CNCRP

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

MANUAL FOR SEISMIC DESIGN OF REINFORCED CONCRETE BUILDINGS

Public Works Department

MANUAL FOR SEISMIC DESIGN OF REINFORCED CONCRETE BUILDINGS

PUBLIC WORKS DEPARTMENT

PREPARED UNDER

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

A TECHNICAL COOPERATION PROJECT BETWEEN PWD AND JICA

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Foreword

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

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

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

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

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

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

1. Manual for Seismic Evaluation of Existing Reinforced Concrete Buildings
2. Manual for Seismic Retrofit Design of Existing Reinforced Concrete Buildings
3. Manual for Retrofit Construction and Supervision of Reinforced Concrete Buildings

4. Guidelines for Quality Control of Design and Construction of Reinforced Concrete Buildings
5. Manual for Seismic Design of Reinforced Concrete Buildings
6. Manual for Vulnerability Assessment and Damage Prediction of Reinforced Concrete Buildings against Non Seismic Hazards

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

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

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

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

Retrofitting a vulnerable building is costly. Also some of the vulnerable buildings may not be retrofitable due to technical and other socio-cultural reasons. So it is important to design and construct new buildings abiding by all the requirements of the latest building code, saving future expenditures and efforts for retrofitting. To make the buildings of our country earthquake disaster resilient, the application of knowledge of seismic building design and construction by the engineers are indispensable. This "Manual for Seismic Design of Reinforced Concrete Building" has been prepared to help the structural engineers to do seismic design of reinforced concrete buildings properly as per BNBC 2015. In Part II of the manual, seismic design methods in Japan, pile foundation design in liquefiable soil, architectural license system of Japan etc. is introduced to share the Japanese experience with the engineers of our country.

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

In order to enhance the safety of the buildings in Bangladesh against natural disasters, to follow Bangladesh National Building Code (BNBC) during their design and construction is the first priority. We should never forget the tragedy of RANA Plaza on April 2013, as well as the need for measures against future earthquakes. BNBC was published in the year 1993, enacted in 2006, and the latest version BNBC 2015 (Final draft, July 2015) is going to be published soon. Engineers of Bangladesh will follow the BNBC 2015 in design and construction of buildings in the coming years.

The purpose of this manual is to design buildings by knowing the steps of seismic design and referring to the detailed background, aiding in the implementation of seismic design of buildings according to the BNBC 2015. The seismic design methodology according to the BNBC 2015 is described in this manual. Construction management, which is covered in another manual produced by CNCRP, is important as is seismic design method for building safety. Both seismic design method and construction management are great helps for buildings against earthquakes.

This Design Manual for new reinforced concrete buildings is divided into two Parts: Part I and Part II. Part I is composed of Chapter 1 to Chapter 6. The basic idea and scope of the manual is described in Chapter 1. In Chapter 2 the seismic design regulations in the new BNBC are explained in accordance with provisions in detail as much as possible. The calculation example of an actual building is shown in Chapter 3 for structural engineers to understand the seismic design method easily. Seismic response of buildings, earthquake load and its impact on structure and reinforcing points for seismic designed buildings are shown in Chapter 4, 5 and 6 respectively. Especially, seismic conceptions expressed in Chapter 4 and 5 are important and structural engineers are expected to understand them correctly. Important items or matters in Chapter 2 for structural engineers to understand the BNBC 2015 are referred to by section numbers in Chapters 4, 5 and 6 such as " See Sec. 4.2.1"or "See Sec. 5.3.1" etc.

Section 2.2.14 in Chapter 2 shows the procedure to calculate the generation of earthquake load following IBC 2006 or ASCE 7-05, to be used in commercial software.

Four appendices are attached as the backup of Part I of the manual. Foundation design in seismic zone, especially pile foundation design in the construction site having liquefaction potential is described in Appendix A. Though the inelastic analysis of pile foundations or interaction analysis with sub and super structure is popular internationally these days, only the basic and elementary design method is introduced. In Appendix B some technical reports concerning unreinforced masonry infill structures are introduced. Unreinforced masonry infill walls (URM or MI) are quite common in Bangladesh. Many research papers published in the world have suggested that it is very important to do precise seismic design considering the rigidity and strength of such brick walls. From that point of view, Appendix B is prepared. In Appendix C, pushover analysis based on static non-linear analysis is described, followed by a case study for "The Dhaka Medical College Extension Building." One of the methods to calculate the collapse mechanism of structures is the pushover analysis.

A collection of photographs of earthquake damaged reinforced concrete structures, depicting different failure modes are given in Appendix D.

Part II of the Manual, introduces seismic design methods in Japan as reference materials. The updated, most popular seismic design method in Japan is introduced in Chapter 1. The history of the Japanese seismic design method is also shown in Chapter 1. A case-study of the seismic design method is shown in Chapter 2. Response and limit capacity calculation which is another seismic design method in Japan and the calculation example for Dhaka Medical College Extension Building are shown in Chapters 3 and 4 respectively. The Part II of the Manual has four appendices. Similar to the Appendix A in the Part I of the Manual, "Pile Foundation Design in Liquefiable Soil" is explained in Appendix I, translated from Japanese regulations.

Establishment of an inspection system as well as an accreditation system is important to properly regulate building construction in Bangladesh. From this point of view, the architectural license system of Japan and the contents of structural design documentation in Japan are introduced as a reference in Appendix II and Appendix III, respectively.

In Appendix IV, the regulations for live load and load combination in the Building Standard Law of Japan are shown, and one way to obtain the live load for machine rooms where machines such as generators are placed is described. This is because one of the most critical reasons for the collapse of the garment building in Bangladesh in April 2013 was that the building was overloaded during operation. It is a very important matter to consider appropriate live load for factory buildings where many machines are concentrated.

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Editorial Advisory Board:

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LIST OF SYMBOLS

A_{ch}	Cross-sectional area of a structural member measured to the outside edges of transverse reinforcement, (mm^2).
A_{cv}	Cross area of concrete section bounded by web thickness and length of section in the direction of shear force considered, (mm^2).
A_g	Gross area of concrete section, (mm^2).
$A_{g,be}$	Gross area of wall boundary containing longitudinal reinforcement $A_{s,be}$(mm^2).
A_r	Wall aspect ratio, measured at roof level
$A_{s,be}$	Total area of longitudinal reinforcement at wall boundary,(mm^2).
A_{sp}	Area of transverse reinforcement in section perpendicular to direction considered
A_{sh}	Total cross-sectional area of transverse reinforcement (including crossties) within spacing and perpendicular to dimension b_c , (mm^2).
A_x	Torsional amplification
b_c	Cross-sectional dimension of member core measured to the outside edges of the transverse reinforcement composing area A_{sh}, (mm).
b_w	Web width or wall thickness, (mm).
c	distance from extreme compression fiber to neutral axis, (mm)
C_d	Deflection amplification factor
C_M	Centre of mass
C_R	Centre of rigidity in torsional response calculations
C_V	Centre of shear strength in torsional response calculations
$C_{T,S,\mu,p}$	Inelastic seismic base shear coefficient
d	Effective depth of section to centroid of tension reinforcement
d_{bl}	Reinforcing bar diameter
d_b	Nominal diameter of bar, wire, or prestressing strand, (in)
d_s	Diameter of spiral, mm (in).
E_s	Strain energy
e_R	Rigidity eccentricity in torsional response calculations
e_v	Strength eccentricity in torsional response calculations
F	Force
F_{el}	Seismic force corresponding to elastic spectrum
F_R	Seismic force reduced from elastic spectrum by force-reduction factor
F_r	Total lateral force acting at level r
f	Cyclic frequency
f_c'	Specified (28 day) concrete compression strength
f_s	Equivalent static force
f_{ye}	Expected yield strength of steel for DDBD
f_u	Steel ultimate stress
f_{yh}	Yield strength of hoop or spiral transverse reinforcement
f_{yt}	Specified yield strength f_y of transverse reinforcement, (MPa)
H	Height
h	Section depth
h_s	Storey height
h_b	Beam section depth
h_c	Column section depth
h_x	Maximum center-to-center horizontal spacing of crossties or hoop legs on all faces of the boundary element, (mm).

h_w	Height of entire wall from base to top, or clear height of wall segment or wall pier considered, (mm).
I	Structural importance factor
l_{be}	Length of boundary element, (mm).
l_c	Length of compression member in a frame, measured center-to-center of the joints in the frame, (mm).
l_d	Development length of straight bar, (mm)
l_{dh}	Development length in tension of deformed bar with a standard hook, measured from critical section to outside end of hook, (mm).
l_{dt}	Development length in tension of headed deformed bar, measured from the critical section to the bearing face of the head, (mm)
l_n	length of clear span measured face-to-face of supports, (in).
l_0	Length, measured from joint face along axis of structural member, over which special transverse reinforcement must be provided, (in).
l_s	Length of the splice, mm(in)
l_w	Length of entire wall or length of wall segment or wall pier considered in direction of shear force, (in).
K	Structure stiffness
k	Element stiffness
K_e	Structure effective stiffness for DDBD
K_{tr}	Transverse reinforcement index, see 12.2.3, Chapter 12 of ACI 318
L_p	Plastic hinge length
L_{sp}	Strain penetration length
M	Moment
M_i	Ideal flexural strength
M_{ta}	Torsional moment due to accidental torsion
M_u	Ultimate moment
M_w	Earthquake moment magnitude
m_e	Effective mass participating in the fundamental mode
$M_{n,CS}$	Value of M_n at the critical section for flexure and axial force, (in.-lb)
$M_{u,CS}$	Value of M_u at the critical section for flexure and axial force, (in.-lb)
n	Number of storey
P	Axial force on section
P_{be}	Ratio of area of boundary element longitudinal reinforcement to gross area boundary element
PS_a	Pseudo spectral acceleration
PS_v	Pseudo spectral velocity
P_u	Factored axial force; to be taken as positive for compression and negative for tension, (lb)
R	Force-reduction factor applied to elastic spectrum in force-based design
S_o	Over strength
S_i	Ideal strength
S_u	Required strength to resist combined actions due to factored loads and forces
s	Spacing along member axis of transverse reinforcement
S_a	Acceleration response spectra
S_v	Velocity response spectra
S_d	Deformation response spectra
S_{gx}, S_{gy}	Section moduli of gross section about x and y axes, (mm^3)
T	Natural vibration period/ Time period

V_E	Shear force in structure corresponding to elastic response
V_y	Shear force in structure corresponding to fully yielded strength
V_S	Shear force in structure corresponding to design lateral force
R	Response modification factor
R_d	Ductility reduction factor
W	Weight/ Seismic dead load
V_c	Nominal shear strength provided by concrete, (N)
V_b	Total base shear applied to a building
V_u	Factored shear force at section, (N)
$V_{u,CS}$	Value of V_u at critical section, (N)
W_{tr}	Total weight at level r
w_i	Floor weight at storey level i
Z	Zone factor
$a_{(T)}$	Period-dependent acceleration from response spectrum
Δ	Displacement
Δ_a	Allowable storey drift
Δ_p	Plastic displacement
Δ_{max}	Maximum displacement
Δ_y	Yield displacement
Δ_u	Ultimate displacement
Φ	Strength reduction factor
$\emptyset_o$	Flexural overstrength factor
Φ_y	Yield curvature of bilinear approximation to moment-curvature curve
Φ_y'	Curvature at first yield of reinforcing steel, or compression strain of 0.002
Φ_m	Ultimate curvature
μ	Ductility factor;
μ_{sys}	Structure displacement ductility factor
μ_Δ	Displacement ductility factor
μ_ϕ	Curvature ductility factor
ε_y	Yield strain of steel
ε_c	Concrete strain
ε_{cm}	Maximum concrete compression strain
ρ_L	Longitudinal reinforcement ratio in concrete section
ρ_s	Area ratio of transverse reinforcement in section
θ	Rotation; drift angle (displacement/height); angle
θ_c	Code drift limit
λ_o	Materials over strength factor
ω	Circular frequency
ξ	Viscous damping ratio
$u(t)$	Displacement or deformation at time t
u_g	Ground displacement
$\ddot{u}_g$	Ground acceleration
u^t	Total displacement
θ_Δ	Stability index in P-Δ design
Ω_o	Structural Overstrength Factor
ρ	Redundancy factor
Δ_i	Storey displacement at storey level i

ψ_e	factor used to modify development length based on reinforcement coating,
ψ_t	Factor used to modify development length based on reinforcement location,
ψ_s	Factor used to modify development length based on reinforcement size,
λ	Modification factor reflecting the reduced mechanical properties of lightweight concrete all relative to normal weight concrete of the same compressive strength
ρ_l	Ratio of area of distributed longitudinal reinforcement to gross concrete area perpendicular to that reinforcement
ρ_t	Ratio of area of distributed transverse reinforcement to gross concrete area perpendicular to that reinforcement
δ_u	Design displacement, (mm)
σ	Normal stress used to determine required boundary elements by Method II, (MPa)

PART-I

CHAPTER 1. GENERAL

1.1 INTRODUCTION

1.1.1 General

This manual focuses to make structural engineers in Bangladesh practically understand the seismic design process of the BNBC 2015 (Final Draft, July 2015). The manual is composed of two Parts: Part-I and Part-II. Part-I includes seismic design codes in the BNBC 2015. The basic and elementary technical knowledge regarding the seismic design method is explained. Since this manual is prepared for structural engineers to master fundamental principles and practices of building design, the buildings are designed with extensive thought rather than direct design from computers. Structural engineers need to verify the loadpath of gravity load, earthquake load and other special loads. It is required to design stable and economical buildings through repeated calculation. (Hereafter, unless noted otherwise, "BNBC" stands for BNBC 2015).

The most important thing for structural engineers is to design well balanced building. Buildings that are well balanced in vertical and horizontal stiffness can resist external forces such as gravity load and earthquake load. Such buildings can endure throughout an earthquake and would not collapse easily.

On the other hand, when weak points are left as they are in unbalanced buildings, the earthquake load emanates concentrically from the weak points. After a local failure or story collapse occurs, human lives, buildings and properties are lost. In order to design well balanced buildings, it is required to accumulate knowledge and information regarding the seismic design method. To design well-balanced buildings, cooperation between the structural engineers, architects, electrical and mechanical engineers are indispensable.

Helmut Jahn who is an internationally famous architect said that his most important job prior to a project is to find an excellent structural engineer and when he could find one, and then his main job would be finished. He also said that modern buildings make the highest demands of technology and it is not the architect who fulfils them but the structural engineers.

Structural engineers should motivate the architects, electrical and mechanical engineers to understand the basic structural design method in order to get their cooperation.

1.1.2 Fundamental Concepts

To design the elements of a reinforced concrete building, a structural engineer should have sound understanding of the properties of materials such as concrete and steel reinforcement. BNBC 2015 specified on their properties in Section 5.2 of Part 5. Combination of concrete and steel reinforcement depends upon the usage of a building, its scale, environment and cost effectiveness.

The combination of concrete and steel reinforcement strength is shown in **Figure 1.1**. The strength of steel reinforcement alone is not good for a building. Even though steel reinforcement has a high strength, if it is combined with a low strength concrete it is not good. The concrete also requires certain strength, conforming to the strength of steel reinforcing bars.

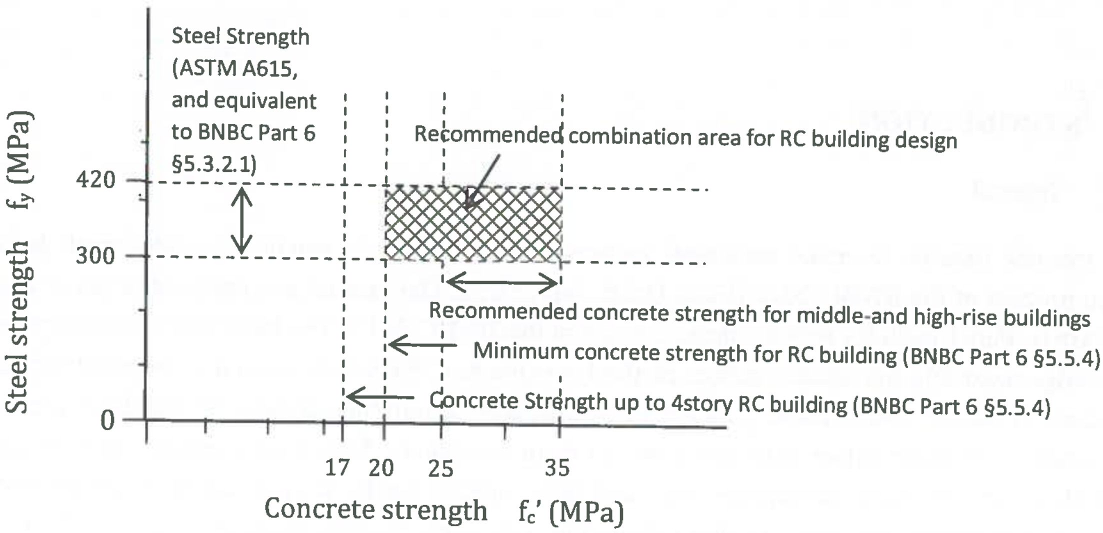


Fig. 1.1 Strength Matrix of Concrete & Steel Reinforcement

A quality management system for the concrete construction is also important for a better building. Although BNBC 2015 has the provisions for detailing of reinforced concrete structures (Part-6) and for construction practices and safety (Part-7), the "Manual for Retrofit Construction and Supervision of Reinforced Concrete Buildings" compiled by CNCRP will be helpful for the engineers to understand the quality control of concrete construction.

1.2 SCOPE

1.2.1 The Manual and BNBC 2015

BNBC 2015 is compiled in ten Parts (Part 1 through Part 10). The thirteen chapters of Part 6 discuss the Structural Design of Buildings. This manual does not address the whole of BNBC 2015 but design method for super-structures of ordinary high-rise reinforced concrete buildings, which is discussed in Part 6, Chapter 6 (Strength Design of Reinforced Concrete Structures) as well as Chapter 5 (Concrete Material), and Chapter 8 (Detailing of Reinforcement in Concrete Structures). The scope of the Manual is mainly seismic load and design for the super-structure of reinforced concrete buildings and not for steel structures, timber structures and pre-stressed concrete structures.

The exceptions are that the design methods for the sub-structure of buildings are referred to Chapter 3 (Part 6) of BNBC 2015. More detailed description for the liquefaction analysis is provided as supplementary material in Appendix A of this manual.

1.2.2 Outline of Seismic Design Method

Outline of seismic design procedure for RC structures is shown in Figure 1.2 and Figure 1.3. Figure 1.2 shows how to obtain the seismic load, modeling and frame analysis. Figure 1.3 shows how to make proportion and detailing of the structural members.

