 - 8	9 - 13		0.115	1.00	0.000	2.00			0.115	0.115
	2 - 9	10 - 13		0.115	1.00	0.000	2.25			0.115	0.115
	2 - 10	11 - 13		0.115	1.00	0.000	2.60			0.115	0.115
	2 - 11	12 - 13		0.115	1.00	0.000	3.00			0.115	0.115
	2 - 12	13 - 13		0.115	1.00	0.000	3.20			0.115	0.115
	3 - 3	4 - 13		0.000	1.10	0.065	1.20			0.078	0.065
	3 - 4	5 - 13		0.000	1.10	0.070	1.27			0.089	0.070
	3 - 5	6 - 13		0.000	1.10	0.070	1.40			0.098	0.070
	3 - 6	7 - 13		0.000	1.10	0.070	1.50			0.105	0.070
	3 - 7	8 - 13		0.000	1.10	0.070	1.75			0.123	0.070
	3 - 8	9 - 13		0.058	1.10	0.000	2.00			0.064	0.058
	3 - 9	10 - 13		0.058	1.10	0.000	2.25			0.064	0.058
	3 - 10	11 - 13		0.058	1.10	0.000	2.60			0.064	0.058
	3 - 11	12 - 13		0.058	1.10	0.000	3.00			0.064	0.058
	3 - 12	13 - 13		0.058	1.10	0.000	3.20			0.064	0.058
	4 - 4	5 - 13		0.000	1.20	0.070	1.27			0.089	0.070
	4 - 5	6 - 13		0.000	1.20	0.070	1.40			0.098	0.070
	4 - 6	7 - 13		0.000	1.20	0.070	1.50			0.105	0.070
	4 - 7	8 - 13		0.000	1.20	0.070	1.75			0.123	0.070
	4 - 8	9 - 13		0.065	1.20	0.000	2.00			0.078	0.065
	4 - 9	10 - 13		0.065	1.20	0.000	2.25			0.078	0.065
	4 - 10	11 - 13		0.065	1.20	0.000	2.60			0.078	0.065
	4 - 11	12 - 13		0.065	1.20	0.000	3.00			0.078	0.065
	4 - 12	13 - 13		0.065	1.20	0.000	3.20			0.078	0.065
	5 - 5	6 - 13		0.000	1.27	0.070	1.40			0.098	0.070
	5 - 6	7 - 13		0.000	1.27	0.070	1.50			0.105	0.070
	5 - 7	8 - 13		0.000	1.27	0.070	1.75			0.123	0.070
	5 - 8	9 - 13		0.070	1.27	0.000	2.00			0.089	0.070
	5 - 9	10 - 13		0.070	1.27	0.000	2.25			0.089	0.070
	5 - 10	11 - 13		0.070	1.27	0.000	2.60			0.089	0.070
	5 - 11	12 - 13		0.070	1.27	0.000	3.00			0.089	0.070
	5 - 12	13 - 13		0.070	1.27	0.000	3.20			0.089	0.070
	6 - 6	7 - 13		0.000	1.40	0.070	1.50			0.105	0.070
	6 - 7	8 - 13		0.000	1.40	0.070	1.75			0.123	0.070
	6 - 8	9 - 13		0.070	1.40	0.000	2.00			0.098	0.070
	6 - 9	10 - 13		0.070	1.40	0.000	2.25			0.098	0.070
	6 - 10	11 - 13		0.070	1.40	0.000	2.60			0.098	0.070
	6 - 11	12 - 13		0.070	1.40	0.000	3.00			0.098	0.070
	6 - 12	13 - 13		0.070	1.40	0.000	3.20			0.098	0.070
	7 - 7	8 - 13		0.000	1.50	0.070	1.75			0.123	0.070
	7 - 8	9 - 13		0.070	1.50	0.000	2.00			0.105	0.070
	7 - 9	10 - 13		0.070	1.50	0.000	2.25			0.105	0.070
	7 - 10	11 - 13		0.070	1.50	0.000	2.60			0.105	0.070
	7 - 11	12 - 13		0.070	1.50	0.000	3.00			0.105	0.070
	7 - 12	13 - 13		0.070	1.50	0.000	3.20			0.105	0.070
	8 - 8	9 - 13		0.070	1.75	0.000	2.00			0.123	0.070
	8 - 9	10 - 13		0.070	1.75	0.000	2.25			0.123	0.070

	GN -1	GN -2	GN -3	C1	F1	C2	F2	C3	F3	E0	Ctu
	8 - 10	11 - 13		0.070	1.75	0.000	2.60			0.123	0.070
	8 - 11	12 - 13		0.070	1.75	0.000	3.00			0.123	0.070
	8 - 12	13 - 13		0.070	1.75	0.000	3.20			0.123	0.070
	9 - 9	10 - 13		0.000	2.00	0.000	2.25			0.000	0.000
	9 - 10	11 - 13		0.000	2.00	0.000	2.60			0.000	0.000
	9 - 11	12 - 13		0.000	2.00	0.000	3.00			0.000	0.000
	9 - 12	13 - 13		0.000	2.00	0.000	3.20			0.000	0.000
	10 - 10	11 - 13		0.000	2.25	0.000	2.60			0.000	0.000
	10 - 11	12 - 13		0.000	2.25	0.000	3.00			0.000	0.000
	10 - 12	13 - 13		0.000	2.25	0.000	3.20			0.000	0.000
	11 - 11	13 - 13		0.000	2.60	0.000	3.20			0.000	0.000
	11 - 12	13 - 13		0.000	2.60	0.000	3.20			0.000	0.000
	12 - 12	13 - 13		0.000	3.00	0.000	3.20			0.000	0.000
	2 - 2	3 - 3	4 - 13	0.064	1.00	0.000	1.10	0.065	1.20	0.101	0.065
	2 - 2	3 - 4	5 - 13	0.064	1.00	0.000	1.10	0.070	1.27	0.110	0.070
	2 - 2	3 - 5	6 - 13	0.064	1.00	0.000	1.10	0.070	1.40	0.117	0.070
	2 - 2	3 - 6	7 - 13	0.064	1.00	0.000	1.10	0.070	1.50	0.123	0.070
*	2 - 2	3 - 7	8 - 13	0.064	1.00	0.000	1.10	0.070	1.75	0.138	0.070
	2 - 2	3 - 8	9 - 13	0.064	1.00	0.058	1.10	0.000	2.00	0.090	0.058
	2 - 2	3 - 9	10 - 13	0.064	1.00	0.058	1.10	0.000	2.25	0.090	0.058
	2 - 2	3 - 10	11 - 13	0.064	1.00	0.058	1.10	0.000	2.60	0.090	0.058
	2 - 2	3 - 11	12 - 13	0.064	1.00	0.058	1.10	0.000	3.00	0.090	0.058
	2 - 2	3 - 12	13 - 13	0.064	1.00	0.058	1.10	0.000	3.20	0.090	0.058
	2 - 3	4 - 4	5 - 13	0.064	1.00	0.000	1.20	0.070	1.27	0.110	0.070
	2 - 3	4 - 5	6 - 13	0.064	1.00	0.000	1.20	0.070	1.40	0.117	0.070
	2 - 3	4 - 6	7 - 13	0.064	1.00	0.000	1.20	0.070	1.50	0.123	0.070
*	2 - 3	4 - 7	8 - 13	0.064	1.00	0.000	1.20	0.070	1.75	0.138	0.070
	2 - 3	4 - 8	9 - 13	0.064	1.00	0.065	1.20	0.000	2.00	0.101	0.065
	2 - 3	4 - 9	10 - 13	0.064	1.00	0.065	1.20	0.000	2.25	0.101	0.065
	2 - 3	4 - 10	11 - 13	0.064	1.00	0.065	1.20	0.000	2.60	0.101	0.065
	2 - 3	4 - 11	12 - 13	0.064	1.00	0.065	1.20	0.000	3.00	0.101	0.065
	2 - 3	4 - 12	13 - 13	0.064	1.00	0.065	1.20	0.000	3.20	0.101	0.065
	2 - 4	5 - 5	6 - 13	0.064	1.00	0.000	1.27	0.070	1.40	0.117	0.070
	2 - 4	5 - 6	7 - 13	0.064	1.00	0.000	1.27	0.070	1.50	0.123	0.070
*	2 - 4	5 - 7	8 - 13	0.064	1.00	0.000	1.27	0.070	1.75	0.138	0.070
	2 - 4	5 - 8	9 - 13	0.064	1.00	0.070	1.27	0.000	2.00	0.110	0.070
	2 - 4	5 - 9	10 - 13	0.064	1.00	0.070	1.27	0.000	2.25	0.110	0.070
	2 - 4	5 - 10	11 - 13	0.064	1.00	0.070	1.27	0.000	2.60	0.110	0.070
	2 - 4	5 - 11	12 - 13	0.064	1.00	0.070	1.27	0.000	3.00	0.110	0.070
	2 - 4	5 - 12	13 - 13	0.064	1.00	0.070	1.27	0.000	3.20	0.110	0.070
	2 - 5	6 - 6	7 - 13	0.064	1.00	0.000	1.40	0.070	1.50	0.123	0.070
*	2 - 5	6 - 7	8 - 13	0.064	1.00	0.000	1.40	0.070	1.75	0.138	0.070
	2 - 5	6 - 8	9 - 13	0.064	1.00	0.070	1.40	0.000	2.00	0.117	0.070
	2 - 5	6 - 9	10 - 13	0.064	1.00	0.070	1.40	0.000	2.25	0.117	0.070
	2 - 5	6 - 10	11 - 13	0.064	1.00	0.070	1.40	0.000	2.60	0.117	0.070
	2 - 5	6 - 11	12 - 13	0.064	1.00	0.070	1.40	0.000	3.00	0.117	0.070
	2 - 5	6 - 12	13 - 13	0.064	1.00	0.070	1.40	0.000	3.20	0.117	0.070
*	2 - 6	7 - 7	8 - 13	0.064	1.00	0.000	1.50	0.070	1.75	0.138	0.070
	2 - 6	7 - 8	9 - 13	0.064	1.00	0.070	1.50	0.000	2.00	0.123	0.070
	2 - 6	7 - 9	10 - 13	0.064	1.00	0.070	1.50	0.000	2.25	0.123	0.070
	2 - 6	7 - 10	11 - 13	0.064	1.00	0.070	1.50	0.000	2.60	0.123	0.070
	2 - 6	7 - 11	12 - 13	0.064	1.00	0.070	1.50	0.000	3.00	0.123	0.070
	2 - 6	7 - 12	13 - 13	0.064	1.00	0.070	1.50	0.000	3.20	0.123	0.070
*	2 - 7	8 - 8	9 - 13	0.064	1.00	0.070	1.75	0.000	2.00	0.138	0.070
*	2 - 7	8 - 9	10 - 13	0.064	1.00	0.070	1.75	0.000	2.25	0.138	0.070
*	2 - 7	8 - 10	11 - 13	0.064	1.00	0.070	1.75	0.000	2.60	0.138	0.070
*	2 - 7	8 - 11	12 - 13	0.064	1.00	0.070	1.75	0.000	3.00	0.138	0.070
	2 - 8	9 - 9	10 - 13	0.115	1.00	0.000	2.00	0.000	2.25	0.115	0.115
	2 - 8	9 - 10	11 - 13	0.115	1.00	0.000	2.00	0.000	2.60	0.115	0.115
	2 - 8	9 - 11	12 - 13	0.115	1.00	0.000	2.00	0.000	3.00	0.115	0.115
	2 - 8	9 - 12	13 - 13	0.115	1.00	0.000	2.00	0.000	3.20	0.115	0.115
	2 - 9	10 - 10	11 - 13	0.115	1.00	0.000	2.25	0.000	2.60	0.115	0.115
	2 - 9	10 - 11	12 - 13	0.115	1.00	0.000	2.25	0.000	3.00	0.115	0.115

	GN -1	GN -2	GN -3	C1	F1	C2	F2	C3	F3	E0	Ctu
	2 - 9	10 - 12	13 - 13	0.115	1.00	0.000	2.25	0.000	3.20	0.115	0.115
	2 - 10	11 - 11	12 - 13	0.115	1.00	0.000	2.60	0.000	3.00	0.115	0.115
	2 - 10	11 - 12	13 - 13	0.115	1.00	0.000	2.60	0.000	3.20	0.115	0.115
	2 - 11	12 - 12	13 - 13	0.115	1.00	0.000	3.00	0.000	3.20	0.115	0.115
	3 - 3	4 - 4	5 - 13	0.000	1.10	0.000	1.20	0.070	1.27	0.089	0.070
	3 - 3	4 - 5	6 - 13	0.000	1.10	0.000	1.20	0.070	1.40	0.098	0.070
	3 - 3	4 - 6	7 - 13	0.000	1.10	0.000	1.20	0.070	1.50	0.105	0.070
	3 - 3	4 - 7	8 - 13	0.000	1.10	0.000	1.20	0.070	1.75	0.123	0.070
	3 - 3	4 - 8	9 - 13	0.000	1.10	0.065	1.20	0.000	2.00	0.078	0.065
	3 - 3	4 - 9	10 - 13	0.000	1.10	0.065	1.20	0.000	2.25	0.078	0.065
	3 - 3	4 - 10	11 - 13	0.000	1.10	0.065	1.20	0.000	2.60	0.078	0.065
	3 - 3	4 - 11	12 - 13	0.000	1.10	0.065	1.20	0.000	3.00	0.078	0.065
	3 - 3	4 - 12	13 - 13	0.000	1.10	0.065	1.20	0.000	3.20	0.078	0.065
	3 - 4	5 - 5	6 - 13	0.000	1.10	0.000	1.27	0.070	1.40	0.098	0.070
	3 - 4	5 - 6	7 - 13	0.000	1.10	0.000	1.27	0.070	1.50	0.105	0.070
	3 - 4	5 - 7	8 - 13	0.000	1.10	0.000	1.27	0.070	1.75	0.123	0.070
	3 - 4	5 - 8	9 - 13	0.000	1.10	0.070	1.27	0.000	2.00	0.089	0.070
	3 - 4	5 - 9	10 - 13	0.000	1.10	0.070	1.27	0.000	2.25	0.089	0.070
	3 - 4	5 - 10	11 - 13	0.000	1.10	0.070	1.27	0.000	2.60	0.089	0.070
	3 - 4	5 - 11	12 - 13	0.000	1.10	0.070	1.27	0.000	3.00	0.089	0.070
	3 - 4	5 - 12	13 - 13	0.000	1.10	0.070	1.27	0.000	3.20	0.089	0.070
	3 - 5	6 - 6	7 - 13	0.000	1.10	0.000	1.40	0.070	1.50	0.105	0.070
	3 - 5	6 - 7	8 - 13	0.000	1.10	0.000	1.40	0.070	1.75	0.123	0.070
	3 - 5	6 - 8	9 - 13	0.000	1.10	0.070	1.40	0.000	2.00	0.098	0.070
	3 - 5	6 - 9	10 - 13	0.000	1.10	0.070	1.40	0.000	2.25	0.098	0.070
	3 - 5	6 - 10	11 - 13	0.000	1.10	0.070	1.40	0.000	2.60	0.098	0.070
	3 - 5	6 - 11	12 - 13	0.000	1.10	0.070	1.40	0.000	3.00	0.098	0.070
	3 - 5	6 - 12	13 - 13	0.000	1.10	0.070	1.40	0.000	3.20	0.098	0.070
	3 - 6	7 - 7	8 - 13	0.000	1.10	0.000	1.50	0.070	1.75	0.123	0.070
	3 - 6	7 - 8	9 - 13	0.000	1.10	0.070	1.50	0.000	2.00	0.105	0.070
	3 - 6	7 - 9	10 - 13	0.000	1.10	0.070	1.50	0.000	2.25	0.105	0.070
	3 - 6	7 - 10	11 - 13	0.000	1.10	0.070	1.50	0.000	2.60	0.105	0.070
	3 - 6	7 - 11	12 - 13	0.000	1.10	0.070	1.50	0.000	3.00	0.105	0.070
	3 - 6	7 - 12	13 - 13	0.000	1.10	0.070	1.50	0.000	3.20	0.105	0.070
	3 - 7	8 - 8	9 - 13	0.000	1.10	0.070	1.75	0.000	2.00	0.123	0.070
	3 - 7	8 - 9	10 - 13	0.000	1.10	0.070	1.75	0.000	2.25	0.123	0.070
	3 - 7	8 - 10	11 - 13	0.000	1.10	0.070	1.75	0.000	2.60	0.123	0.070
	3 - 7	8 - 11	12 - 13	0.000	1.10	0.070	1.75	0.000	3.00	0.123	0.070
	3 - 7	8 - 12	13 - 13	0.000	1.10	0.070	1.75	0.000	3.20	0.123	0.070
	3 - 8	9 - 9	10 - 13	0.058	1.10	0.000	2.00	0.000	2.25	0.064	0.058
	3 - 8	9 - 10	11 - 13	0.058	1.10	0.000	2.00	0.000	2.60	0.064	0.058
	3 - 8	9 - 11	12 - 13	0.058	1.10	0.000	2.00	0.000	3.00	0.064	0.058
	3 - 8	9 - 12	13 - 13	0.058	1.10	0.000	2.00	0.000	3.20	0.064	0.058
	3 - 9	10 - 10	11 - 13	0.058	1.10	0.000	2.25	0.000	2.60	0.064	0.058
	3 - 9	10 - 11	12 - 13	0.058	1.10	0.000	2.25	0.000	3.00	0.064	0.058
	3 - 9	10 - 12	13 - 13	0.058	1.10	0.000	2.25	0.000	3.20	0.064	0.058
	3 - 10	11 - 11	12 - 13	0.058	1.10	0.000	2.60	0.000	3.00	0.064	0.058
	3 - 10	11 - 12	13 - 13	0.058	1.10	0.000	2.60	0.000	3.20	0.064	0.058
	3 - 11	12 - 12	13 - 13	0.058	1.10	0.000	3.00	0.000	3.20	0.064	0.058
	4 - 4	5 - 5	6 - 13	0.000	1.20	0.000	1.27	0.070	1.40	0.098	0.070
	4 - 4	5 - 6	7 - 13	0.000	1.20	0.000	1.27	0.070	1.50	0.105	0.070
	4 - 4	5 - 7	8 - 13	0.000	1.20	0.000	1.27	0.070	1.75	0.123	0.070
	4 - 4	5 - 8	9 - 13	0.000	1.20	0.070	1.27	0.000	2.00	0.089	0.070
	4 - 4	5 - 9	10 - 13	0.000	1.20	0.070	1.27	0.000	2.25	0.089	0.070
	4 - 4	5 - 10	11 - 13	0.000	1.20	0.070	1.27	0.000	2.60	0.089	0.070
	4 - 4	5 - 11	12 - 13	0.000	1.20	0.070	1.27	0.000	3.00	0.089	0.070
	4 - 4	5 - 12	13 - 13	0.000	1.20	0.070	1.27	0.000	3.20	0.089	0.070
	4 - 5	6 - 6	7 - 13	0.000	1.20	0.000	1.40	0.070	1.50	0.105	0.070
	4 - 5	6 - 7	8 - 13	0.000	1.20	0.000	1.40	0.070	1.75	0.123	0.070
	4 - 5	6 - 8	9 - 13	0.000	1.20	0.070	1.40	0.000	2.00	0.098	0.070
	4 - 5	6 - 9	10 - 13	0.000	1.20	0.070	1.40	0.000	2.25	0.098	0.070
	4 - 5	6 - 10	11 - 13	0.000	1.20	0.070	1.40	0.000	2.60	0.098	0.070
	4 - 5	6 - 11	12 - 13	0.000	1.20	0.070	1.40	0.000	3.00	0.098	0.070