START

Design Data

- Building Location (BNBC 2015 Part 6 §2.5.6.1 ; Sec. 2.2.3*)
- Material Properties ---Concrete & Reinforcement (BNBC 2015 Part 5 Chapter 2)
- Service Loads --- Live loads & Dead Loads (BNBC 2015 Part 6 Chapter 2)
- Seismic Design Data (BNBC 2015 Part 6 Chapter 2)
- Wind Design Data (BNBC 2015 Part 6 Chapter 2)
- Member Dimension --- Slab, Beams, Columns &Walls

Seismic Load Analysis (See Figure. 2.1)

- Selection of SDC (Seismic Design Category) (BNBC 2015 Part 6 §2.5.5.2 ; Sec. 2.2.11*)
- Selection of structural system (BNBC 2015 Part 6 §2.5.5.4 ; Sec. 2.2.12*)
- Determination of Seismic Forces - Seismic base shear (Sec. 2.4* See Fig. 2.1*)

Vertical distribution of seismic forces (BNBC Part 6 §2.5.7.4 ; Sec. 2.3 and See Eq.

$$F_x = V\frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

Method of Analysis

- A three dimensional analysis using computer Program such as STAAD Pro, ETABS and SAP (Sec. 2.2.14*)
- Rigid diaphragms at each floor, rigid-end of horizontal members (Sec. 2.10.1*)
- Stiffness properties of members assumed crack sections (BNBC 2015, Part 6 §6.3.5.2 and §6.3.10.4)

Storey Drift & P -delta Effects

- Storey drift determination (BNBC 2015 Part 6 §2.5.14 ; Sec. 2.6*)
- P -delta effects (BNBC 2015 Part 6 §2.5.7.9 ; Sec. 2.8*)

Soft Storey, Torsional Effects & Overturning Effects

- Soft Storey (BNBC 2015 Part 6 §2.5.19 ; Sec. 2.9.2.1*)
- Torsional Effects (BNBC 2015 Part 6§2.5.9.6 ; Sec. 2.5*)
- Overturning Effects (BNBC 2015 Part 6§2.5.9.8 ; Sec. 2.7*)

Wind Load Analysis

- See BNBC 2015 Part 6 §2.4

Design for Combined Load Effects

- Load Combination (BNBC 2015 Part 6 §2.5.13.1 ; Sec. 2.11*)

Design of Members

Figure 1.2 Procedure to Obtain Loads, Modeling and Frame Analysis

Design of Members

Design of Beams

- Flexural Design (BNBC 2015 Part 6 §6.3.15 ; Sec. 2.12.2.2*)
- Shear Design (BNBC 2015 Part 6 §6.4.1 ; Sec. 2.12.2.4*)
- Reinforcing Bar Cut-off Points
- Flexural Reinforcement Splices (BNBC 2015 Part 6 §8.3.5 ; Sec. 2.12.2.2.*)
- Check embedment or anchorage of beam rebar

Design of Column

- Design for Axial Force and Bending (BNBC 2015 Part 6 §6.3 ; Sec. 2.12.2.6*)
- Relative Flexural Strength of Columns and Beams (BNBC 2015 Part 6 §8.3.5.2 ; Sec. 2.12.2.6*)
- Design of Shear (BNBC 2015 Part 6 §6.4.1 ; Sec. 2.12.2.8*)
- Check strong column-weak beam (BNBC 2015 Part 6 §8.3.5.2 ; Sec. 2.12.2.6*)
- Confinement reinforcement (BNBC 2015 Part 6 §8.3.5.4 ; Sec. 2.12.2.8*)
- Transverse reinforcement for shear (BNBC 2015 Part 6 §8.3.5.4 ; Sec. 2.12.2.8*)
- Splice length of Longitudinal Reinforcement (BNBC 2015 Part 6 §8.3.5.4 ; Sec. 2.12.2.8*)

Design of Beam-Column Joint

- Transverse reinforcement for confinement (BNBC 2015 Part 6 §8.3.7 ; Sec. 2.12.3.2*)
- Shear strength of Joint (BNBC 2015 Part 6 §8.3.7 ; Sec. 2.12.3.2*)

Design of Shear Wall (if needed)

FINISH

Note: The steps shaded are not covered in this Manual.
Sec. numbers with asterisk (*) refers to this Manual.

Figure 1.3 Procedure to Obtain Members' Proportion

1.3 DEFINITION

BUILDINGS: Structures that enclose a space and are used for various occupancies.

BUILDING FRAME SYSTEM: An essentially complete space frame which provides support for loads.

DAMPING: The effect of inherent energy dissipation mechanisms in a structure (due to sliding, friction, etc.) that results in reduction of effect of vibration, expressed as a percentage of the critical damping for the structure.

DEAD LOAD: The load due to the weight of all permanent structural and nonstructural components of a building or a structure, such as walls, floors, roofs and fixed service equipment.

DIAPHRAGM: A horizontal or nearly horizontal system of structures acting to transmit lateral forces to the vertical resisting elements. The term "diaphragm" includes reinforced concrete floor slabs as well as horizontal bracing systems.

DESIGN ACCELERATION RESPONSE SPECTRUM: Smoothened idealized plot of maximum acceleration of a single degree of freedom structure as a function of structure period for design earthquake ground motion.

DESIGN EARTHQUAKE: The earthquake ground motion considered (for normal design) as two-thirds of the corresponding Maximum Considered Earthquake (MCE).

DUCTILITY: Capacity of a structure, or its members to undergo large inelastic deformations without significant loss of strength or stiffness.

EPICENTRE: The point on the surface of earth vertically above the focus (point of origin) of the earthquake.

ESSENTIAL FACILITIES: Buildings and structures which are necessary to remain functional during an emergency or a post disaster period.

FLEXIBLE DIAPHRAGM: A floor or roof diaphragm shall be considered flexible, for purposes of this provision, when the maximum lateral deformation of the diaphragm is more than two times the average story drift of the associated story. This may be determined by comparing the computed midpoint in-plane deflection of the diaphragm under lateral load with the storey drift of adjoining vertical resisting elements under equivalent tributary lateral load.

FLEXIBLE ELEMENT OR SYSTEM: An element or system whose deformation under lateral load is significantly larger than adjoining parts of the system.

IMPORTANCE FACTOR: It is a factor used to increase the design seismic forces for structures of importance.

INTENSITY OF EARTHQUAKE: It is a measure of the amount of ground shaking at a particular site due to an earthquake.

INTERMEDIATE MOMENT FRAME (IMF): A concrete or steel frame designed in accordance with Sec. 8.3.10 or Sec.10.20.10 of BNBC 2015 respectively.

LIQUEFACTION: State in saturated cohesion less soil wherein the effective shear strength is reduced to negligible value due to pore water pressure generated by earthquake vibrations, when the pore water pressure approaches the total confining pressure. In this condition, the soil tends to behave like a liquid.

LIVE LOAD: The load superimposed by the use and occupancy of a building.

MAGNITUDE OF EARTHQUAKE: The magnitude of earthquake is a number, which is a measure of energy released in an earthquake.

MAXIMUM CONSIDERED EARTHQUAKE (MCE): The most severe earthquake ground motion considered by this code.

MODAL MASS: Part of the total seismic mass of the structure that is effective in mode k of vibration.

MODAL PARTICIPATION FACTOR: Amount by which mode k contributes to the overall vibration of the structure under horizontal and vertical earthquake ground motions.

MODAL SHAPE COEFFICIENT: When a system is vibrating in a normal mode, at any particular instant of time, the vibration amplitude of mass i expressed as a ratio of the vibration amplitude of one of the masses of the system, is known as modal shape coefficient

MOMENT FRAME: A frame in which members and joints are capable of resisting lateral forces primarily by flexure. Moment resisting frames are classified as ordinary moment frames (OMF), intermediate moment frames (IMF) and special moment frames (SMF).

NUMBER OF STOREYS (n): Number of story's of a building is the number of levels above the base. This excludes the basement storey's, where basement walls are connected with ground floor deck or fitted between the building columns. But, it includes the basement storeys, when they are not so connected.

ORDINARY MOMENT FRAME (OMF): A moment resisting frame not meeting special detailing requirements for ductile behavior.

P–DELTA EFFECT: It is the secondary effect on shears and moments of frame members due to action of the vertical loads due to the lateral displacement of building resulting from seismic forces.

PERIOD OF BUILDING: Fundamental period (for 1st mode) of vibration of building for lateral motion in direction considered.

PRIMARY FRAMING SYSTEM: That part of the structural system assigned to resist lateral forces.

RESPONSE REDUCTION FACTOR: It is the factor by which the actual base shear force that would develop if the structure behaved truly elastic during earthquake, is reduced to obtain design base shear. This reduction is allowed to account for the beneficial effects of inelastic deformation (resulting in energy dissipation) that can occur in a structure during a major earthquake, still ensuring acceptable response of the structure.

SEISMIC DESIGN CATEGORY: A classification assigned to a structure based on its importance factor and the severity of the design earthquake ground motion at the site.

SEISMIC-FORCE-RESISTING SYSTEM: That part of the structural system that has been considered in the design to provide the required resistance to the seismic forces.

SHEAR WALL: A wall designed to resist lateral forces parallel to the plane of the wall (sometimes referred to as a vertical diaphragm or a structural wall).

SITE CLASS: Site is classified based on soil properties of upper 30 meters.

SLENDER BUILDINGS AND STRUCTURES : Buildings and structures having a height exceeding five times the least horizontal dimension, or having a fundamental natural frequency less than 1 Hz. For those cases where the horizontal dimensions vary with height, the least horizontal dimension at mid height shall be used.

SOFT STOREY: Storey in which the lateral stiffness is less than 70 per cent of the stiffness of the storey above.

SPACE FRAME: A three-dimensional structural system without bearing walls composed of members interconnected so as to function as a complete self contained unit with or without the aid of horizontal diaphragms or floor bracing systems.

SPECIAL MOMENT FRAME (SMF): A moment resisting frame specially detailed to provide ductile behavior complying with the requirements of BNBC 2015 Chapter 8 or 10 for concrete or steel frames respectively.

STOREY: The space between consecutive floor levels. Storey-x is the storey below level-x.

STOREY SHEAR: The total horizontal shear force at a particular storey (level).

STOREY DRIFT: The horizontal deflection at the top of the storey relative to bottom of the storey.

STIFFNESS: Stiffness is the rigidity of an object - the extent to which it resists deformation in response to an applied force. The stiffness, k, of a body is a measure of the resistance offered by an elastic body to deformation.

STRENGTH: The usable capacity of an element or a member to resist the load as prescribed in these provisions.

VERTICAL LOAD-CARRYING FRAME: A space frame designed to carry all vertical gravity loads.

WEAK STOREY: Storey in which the lateral strength is less than 80 per cent of that of the storey

CHAPTER 2. EARTHQUAKE PROVISIONS ACCORDING TO BANGLADESH NATIONAL BUILDING CODE (BNBC 2015)

2.1 EARTHQUAKE RELATED DESIGN & BASIC CONCEPTS

2.1.1 General

The purpose of earthquake resistant design provisions is to allow inelastic deformation & structural damage at preferred locations in the structure without endangering structural integrity & to prevent structural collapse during a major earthquake.

In general most earthquake code provisions implicitly require that structures be able to resist

1. Minor earthquakes without any damage.
2. Moderate earthquakes with negligible structural damage & some nonstructural damage.
3. Major earthquakes with some structural & some nonstructural damage but without collapse. The structure is capable to undergo fairly large deformations by yielding in some members.

Each seismic zone coefficient provides expected peak ground acceleration values on rock/firm soil corresponding to the maximum considered earthquake (MCE). The design basis earthquake is taken as 2/3 of maximum considered earthquake (MCE).

A code-designed structures is thought highly unlikely to collapse under ground motion that is one-and-one-half (1.5) times as strong as the design earthquake ground motion. 2/3 multiplied to maximum considered earthquake (MCE) is reciprocal of 1.5.

The earthquake forces are reduced using the response modification factor, R, because inelastic energy dissipation due to inherent ductility & redundancy in the structure as well as material over-strength is considered.

The elastic deformation calculated under these reduced design forces is multiplied by the deflection amplification factor C_d, to estimate inelastic deformation at ultimate stage. (BNBC 2015, § 2.5.2.1)

2.1.2 Main Contents of Seismic Design Process

Seismic design is a subset of structural design and is the calculation of the response of a building (or non building) structure to earthquakes. A building has the potential to 'wave' back and forth during an earthquake (or even a severe wind storm). This is called the 'fundamental mode', and is the lowest frequency of building response. Most buildings, however, have higher modes of response, which are uniquely activated during earthquakes.

Earthquake engineering has developed a lot since the early days. Structural analysis methods can be divided into the following five categories.

Equivalent static analysis
Response spectrum analysis
Linear dynamic analysis
Nonlinear static analysis
Nonlinear dynamic analysis

The expected earthquake ground motion at the site due to all probable earthquakes may be evaluated in deterministic or probabilistic terms. The ground motion at the site due to an earthquake is a complex phenomenon and depends on several parameters such as earthquake magnitude, focal depth, earthquake source characteristics, distance from earthquake epicentre, wave path characteristics, and geometry of the building as well as local soil conditions at the site. The seismic zoning map divides the country into four seismic zones with different expected levels of intensity of ground motion. Each seismic zone has a zone coefficient which provides expected peak ground acceleration values on rock/firm soil corresponding to the maximum considered earthquake (MCE). The design basis earthquake is taken as 2/3 of the maximum considered earthquake.

The effects of the earthquake ground motion on the structure is expressed in terms of an idealized elastic design acceleration response spectrum, which depends on (a) seismic zone coefficient and local soil conditions defining ground motion and (b) importance factor and response reduction factor representing building considerations.

The Seismic Design is composed of three procedures, first "Base Shear Calculation", second "Horizontal and Vertical Base shear Distribution", and the third is "Frame (Column, Beam, Slab, Wall and Joints) arrangement".

2.2 DESIGN BASE SHEAR

Design base shear is the total design lateral force or shear due to earthquake at the base of a structure. The seismic design base shear force in a given direction shall be determined from the following relation (BNBC 2015 Equation 6.2.37):

$$V = S_a W \tag{2.1}$$

Where,

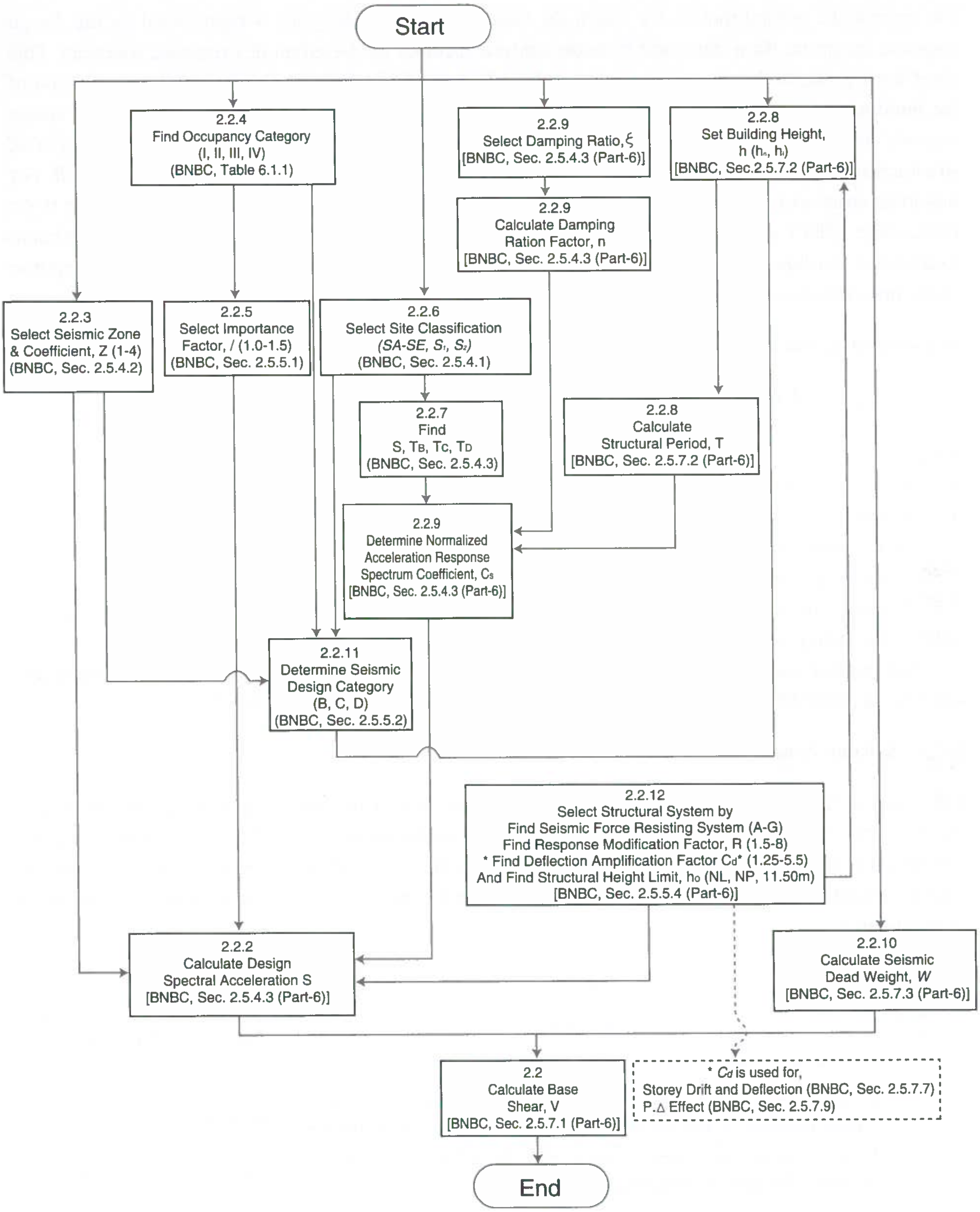
S_a = Lateral seismic force coefficient defined in section 2.2.2 (BNBC 2015, § 2.5.4.3). It is also called design spectral acceleration (in units of g) corresponding to building period T(s) computed as per section 2.2.8 (BNBC 2015, § 2.5.7.2).

W = Total seismic weight of the building defined in section 2.2.10 (BNBC 2015, § 2.5.7.3).

2.2.1 Flowchart for Calculation of Base Shear

When earthquakes occur, a building undergoes dynamic motion. This is because the building is subjected to inertia forces that act in opposite direction to the acceleration of earthquake excitations. These inertia forces, called seismic loads, are usually dealt with by assuming forces external to the building. Since earthquake motions vary with time and inertia forces vary with time and direction, seismic loads are not constant in terms of time and space. In designing buildings, the maximum storey shear force is considered to be the most influential; therefore in this section seismic loads are the static loads to give the maximum storey shear force for each storey, i.e. equivalent static seismic loads.

One of the most basic process of seismic design following BNBC 2015 is to calculate base shear. Figure 2.1 shows the flow chart that illustrates the relation and order of each item and section. The section number is written in each box for easy reference.



Note : (a) BNBC – Bangladesh National Building Code (BNBC 2015)
(b) Section numbers are shown on the top of textbox

Figure 2.1 Flow Chart for Calculation of Base Shear

2.2.2 Design Spectral Acceleration S_a

The earthquake ground motion for which the building has to be designed is represented by the design response spectrum. Both static and dynamic analysis methods are based on this response spectrum. This spectrum represents the spectral acceleration for which the building has to be designed as a function of the building period, taking into account the ground motion intensity. The spectrum is based on elastic analysis but in order to account for energy dissipation due to inelastic deformation and benefits of structural redundancy, the spectral accelerations are reduced by the response modification factor *R*. For important structures, the spectral accelerations are increased by the importance factor *I*. The Design Basis Earthquake (DBE) ground motion is selected at a ground shaking level that is 2/3 of the Maximum Considered Earthquake (MCE) ground motion. The effect of local soil conditions on the response spectrum is incorporated in the normalized acceleration response spectrum C_s.

The spectral acceleration for the design earthquake is given by the following equation:

$$S_a = \frac{2}{3}\frac{Z\,I\,C_S}{R} \geq \frac{2}{3} Z\,I\,\beta \qquad (2.2)$$

Where,

S_a = Design spectral acceleration defined in units of g.

β = coefficient used to calculate lower bound for S_a. Recommended value for β is 0.15.

Z = Seismic zone coefficient defined in section 2.2.3

I = Structure importance factor defined in section 2.2.5

R = Response reduction factor (for more detail see section 5.3.6.3) which depends on the type of structural system given in Table 2.9. The ratio I/R < 1

C_s = Normalized acceleration response spectrum which is a function of structure (building) period and soil type as defined by Equation 2.10 to 2.13 (section 2.2.9) (BNBC 2015, § 2.5.4.3)

2.2.3 Seismic Zone Coefficient, *Z*

The intent of the seismic zoning map is to give an indication of the Maximum Considered Earthquake (MCE) motion at different parts of the country. In probabilistic terms, the MCE motion may be considered to have a probability of exceedance by 2% within a period of 50 years, which corresponds to a return period of 2475 years. The country has been divided into four seismic zones with different levels of ground motion.

Table2.1 Seismic Zones

Seismic Zone	Location	Seismic Intensity	Seismic Zone Coefficient, Z
1	Southwestern part including Barisal, Khulna, Jessore, Rajshahi	Low	0.12
2	Lower Central and Northwestern part including Noakhali, Dhaka, Pabna, Dinajpur, as well as Southwestern corner including Sundarbans	Moderate	0.20
3	Upper Central and Northwestern part including Brahmanbaria, Sirajganj, Rangpur, Chittagong, Cox's Bazar.	Severe	0.28
4	Northeastern part including Sylhet, Mymensingh, Kurigram	Very Severe	0.36

Table2.2 Seismic Zone Coefficient *Z* for Some Important Towns of Bangladesh (BNBC 2015, Table 6.2.15)

Town	*Z*	Town	*Z*	Town	*Z*	Town	*Z*
Bagerhat	0.12	Gaibandha	0.28	Magura	0.12	Patuakhali	0.12
Bandarban	0.28	Gazipur	0.20	Manikganj	0.20	Pirojpur	0.12
Barguna	0.12	Gopalganj	0.12	Maulvibazar	0.36	Rajbari	0.20
Barisal	0.12	Habiganj	0.36	Meherpur	0.12	Rajshahi	0.12
Bhola	0.12	Jaipurhat	0.20	Mongla	0.12	Rangamati	0.28
Bogra	0.28	Jamalpur	0.36	Munshiganj	0.20	Rangpur	0.28
Brahmanbaria	0.28	Jessore	0.12	Mymensingh	0.36	Satkhira	0.12
Chandpur	0.20	Jhalokati	0.12	Narail	0.12	Shariatpur	0.20
Chapainababganj	0.12	Jhenaidah	0.12	Narayanganj	0.20	Sherpur	0.36
Chittagong	0.28	Khagrachari	0.28	Narsingdi	0.28	Sirajganj	0.28
Chuadanga	0.12	Khulna	0.12	Natore	0.20	Srimangal	0.36
Comilla	0.20	Kishoreganj	0.36	Naogaon	0.20	Sunamganj	0.36
Cox's Bazar	0.28	Kurigram	0.36	Netrakona	0.36	Sylhet	0.36
Dhaka	0.20	Kushtia	0.20	Nilphamari	0.12	Tangail	0.28
Dinajpur	0.20	Lakshmipur	0.20	Noakhali	0.20	Thakurgaon	0.20
Faridpur	0.20	Lalmanirhat	0.28	Pabna	0.20		
Feni	0.20	Madaripur	0.20	Panchagarh	0.20		

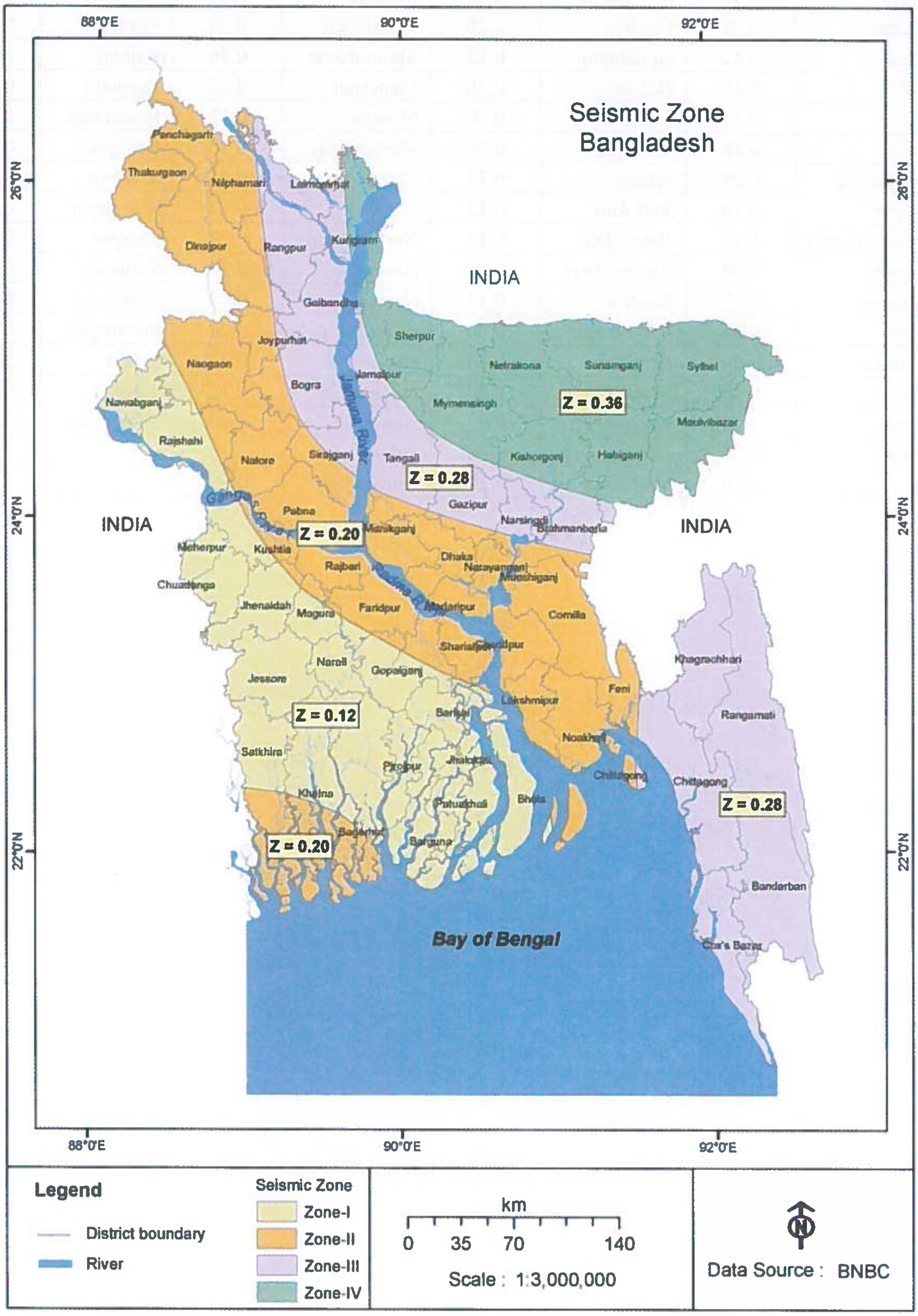


Figure 2.2 Seismic Zoning Map of Bangladesh (BNBC 2015, Figure 6.2.24)

2.2.4 Occupancy Category of Buildings and Other Structures

Buildings and other structures shall be classified, based on the nature of occupancy, according to Table 2.3. The Occupancy Categories range from I to IV where occupancy I represents buildings and structures with low hazard to human life in the event of failure and Occupancy Category IV represents essential facilities. In earthquake analysis Occupancy Category is needed to determine the importance factor of the building.

Table2.3 Occupancy Category of Buildings and other Structures for Flood, Surge, Wind and Earthquake load. (BNBC 2015, Table 6.1.1)

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
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 stations[a] • Water treatment facilities. • Sewage treatment facilities. • Telecommunication centers.	III
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
[a] Cogeneration power plants that do not supply power on the national grid shall be designated Occupancy Category II.	

2.2.5 Structural Importance Factor, *I*

Building are classified in four categories depending on the consequences of collapse for human life, their importance on public life civil protection in the immediate post-earthquake period and on the social and economic consequences of collapse. Depending on occupancy category, buildings may be designed for higher seismic forces using importance factor greater than one. Table 2.4 defines different occupancy categories and corresponding importance factor.

Table 2.4 Importance Factors for Buildings and Structures for Earthquake design (BNBC 2015, Table 6.2.17)

Occupancy Category	Importance factor *I*
I or II	1.0
III	1.25
IV	1.5

2.2.6 Site Classification

Site Classification will be done in accordance with Table 2.5 based on the soil properties of upper 30 meters of the site profile. Average soil properties will be determined by the following equations:

$$\overline{V_s} = \Sigma_{i=1}^{n} d_i / \Sigma_{i=1}^{n} \frac{d_i}{V_{si}} \tag{2.3}$$

$$\overline{N} = \Sigma_{i=1}^{n} d_i / \Sigma_{i=1}^{n} \frac{d_i}{N_i} \tag{2.4}$$

$$\overline{S_u} = \Sigma_{i=1}^{k} d_{ci} / \Sigma_{i=1}^{k} \frac{d_{ci}}{S_{ui}} \tag{2.5}$$

Where,

n = number of soil layers in upper 30 m
d_i= thickness of layer i
V_{si}= shear wave velocity of layer i
N_i= Field (uncorrected) Standard Penetration Test Value for layer i
k = number of cohesive soil layers in upper 30m
d_{ci}= thickness of cohesive layer i
S_{ui}= Undrained shear strength of cohesive layer i

The site profile up to a depth of 30 m is divided into n number of distinct soil or rock layers. Where some of the layers are cohesive, k is the number of cohesive layers. Hence $\sum_{i=1}^{n} d_i = 30$ m, while $\sum_{i=1}^{k} d_{ci} < 30$ m if $k < n$ in other words if there are both cohesionless and cohesive layers. The standard penetration value N as directly measured in the field without correction will be used.

The site classification should be done using average shear wave velocity $\bar{V_s}$ if this can be estimated, otherwise the value of $\bar{N}$ may be used.

Table 2.5 Site Classification Based on Soil Properties (BNBC 2015, Table 6.2.13)

Site Class	Description of soil profile up to 30 meters depth	Average Soil Properties in top 30 meters		
		Shear wave Velocity, $\overline{V_s}$ (m/s)	**Standard Penetration Value, $\overline{N}$ (blows/30cm)**	**Undrained Shear strength, $\overline{S_u}$ (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, characterized 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 SC or SD and thickness varying between about 5m and 20m, underlain by stiffer material with V_s>800m/s.	--	--	--
S1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index(PI> 40) and high water content.	< 100 (indicative)	--	10 - 20
S2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types SA to SE or S_1.	--	--	--

2.2.7 Determination of S, T_B, T_C, T_D

Table 2.6 shows the site dependent soil factor and other Parameters defining Elastic Response Spectrum

Table 2.6 Site Dependent Soil Factor and other Parameters Defining Elastic Response Spectrum (BNBC 2015, Table 6.2.16)

Soil Type	S	T_B (s)	T_C (s)	T_D (s)
SA	1.0	0.15	0.40	2.0
SB	1.2	0.15	0.50	2.0
SC	1.15	0.20	0.60	2.0
SD	1.35	0.20	0.80	2.0
SE	1.4	0.15	0.50	2.0

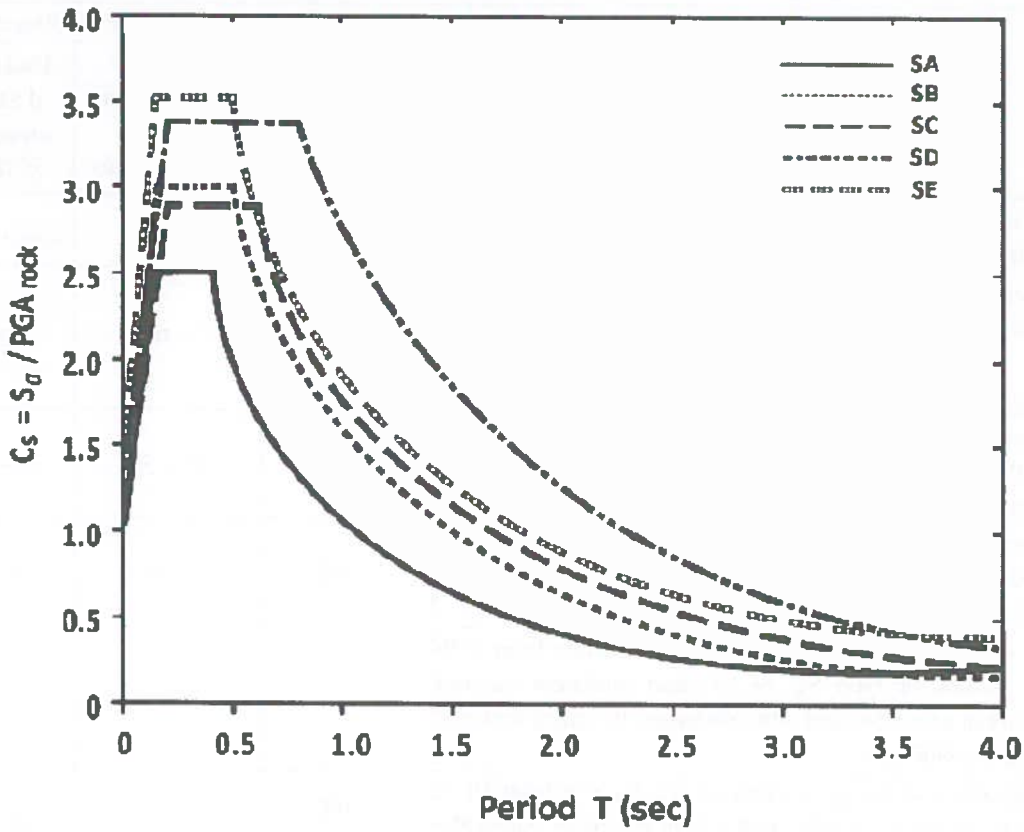


Figure 2.3 Normalized Design Acceleration Response Spectrum for Different Site Classes. (BNBC 2015, Figure 6.2.26)

2.2.8 Building Period, *T*

The building period *T* (sec) may be approximated by the following formula:

$$T = C_t (h_n)^m \tag{2.6}$$

Where,

h_n = Height of building in metres from foundation or from top of rigid basement.

C_t and coefficient *m* are obtained from **Table 2.7.**

Table 2.7 Values for Coefficient to Estimate Approximate Building Period (BNBC 2015, Table 6.2.20)

Structure type	C_t (for SI)	*m*	C_t (for Fps)
Concrete moment-resisting frames	0.0466	0.9	(0.016)
Steel moment-resisting frames	0.0724	0.8	(0.0280)
Eccentrically braced steel frame	0.0731	0.75	(0.03)
All other structural systems	0.0488	0.75	(0.02)
Note: Consider moment resisting frames as frames which resist 100% of seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting under seismic forces			

For masonry or concrete shear wall structures, the approximate fundamental period, T (sec) may be determined as follows:

$$T = \frac{0.0062h_n}{\sqrt{C_w}} \qquad T = \frac{0.0019h_n}{\sqrt{C_w}} \qquad (2.7)$$

When h_n is in metre (m) (When h_n is in ft)

$$C_w = \frac{100}{A_B}\sum_{i=1}^{x}\frac{\left(h_n/h_i\right)^2 A_i}{\left[1+0.83\left(h_i/D_i\right)^2\right]} \qquad (2.8)$$

Where,

A_B= area of base of the structure, m^2 (or ft^2)

A_i= web area of shear wall "i", m^2 (or ft^2)

D_i= length of shear wall "i", m (or ft)

h_i= height of shear wall "i", m (or ft)

x = number of shear walls in the building effective in resisting lateral forces in the direction under consideration.

Structural dynamics procedures (such as Rayleigh method), using structural properties and deformation characteristics of resisting elements, may be used to determine the fundamental period T of the building in the direction under consideration. This period shall not exceed the approximate fundamental period determined by equation 2.7 by more than 40%.