	GN -1	GN -2	GN -3	C1	F1	C2	F2	C3	F3	E0	Ctu
	4 - 5	6 - 12	13 - 13	0.000	1.20	0.070	1.40	0.000	3.20	0.098	0.070
	4 - 6	7 - 7	8 - 13	0.000	1.20	0.000	1.50	0.070	1.75	0.123	0.070
	4 - 6	7 - 8	9 - 13	0.000	1.20	0.070	1.50	0.000	2.00	0.105	0.070
	4 - 6	7 - 9	10 - 13	0.000	1.20	0.070	1.50	0.000	2.25	0.105	0.070
	4 - 6	7 - 10	11 - 13	0.000	1.20	0.070	1.50	0.000	2.60	0.105	0.070
	4 - 6	7 - 11	12 - 13	0.000	1.20	0.070	1.50	0.000	3.00	0.105	0.070
	4 - 6	7 - 12	13 - 13	0.000	1.20	0.070	1.50	0.000	3.20	0.105	0.070
	4 - 7	8 - 8	9 - 13	0.000	1.20	0.070	1.75	0.000	2.00	0.123	0.070
	4 - 7	8 - 9	10 - 13	0.000	1.20	0.070	1.75	0.000	2.25	0.123	0.070
	4 - 7	8 - 10	11 - 13	0.000	1.20	0.070	1.75	0.000	2.60	0.123	0.070
	4 - 7	8 - 11	12 - 13	0.000	1.20	0.070	1.75	0.000	3.00	0.123	0.070
	4 - 7	8 - 12	13 - 13	0.000	1.20	0.070	1.75	0.000	3.20	0.123	0.070
	4 - 8	9 - 9	10 - 13	0.065	1.20	0.000	2.00	0.000	2.25	0.078	0.065
	4 - 8	9 - 10	11 - 13	0.065	1.20	0.000	2.00	0.000	2.60	0.078	0.065
	4 - 8	9 - 11	12 - 13	0.065	1.20	0.000	2.00	0.000	3.00	0.078	0.065
	4 - 8	9 - 12	13 - 13	0.065	1.20	0.000	2.00	0.000	3.20	0.078	0.065
	4 - 9	10 - 10	11 - 13	0.065	1.20	0.000	2.25	0.000	2.60	0.078	0.065
	4 - 9	10 - 11	12 - 13	0.065	1.20	0.000	2.25	0.000	3.00	0.078	0.065
	4 - 9	10 - 12	13 - 13	0.065	1.20	0.000	2.25	0.000	3.20	0.078	0.065
	4 - 10	11 - 11	12 - 13	0.065	1.20	0.000	2.60	0.000	3.00	0.078	0.065
	4 - 10	11 - 12	13 - 13	0.065	1.20	0.000	2.60	0.000	3.20	0.078	0.065
	4 - 11	12 - 12	13 - 13	0.065	1.20	0.000	3.00	0.000	3.20	0.078	0.065
	5 - 5	6 - 6	7 - 13	0.000	1.27	0.000	1.40	0.070	1.50	0.105	0.070
	5 - 5	6 - 7	8 - 13	0.000	1.27	0.000	1.40	0.070	1.75	0.123	0.070
	5 - 5	6 - 8	9 - 13	0.000	1.27	0.070	1.40	0.000	2.00	0.098	0.070
	5 - 5	6 - 9	10 - 13	0.000	1.27	0.070	1.40	0.000	2.25	0.098	0.070
	5 - 5	6 - 10	11 - 13	0.000	1.27	0.070	1.40	0.000	2.60	0.098	0.070
	5 - 5	6 - 11	12 - 13	0.000	1.27	0.070	1.40	0.000	3.00	0.098	0.070
	5 - 5	6 - 12	13 - 13	0.000	1.27	0.070	1.40	0.000	3.20	0.098	0.070
	5 - 6	7 - 7	8 - 13	0.000	1.27	0.000	1.50	0.070	1.75	0.123	0.070
	5 - 6	7 - 8	9 - 13	0.000	1.27	0.070	1.50	0.000	2.00	0.105	0.070
	5 - 6	7 - 9	10 - 13	0.000	1.27	0.070	1.50	0.000	2.25	0.105	0.070
	5 - 6	7 - 10	11 - 13	0.000	1.27	0.070	1.50	0.000	2.60	0.105	0.070
	5 - 6	7 - 11	12 - 13	0.000	1.27	0.070	1.50	0.000	3.00	0.105	0.070
	5 - 6	7 - 12	13 - 13	0.000	1.27	0.070	1.50	0.000	3.20	0.105	0.070
	5 - 7	8 - 8	9 - 13	0.000	1.27	0.070	1.75	0.000	2.00	0.123	0.070
	5 - 7	8 - 9	10 - 13	0.000	1.27	0.070	1.75	0.000	2.25	0.123	0.070
	5 - 7	8 - 10	11 - 13	0.000	1.27	0.070	1.75	0.000	2.60	0.123	0.070
	5 - 7	8 - 11	12 - 13	0.000	1.27	0.070	1.75	0.000	3.00	0.123	0.070
	5 - 7	8 - 12	13 - 13	0.000	1.27	0.070	1.75	0.000	3.20	0.123	0.070
	5 - 8	9 - 9	10 - 13	0.070	1.27	0.000	2.00	0.000	2.25	0.089	0.070
	5 - 8	9 - 10	11 - 13	0.070	1.27	0.000	2.00	0.000	2.60	0.089	0.070
	5 - 8	9 - 11	12 - 13	0.070	1.27	0.000	2.00	0.000	3.00	0.089	0.070
	5 - 8	9 - 12	13 - 13	0.070	1.27	0.000	2.00	0.000	3.20	0.089	0.070
	5 - 9	10 - 10	11 - 13	0.070	1.27	0.000	2.25	0.000	2.60	0.089	0.070
	5 - 9	10 - 11	12 - 13	0.070	1.27	0.000	2.25	0.000	3.00	0.089	0.070
	5 - 9	10 - 12	13 - 13	0.070	1.27	0.000	2.25	0.000	3.20	0.089	0.070
	5 - 10	11 - 11	12 - 13	0.070	1.27	0.000	2.60	0.000	3.00	0.089	0.070
	5 - 10	11 - 12	13 - 13	0.070	1.27	0.000	2.60	0.000	3.20	0.089	0.070
	5 - 11	12 - 12	13 - 13	0.070	1.27	0.000	3.00	0.000	3.20	0.089	0.070
	6 - 6	7 - 7	8 - 13	0.000	1.40	0.000	1.50	0.070	1.75	0.123	0.070
	6 - 6	7 - 8	9 - 13	0.000	1.40	0.070	1.50	0.000	2.00	0.105	0.070
	6 - 6	7 - 9	10 - 13	0.000	1.40	0.070	1.50	0.000	2.25	0.105	0.070
	6 - 6	7 - 10	11 - 13	0.000	1.40	0.070	1.50	0.000	2.60	0.105	0.070
	6 - 6	7 - 11	12 - 13	0.000	1.40	0.070	1.50	0.000	3.00	0.105	0.070
	6 - 6	7 - 12	13 - 13	0.000	1.40	0.070	1.50	0.000	3.20	0.105	0.070
	6 - 7	8 - 8	9 - 13	0.000	1.40	0.070	1.75	0.000	2.00	0.123	0.070
	6 - 7	8 - 9	10 - 13	0.000	1.40	0.070	1.75	0.000	2.25	0.123	0.070
	6 - 7	8 - 10	11 - 13	0.000	1.40	0.070	1.75	0.000	2.60	0.123	0.070
	6 - 7	8 - 11	12 - 13	0.000	1.40	0.070	1.75	0.000	3.00	0.123	0.070
	6 - 7	8 - 12	13 - 13	0.000	1.40	0.070	1.75	0.000	3.20	0.123	0.070
	6 - 8	9 - 9	10 - 13	0.070	1.40	0.000	2.00	0.000	2.25	0.098	0.070
	6 - 8	9 - 10	11 - 13	0.070	1.40	0.000	2.00	0.000	2.60	0.098	0.070

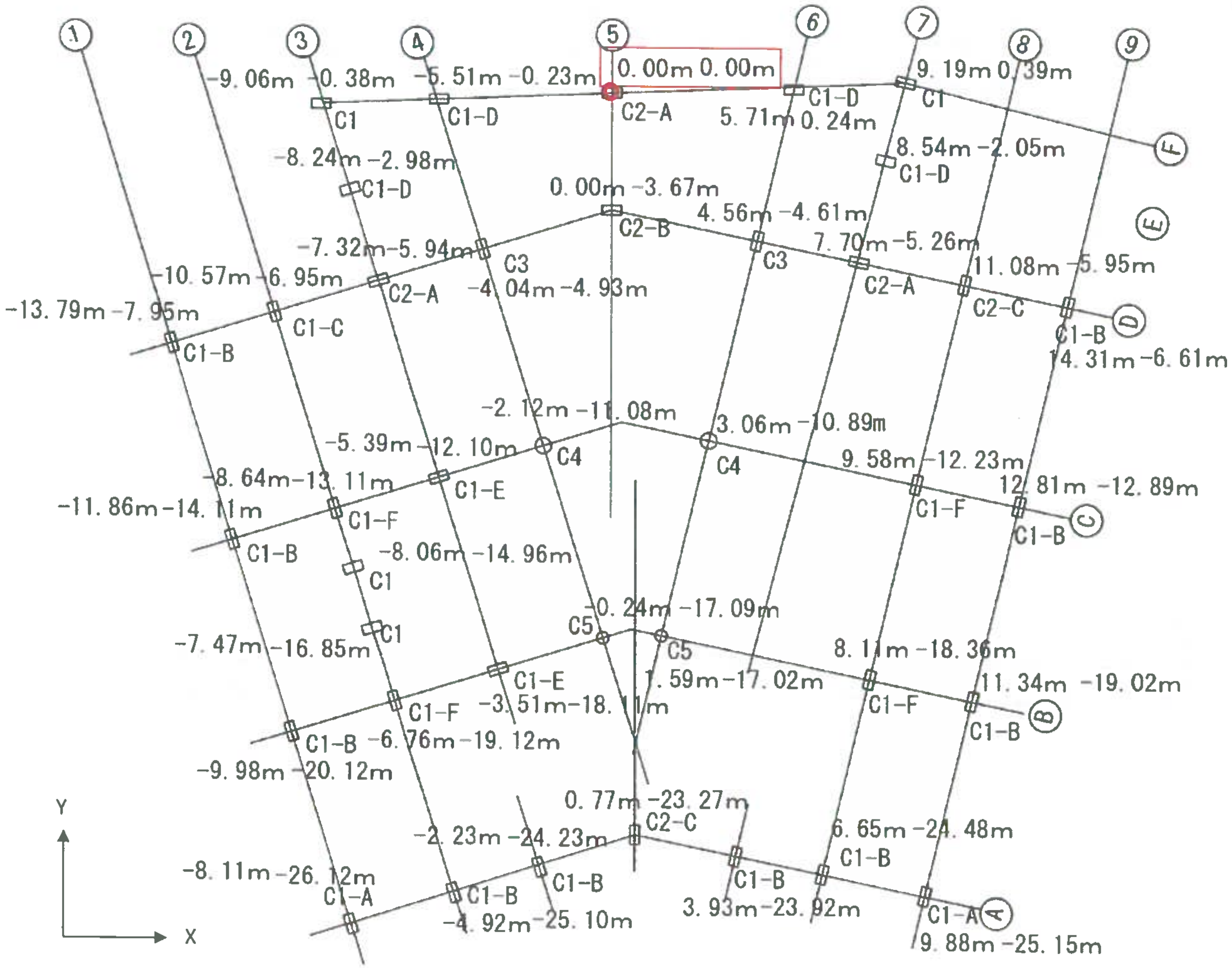
	GN -1	GN -2	GN -3	C1	F1	C2	F2	C3	F3	E0	Ctu
	6 - 8	9 - 11	12 - 13	0.070	1.40	0.000	2.00	0.000	3.00	0.098	0.070
	6 - 8	9 - 12	13 - 13	0.070	1.40	0.000	2.00	0.000	3.20	0.098	0.070
	6 - 9	10 - 10	11 - 13	0.070	1.40	0.000	2.25	0.000	2.60	0.098	0.070
	5 - 9	10 - 11	12 - 13	0.070	1.27	0.000	2.25	0.000	3.00	0.089	0.070
	6 - 9	10 - 12	13 - 13	0.070	1.40	0.000	2.25	0.000	3.20	0.098	0.070
	6 - 10	11 - 11	12 - 13	0.070	1.40	0.000	2.60	0.000	3.00	0.098	0.070
	6 - 10	11 - 12	13 - 13	0.070	1.40	0.000	2.60	0.000	3.20	0.098	0.070
	6 - 11	12 - 12	13 - 13	0.070	1.40	0.000	3.00	0.000	3.20	0.098	0.070
	7 - 7	8 - 8	9 - 13	0.000	1.50	0.070	1.75	0.000	2.00	0.123	0.070
	7 - 7	8 - 9	10 - 13	0.000	1.50	0.070	1.75	0.000	2.25	0.123	0.070
	7 - 7	8 - 10	11 - 13	0.000	1.50	0.070	1.75	0.000	2.60	0.123	0.070
	7 - 7	8 - 11	12 - 13	0.000	1.50	0.070	1.75	0.000	3.00	0.123	0.070
	7 - 7	8 - 12	13 - 13	0.000	1.50	0.070	1.75	0.000	3.20	0.123	0.070
	7 - 8	9 - 9	10 - 13	0.070	1.50	0.000	2.00	0.000	2.25	0.105	0.070
	7 - 8	9 - 10	11 - 13	0.070	1.50	0.000	2.00	0.000	2.60	0.105	0.070
	7 - 8	9 - 11	12 - 13	0.070	1.50	0.000	2.00	0.000	3.00	0.105	0.070
	7 - 8	9 - 12	13 - 13	0.070	1.50	0.000	2.00	0.000	3.20	0.105	0.070
	7 - 9	10 - 10	11 - 13	0.070	1.50	0.000	2.25	0.000	2.60	0.105	0.070
	7 - 9	10 - 11	12 - 13	0.070	1.50	0.000	2.25	0.000	3.00	0.105	0.070
	7 - 9	10 - 12	13 - 13	0.070	1.50	0.000	2.25	0.000	3.20	0.105	0.070
	7 - 10	11 - 11	12 - 13	0.070	1.50	0.000	2.60	0.000	3.00	0.105	0.070
	7 - 10	11 - 12	13 - 13	0.070	1.50	0.000	2.60	0.000	3.20	0.105	0.070
	7 - 11	12 - 12	13 - 13	0.070	1.50	0.000	3.00	0.000	3.20	0.105	0.070
	8 - 8	9 - 9	10 - 13	0.070	1.75	0.000	2.00	0.000	2.25	0.123	0.070
	8 - 8	9 - 10	11 - 13	0.070	1.75	0.000	2.00	0.000	2.60	0.123	0.070
	8 - 8	9 - 11	12 - 13	0.070	1.75	0.000	2.00	0.000	3.00	0.123	0.070
	8 - 8	9 - 12	13 - 13	0.070	1.75	0.000	2.00	0.000	3.20	0.123	0.070
	8 - 9	10 - 10	11 - 13	0.070	1.75	0.000	2.25	0.000	2.60	0.123	0.070
	8 - 9	10 - 11	12 - 13	0.070	1.75	0.000	2.25	0.000	3.00	0.123	0.070
	8 - 9	10 - 12	13 - 13	0.070	1.75	0.000	2.25	0.000	3.20	0.123	0.070
	8 - 10	11 - 11	12 - 13	0.070	1.75	0.000	2.60	0.000	3.00	0.123	0.070
	8 - 10	11 - 12	13 - 13	0.070	1.75	0.000	2.60	0.000	3.20	0.123	0.070
	8 - 11	12 - 12	13 - 13	0.070	1.75	0.000	3.00	0.000	3.20	0.123	0.070
	9 - 9	10 - 10	11 - 13	0.000	2.00	0.000	2.25	0.000	2.60	0.000	0.000
	9 - 9	10 - 11	12 - 13	0.000	2.00	0.000	2.25	0.000	3.00	0.000	0.000
	9 - 9	10 - 12	13 - 13	0.000	2.00	0.000	2.25	0.000	3.20	0.000	0.000
	9 - 10	11 - 11	12 - 13	0.000	2.00	0.000	2.60	0.000	3.00	0.000	0.000
	9 - 10	11 - 12	13 - 13	0.000	2.00	0.000	2.60	0.000	3.20	0.000	0.000
	9 - 11	12 - 12	13 - 13	0.000	2.00	0.000	3.00	0.000	3.20	0.000	0.000
	10 - 10	11 - 11	12 - 13	0.000	2.25	0.000	2.60	0.000	3.00	0.000	0.000
	10 - 10	11 - 12	13 - 13	0.000	2.25	0.000	2.60	0.000	3.20	0.000	0.000
	10 - 11	12 - 12	13 - 13	0.000	2.25	0.000	3.00	0.000	3.20	0.000	0.000