According to Rayleigh method (See Sec.5.3.1.3.3)

$$T = 2\pi\sqrt{\sum_{i=1}^{n} w_i\delta_i^2 \Big/ g\sum_{i=1}^{n} f_i\delta_i} \qquad (2.9)$$

Where,

f_i = lateral force applied at levels i=1 to n

δ_i = corresponding lateral displacements

W_i =corresponding floor weight

(BNBC 2015, § 2.5.7.2)

2.2.9 Determination of C_s

C_s is Normalized acceleration response spectrum, which is a function of structure (building) period and soil type (site class) (Figure 2.4). In other word this is the percentage of seismic weight which will generate earthquake induced force. It can be calculated by following equations:

$$C_s = S\left(1+\frac{T}{T_B}(2.5\eta-1)\right) \quad \text{for } 0 \le T \le T_B \qquad (2.10)$$

$$C_s = 2.5S\eta \quad \text{for } T_B \le T \le T_C \qquad (2.11)$$

$$C_s = 2.5S\eta\left(\frac{T_C}{T}\right) \quad \text{for } T_C \le T \le T_D \qquad (2.12)$$

$$C_s = 2.5S\eta\left(\frac{T_C T_D}{T^2}\right) \quad \text{for } T_D \le T \le 4\,\text{sec} \qquad (2.13)$$

Where,

S = Soil factor given in Table 2.6

T = Structure (building) period defined in section 2.2.8

T_B= Lower limit of the period of the constant spectral acceleration branch given in Table 2.6

T_C= Upper limit of the period of the constant spectral acceleration branch given in Table 2.6

T_D= Lower limit of the period of the constant spectral acceleration branch given in Table 2.6

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

$$\eta = \sqrt{\frac{10}{(5+\xi)}} \geq 0.55 \tag{2.14}$$

Where, ξ is the viscous damping ratio of the structure expressed in percentage of critical damping. (BNBC 2015 Part 6, § 2.5.4.3)

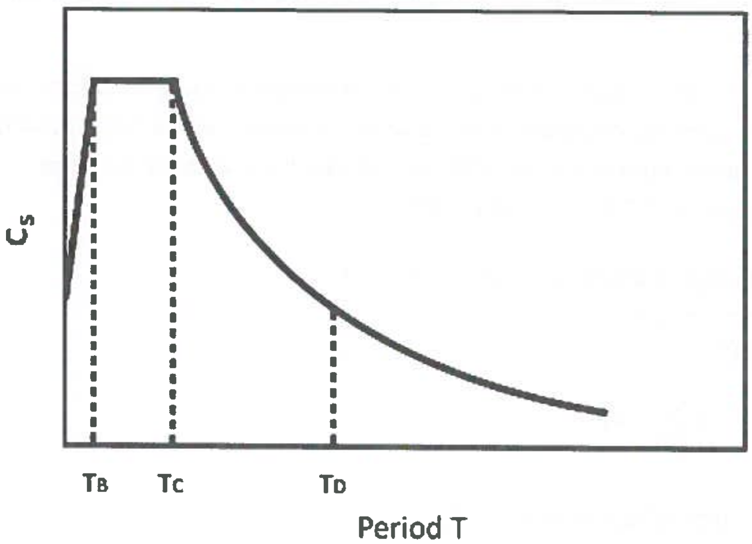


Figure 2.4 Typical shape of the Elastic Response Spectrum Coefficient, C_s

(BNBC 2015 Part 6, Figure 6.2.25)

2.2.10 Seismic Weight, *W* (See Sec.5.3.6.7)

Seismic weight, W, is the total dead load of a building or a structure, including partition walls and applicable portions of other imposed loads listed below:

a) For live load ≤ 3kN/m², a minimum of 25% of the live load shall be applicable.
b) For live load > 3kN/m², a minimum of 50% of the live load shall be applicable.
c) Total weight (100%) of permanent heavy equipment or retained liquid or any imposed load sustained in nature shall be included.

Where the probable imposed loads (mass) at the time of earthquake are more correctly assessed, the designer may go for higher percentage of live load. (BNBC 2015 Part 6, § 2.5.7.3)

2.2.11 Seismic Design Category

Design requirements for an earthquake-resistant structure are determined by the seismic design category to which the structure is assigned. Seismic design category relates to seismic hazard level, soil type, occupancy & use of the building. Building has to be assigned a seismic design category based on;

- Seismic zone
- Local site condition
- Importance class

Seismic design category D has the most stringent seismic design detailing, while seismic design category B has the least seismic design detailing requirements.

Table 2.8 Seismic Design Category of Buildings (BNBC 2015, Table 6.2.18)

Site Class	Occupancy Category I, II and III				Occupancy Category IV			
	Seismic Zone 1	Seismic Zone 2	Seismic Zone 3	Seismic Zone 4	Seismic Zone 1	Seismic Zone 2	Seismic Zone 3	Seismic Zone 4
SA	B	C	C	D	C	D	D	D
SB	B	C	D	D	C	D	D	D
SC	B	C	D	D	C	D	D	D
SD	C	D	D	D	D	D	D	D
SE, S1, S2	D	D	D	D	D	D	D	D

(BNBC 2015 Part 6, § 2.5.5.2)

2.2.12 Selection of Structural Systems

The basic lateral & vertical seismic-force-resisting system shall conform to one of the types indicated in Table 2.9. Each type is subdivided by the types of vertical elements used to resist lateral seismic forces. The structural system used shall be in accordance with the seismic design category & height limitations. The response modification factor, R (see section 5.3.6.3 for more detail) & the deflection amplification factor, C_d, (see section 5.3.6.4 for more detail) indicated in Table 2.9 shall be used in determining the base shear & design storey drift. The selected structural system shall be designed and detailed in accordance with specific requirements for the system given in section 2.12.1 to section 2.12.6.

2.2.12.1 Combinations of Framing Systems in Different Directions

Different seismic force-resisting systems are permitted to be used to resist seismic forces along each of the two orthogonal axes of the structure. Where different systems are used, the respective R and C_d coefficient shall apply to each system, including the limitations on system.

2.2.12.2 Combinations of Structural Systems in the Same Direction

Where different seismic force–resisting systems are used in combination to resist seismic forces in the same direction of structural response, other than those combinations considered as dual systems, the more stringent system limitation contained in Table 2.9 shall apply. The value of R used for design in that direction shall not be greater than the least value of R for any of the systems utilized in that direction. The deflection amplification factor, C_d in the direction under consideration at any storey shall not be less than the largest value of this factor for the R factor used in the same direction being considered.

(BNBC 2015, § 2.5.5.4)

Table 2.9 Response Reduction Factor, Deflection Amplification Factor and Height Limitations for Different Structural Systems

Seismic Force–Resisting System	Response Reduction Factor, R	System Overstrength Factor, Ω_o	Deflection Amplification Factor, C_d	Seismic Design Category B	Seismic Design Category C	Seismic Design Category D
				Height limit (m)		
A. BEARING WALL SYSTEMS (no frame)						
1. Special reinforced concrete shear walls	5	2.5	5	NL	NL	50
2. Ordinary reinforced concrete shear walls	4	2.5	4	NL	NL	NP
3. Ordinary reinforced masonry shear walls	2	2.5	1.75	NL	50	NP
4. Ordinary plain masonry shear walls	1.5	2.5	1.25	18	NP	NP
B. BUILDING FRAME SYSTEMS (with bracing or shear wall)						
1. Steel eccentrically braced frames, moment resisting connections at columns away from links	8	2	4	NL	NL	50
2.Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links	7	2	4	NL	NL	50
3. Special steel concentrically braced frames	6	2	5	NL	NL	50
4. Ordinary steel concentrically braced frames	3.25	2	3.25	NL	NL	11
5. Special reinforced concrete shear walls	6	2.5	5	NL	NL	50
6. Ordinary reinforced concrete shear walls	5	2.5	4.25	NL	NL	NP
7. Ordinary reinforced masonry shear walls	2	2.5	2	NL	50	NP
8. Ordinary plain masonry shear walls	1.5	2.5	1.25	18	NP	NP
C. MOMENT RESISTING FRAME SYSTEMS (no shear wall)						
1. Special steel moment frames	8	3	5.5	NL	NL	NL
2. Intermediate steel moment frames	4.5	3	4	NL	NL	35
3. Ordinary steel moment frames	3.5	3	3	NL	NL	NP
4. Special reinforced concrete moment frames	8	3	5.5	NL	NL	NL
5. Intermediate reinforced concrete moment frames	5	3	4.5	NL	NL	NP
6. Ordinary reinforced concrete moment frames	3	3	2.5	NL	NP	NP

Seismic Force–Resisting System	Response Reduction Factor, R	System Overstrength Factor, Ω_o	Deflection Amplification Factor, C_d	Seismic Design Category B	Seismic Design Category C	Seismic Design Category D
				Height limit (m)		
D. DUAL SYSTEMS: SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall)						
1. Steel eccentrically braced frames	8	2.5	4	NL	NL	NL
2. Special steel concentrically braced frames	7	2.5	5.5	NL	NL	NL
3. Special reinforced concrete shear walls	7	2.5	5.5	NL	NL	NL
4. Ordinary reinforced concrete shear walls	6	2.5	5	NL	NL	NP
E. DUAL SYSTEMS: INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall)						
1. Special steel concentrically braced frames	6	2.5	5	NL	NL	11
2. Special reinforced concrete shear walls	6.5	2.5	5	NL	NL	50
3. Ordinary reinforced masonry shear walls	3	3	3	NL	50	NP
4. Ordinary reinforced concrete shear walls	5.5	2.5	4.5	NL	NL	NP
F. DUAL SHEAR WALL-FRAME SYSTEM: ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS	4.5	2.5	4	NL	NP	NP
G. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE	3	3	3	NL	NL	NP

Notes:

1. Seismic design category, NL = No height restriction, NP = Not permitted. Number represents maximum allowable height (m).
2. Dual Systems include buildings which consist of both moment resisting frame and shear walls (or braced frame) where both systems resist the total design forces in proportion to their lateral stiffness.
3. See BNBC 2015 Sec. 10.20 of Chapter 10 of this Part for additional values of R and $C_d$and height limits for some other types of steel structures not covered in this Table.
4. Where data specific to a structure type is not available in this Table, reference may be made to Table 12.2-1 of ASCE 7-05.

2.2.13 Ground Motion Factor as per ASCE 7-05

For seismic analysis of a structure by commercial software it is easier or handy to generate the earthquake force following one of the Model Building Code. There is no option to choose BNBC 2015 in software like STAAD-Pro, ETABS, SAP etc. Previously UBC94 had been selected for earthquake load generation since it had close similarity with BNBC 1993. Though BNBC 2015 is not directly similar to none of foreign Model Building Code, it is possible to generate earthquake load following IBC-06 or ASCE 7-05. To do that one has to understand ground motion factors of ASCE 7-05.

S_s and S_1 are the two parameters that play a key role in the determination of ground motion values used in seismic design. S_s is the mapped value of the 5% damped Maximum Considered Earthquake (MCE) spectral response acceleration, for short period (0.2 sec) structures founded on firm rock. MCE is the most severe earthquake considered in ASCE 7-05, standard. Similarly S_1 is the mapped value of the 5% damped MCE spectral response acceleration at a period of 1 sec on firm (soil type SA in case of BNBC 2015). Consequently, these two response acceleration parameters, S_s and S_1 are sufficient to define an entire response spectrum for the period range of importance for most buildings.

To obtain acceleration response parameters that are appropriate for sites with characteristics, other than those for firm rock, it is necessary to modify the S_s and S_1 values. This modification is performed with the use of two co-efficient, F_a and F_v. The MCE spectral response accelerations adjusted for site class effects are designated S_{MS} and S_{M1}, respectively, for short-period and 1sec – period responses.

Thus $S_{MS} = F_a \,.\, S_s$

And $S_{M1} = F_y \,.\, S_1$

For design purpose MCE spectral acceleration parameters, S_{MS} and S_{M1} are reduced to two-thirds to determine design spectral acceleration parameters, S_{DS} and S_{D1}.

Thus $S_{DS} = 2/3 S_{MS}$

and $S_{D1} = 2/3 S_{M1}$

In BNBC 2015, spectral acceleration (S_a) at design level is found directly based on zone factors at different zones, no need to determine S_s, S_1, F_a and F_v. But for earthquake load generation by commercial software one has to find out these parameters. Following tables are given as design aid to determine above parameters.

Table2.9 Site Coefficient S_S and S_1 for different seismic zone

	Zone-1	Zone-2	Zone-3	Zone-4
S_S	0.3	0.5	0.7	0.9
S_1	0.12	0.2	0.28	0.36

Table2.10 Site Coefficient F_a for different seismic zone and soil type

Soil Type	Zone-1	Zone-2	Zone-3	Zone-4
SA	1.0	1.0	1.0	1.0
SB	1.2	1.2	1.2	1.2
SC	1.15	1.15	1.15	1.15
SD	1.35	1.35	1.35	1.35
SE	1.2	1.2	1.2	1.2

Table2.11 Site Coefficient F_y for different seismic zone and soil type

Soil Type	Zone-1	Zone-2	Zone-3	Zone-4
SA	1.0	1.0	1.0	1.0
SB	1.5	1.5	1.5	1.5
SC	1.725	1.725	1.725	1.725
SD	2.7	2.7	2.7	2.7
SE	1.75	1.75	1.75	1.75

Table2.12 Site Coefficient S_{DS} for different seismic zone and soil type

Soil Type	Zone-1	Zone-2	Zone-3	Zone-4
SA	0.2	0.333	0.466	0.6
SB	0.24	0.4	0.56	0.72
SC	0.23	0.383	0.536	0.69
SD	0.27	0.45	0.63	0.81
SE	0.28	0.466	0.653	0.84

Table2.13 Site Coefficient S_{D1} for different seismic zone and soil type

Soil Type	Zone-1	Zone-2	Zone-3	Zone-4
SA	0.08	0.133	0.186	0.24
SB	0.12	0.2	0.28	0.36
SC	0.138	0.23	0.322	0.414
SD	0.216	0.36	0.504	0.648
SE	0.14	0.233	0.326	0.42

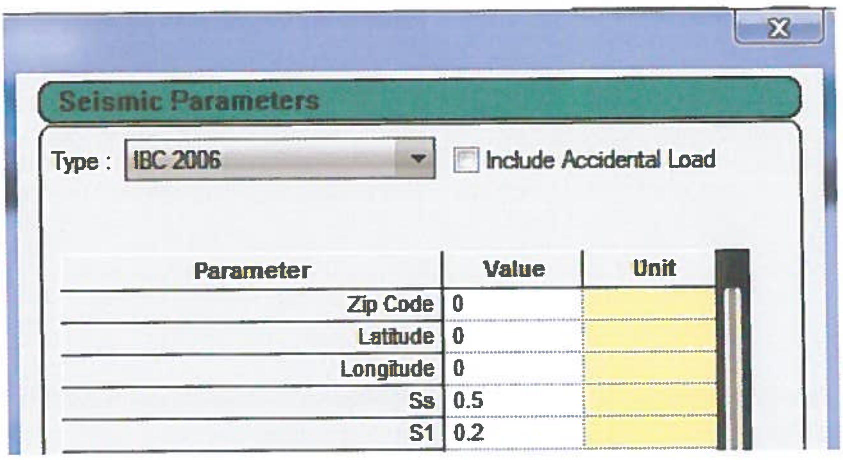
It is important to note that base shear thus obtained from software shall be checked manually because minimum base shear defined in BNBC is higher than that of ASCE 7-05. It usually governs in high rise building. The main advantage of this generation is automatic distribution of base shear at different joints of various floor level.

2.2.14 Analysis by Commercial Software

During analysis and design of a structure by commercial software, it is reasonable to generate the earthquake load by the software and optimize the structural member accordingly. BNBC 2015 is not included in any structural software like ETABS or STAAD Pro for earthquake load generation. But it is possible to generate earthquake load prescribed by BNBC 2015 following input parameter of IBC 2006.

Procedure for determining the input values of the IBC 2006 parameters are as follows:
The values of S_s and S_1 can be chosen manually as per seismic zone, which are given in Table 2.10. If the values of S_s and S_1 are provided, then no need to provide zip code or latitude and longitude of the location. For example, if the building is located in zone-2, then $S_s = 0.5$ and $S_1 = 0.2$.

Table 2.15 Seismic Parameters (Ss & S1) in STAAD Pro V8i



- T_L = Long-Period transition period in seconds. This is the value usually required for super tall buildings. There is no guideline in BNBC 2015 about T_L. The suggested value of T_L according to IBC/ASCE 7-05 is 12 second.
- From Table 2.3 (BNBC 2015 Table 6.1.1), occupancy category is determined. Next, from Table 2.4 (BNBC 2015 Table 6.2.17) Importance factor, I is obtained.
- Response Modification factor, R is determined from Table 2.9 (BNBC 2015 Tab. 6.2.19). It depends on type of resisting structures. R values may be different in both principal directions for same structure. For example if the structure is Special Moment Resisting Frame and shear wall is provided only in X-direction but not in Z-direction, then RX is 7 & RZ is 8.
- Site Classification is given in Table 2.5. But it should be provided carefully because IBC 2006 (or ASCE 7-05) defined site class is different from that of BNBC 2015. If Ss, S_1, F_a, F_v (discussed below) are provided, then it is not mandatory to provide appropriate site class. These values will automatically determine site class.
- The values of F_a and F_v are given in Table 2.11 and Table 2.12. These values are determined based on soil type and seismic zone.
- Determine C_t and x. C_t is building period coefficient and x is the exponent used in the calculation of approximate time period by Equation 2.6 (BNBC 2015 Equation 6.2.38). These two parameters depend on the type of structure and are explained in Table 2.7 (BNBC 2015 Table 6.2.20). Before providing C_t value, input unit has to be checked. For example, if the structure is Concrete moment-resisting frames then $C_t = 0.016$ in FPS unit but $C_t = 0.0466$ in SI unit.
- Period (T) in both principal directions can be determined from Equation 2.6 (BNBC 2015 Equation 6.2.38). Users can define period in their own judgment. If C_t and x values are provided, software will automatically calculate T. But user defined T will be used for analysis if T is defined by the user.

Users are encouraged to check software generated design base shear with manual calculation.

2.3 VERTICAL DISTRIBUTION OF SEISMIC FORCES

The lateral seismic forces (F_x) induced at any floor level shall be determined from the following equations:

$$F_x = V \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{2.15}$$

Where,

F_x = part of base shear force induced at level x, kN (or kip)

w_i and w_x= part of the total effective seismic weight of the structure (W) assigned to level i or x (kN)(or kip)

h_i *and* h_x= the height from the base to level i or x, m (or ft)

k = 1 for structure period $\leq$ 0.5s

= 2 for structure period $\geq$ 2.5s

= linear interpolation between 1 and 2 for other periods.

n = number of stories

(BNBC 2015, § 2.5.16.2.3)

2.4 STOREY SHEAR

The design storey shear V_x, at any storey x is the sum of the forces F_x in that storey and all other stories above it, given by following equation:

$$V_x = \sum_{i=1}^{n} F_i \tag{2.16}$$

Where,

F_i= Portion of base shear induced at level i

The seismic design storey shear (V_x) shall be distributed to the various elements of the lateral force resisting system in the storey under consideration based on the relative lateral stiffness of the vertical resisting elements & the diaphragm.

(BNBC 2015, § 2.5.7.5)

2.5 HORIZONTAL TORSIONAL MOMENT

Design shall accommodate increase in storey shear forces resulting from probable horizontal torsional moments on rigid floor diaphragms. Computation of such moments shall be as follows (BNBC 2015, § 2.5.7.6):

2.5.1 In Built Torsional Effects

When there is in-built eccentricity between centre of mass and centre of rigidity (lateral resistance) at the floor levels, rigid diaphragms at each level will be subjected to torsional moment M_t.

2.5.2 Accidental Torsional Effects

Accidental Torsion that occur due to uncertainties in the building's mass and stiffness distribution must be added to the calculated eccentricity This is done by adding a torsional moment at each floor equal to the storey force multiplied by 5% of the floor dimension, perpendicular to the direction of the force. This procedure is equivalent to moving the center of mass by 5% of the plan dimension, in a direction perpendicular to the force. If the lateral deflection at either end of a building is more than 20% greater than the average deflection, the building is classified as torsional irregular. The accidental torsional moment M_{tai} at level i is given as:

$$M_{tai} = e_{ai} F_i \tag{2.17}$$

Where,

e_{ai}= accidental eccentricity of floor mass at level i in the same direction at all floors = $\pm 0.05 L_i$

L_i= floor dimension perpendicular to the direction of seismic force considered.