The Basic seismic capacity index E_0 is calculated by ductility dominant equation of each floor can be calculated in the same procedure.

3.4 The Irregularity Index S_D

(1) The center of gravity

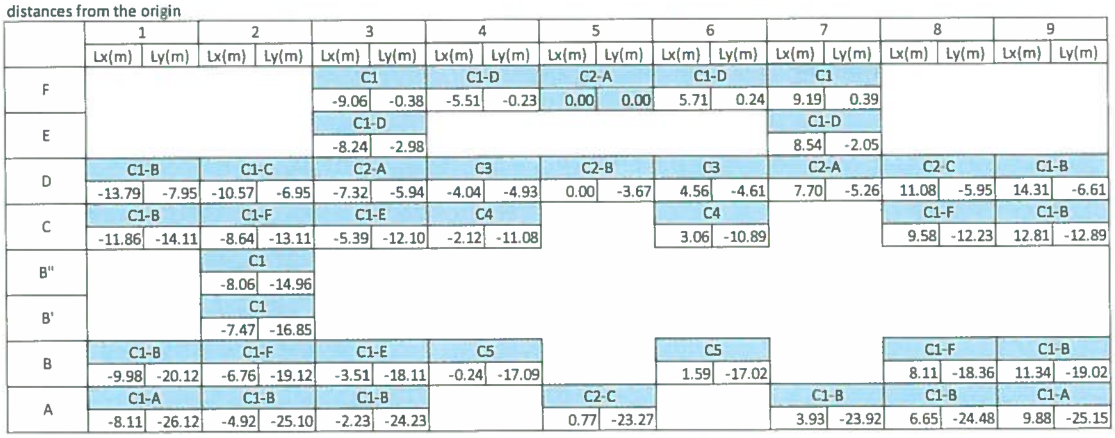
F-5 is set as an origin of coordinates. Below shows the distances from the origin and the axial forces of the columns.



N COLUMN LAY OUT

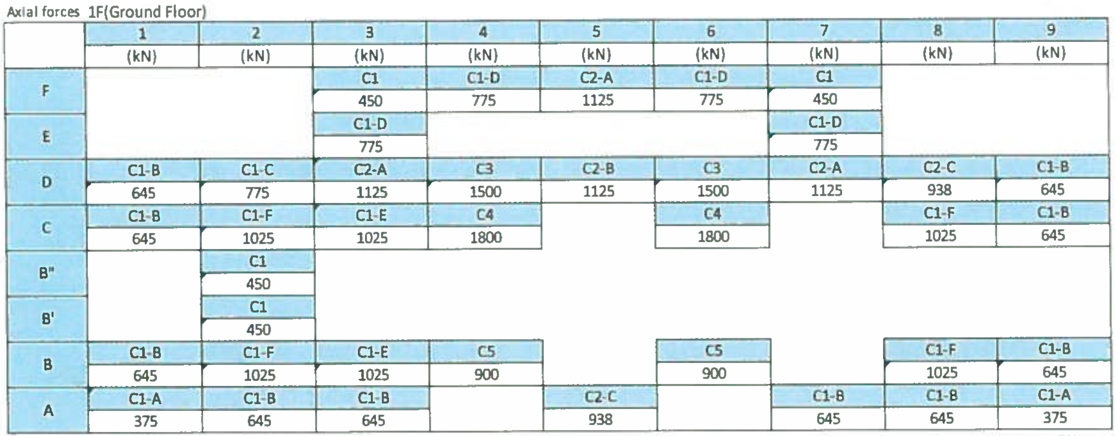
Supplement 9

distances from the origin



	1		2		3		4		5		6		7		8		9	
	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)
F					C1		C1-D		C2-A		C1-D		C1					
					-9.06	-0.38	-5.51	-0.23	0.00	0.00	5.71	0.24	9.19	0.39				
E					C1-D								C1-D					
					-8.24	-2.98							8.54	-2.05				
D	C1-B		C1-C		C2-A		C3		C2-B		C3		C2-A		C2-C		C1-B	
	-13.79	-7.95	-10.57	-6.95	-7.32	-5.94	-4.04	-4.93	0.00	-3.67	4.56	-4.61	7.70	-5.26	11.08	-5.95	14.31	-6.61
C	C1-B		C1-F		C1-E		C4				C4				C1-F		C1-B	
	-11.86	-14.11	-8.64	-13.11	-5.39	-12.10	-2.12	-11.08			3.06	-10.89			9.58	-12.23	12.81	-12.89
B''			C1															
			-8.06	-14.96														
B'			C1															
			-7.47	-16.85														
B	C1-B		C1-F		C1-E		C5				C5				C1-F		C1-B	
	-9.98	-20.12	-6.76	-19.12	-3.51	-18.11	-0.24	-17.09			1.59	-17.02			8.11	-18.36	11.34	-19.02
A	C1-A		C1-B		C1-B				C2-C				C1-B		C1-B		C1-A	
	-8.11	-26.12	-4.92	-25.10	-2.23	-24.23			0.77	-23.27			3.93	-23.92	6.65	-24.48	9.88	-25.15

Axial forces 1F(Ground Floor)



	1	2	3	4	5	6	7	8	9
	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
F			C1	C1-D	C2-A	C1-D	C1		
			450	775	1125	775	450		
E			C1-D				C1-D		
			775				775		
D	C1-B	C1-C	C2-A	C3	C2-B	C3	C2-A	C2-C	C1-B
	645	775	1125	1500	1125	1500	1125	938	645
C	C1-B	C1-F	C1-E	C4		C4		C1-F	C1-B
	645	1025	1025	1800		1800		1025	645
B"		C1							
		450							
B'		C1							
		450							
B	C1-B	C1-F	C1-E	C5		C5		C1-F	C1-B
	645	1025	1025	900		900		1025	645
A	C1-A	C1-B	C1-B		C2-C		C1-B	C1-B	C1-A
	375	645	645		938		645	645	375

ΣN= 33801 kN

■N・Lx 1F(Ground Floor)

	1 (kN・m)	2 (kN・m)	3 (kN・m)	4 (kN・m)	5 (kN・m)	6 (kN・m)	7 (kN・m)	8 (kN・m)	9 (kN・m)
F			C1 450*-9.06	C1-D 775*-5.51	C2-A 1125*0	C1-D 775*5.71	C1 450*9.19		
E			C1-D 775*-8.24				C1-D 775*8.54		
D	C1-B 645*-13.79	C1-C 775*-10.57	C2-A 1125*-7.32	C3 1500*-4.04	C2-B 1125*0	C3 1500*4.56	C2-A 1125*7.7	C2-C 938*11.08	C1-B 645*14.31
C	C1-B 645*-11.86	C1-F 1025*-8.64	C1-E 1025*-5.39	C4 1800*-2.12		C4 1800*3.06		C1-F 1025*9.58	C1-B 645*12.81
B"		C1 450*-8.06							
B'		C1 450*-7.47							
B	C1-B 645*-9.98	C1-F 1025*-6.76	C1-E 1025*-3.51	C5 900*-0.24		C5 900*1.59		C1-F 1025*8.11	C1-B 645*11.34
A	C1-A 375*-8.11	C1-B 645*-4.92	C1-B 645*-2.23		C2-C 938*0.77		C1-B 645*3.93	C1-B 645*6.65	C1-A 375*9.88
	-26023 sum Grid 1	-34139 sum Grid 2	-29259 sum Grid 3	-14362 sum Grid 4	722 sum Grid 5	18204 sum Grid 6	21951 sum Grid 7	32815 sum Grid 8	28512 sum Grid 9

ΣN・Lx= -1578 kN・m

■N・Ly 1F(Ground Floor)