Where torsional irregularity exists for Seismic Design Category C or D,(*see section 2.2.11*) the accidental torsion M_{ta} at each level must be amplified by a torsional amplification factor, A_x as determined from the following equation:

$$A_x = \left[\frac{\delta_{max}}{1.2\delta_{avg}}\right]^2 \leq 3.0 \tag{2.18}$$

This torsional amplification factor (A_x) should not exceed 3.0.

And δ_{max}= Maximum displacement at level-x computed assuming A_x=1.

δ_{avg}= Average of the displacements at extreme points of the building at level-x computed assuming A_x= 1.

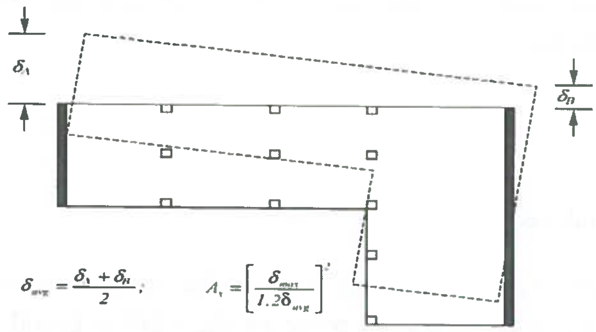


Figure 2.5 TorsionalAmplification Factor A_x for Plan Irregularity.(BNBC 2015, Figure 6.2.29)

2.5.3 Design for Torsional Effects

The torsional design moment at a given story shall be equal to the accidental torsional moment M_{ta} plus the inbuilt torsional moment M_t (if any). Where earthquake forces are applied concurrently in two orthogonal directions, the required 5 percent displacement of the center of mass (for accidental torsion) need not be applied in both of the orthogonal directions at the same time, but shall be applied in only one direction that produces the greater effect

(BNBC 2015, § 2.5.7.6.3)

2.6 STOREY DRIFT AND DEFLECTION

The design storey drift (Δ) (for more detail see section 5.3.4) at storey x shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the storey under consideration:

$$\Delta_x = \delta_x - \delta_{x-1} \tag{2.19}$$

The deflections (δ_x) of level x at the center of the mass shall be determined in accordance with the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \tag{2.20}$$

Where,
C_d= Deflection amplification factor given in Table 2.9
δ_{xe}= Deflection determined by an elastic analysis
I = Importance factor given in Tab. 2.4.
(BNBC 2015, § 2.5.7.7)

2.6.1 Storey Drift Limit

The design story drift (Δ) of each story shall not exceed the allowable story drift (Δ_a) as obtained from Table 2.10. For structures with significant torsional deflections, the maximum drift shall include torsional effects. For structures assigned to Seismic Design Category C or D having torsional irregularity, the design story drift, shall be computed as the largest difference of the deflections along any of the edges of the structure at the top and bottom of the story under consideration. For seismic force–resisting systems comprised solely of moment frames in Seismic Design Categories D, the allowable storey drift for such linear elastic analysis procedures shall not exceed Δ_a/ρ where ρ is termed as a structural redundancy factor (for more detail see section 5.3.6.5). The value of redundancy factor ρ may be considered as 1.0 with the exception of structures with very low level of redundancy where ρ may be considered as 1.3.

Table 2.16 Allowable Storey Drift Limit (Δ_a) [BNBC 2015, Table 6.2.21]

Structure	Occupancy Category		
	I and II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the storey drifts.	$0.025h_{sx}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

1. h_{sx} is the storey height below Level x.
2. There shall be no drift limit for single-storey structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the storey drifts.
3. Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.
4. Occupancy categories are given in Table 2.3

2.6.2 Separation Between Adjacent Structures

BNBC 2015 requires buildings to be protected from earthquake induced pounding from adjacent structures or between structurally independent units of the same building maintaining safe distance between such structures as follows:

(i) for buildings, or structurally independent units, that do not belong to the same property, the distance from the property line to the potential points of impact shall not be less than the computed maximum horizontal displacement (BNBC 2015 § 2.5.7.7) of the building at the corresponding level.

(ii) for buildings, or structurally independent units, that do not belong to the same property, if the distance between them is not less than the square root of the sum of the squares (SRSS) of the computed maximum horizontal displacement (BNBC 2015 § 2.5.7.7) of the two buildings or units at the corresponding level.

(iii) if the floor elevation of the building or independent unit under design are the same as those of the adjscent building or unit, the above referred minimum distance may be reduced by a factor of 0.7.

2.7 OVERTURNING EFFECTS

The structure shall be designed to resist overturning effects caused by the seismic forces determined in section 2.3. At any storey, the increment of overturning moment in the storey under consideration shall be distributed to the various vertical force resisting elements in the same proportion as the distribution of the horizontal shears to those elements. The overturning moments M_x, at level x shall be determined as follows:

$$M_x = \sum_{i=x}^{n} F_i (h_i - h_x) \tag{2.21}$$

Where,

F_i= the portion of the seismic base shear, V, induced at level i

h_i, h_x= the height from the base to level $i$or x.

The foundations of structures, except inverted pendulum-type structures, shall be permitted to be designed for three-fourths of the foundation overturning design moment, M_o, determined using above equation.

(BNBC 2015, § 2.5.7.8)

When the overturning-to-resisting moment ratio is less than 1, the foundation must provide resistance to uplift. One viable option is to tie adjacent footing together with a grade beam. When computing the resisting moments, the dead load will be multiplied by the factor (0.9-0.2 S_{DS}) in accordance with ASCE Equation 12.4-2, which applies when the effects of gravity and seismic ground motion counteract (ASCE 12.4.2).

2.8 P-Δ EFFECT

The P-delta effects (for more detail see Sec. 5.3.5) on story shears and moments, the resulting member forces and moments, and the storey drifts induced by these effects are not required to be considered if the stability coefficient (θ) determined by the following equation is not more than 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \tag{2.22}$$

Where,

P_x= the total vertical design load at and above level x(kN)(or kip); where computing P_x, no individual load factor need exceed 1.0

Δ = the design story drift occurring simultaneously with V_x(mm)(or in)

V_x= the storey shear force acting between levels x and $x-1$(kN)(or kip)

h_{sx}= the storey height below level x(mm)(or in)

C_d= the deflection amplification factor given in Table 2.9

The stability coefficient (θ) shall not exceed θ_{max} determined as follows:

$$\theta_{\max} = \frac{0.5}{\beta C_d} \le 0.25 \tag{2.23}$$

Where,
β = ratio of shear demand to shear capacity for the storey between levels x and x -1. Conservatively β = 1.0.
If $0.10 < \theta \leq \theta_{max}$, the incremental factor related to P-delta effects on displacement and member forces shall be increased by rational analysis or multiplied by a factor $1.0/(1-\theta)$.
(BNBC 2015, § 2.5.7.9)
But usually, in the case of conventional RC buildings P-delta effects need not be considered at any of the floor levels, because θ-values is considerably smaller than 0.1. (See again Sec. 5.3.5)

2.9 BUILDING IRREGULARITY

Buildings with irregularity in plan or elevation suffer much more damage in earthquakes than buildings with regular configuration. A building may be considered as irregular, if at least one of the conditions given below is applicable: (BNBC 2015, § 2.5.5.3)

2.9.1 Plan Irregularity

2.9.1.1 Torsion Irregularity

To be considered for rigid floor diaphragms, when the maximum storey drift (Δ_{max}) including accidental torsion, at one end of the structure is more than 1.2 times the average ($\Delta_{avg} = (\Delta_{max}+\Delta_{min})/2$) of the storey drifts at the two ends of the structure. If $\Delta_{max} > 1.4\Delta_{avg}$ then the irregularity is termed as extreme torsional irregularity. (BNBC 2015, § 2.5.5.3.1 (i))

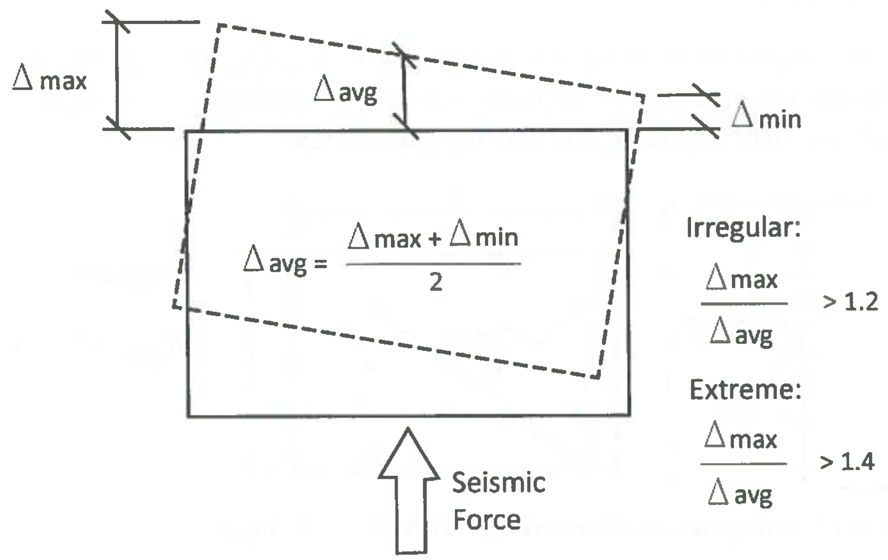


Figure 2.6 Torsional Irregularity [BNBC 2015, Figure 6.2.27(a)]

2.9.1.2 Re-entrant Corners

Both projections of the structure beyond a re-entrant are greater than 15 percent of its plan dimension in the given direction.

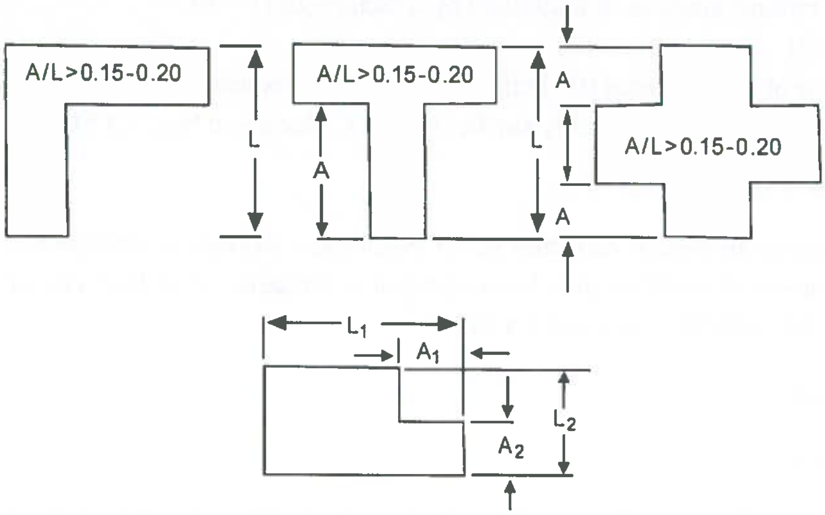


Figure 2.7 Re-entrant corners (A/L>0.15) [BNBC 2015, Figure 6.2.27(b)]

2.9.1.3 Diaphragm Discontinuity

Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 percent of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50 percent from one storey to the next.

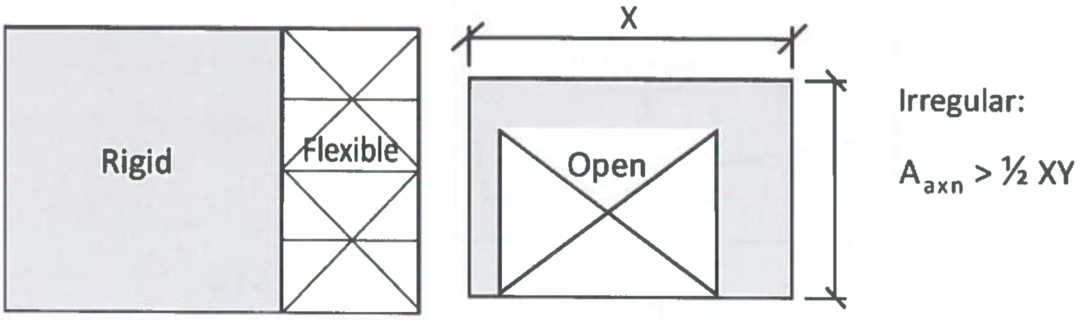


Figure 2.8 Diaphragm Discontinuity (BNBC 2015, Figure 6.2.27)

For structures assigned to Seismic Design Category D and having Diaphragm Discontinuity Irregularity design forces determined from Section of design base shear shall be increased 25 percent. [BNBC 2015, § 2.5.5.3.1 (iii)]

2.9.1.4 Out-of-Plane Offsets

Discontinuities in a lateral force resistance path, such as out of-plane offsets of vertical elements.

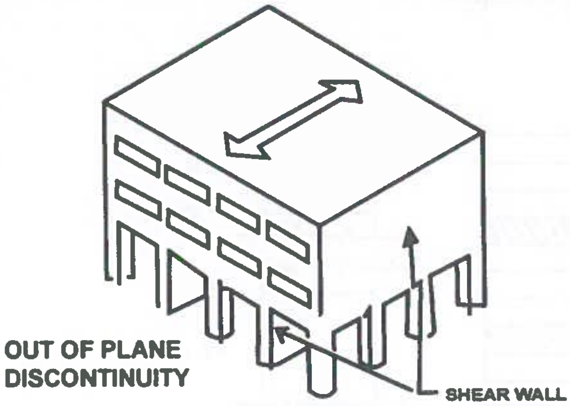


Figure 2.9 Out-Of-Plane Offsets of Shear Wall [BNBC 2015, Figure 6.2.27(d)]

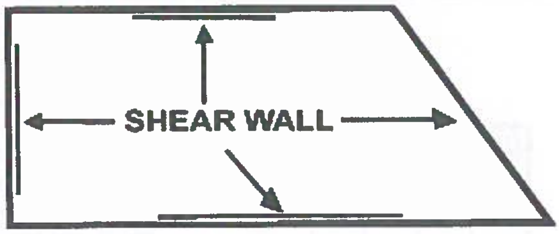


Figure 2.10 Non-parallel Systems of Shear Wall [BNBC 2015, Figure 6.2.27(e)]

For structures assigned to seismic design category D and having out-of-plane offsets irregularity design forces determined from section of design base shear shall be increased 25 percent. [BNBC 2015, § 2.5.5.3.1 (iv)]

2.9.1.5 Non-parallel Systems

The vertical elements resisting the lateral force are not parallel to or symmetric about the major orthogonal axes of the lateral force resisting elements. [BNBC 2015, § 2.5.5.3.1 (v)]

2.9.2 Vertical Irregularity

2.9.2.1 Stiffness Irregularity-Soft Storey

A soft storey is one in which the lateral stiffness is less than 70% of that in the storey above or less than 80% of the average lateral stiffness of the three storey above irregularity. An extreme soft storey is defined where its lateral stiffness is less than 60% of that in the storey above or less than 70% of the average lateral stiffness of the three storey above. [BNBC 2015, § 2.5.5.3.2 (i)]

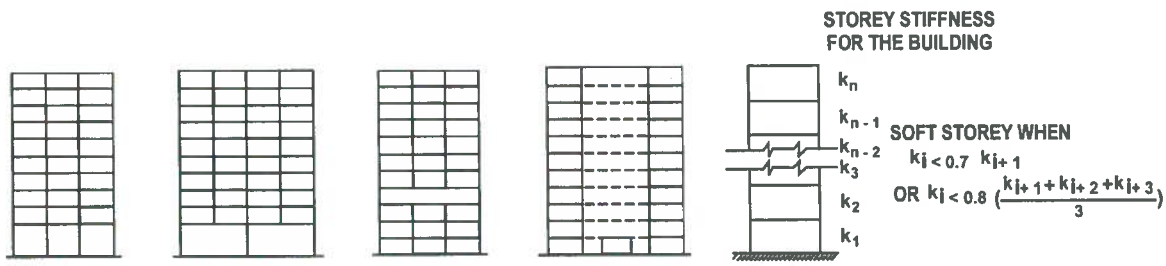


Figure 2.11 Soft Storey [BNBC 2015, Figure 6.2.28(a)]

2.9.2.2 Mass Irregularity

The seismic weight of any story is more than twice of that of its adjacent stories. This irregularity need not be considered in case of roofs. [BNBC 2015, § 2.5.5.3.2 (ii)]

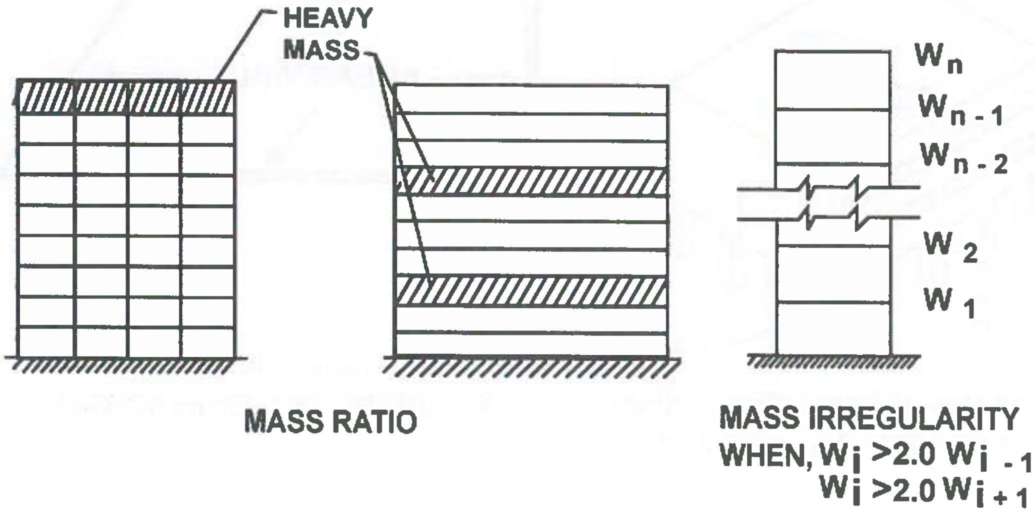


Figure 2.12 Mass Irregularity [BNBC 2015, Figure 6.2.28(b)]

2.9.2.3 Vertical Geometric Irregularity

This irregularity exists for buildings with setbacks with dimensions given in Figure below. [BNBC 2015, § 2.5.5.3.2 (iii)]

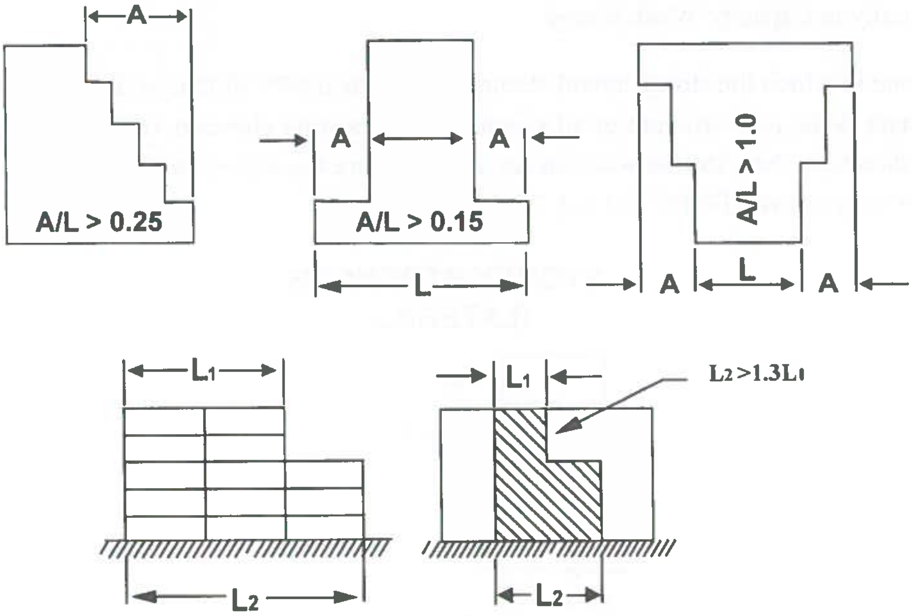


Figure 2.13 Vertical Geometric Irregularity (Setback Structures) [BNBC 2015, Figure 6.2.28(c)]

2.9.2.4 Vertical In-Plane Discontinuity in Vertical Elements Resisting Lateral Force

An in-plane offset of the lateral force resisting elements greater than the length of those elements.

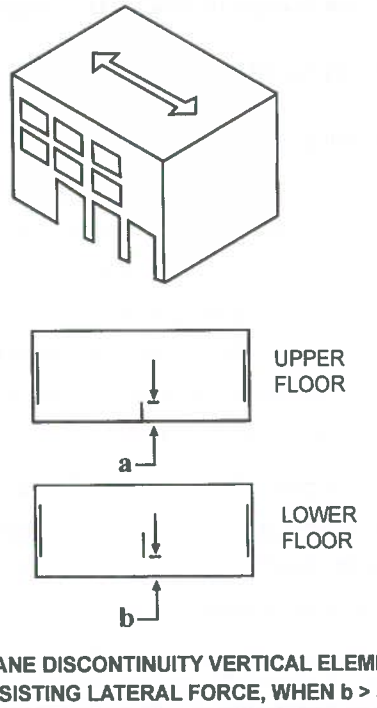


Figure 2.14 Vertical In-Plane Discontinuity in Vertical Elements Resisting Lateral Force [BNBC 2015, Figure 6.2.28(d)]

For structures assigned to Seismic Design Category D and having this type of irregularity design forces determined from Section of design base shear shall be increased 25 percent. [BNBC 2015, § 2.5.5.3.2 (iv)]

2.9.2.5 Discontinuity in Capacity-Weak Storey

A weak storey is one in which the storey lateral strength is less than 80% of that in the storey above. The storey lateral strength is the total strength of all seismic force resisting elements sharing the storey shear in the considered direction. An extreme weak storey is one where the storey lateral strength is less than 65% of that in the storey above. [BNBC 2015, § 2.5.5.3.2 (v)]

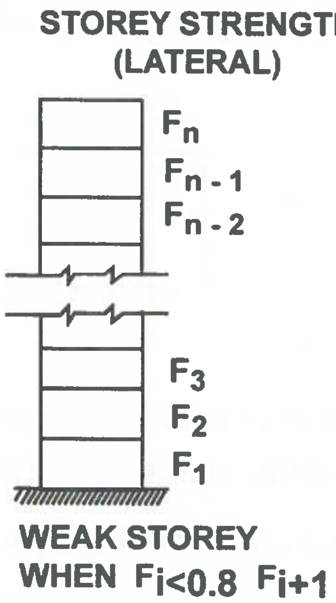


Figure 2.15 Weak Storey [BNBC 2015, Figure 6.2.28(e)]

2.9.3 Design Considerations for Irregular Structures

1) Structures having irregularities

i) For structures assigned to Seismic Design Category D and having a plan irregularity of Type I, II, III, or IV in Table 1.3.3 or a vertical structural irregularity of Type IV in Table 1.3.2, the design forces determined from Section 2.5.9 shall be increased 25 percent for connections of diaphragms to vertical elements and to collectors and for connections of collectors to the vertical elements. Collectors and their connections also shall be designed for these increased forces unless they are designed for the load combinations with overstrength factor.

ii) For structures having a plan irregularity of Type II in Table 1.3.3, diaphragm chords and collectors shall be designed considering independent movement of any projecting wings of the structure. Each of these diaphragm elements shall be designed for the more severe of the following cases:

(a) Motion of the projecting wings in the same direction.

(b) Motion of the projecting wings in opposing directions.

(BNBC 2015 §1.7.3.8)

2) Requirement for Dynamic Analysis

Dynamic analysis should be performed to obtain the design seismic force, and its distribution to different levels along the height of the building and to the various lateral load resisting elements, for the following buildings:

a) Regular buildings with height greater than 40 m in Zones 2, 3, 4 and greater than 90 m in Zone 1.

b) Irregular buildings (as defined in Section 2.5.7.3) with height greater than 12 m in Zones 2, 3, 4 and greater than 40 m in Zone 1.

For irregular buildings, smaller than 40 m in height in Zone 1, dynamic analysis, even though not mandatory, is recommended. **(BNBC 2015 §2.5.8.1)**

3) Buildings with Soft Storey

For design considerations for buildings with soft storey refer to Section 2.10.3 of this manual (BNBC 2015 2.5.17).

4) **Torsional Effects:**

For design considerations of accidental torsional effects refer to Section 2.5.2 and 2.5.3 of this manual (BNBC 2015 2.5.7.6.2 and 2.5.7.6.3).

5) Storey drift limit

For storey drift limits refer to Section 2.6.1 of this manual.

6) Structural system and configuration requirements.

Seismic design provisions impose the following limitations on the use of structural systems and configurations:

(a) The structural system used shall satisfy requirements of the Seismic Design Category (defined in BNBC 2015 Sec. 2.5.5.2) and height limitations given in BNBC 2015 Sec 2.5.5.4.

(b) Structures assigned to Seismic Design Category D having vertical irregularity Type Vb of BNBC 2015 Table 6.1.4 shall not be permitted. Structures with such vertical irregularity may be permitted for Seismic Design Category B or C but shall not be over two stories or 9 m in height.

(c) Structures having irregular features described in BNBC 2015 Table 6.1.4 or Table 6.1.5 shall be designed in compliance with the additional requirements of the Sections referenced in these Tables.

(d) Special Structural Systems defined in BNBC 2015 Sec 1.3.2.5 may be permitted if it can be demonstrated by analytical and test data to be equivalent, with regard to dynamic characteristics, lateral force resistance and energy absorption, to one of the structural systems listed in BNBC 2015 Table 6.2.19, for obtaining an equivalent R and C_d value for seismic design.

(BNBC 2015 §1.5.4.3)

7) Earthquake Load Combination

For earthquake load combination, refer to Section 2.11.1 of this manual (BNBC 2015 2.5.13.1).

2.10 MODELING AND ANALYSIS METHODS

2.10.1 Modeling Criteria

Computers have been used extensively in structural engineering. They are very useful and convenient tools to analyze even the most complicated building frames. There are cases that structural engineers accept the computer results as correct without raising questions even if analytical modeling and assumption are not correct and computer results are not compatible with the actual structural behaviors. Therefore, correct modeling and assumptions are indispensable toward accurate building analysis and design.

In order to analyze the behavior of building during earthquake properly for seismic resistant design, a suitable building model should be established based on the relevant data for the appropriate analytical method. There are some assumptions of building modeling shown below:

1) Floor and roof diaphragm in an ordinary reinforced concrete structure are assumed to be rigid and does not change its shape when subjected to lateral load like earthquake or wind. The lateral story shear is assumed to be distributed to the resisting elements like frames or bearing walls in proportion to the rigidity of those elements.

2) For the purpose of frame analysis for seismic load, it is permitted to assume fixed support at base level under the condition that the grade beam's rigidity is 2 or 3 times greater than column's rigidity. Alternatively, where foundation flexibility is considered, it shall be in accordance with Chapter 3, Part 6 in BNBC 2015.

3) When columns of a building are supported by independent spread footings but not inter connected by grade beams, then column base support shall be modeled as pin support.

4) Columns and beams are modeled by structural members with rigid ends. Reinforced concrete walls are modeled by finite elements. Alternatively, they are modeled by braces or column & beam elements.

2.10.2 Analysis Methods

BNBC 2015 provides roughly four kinds of Analysis methods in Chapter 2, Part 6. Those are static linear analysis, dynamic linear analysis, static non-linear analysis and dynamic non-linear analysis as figured below.

Table 2.17 Matrix of Analysis Methods in BNBC 2015 Part 6

Hysteresis / Time	Linear	Non-Linear
Static	Linear Static Analysis BNBC 2015 Part 6 §2.5.7	Non-Linear Static Analysis BNBC 2015 Part 6 §2.5.12
Dynamic ・Response Spectrum Analysis Method ・Time History Analysis Method	Linear Time History Analysis BNBC 2015 Part 6 §2.5.10	Non-Linear Time History Analysis BNBC 2015 Part 6 §2.5.11

This Manual includes mainly linear static analysis based on the design response spectrum, because it is most popularly used in Bangladesh. As per BNBC 2015, dynamic analysis should be applied for the structures below:

a) Regular buildings with height greater than 40m in Zones 2, 3, 4 and greater than 90m in Zone 1.

b) Irregular buildings (as defined in BNBC 2015, Part 6 §2.5.5.3) with height greater than 12m in Zones 2, 3, 4 and greater than 40m in Zone 1. A Case-study of pushover analysis in Appendix-C of the Main Manual is shown in accordance with Non-Linear Static Analysis Method.

2.10.3 Buildings with Soft Storey

Buildings with possible soft storey action at ground level for providing open parking spaces belong to structures with major vertical irregularity [Figure 6.2.28(a)]. Special arrangement is needed to increase the lateral strength and stiffness of the soft/open storey. The following two approaches may be considered:

(1) Dynamic analysis of such building may be carried out incorporating the strength and stiffness of infill walls and inelastic deformations in the members, particularly those in the soft storey, and the members designed accordingly.

(2) Alternatively, the following design criteria are to be adopted after carrying out the earthquake analysis, neglecting the effect of infill walls in other storeys. Structural elements (e.g columns and beams) of the soft storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads neglecting effect of infill walls. Shear walls placed symmetrically in both directions of the building as far away from the centre of the building as feasible are to be designed exclusively for 1.5 times the lateral shear force calculated before.

2.11 EARTHQUAKE LOAD COMBINATIONS

When earthquake effect is included in the analysis and design of a building or structure, the provisions set forth in section of Load Combinations described below shall be followed to combine earthquake load effects with other loading effects to obtain design forces. (BNBC 2015, § 2.5.13.1)

2.11.1 Horizontal Earthquake Loading

Earthquake forces act in both principal directions of the building simultaneously. In order to account for that:

(a) For structures of Seismic Design Category B, the design seismic forces are permitted to be applied independently in each of two orthogonal directions and orthogonal interaction effects are permitted to be neglected.

(b) Structures of Seismic Design Category C and D shall, as a minimum, conform to the requirements of (a) for Seismic Design Category B and in addition the requirements of this section. The structure of Seismic Design Category C with plan irregularity type V and Seismic Design Category D shall be designed for 100% of the seismic forces in one principal direction combined with 30% of the seismic forces in the orthogonal direction. Possible combinations are:

"±100% in x-direction ±30% in y-direction" or
"±30% in x-direction ±100% in y-direction"

The combination which produces most unfavorable effect for the particular action effect shall be considered.

2.11.2 Vertical Earthquake Loading

The maximum vertical ground acceleration shall be taken as 50% of the expected horizontal peak ground acceleration (PGA). The vertical seismic load effect E_v may be determined as:

$$E_v = 0.5(a_h)\,D \qquad (2.24)$$

Where,
a_h = expected horizontal peak ground acceleration (in g) for design = $(2/3)ZS$
D = effect of dead load

2.11.3 Combination of Earthquake Loading with Other Loadings

Ultimate Strength Design:

For ultimate strength design, earthquake loading effects shall be considered in combination with effects of dead loads and live loads. So,

1. $1.4D$
2. $1.2D + 1.6L$
3. $(1.2\,D + E_v) + 1.0E + 1.0L$
4. $(0.9D – E_v) + 1.0E$
5. $(1.2D + E_v) + 1.0L + 1.0E(X) + 0.3E(Y)$ (2.24)
6. $(1.2\,D + E_v) + 1.0L + 1.0E(Y) + 0.3E(X)$
7. $(0.9D – E_v) + 1.0E(X) + 0.3E(Y)$
8. $(0.9D – E_v) + 1.0E(Y) + 0.3E(X)$

where, D, L, E, E_v represent effects of dead load, live load, horizontal earthquake load and vertical earthquake load respectively. X,Y represents two orthogonal horizontal axis.
In load combinations, earthquake load shall not be considered to occur simultaneously with wind load.

2.12 GENERAL REQUIREMENTS & REINFORCEMENT DETAILING OF EARTHQUAKE RESISTANT BUILDING FRAME

2.12.1 General Requirements

2.12.1.1 Scope

a) This section contains special requirements for design and construction of reinforced concrete members of a structure for which the design forces, related to earthquake motions, have been determined on the basis of energy dissipation in the nonlinear range of response.
b) The provisions of Chapter 6 (Part 6) of BNBC 2015, shall apply for typical proportioning of structural members except as modified by the provisions of this section.
c) Structures assigned to seismic design category SDC D (*see section 2.2.11*), all reinforced concrete structures shall satisfy the requirements of special moment frames as given in *section 2.12.2 through 2.12.4* in addition to the requirements of Chapter 6(Part 6) of BNBC 2015.
d) Structures assigned to SDC C (*see section 2.2.11*), all reinforced concrete structures shall be built to satisfy the requirements of intermediate moment frames as given in *section 2.12.6* in addition to the requirements of Chapter 6 (Part 6) of BNBC 2015.

e) Structures assigned to SDC B (*see section 2.2.11*), all reinforced concrete structures shall be built to satisfy the requirements of ordinary moment frames as given in *section 2.12.5* in addition to the requirements of Chapter 6 (Part 6) of BNBC 2015.

f) Structures in lower SDCs are permitted to design with detailing provisions of higher SDCs to take advantage of lower design force levels.

2.12.1.2 Limitation on Materials in Special Moment Frame & Special Structural Walls

Requirements of concrete & reinforcement as per BNBC 2015, § 8.3.3.3 and § 8.3.3.4 are applicable to special moment frame & special structural walls & coupling beams. For intermediate and ordinary moment frames, there is no such limitation. From Seismic Design Category perspective, these limitations are applicable for SDC D.

2.12.1.3 Requirements for Concrete in Special Moment Frame & Special Structural Walls

- A minimum specified compressive strength of concrete (f_c') is 21N/mm^2 (3,000psi).
- The maximum specified compressive strength of lightweight concrete shall not exceed 35 N/mm^2 (5000psi). However, this limit may be increased to a level justified by the evidence. [BNBC 2015, § 8.3.3.3] [ACI 318-11, § 21.1.4]

2.12.1.4 Requirements for Reinforcement in Special Moment Frame & Special Structural Walls

a) Deformed reinforcement resisting earthquake-induced flexural and axial force, or both, shall comply with ASTM A706, Grade 60. BDS ISO 6935-2: 2007(E) Grade 300 and 400 (not Grade 500), and ASTM A615 Grades 40 and 60 shall be permitted if:

i. actual $f_y \leq$ specified f_y + 125N/mm^2 (18,000psi) &

ii. $$\frac{\text{actual ultimate tensile strength}}{\text{actual tensile yield strength}} \geq 1.25$$

b) The value of f_{yt} used to compute the amount of confinement reinforcement shall not exceed 700MPa (100,000psi).
[BNBC 2015, § 8.3.3.4] [ACI 318-11, § 21.1.5]

c) The value of f_y or f_{yt} used in the design of shear reinforcement shall not exceed 420MPa (60,000psi), except the value shall not exceed 550MPa (79,770psi) for welded deformed wire reinforcement. [BNBC 2015, § 6.4.3.2] [ACI 318-11, § 11.4.2]

2.12.2 Special Moment Frames

2.12.2.1 General Requirements: Frame Beams

Flexural frame members shall satisfy the following conditions:

a) Factored axial compressive force on the member $\leq A_g f_c'/10$

b) Clear span for member $l_n \geq 4 \times$ effective depth

c) Width, $b_w \geq 0.3 \times$ depth, h

d) Width $b_w \geq$ 250mm(10in)

e) Width $b_w \leq$ width of supporting member(measured on a plane perpendicular to the longitudinal axis of the flexural member) c_2 + distances on each side of the supporting member equal to the smaller of the width of the supporting member c_2 or 0.75 times the overall dimension of supporting member c_1 i.e. $b_w \leq \min(3c_2, c_2+1.5c_1)$. (see Figure 2.16) [BNBC 2015, § 8.3.4] [ACI 318-11, § 21.5.1]

2.12.2.2 Flexural Reinforcement Requirements: Frame Beams

- At any section of a flexural member and for the top as well as for the bottom reinforcement, the amount of reinforcement shall be not less than that provided by following equation:

$$A_s = \frac{0.25\sqrt{f_c'}b_w d}{f_y} \leq \frac{1.4 b_w d}{f_y} \text{ in SI} \tag{2.25a}$$

$$\left(A_s = \frac{3\sqrt{f_c'}b_w d}{f_y} \leq \frac{200 b_w d}{f_y} \text{ in fps} \right) \tag{2.25b}$$

- The reinforcement ratio ρ shall not exceed 0.025.
- At least two bars must be provided continuously at both top and bottom of section.
- Positive moment strength at joint face $\geq$ 1/2 negative moment strength at that face of the joint.
- Neither the negative nor the positive moment strength at any section along the member length shall be less than 1/4 the maximum moment strength provided at the face of either joint (see Figure 2.17).
- Lap splices of flexural reinforcement are permitted only if hoop or spiral reinforcement is provided over the lap length. Hoop and spiral reinforcement spacing shall not exceed smaller of $d/4$ and 100mm (4in).
- Lap splices shall not be used(see Figure 2.18)
  - Within joints.
  - Within a distance of $2h$ from the face of the joint.
  - At locations where analysis indicates flexural yielding caused by inelastic lateral displacements of the frame. [BNBC 2015 Part 6, § 8.3.4.2] [ACI 318-11, § 21.5.2]