	1 (kN・m)	2 (kN・m)	3 (kN・m)	4 (kN・m)	5 (kN・m)	6 (kN・m)	7 (kN・m)	8 (kN・m)	9 (kN・m)	
F			C1 450*-0.38	C1-D 775*-0.23	C2-A 1125*0	C1-D 775*0.24	C1 450*0.39			12 sum GridF
E			C1-D 775*-2.98				C1-D 775*-2.05			-3898 sum GridE
D	C1-B 645*-7.95	C1-C 775*-6.95	C2-A 1125*-5.94	C3 1500*-4.93	C2-B 1125*-3.67	C3 1500*-4.61	C2-A 1125*-5.26	C2-C 938*-5.95	C1-B 645*-6.61	-51397 sum GridD
C	C1-B 645*-14.11	C1-F 1025*-13.11	C1-E 1025*-12.1	C4 1800*-11.08		C4 1800*-10.89		C1-F 1025*-12.23	C1-B 645*-12.89	-95337 sum GridC
B"		C1 450*-14.96								-6732 sum GridB"
B'		C1 450*-16.85								-7583 sum GridB'
B	C1-B 645*-20.12	C1-F 1025*-19.12	C1-E 1025*-18.11	C5 900*-17.09		C5 900*-17.02		C1-F 1025*-18.36	C1-B 645*-19.02	-112924 sum GridB
A	C1-A 375*-26.12	C1-B 645*-25.1	C1-B 645*-24.23		C2-C 938*-23.27		C1-B 645*-23.92	C1-B 645*-24.48	C1-A 375*-25.15	-104089 sum GridA

ΣN・Ly= -381948.21 kN・m

Supplement 9

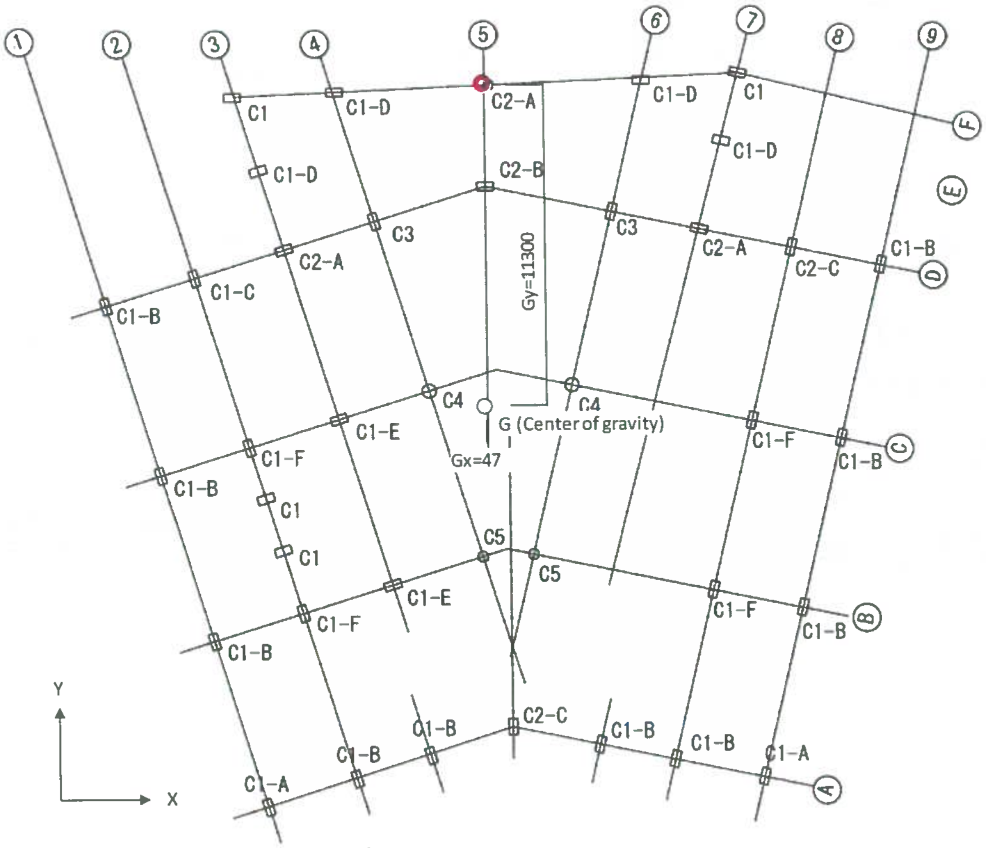
Distance to the center of gravity from the origin of each direction is obtained by dividing the total value of all directions with the total weight, which the floor is holding.

ΣN・Lx= -1578.25 kN・m

ΣN・Ly= -381948.21 kN・m

Gx= ΣN・Lx/ΣN= -1578.25 / 33801 = -0.047 m

Gy= ΣN・Ly/ΣN= -381948.21 / 33801 = -11.300 m

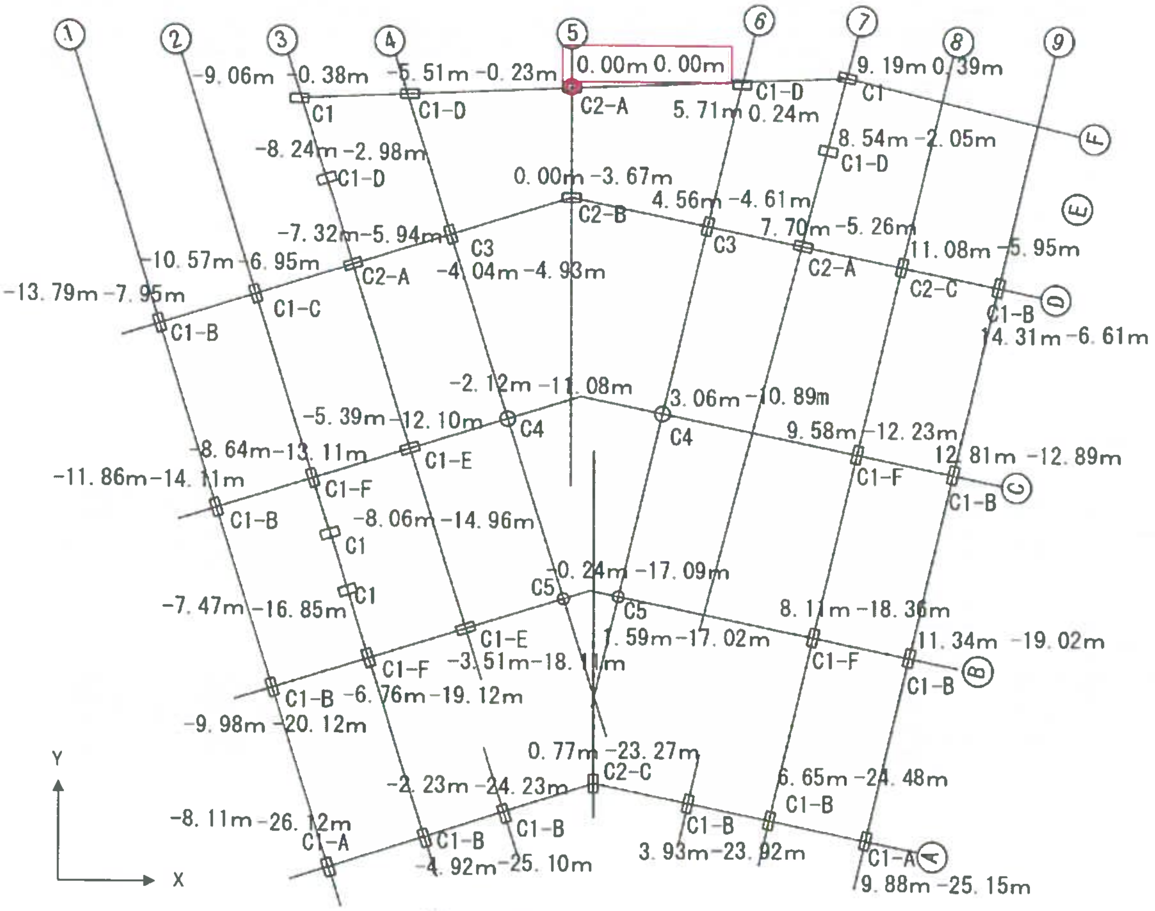


N COLUMN LAY OUT

(2) The center of stiffness

This calculation method is a simple one that takes the sectional area and the stiffness of column into consideration.

Below shows the distances from the origin and the stiffness of each column is shown below



N COLUMN LAY OUT

distances from the origin

	1		2		3		4		5		6		7		8		9	
	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)
F					C1		C1-D		C2-A		C1-D		C1					
					-9.06	-0.38	-5.51	-0.23	0.00	0.00	5.71	0.24	9.19	0.39				
E					C1-D								C1-D					
					-8.24	-2.98							8.54	-2.05				
D	C1-B		C1-C		C2-A		C3		C2-B		C3		C2-A		C2-C		C1-B	
	-13.79	-7.95	-10.57	-6.95	-7.32	-5.94	-4.04	-4.93	0.00	-3.67	4.56	-4.61	7.70	-5.26	11.08	-5.95	14.31	-6.61
C	C1-B		C1-F		C1-E		C4				C4				C1-F		C1-B	
	-11.86	-14.11	-8.64	-13.11	-5.39	-12.10	-2.12	-11.08			3.06	-10.89			9.58	-12.23	12.81	-12.89
B"			C1															
			-8.06	-14.96														
B'			C1															
			-7.47	-16.85														
B	C1-B		C1-F		C1-E		C5				C5				C1-F		C1-B	
	-9.98	-20.12	-6.76	-19.12	-3.51	-18.11	-0.24	-17.09			1.59	-17.02			8.11	-18.36	11.34	-19.02
A	C1-A		C1-B		C1-B				C2-C				C1-B		C1-B		C1-A	
	-8.11	-26.12	-4.92	-25.10	-2.23	-24.23			0.77	-23.27			3.93	-23.92	6.65	-24.48	9.88	-25.15

section area of column Dx=Dy (m^2)

	1	2	3	4	5	6	7	8	9
	(m^2)	(m^2)	(m^2)	(m^2)	(m^2)	(m^2)	(m^2)	(m^2)	(m^2)
F			C1	C1-D	C2-A	C1-D	C1		
			0.1800	0.1800	0.1800	0.1800	0.1800		
E			C1-D				C1-D		
			0.1800				0.1800		
D	C1-B	C1-C	C2-A	C3	C2-B	C3	C2-A	C2-C	C1-B
	0.1800	0.1800	0.1800	0.1800	0.1800	0.1800	0.1800	0.1800	0.1800
C	C1-B	C1-F	C1-E	C4		C4		C1-F	C1-B
	0.1800	0.1800	0.1800	0.1936		0.1936		0.1800	0.1800
B"		C1							
		0.1800							
B'		C1							
		0.1800							
B	C1-B	C1-F	C1-E	C5		C5		C1-F	C1-B
	0.1800	0.1800	0.1800	0.1089		0.1089		0.1800	0.1800
A	C1-A	C1-B	C1-B		C2-C		C1-B	C1-B	C1-A
	0.1800	0.1800	0.1800		0.1800		0.1800	0.1800	0.1800

$\Sigma Dx=\Sigma Dy=$ 6.905 m^2

Multiply stiffness (section area of column) and distance in X and Y direction respectively and then sum them in each X and Y direction separately. Note that when calculating the center of stiffness that affects in X direction, it is to multiply the stiffness in X direction and the distance in Y direction. Vice versa for Y direction.