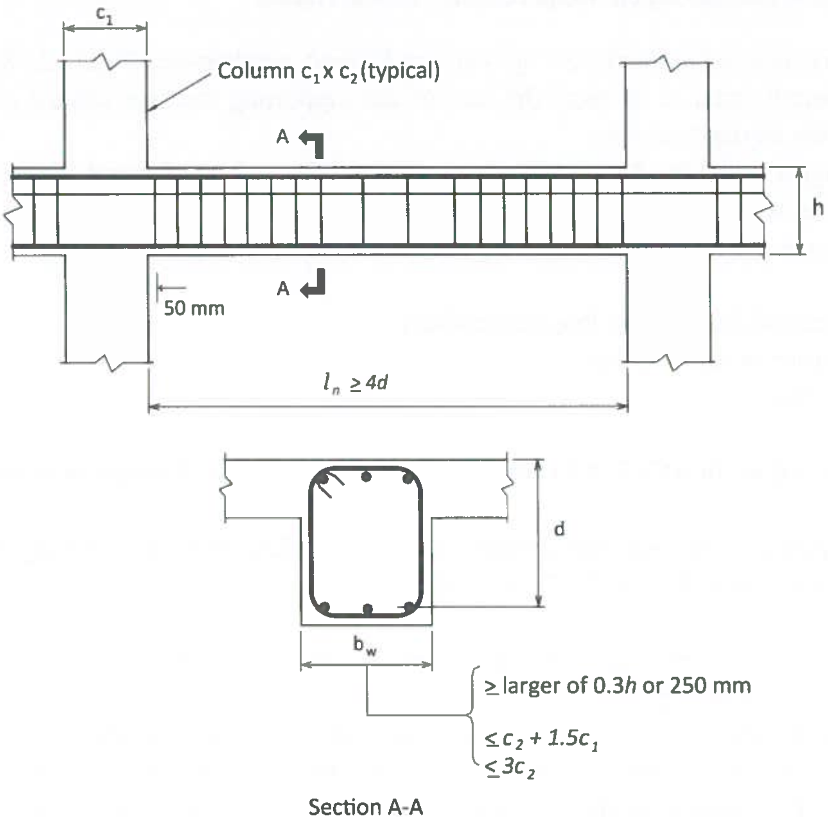


Figure 2.16 Frame Beam, General Requirement for Special Moment Frame (BNBC 2015, Figure 6.8.1)

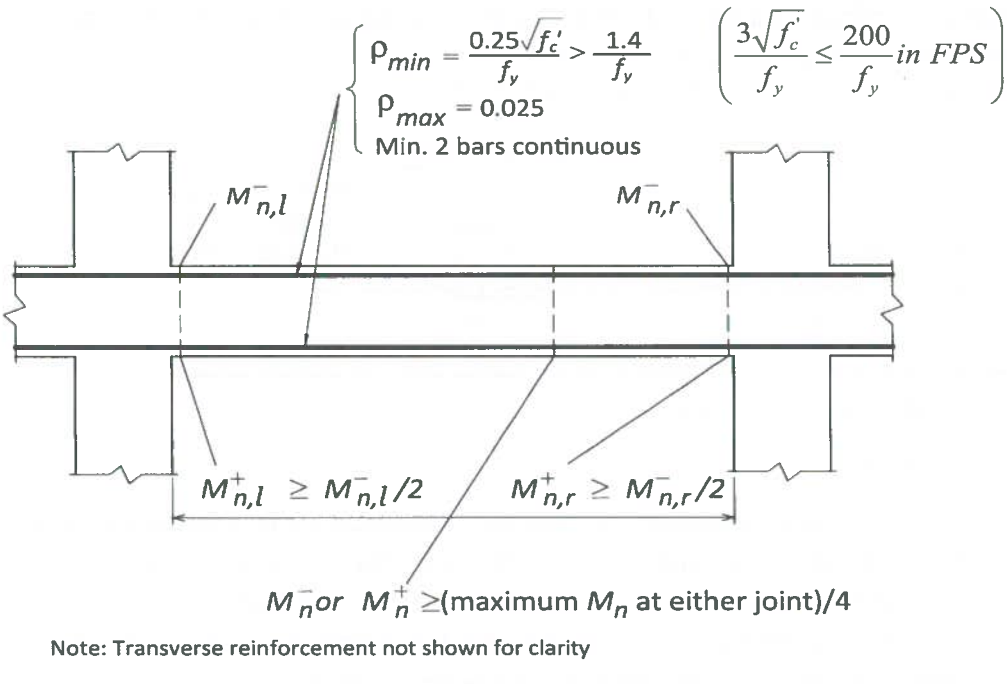


Figure 2.17 Flexural Requirements of Beams of Special Moment Frame (BNBC 2015, Figure 6.8.2)

2.12.2.3 Transverse Reinforcement Requirements: Frame Beams

a) Hoops are required in the following regions of frame members(see Figure 2.18):
   I. Over a length equal to $2h$ from the face of the supporting member toward mid span at both ends of the flexural member.
   II. Over lengths equal to $2h$ on both sides of a section where flexural yielding may occur in connection with inelastic lateral displacements of the frame.

b) Where hoops are required, the spacing shall not exceed(see Figure 2.19):
   I. $d/4$.
   II. 8 × diameter of the smallest longitudinal bars
   III. 24 × diameter of the hoop bars
   IV. 300mm(12in)

This provision according to ACI 318-11 at section 21.5.3.2, spacing of hoops shall not exceed

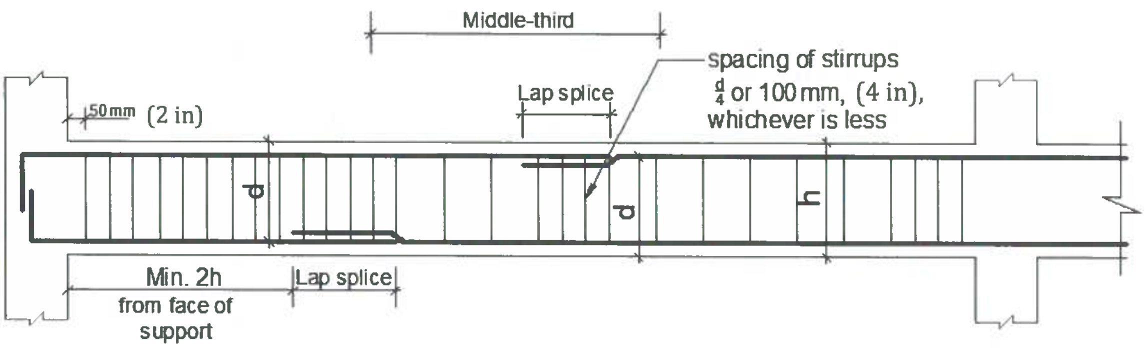
I. $d/4$
II. Six × diameter of the smallest primary flexural reinforcing bars excluding longitudinal skin reinforcement required by ACI 318-11 section 10.6.7
III. 6 in.

c) The first hoop shall be located no more than 50 mm (2 in.) from the face of the supporting member (see Figure 2.19).

d) Where hoops are required, longitudinal bars on the perimeter shall have lateral support conforming to the following provisions of ties. Where hoops are not required, stirrups with seismic hooks at both ends shall be spaced at a distance not more than $d/2$ throughout the length of the member. (See Figure 2.19).

e) Hoops in flexural members are allowed to be made up of two pieces of reinforcement consisting of a U-stirrup having hooks not less than 135 deg with 6 diameter but not less than 75mm(3in) extension anchored in the confined core and a cross tie to make a closed hoop(see Figure 2.20). Consecutive crossties engaging the same longitudinal bar shall have their 90 deg hooks at opposite sides of the flexural member. If the longitudinal reinforcing bars secured by the cross ties are confined by a slab only on one side of the flexural frame member, the 90 deg hooks of the cross ties shall all be placed on that.

[BNBC 2015, § 8.3.4.3] [ACI 318-11, § 21.5.3]



Notes: (i) For beam bottom bars lap shall not be provided within a distance of twice the member depth from the face of the support; (ii) Preferred lap location of top bar is within middle third of the span but may be provided beyond 2h from the face of the support; (iii) Not more than 50% of the bars shall be spliced at one location; (iv) Lap splices are to be confined by stirrups with maximum spacing d/4 or 100 mm(4 in.) whichever is smaller.

Figure 2.18 Lap Splice Requirements for Flexural Members of Special Moment Frames

[BNBC 2015, Figure 6.8.3]

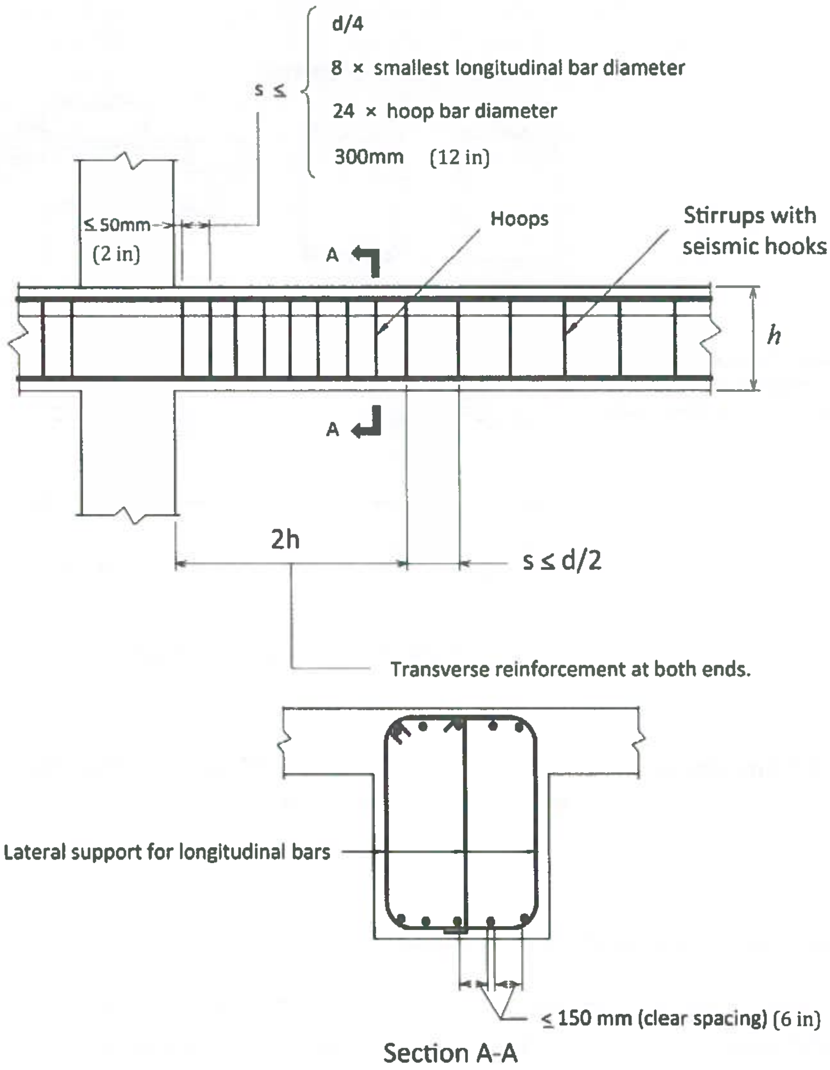


Figure 2.19 Transverse Reinforcement Requirements for Flexural Members of Special Moment Frames [BNBC 2015, Figure 6.8.4]

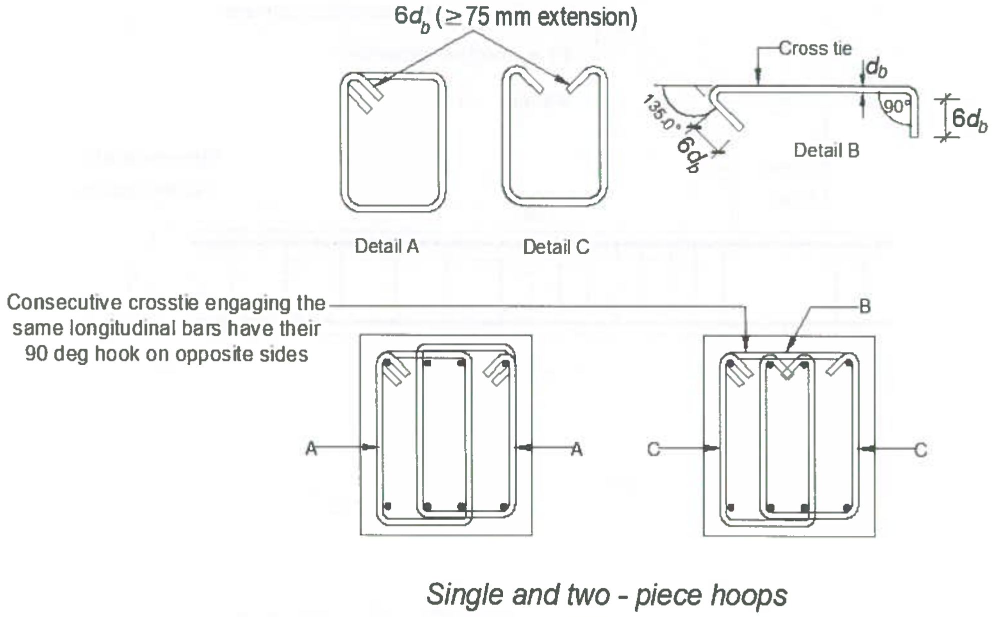


Figure 2.20 Hoop Reinforcement Requirements for Flexural Members of Special Moment Frames [BNBC 2015, Figure 6.8.5]

2.12.2.4 Shear Strength Requirements: Frame Beam

Adequate shear reinforcement must be provided so as to prevent shear failure prior to the development of plastic hinges at the ends of the beam. Thus, a flexural member of a special moment frame must be designed for the shear forces associated with probable moment strengths M_{pr}(Equation 2.26) of opposite sign acting at the ends and the factored tributary gravity load along its span (see Figure 2.21).

$$M_{pr} = A_s\left(1.25 f_y\right)\left(d - \frac{a}{2}\right) \tag{2.26}$$

$$a = \frac{A_s\left(1.25 f_y\right)}{0.85 f_c' b} \tag{2.27}$$

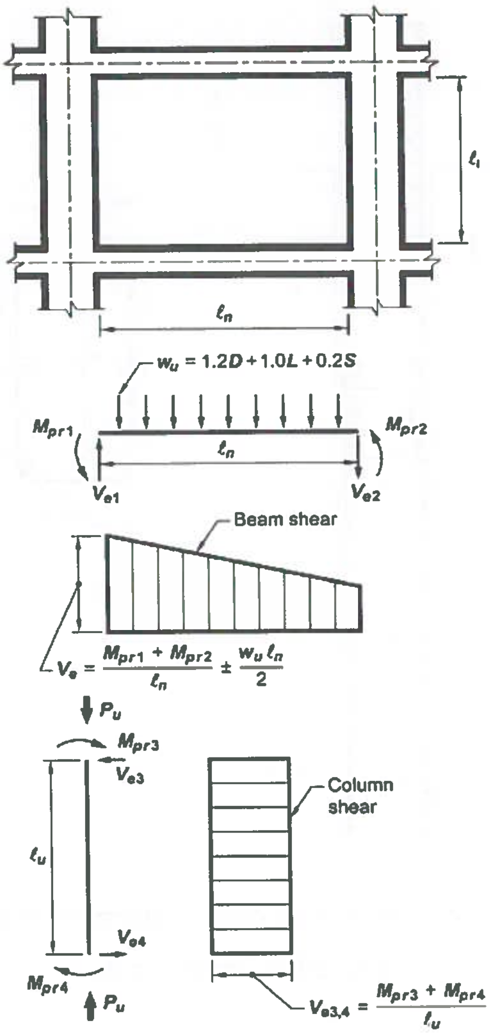
In determining the required shear reinforcement over the lengths identified in section 2.12.2.3,the contribution of the shear strength of the concrete V_c is taken as zero if the shear force from seismic loading is one-half or more of the required shear strength and the factored axial compressive force including earthquake effects is less than $A_g f_c'/20$.
[BNBC 2015, § 8.3.8.1 & 8.3.8.2] [ACI 318-11, § 21.5.4]

Notes on Figure 2.21:

1. Direction of Shear force V_e depends on relative magnitudes of gravity loads and shear generated by end moments.

2. End moments M_{pr} based on tensile stress of $1.25f_y$, where f_y is specified yield strength. (Both end moments should be considered in both directions, clockwise and counter-clockwise)

3. End moments M_{pr} for columns need not be greater than moments generated by M_{pr} of the beams framing into the beam column joints. V_e should not be less than that required by analysis of the structure



M_{pr} = Probable Flexural strength of members, with or without axial load, determined using the proper ties of the member at the joint faces assuming a tensile stress in the longitudinal bars at least $1.25f_y$ and strength reduction factor, φ, of 1.0 in Nmm.

V_e = Design shear force for load combination including earthquake effects, N.

W_u = factored load per unit length of beam or one way slab.

Figure 2.21 Design Shears for Beams &Columns(After Ref. 2.4)

2.12.2.5 General Requirements: Frame Columns

a) Factored axial compressive force $> A_g f_c{'}/10$.
b) Shortest cross-sectional dimension measured on a straight line passing through thegeometric centre $\geq$ 300mm(12in)
c) Ratio of the shortest cross-sectional dimension to the perpendicular dimension $\geq$ 0.4

These provisions are shown in Figure 2.22
[BNBC 2015, § 8.3.5.1] [ACI 318-11, § 21.6.1]

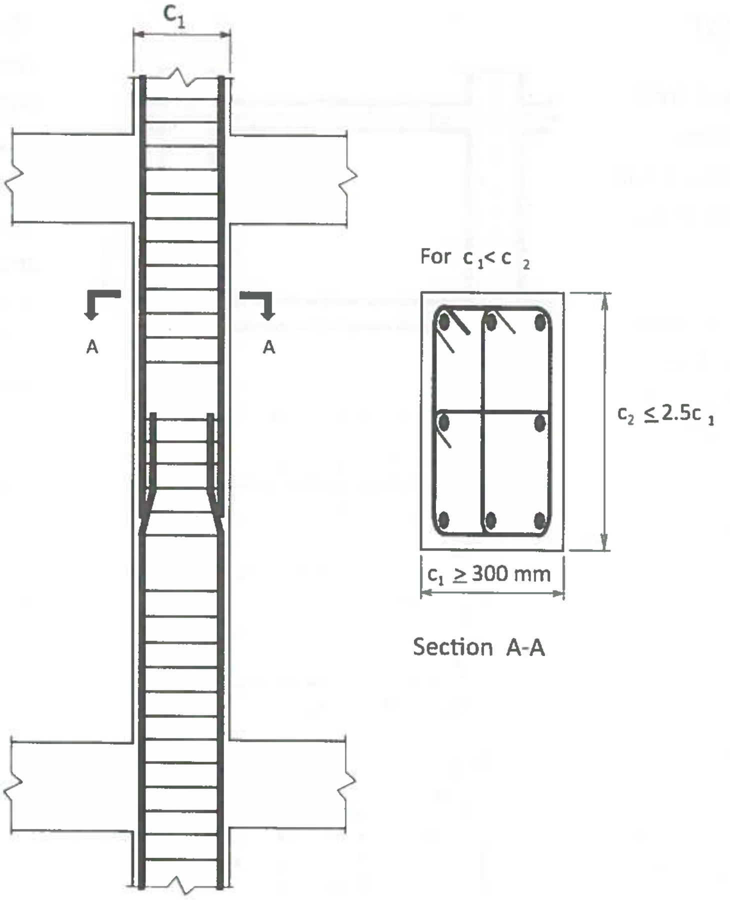


Figure 2.22 General Requirements for Frame Column of Special Moment Frames [BNBC 2015, Figure 6.8.6]

2.12.2.6 Minimum Flexural Strength: Frame Columns

Yielding of the columns prior to the beams could result in total collapse of the structure. For this reason, columns are designed with 20% higher flexural strength as compared to beams meeting at the same joint. The flexural strengths of columns shall satisfy the following Equation (BNBC 2015, Equation 6.8.5):

$$\sum M_{nc} \geq \frac{6}{5}\sum M_{nb} \tag{2.28}$$

Where,

$\sum M_{nc}$ = sum of moments at the faces of the joint, corresponding to the nominal flexural strength of the columns framing into that joint. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength.