Effect of Y direction

■Dy・Lx 1F(Ground Floor)

	1 (m^2・m)	2 (m^2・m)	3 (m^2・m)	4 (m^2・m)	5 (m^2・m)	6 (m^2・m)	7 (m^2・m)	8 (m^2・m)	9 (m^2・m)
F			C1 0.18*-9.06	C1-D 0.18*-5.51	C2-A 0.18*0	C1-D 0.18*5.71	C1 0.18*9.19		
E			C1-D 0.18*-8.24				C1-D 0.18*8.54		
D	C1-B 0.18*-13.79	C1-C 0.18*-10.57	C2-A 0.18*-7.32	C3 0.18*-4.04	C2-B 0.18*0	C3 0.18*4.56	C2-A 0.18*7.7	C2-C 0.18*11.08	C1-B 0.18*14.31
C	C1-B 0.18*-11.86	C1-F 0.18*-8.64	C1-E 0.18*-5.39	C4 0.1936*-2.12		C4 0.1936*3.06		C1-F 0.18*9.58	C1-B 0.18*12.81
B"		C1 0.18*-8.06							
B'		C1 0.18*-7.47							
B	C1-B 0.18*-9.98	C1-F 0.18*-6.76	C1-E 0.18*-3.51	C5 0.1089*-0.24		C5 0.1089*1.59		C1-F 0.18*8.11	C1-B 0.18*11.34
A	C1-A 0.18*-8.11	C1-B 0.18*-4.92	C1-B 0.18*-2.23		C2-C 0.18*0.77		C1-B 0.18*3.93	C1-B 0.18*6.65	C1-A 0.18*9.88
	-7.87 sum Grid 1	-8.36 sum Grid 2	-6.44 sum Grid 3	-2.16 sum Grid 4	0.14 sum Grid 5	2.61 sum Grid 6	5.28 sum Grid 7	6.38 sum Grid 8	8.70 sum Grid 9

ΣDy・Ly= -1.71 m^2・m

Effect of X direction

■Dx・Ly 1F(Ground Floor)

	1 (m^2・m)	2 (m^2・m)	3 (m^2・m)	4 (m^2・m)	5 (m^2・m)	6 (m^2・m)	7 (m^2・m)	8 (m^2・m)	9 (m^2・m)	
F			C1 0.18*-0.38	C1-D 0.18*-0.23	C2-A 0.18*0	C1-D 0.18*0.24	C1 0.18*0.39			0.00 sum GridF
E			C1-D 0.18*-2.98				C1-D 0.18*-2.05			-0.91 sum GridE
D	C1-B 0.18*-7.95	C1-C 0.18*-6.95	C2-A 0.18*-5.94	C3 0.18*-4.93	C2-B 0.18*-3.67	C3 0.18*-4.61	C2-A 0.18*-5.26	C2-C 0.18*-5.95	C1-B 0.18*-6.61	-9.34 sum GridD
C	C1-B 0.18*-14.11	C1-F 0.18*-13.11	C1-E 0.18*-12.1	C4 0.1936*-11.08		C4 0.1936*-10.89		C1-F 0.18*-12.23	C1-B 0.18*-12.89	-15.85 sum GridC
B"		C1 0.18*-14.96								-2.69 sum GridB"
B'		C1 0.18*-16.85								-3.03 sum GridB'
B	C1-B 0.18*-20.12	C1-F 0.18*-19.12	C1-E 0.18*-18.11	C5 0.1089*-17.09		C5 0.1089*-17.02		C1-F 0.18*-18.36	C1-B 0.18*-19.02	-20.77 sum GridB
A	C1-A 0.18*-26.12	C1-B 0.18*-25.1	C1-B 0.18*-24.23		C2-C 0.18*-23.27		C1-B 0.18*-23.92	C1-B 0.18*-24.48	C1-A 0.18*-25.15	-31.01 sum GridA

ΣDy・Ly= -83.59 m^2・m

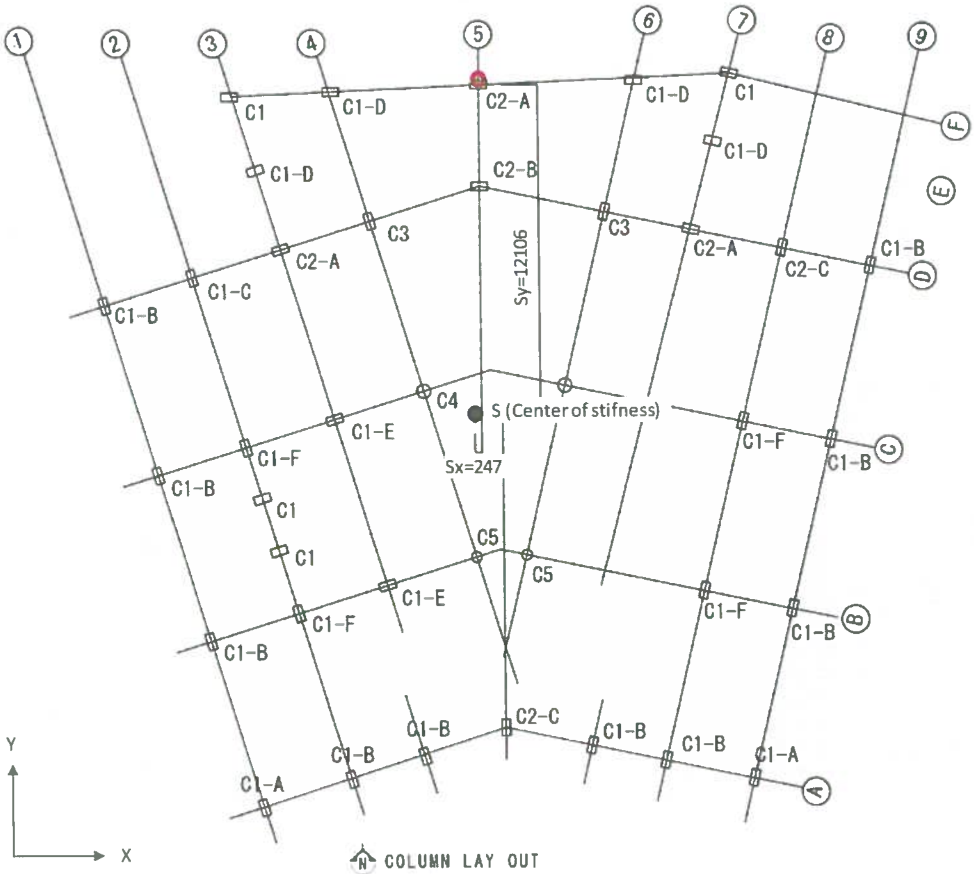
Distance from the origin of each direction to the center of rigidity of a building is obtained by dividing the total value of one direction with the total rigidity of that direction.

$\Sigma Dy \cdot Lx =$ -1.71 $m^2 \cdot m$

$\Sigma Dx \cdot Ly =$ -83.59 $m^2 \cdot m$

$Sx = \Sigma Dy \cdot Lx / \Sigma Dy =$ -1.71 / 6.905 = -0.247 m

$Sy = \Sigma Dx \cdot Ly / \Sigma Dx =$ -83.59 / 6.905 = -12.106 m



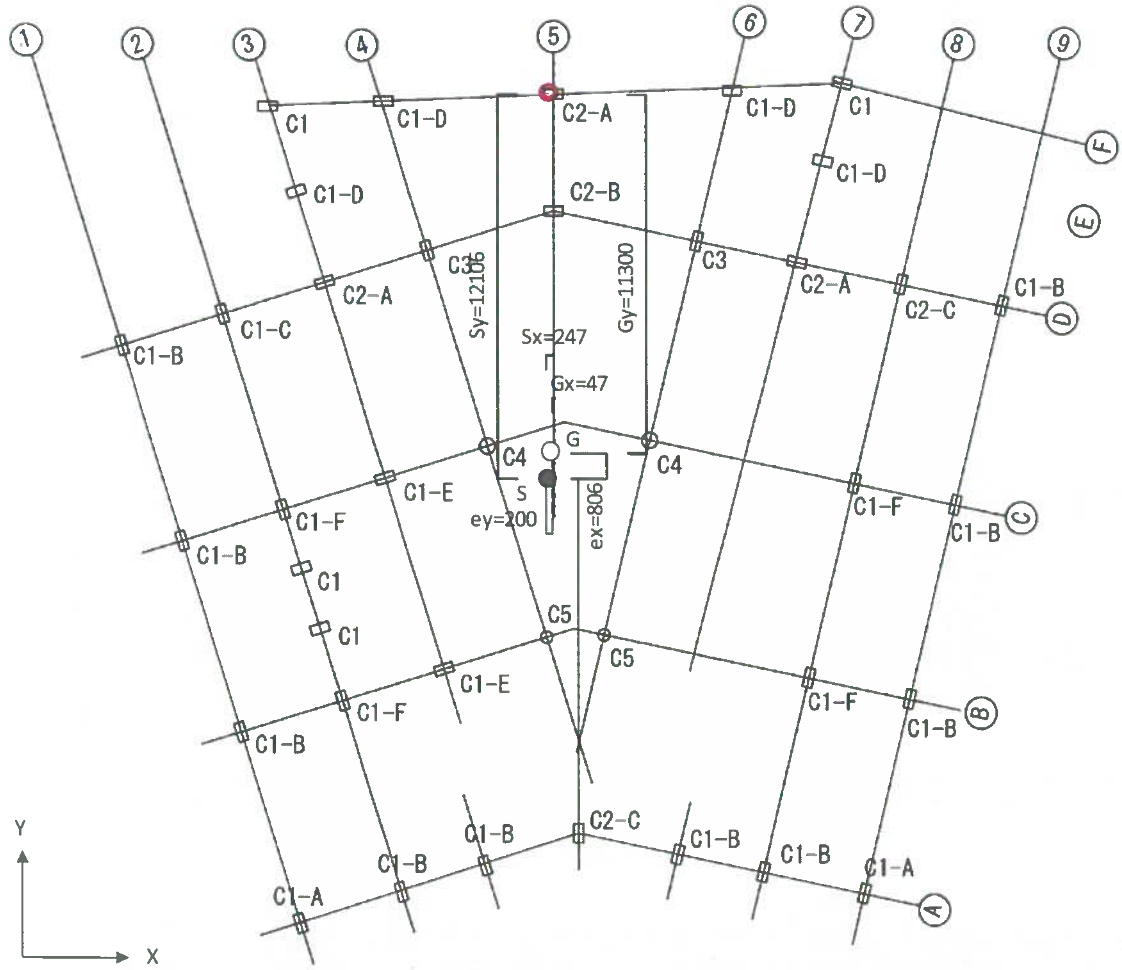
COLUMN LAY OUT

(3) The Eccentricity ratio

Gx= -0.047 m Sx= -0.247 m
Gy= -11.3 m Sy= -12.106 m

● S: Center of stiffness
○ G: Center of gravity

ex= | Gy - Sy | = | -12.106 | - | -11.300 | = 0.806 m
ey= | Gx - Sx | = | -0.247 | - | -0.047 | = 0.200 m



COLUMN LAY OUT

distances from the center of stiffness

	1		2		3		4		5		6		7		8		9	
	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)	Lx(m)	Ly(m)
F					C1		C1-D		C2-A		C1-D		C1					
					8.81	11.73	5.26	11.88	0.25	12.11	5.96	12.35	9.44	12.50				
E					C1-D								C1-D					
					7.99	9.13							8.79	10.06				
D	C1-B		C1-C		C2-A		C3		C2-B		C3		C2-A		C2-C		C1-B	
	13.54	4.16	10.32	5.16	7.07	6.17	3.79	7.18	0.25	8.44	4.81	7.50	7.95	6.85	11.33	6.16	14.56	5.50
C	C1-B		C1-F		C1-E		C4				C4				C1-F		C1-B	
	11.61	-2.00	8.39	-1.00	5.14	0.01	1.87	1.03			3.31	1.22			9.83	-0.12	13.06	-0.78
B"			C1															
			7.81	-2.85														
B'			C1															
			7.22	-4.74														
B	C1-B		C1-F		C1-E		C5				C5				C1-F		C1-B	
	9.73	-8.01	6.51	-7.01	3.26	-6.00	0.01	-4.98			1.84	-4.91			8.36	-6.25	11.59	-6.91
A	C1-A		C1-B		C1-B				C2-C				C1-B		C1-B		C1-A	
	7.86	-14.01	4.67	-12.99	1.98	-12.12			1.02	-11.16			4.18	-11.81	6.90	-12.37	10.13	-13.04

Dx=Dy (m^2)

	1	2	3	4	5	6	7	8	9
	(m^2)	(m^2)	(m^2)	(m^2)	(m^2)	(m^2)	(m^2)	(m^2)	(m^2)
F			C1 0.1800	C1-D 0.1800	C2-A 0.1800	C1-D 0.1800	C1 0.1800		
E			C1-D 0.1800				C1-D 0.1800		
D	C1-B 0.1800	C1-C 0.1800	C2-A 0.1800	C3 0.1800	C2-B 0.1800	C3 0.1800	C2-A 0.1800	C2-C 0.1800	C1-B 0.1800
C	C1-B 0.1800	C1-F 0.1800	C1-E 0.1800	C4 0.1936		C4 0.1936		C1-F 0.1800	C1-B 0.1800
B"		C1 0.1800							
B'		C1 0.1800							
B	C1-B 0.1800	C1-F 0.1800	C1-E 0.1800	C5 0.1089		C5 0.1089		C1-F 0.1800	C1-B 0.1800
A	C1-A 0.1800	C1-B 0.1800	C1-B 0.1800		C2-C 0.1800		C1-B 0.1800	C1-B 0.1800	C1-A 0.1800

ΣDx=ΣDy= 6.905 m^2

■Dx・Ly^2 1F(Ground Floor)

	1	2	3	4	5	6	7	8	9	
	(m^2・m^2)	(m^2・m^2)	(m^2・m^2)	(m^2・m^2)	(m^2・m^2)	(m^2・m^2)	(m^2・m^2)	(m^2・m^2)	(m^2・m^2)	
F			C1 0.18*11.73^2	C1-D 0.18*11.88^2	C2-A 0.18*12.11^2	C1-D 0.18*12.35^2	C1 0.18*12.5^2			132.15 sum GridF
E			C1-D 0.18*9.13^2				C1-D 0.18*10.06^2			33.22 sum GridE
D	C1-B 0.18*4.16^2	C1-C 0.18*5.16^2	C2-A 0.18*6.17^2	C3 0.18*7.18^2	C2-B 0.18*8.44^2	C3 0.18*7.5^2	C2-A 0.18*6.85^2	C2-C 0.18*6.16^2	C1-B 0.18*5.5^2	67.71 sum GridD
C	C1-B 0.18*-2^2	C1-F 0.18*-1^2	C1-E 0.18*0.01^2	C4 0.1936*1.03^2		C4 0.1936*1.22^2		C1-F 0.18*-0.12^2	C1-B 0.18*-0.78^2	1.51 sum GridC
B"		C1 0.18*-2.85^2								1.46 sum GridB"
B'		C1 0.18*-4.74^2								4.04 sum GridB'
B	C1-B 0.18*-8.01^2	C1-F 0.18*-7.01^2	C1-E 0.18*-6^2	C5 0.1089*-4.98^2		C5 0.1089*-4.91^2		C1-F 0.18*-6.25^2	C1-B 0.18*-6.91^2	47.83 sum GridB
A	C1-A 0.18*-14.01^2	C1-B 0.18*-12.99^2	C1-B 0.18*-12.12^2		C2-C 0.18*-11.16^2		C1-B 0.18*-11.81^2	C1-B 0.18*-12.37^2	C1-A 0.18*-13.04^2	197.82 sum GridA

Ix=ΣDx・Ly^2= 485.73 m^2・m^2

■Dy・Lx^2 1F(Ground Floor)

	1	2	3	4	5	6	7	8	9
	(m^2・m^2)	(m^2・m^2)	(m^2・m^2)	(m^2・m^2)	(m^2・m^2)	(m^2・m^2)	(m^2・m^2)	(m^2・m^2)	(m^2・m^2)
F			C1 0.18*8.81^2	C1-D 0.18*5.26^2	C2-A 0.18*0.25^2	C1-D 0.18*5.96^2	C1 0.18*9.44^2		
E			C1-D 0.18*7.99^2				C1-D 0.18*8.79^2		
D	C1-B 0.18*13.54^2	C1-C 0.18*10.32^2	C2-A 0.18*7.07^2	C3 0.18*3.79^2	C2-B 0.18*0.25^2	C3 0.18*4.81^2	C2-A 0.18*7.95^2	C2-C 0.18*11.33^2	C1-B 0.18*14.56^2
C	C1-B 0.18*11.61^2	C1-F 0.18*8.39^2	C1-E 0.18*5.14^2	C4 0.1936*1.87^2		C4 0.1936*3.31^2		C1-F 0.18*9.83^2	C1-B 0.18*13.06^2
B"		C1 0.18*7.81^2							
B'		C1 0.18*7.22^2							
B	C1-B 0.18*9.73^2	C1-F 0.18*6.51^2	C1-E 0.18*3.26^2	C5 0.1089*0.01^2		C5 0.1089*1.84^2		C1-F 0.18*8.36^2	C1-B 0.18*11.59^2
A	C1-A 0.18*7.86^2	C1-B 0.18*4.67^2	C1-B 0.18*1.98^2		C2-C 0.18*1.02^2		C1-B 0.18*4.18^2	C1-B 0.18*6.9^2	C1-A 0.18*10.13^2
	85.42 sum Grid 1	63.76 sum Grid 2	41.83 sum Grid 3	8.24 sum Grid 4	0.21 sum Grid 5	13.05 sum Grid 6	44.47 sum Grid 7	61.65 sum Grid 8	111.51 sum Grid 9

Iy=ΣDy・Lx^2= 430.14 m^2・m^2

Torsional stiffness

$K_T=Ix+Iy$

K_T: Torsional stiffness

Ix: $\Sigma Dx \cdot Ly^2$

Iy: $\Sigma Dy \cdot Lx^2$

$Ix=\Sigma Dx \cdot Ly^2=$ 485.73 $m^2 \cdot m^2$

$Iy=\Sigma Dy \cdot Lx^2=$ 430.14 $m^2 \cdot m^2$

$K_T=$ Ix+Iy= 485.73 + 430.14 = 915.88 $m^2 \cdot m^2$

Spring radius

Obtain the spring radius to be used for evaluating the earthquake-load of X and Y directions from the torsional stiffness.

$$r_{ex} = \sqrt{K_T/\Sigma Dx} \qquad r_{ey} = \sqrt{K_T/\Sigma Dy}$$

rex: Spring radious for X direction

rey: Spring radious for Y direction

$$r_{ex} = \sqrt{915.88/6.905} \quad = \quad 11.517 \quad m$$

$$r_{ey} = \sqrt{915.88/6.905} \quad = \quad 11.517 \quad m$$

Eccentricity ratio

Eccentricity is evaluated by the ratio of eccentric distance and spring radius that obtained.Calculation example is as shown below.

$R_{ex}=$ $ex/r_{ex}=$ 0.806 / 11.517 = 0.070 $\leqq 0.15$

$R_{ey}=$ $ey/r_{ey}=$ 0.200 / 11.517 = 0.017 $\leqq 0.15$

Rex : Eceentricity ratio for Xdirection

Rey : Eceentricity ratio for Ydirection

Eccentricity i Gi=1.0 for Re≦0.15

Gi=0.8 for Re≧0.30

Gi is set by interporation for 0.15<Re<0.3

The eccentricity ratio can be calculated in the same procedure.

Table S 9.9 The Eccentricity Ratio and G_l

		R_{ex}		Gl
6F	5th Floor	0.437	> 0.15	Gl=0.8
5F	4th Floor	0.136	≦ 0.15	Gl=1.0
4F	3rd Floor	0.102	≦ 0.15	Gl=1.0
3F	2nd Floor	0.086	≦ 0.15	Gl=1.0
2F	1st Floor	0.076	≦ 0.15	Gl=1.0
1F	Ground Floor	0.070	≦ 0.15	Gl=1.0

(4) The (Stiffness/mass) Ratio of Above and Below Stories

Stiffness balance of upper and lower floors of the building can be evaluates by comparing the stiffness/mass-ratio of upper and lower floors. Calculation is shown below.

	ΣWi(kN)	N	β (N-1)/N
5th Floor 6	3210	1	2.000
4th Floor 5	9949	2	0.500
3rd Floor 4	15918	3	0.667
2nd Floor 3	21879	4	0.750
1st Floor 2	27842	5	0.800
Ground Floor 1	33801	6	0.833

N: The number of floors sustained by the story concerned.

n=(the ratio of the stiffness to the weight of the story above) / (the ratio of the stiffness to the weight of the story concerned) x β
β=(N-1)/N

(Stiffness/mass)Ratio=(Stiffness of the story)/(the weigh of the story above)
Stiffnes of the story=(the sum of column area +the sum of wall area x α)/the story height

Stiffness of the ground floor= 6.905/3.6 = 1.918
Stiffness of the first floor= 6.905/3.6 = 1.918

(Stiffness/mass)Ratio of ground floor= 1.918 / 33801 = 0.000057
(Stiffness/mass)Ratio of first floor= 1.918 / 27842 = 0.000069

n1= 0.000069 / 0.000057 * 0.833 = 1.012 n≦1.2

↓

$\boxed{\text{Gl=1.0}}$

(Both direction is same)

The (Stiffness/mass) Ratio of above and below stories can be calculated in the same procedure.

Table S 9.10 The (Stiffness/mass) Ratio of Above and Below Stories and G_n

		ΣWi(kN)	height(m)	N	β=(N-1)/N	Total Cross Sectional area of column	stiffness	Stiffness/mass	n	Check	Gn
6F	5th Floor	3210	3.600	1	2.000	2.956	0.821	0.000256	1.471	1.2<n≦1.7	Gn=0.9
5F	4th Floor	9949	3.600	2	0.500	6.736	1.871	0.000188	0.680	n≦1.2	Gn=1.0
4F	3rd Floor	15918	3.600	3	0.667	6.736	1.871	0.000118	1.067	n≦1.2	Gn=1.0
3F	2nd Floor	21879	3.600	4	0.750	6.736	1.871	0.000086	1.031	n≦1.2	Gn=1.0
2F	1st Floor	27842	3.600	5	0.800	6.905	1.918	0.000069	0.993	n≦1.2	Gn=1.0
1F	Ground Floor	33801	3.600	6	0.833	6.905	1.918	0.000057	1.012	n≦1.2	Gn=1.0

(5) The Irregularity Index S_D

The result of the irregularity index S_D is shown below.

Table S 9.11 The Irregularity Index S_D

				Gi (Grade)			SD_2	Note
				1.0	0.9	0.8		
Horizontal balance	a	Regularity		[✓ Regular a1]	Nearly regular a2	Irregular a3	1.000	1.0
	b	Aspect ratio of plan		[✓ b≦5]	5<b≦8	8<b	1.000	1.0
	c	Narrow part		[✓ 0.8≦c]	0.5≦c<0.8	c<0.5	1.000	1.0
	d	Expansion joint		[✓ 1/100≦d]	1/200≦d<1/100	d<1/200	1.000	1.0
	e	Well-style area		[✓ e≦0.1]	0.1<e≦0.3	0.3<e	1.000	1.0
	f	Eccentric well-style area		[✓ f1≦0.4 & f2≦0.1]	f1≦0.4 & 0.1<f2≦0.3	0.4<f1 or 0.3<f2	1.000	1.0
	g						1.000	1.0
Elevation balance	h	Underground floor		1.0≦h	0.5≦h<1.0	[✓ h<0.5]	1.000	0.8
	i	Story height uniformity		[✓ 0.8≦i]	0.7≦i<0.8	i<0.7	1.000	1.0
	j	Soft story		[✓ No soft story]	Soft story	Eccentric soft story	1.000	1.0
	k						1.000	1.0
Second level irregurity index SD_2'(A to K)							1.000	
Eccentricity	l	Eccentricity	6th	[✓ l≦0. 1]	0. 1<l≦0. 15	0. 15<l	0.800	
			5th	[✓ l≦0. 1]	0. 1<l≦0. 15	0. 15<l	1.000	
			4th	[✓ l≦0. 1]	0. 1<l≦0. 15	0. 15<l	1.000	
			3rd	[✓ l≦0. 1]	0. 1<l≦0. 15	0. 15<l	1.000	
			2nd	[✓ l≦0. 1]	0. 1<l≦0. 15	0. 15<l	1.000	
			1st	[✓ l≦0. 1]	0. 1<l≦0. 15	0. 15<l	1.000	
Stiffness	n	(Stiffness/mass) Ratio of above and below stories	6th	[✓ n≦1. 2]	1. 2<n≦1. 7	1. 7<n	0.900	
			5th	[✓ n≦1. 2]	1. 2<n≦1. 7	1. 7<n	1.000	
			4th	[✓ n≦1. 2]	1. 2<n≦1. 7	1. 7<n	1.000	
			3rd	[✓ n≦1. 2]	1. 2<n≦1. 7	1. 7<n	1.000	
			2nd	[✓ n≦1. 2]	1. 2<n≦1. 7	1. 7<n	1.000	
			1st	[✓ n≦1. 2]	1. 2<n≦1. 7	1. 7<n	1.000	

Second level irregurity index SD_2 (SD_2' × Eccentricity × Stiffness)	6th	0.720
	5th	1.000
	4th	1.000
	3rd	1.000
	2nd	1.000
	1st	1.000

3.5 The Time Index *T*

The result of the time index *T* is shown below.

Table S 9.12 The Time Index *T*

Item to be checked		Degree	T value (Check circle at relevant degree)	check ○ at relevant degree
I.Deflection		Tilting of a building or obvious uneven settlement is observed	0.70	
		Landfill site of former rice field	0.90	
		Deflection of beam or column is observed visually	0.90	
		No correspondence to the foregoing	1.00	○
II.Cracking in walls and columns		Rain leak with rust of reinforcing bar is observed	0.80	
		Inclined cracking in columns is obviously observed	0.90	
		Countless cracking is observed in external wall	0.90	
		Rain leak without rust of reinforcing bar is observed	0.90	
		No correspondence to the foregoing	1.00	○
III.Fire experience		Trace	0.70	
		Experience but traceless	0.80	
		No experience	1.00	○
IV.Deterioration	IV-i.Rust of reinforceing bar	Grade iii or more	0.90	
		Grade iior more	0.95	
		Grade i	1.00	○
	IV-ii.Depth of carbonation	Over the thickness of concrete(Grade ii)	0.90	
		Same or less the thickness of cover concrete(Grade i)	0.95	
		No correspondence to the foregoing	1.00	○
V.Finishing condition		Significant spalling of external finishing due to aging is observed	0.90	
		Significant spalling and deterioration of internal finishing is observed	0.90	
		No problem	1.00	○

Time index T by the first level inspection T_1	1.0

3.6 The seismic index of structure I_s and $C_{TU} \cdot S_D$

Result of I_S and $C_{TU} \times S_D$ is shown below.

Table S 9.13 Result of Seismic Evaluation

Results of the secondLevel screening				
Name of build	E BLOCK FOR MAG OSMANI MEDICAL COLLEGE HOSPITAL	Construction year		
Address				
Evaluated Engineer	PWD	Data ofevaluation		
Seismic demand index	Iso=	0.43	$C_{TU} \cdot S_D$=	0.22

Direction	Story	C	F	Failure Mode	Eo	T	S_D	Is	$C_{TU} \cdot S_D$	Result	Adoption	Eq
X	6	0.387	1.75		0.40	1.000	0.720	0.28	0.16	NG		5
		0.387	1.27		0.287			0.206	0.2064	NG		
		0.279	1.00		[0.163]			0.117	[0.117]	NG		
		0.000	1.00		0.395			0.284	0.163	NG	●	4
		0.387	1.75									
	5	0.282	1.75		0.31	1.000	1.000	0.31	0.18	NG		5
		0.282	1.27		0.228			0.228	0.2276	NG		
		0.218	1.00		[0.139]			0.139	[0.139]	NG		
		0.016	1.00		0.314			0.314	0.179	NG	●	4
		0.282	1.75									
	4	0.220	1.75		0.27	1.000	1.000	0.27	0.15	NG		5
		0.220	1.27		0.195			0.195	0.1953	NG		
		0.169	1.00		[0.118]			0.118	[0.118]	NG		
		0.011	1.00		0.269			0.269	0.154	NG	●	4
		0.220	1.75									
	3	0.157	1.75		0.21	1.000	1.000	0.21	0.12	NG		5
		0.157	1.27		0.156			0.156	0.1555	NG		
		0.139	1.00		[0.108]			0.108	[0.108]	NG		
		0.026	1.00		0.215			0.215	0.122	NG	●	4
		0.157	1.75									
	2	0.134	1.75		0.20	1.000	1.000	0.20	0.12	NG	●	5
		0.134	1.27		0.149			0.149	0.1486	NG		
		0.124	1.00		[0.108]			0.108	[0.108]	NG		
		0.028	1.00		0.206			0.206	0.117	NG		4
		0.134	1.75									
	1	0.070	1.75		0.12	1.000	1.000	0.12	0.07	NG		5
		0.070	1.27		0.089			0.089	0.0890	NG		
		0.115	1.00		[0.115]			0.115	[0.115]	NG		
		0.064	1.00		0.138			0.138	0.070	NG	●	4
		0.070	1.75									

$$I_{S0} = 0.8 \cdot 2/3 \cdot Z \cdot I \cdot C_s$$
$$Z = 0.20, I = 1.0\ Soil\ SD(S3), h_n = 22.50m$$
$$T = 0.0446 \cdot (h_n)^{0.9} = 0.768s$$
$$C_s = 2.246$$
$$I_{S0} = 0.8 \cdot 2/3 \cdot 0.36 \cdot 1.0 \cdot 2.246 = 0.431$$

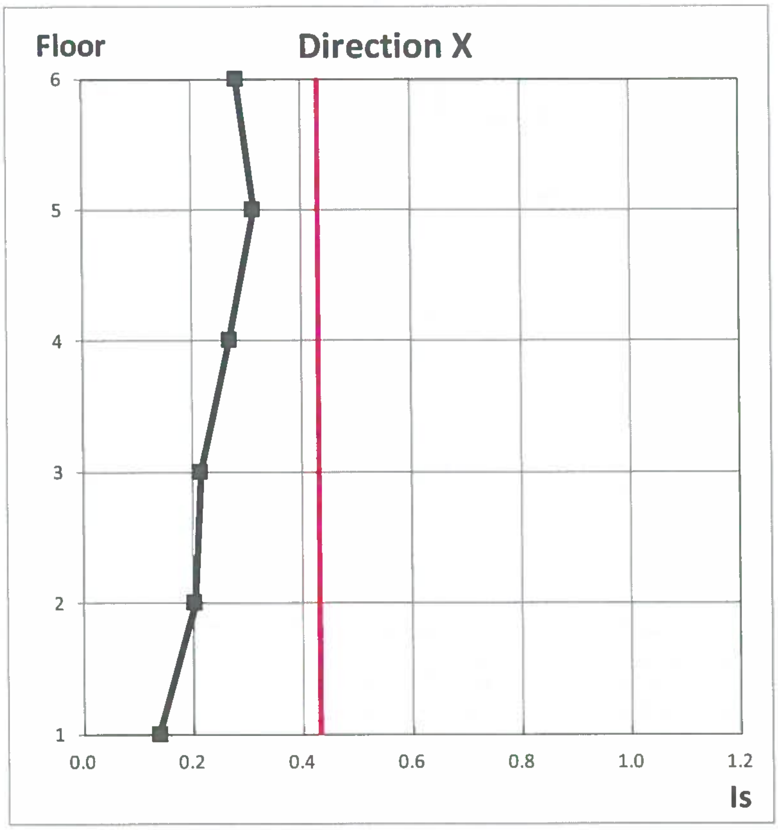


Figure S 9.6 I_S Distribution

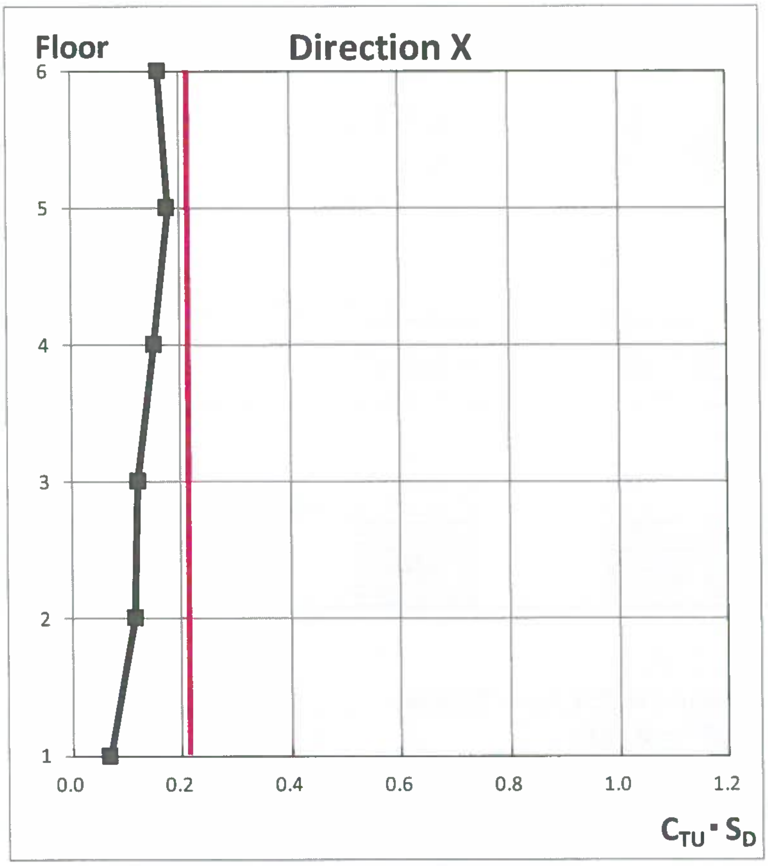


Figure S 9.7 $C_{TU} \cdot S_D$ Distribution

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