$\sum M_{nb}$ = sum of moments at the faces of the joint, corresponding to the nominal flexural strength of the girders framing into that joint. In T-beam construction, slab reinforcement within an effective slab width shall contribute to flexural strength.

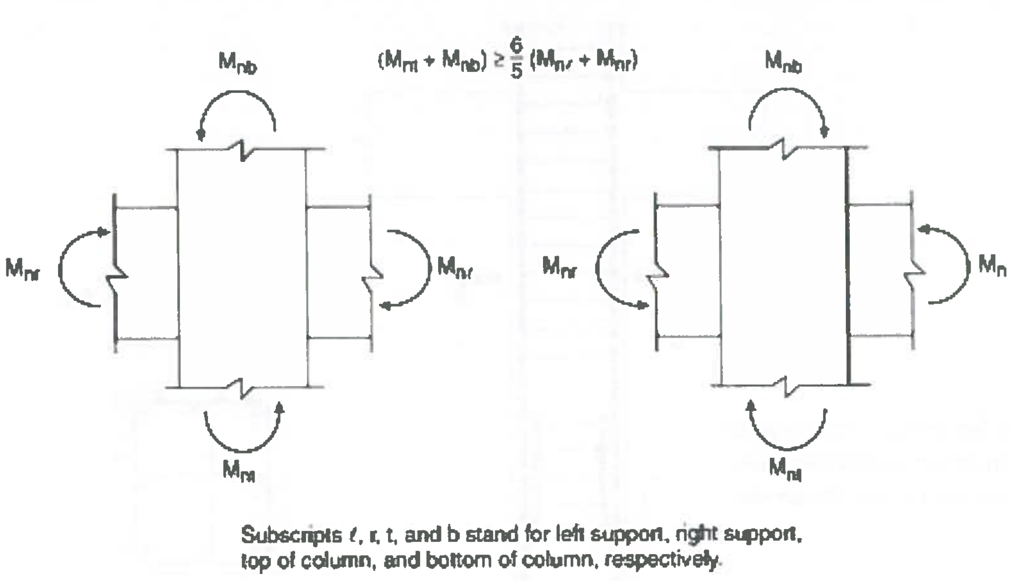


Figure 2.23 Strong column-weak beam requirement of special moment resisting frame

- The nominal flexural capacities of the members are summed such that column moments oppose the beam moments. The column strengths must satisfy the relationship for beam moments acting in both directions.

- If the Equation is not satisfied, any positive contribution of the column or columns involved to the lateral strength and stiffness of the structure is to be ignored. Negative contributions of the column or columns should not be ignored.

[BNBC 2015, § 8.3.5.2] [ACI 318-11, § 21.6.2]

2.12.2.7 Longitudinal Reinforcement Requirements: Frame Column

a) The reinforcement ratio (ρ_g) shall not be less than 0.01 and shall not exceed 0.06.

b) Lap splices are permitted only within the center half of the member length, must be tension lap splices, and shall be enclosed within transverse reinforcement. Welded splices and mechanical connections conforming to *section 2.12.2.9 and 2.12.2.10* are allowed for splicing the reinforcement at any section provided not more than alternate longitudinal bars are spliced at a section and the distance between splices is 600 mm (24 in) or more along the longitudinal axis of the reinforcement.

These provisions are shown in Figure 2.24
[BNBC 2015, § 8.3.5.3][ACI 318-11, § 21.6.3]
According to ACI 318-11 at section 21.6.3.2, in column with circular hoops the minimum number of longitudinal bars shall be 6.

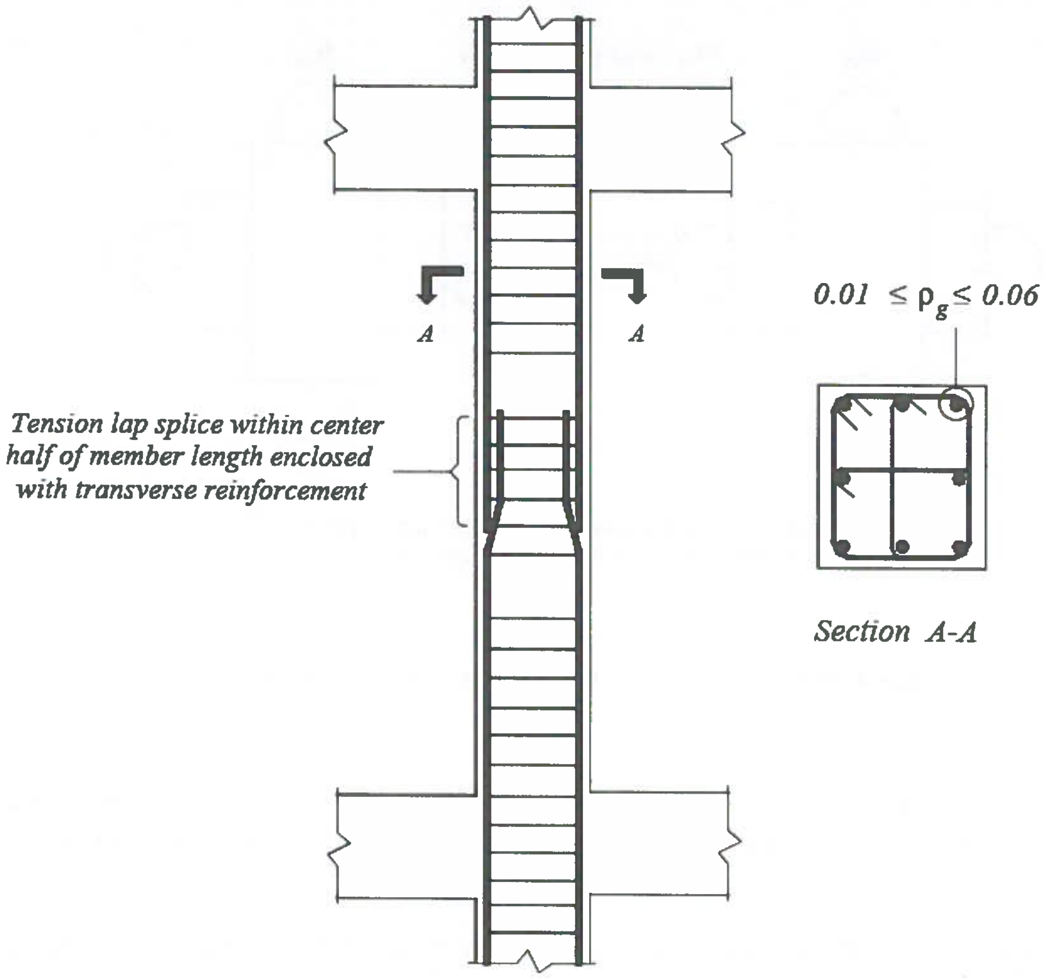


Figure 2.24 Longitudinal Reinforcement Requirements [BNBC 2015 Part 6, Figure 6.8.7]

2.12.2.8 Transverse Reinforcement: Frame Columns

a) The transverse reinforcement requirements discussed in the following need be provided only over a length ℓ_o from each joint face and on both sides of any section where flexural yielding is likely to occur. The length ℓ_o shall not be less than:
   - Depth of member
   - One-sixth of the clear span of the member
   - 450 mm (18 in)

b) Transverse reinforcement shall be provided by either single or overlapping hoops. Crossties of the same bar size and spacing as the hoops are permitted, with each end of the crosstie engaging a peripheral longitudinal reinforcing bar. Consecutive crossties shall be alternated end for end along the longitudinal reinforcement. Crossties or legs of overlapping hoops shall not be spaced more than 350mm (14 in) on center in the direction perpendicular to the longitudinal axis of the structural member.

c) Ratio of spiral or circular hoop reinforcement (ρ_s) shall not be less than that given by the following equation:

$$\rho_s = 0.12 f_c' / f_{yt} \geq 0.45 \left(A_g / A_{ch} - 1 \right) f_c' / f_{yt} \tag{2.29}$$

Where,
A_{ch} = cross-sectional area of member measured out-to-out of transverse reinforcement, mm^2 (or in^2)
A_g = gross area of section, mm^2 (or in^2)
f_c' = specified compressive strength of concrete, MPa(or psi)
f_{yt} = specified yield stress of transverse reinforcement, MPa(or psi)

d) Total cross-sectional area of rectangular hoop reinforcement for confinement A_{sh} shall not be less than that given by the following two equations:

$$A_{sh} = 0.3\left(sb_c \frac{f_c'}{f_{yt}}\right)\left(\frac{A_g}{A_{ch}} - 1\right) \tag{2.30}$$

$$A_{sh} = 0.09\left(sb_c \frac{f_c'}{f_{yt}}\right) \tag{2.31}$$

Where, b_c = cross-sectional dimension of column core measured center-to-center of outer legs of the transverse reinforcement comprising area A_{sh}

e) Transverse reinforcement along the length ℓ_o shall be spaced at distances not exceeding
- Minimum member dimension/4.
- 6 × smallest longitudinal bar diameter.
- s_0

Where, $$100mm \le s_o = 100 + \left(\frac{350 - h_x}{3}\right) \le 150mm \quad \text{(in SI)}$$

$$4in. \le s_o = 4 + \left(\frac{14 - h_x}{3}\right) \le 6in. \quad \text{(in FPS)}$$

h_x = maximum hoizontal spacing of hoop or crosstie legs on all faces of the column, mm^2(or in^2)

f) Where the transverse reinforcement as discussed above is no longer required, the remainder of the column shall contain spiral or hoop reinforcement spaced at distances not to exceed
- 6 × smallest longitudinal bar diameter
- 150mm(6in)

g) Columns supporting reactions from discontinued stiff members, such as walls, shall satisfy the following
- Transverse reinforcement shall be provided over their full height, if the factored axial compressive force, related to earthquake effects exceeds ($A_g f_c'/10$). Where design forces have been magnified to account for the over strength of the vertical elements of the seismic-force-resisting system, the limit of $A_g f_c'/10$ shall be increased to $A_g f_c'/4$.
- This transverse reinforcement shall extend into the discontinued member for at least the development length of the largest longitudinal reinforcement in the column. If the column terminates on a footing or mat, the transverse reinforcement shall extend at least 300mm (12in.) into the footing or mat (see Figure 2.27)

[BNBC 2015, § 8.3.5.4] [ACI 318-11, § 21.6.4]

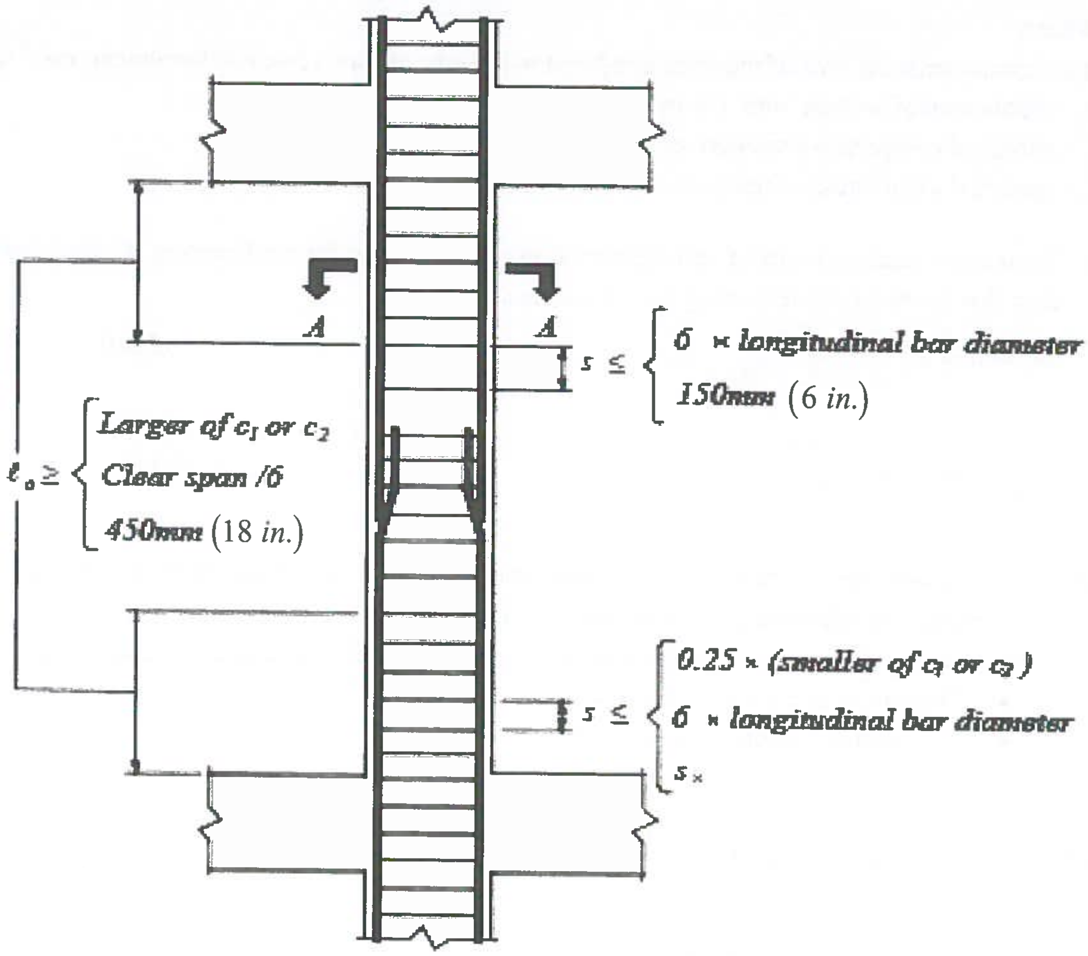


Figure 2.25 Transverse Reinforcement Requirements for Columns of SMF

[BNBC 2015, § 8.3.5.4]

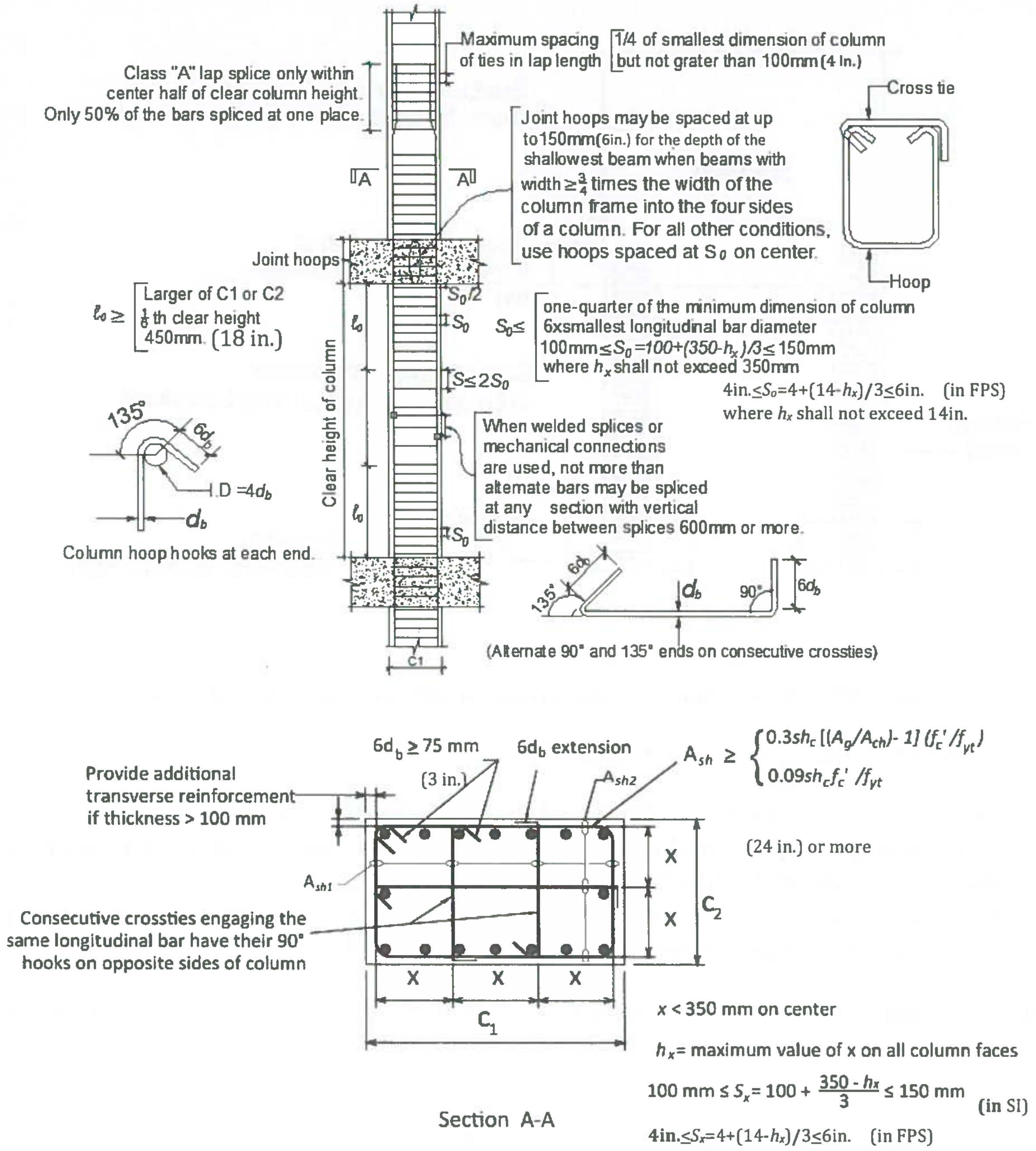


Figure 2.26 Transverse Reinforcement Requirements - Rectangular Hoop for Columns [BNBC 2015, Figure 6.8.8]

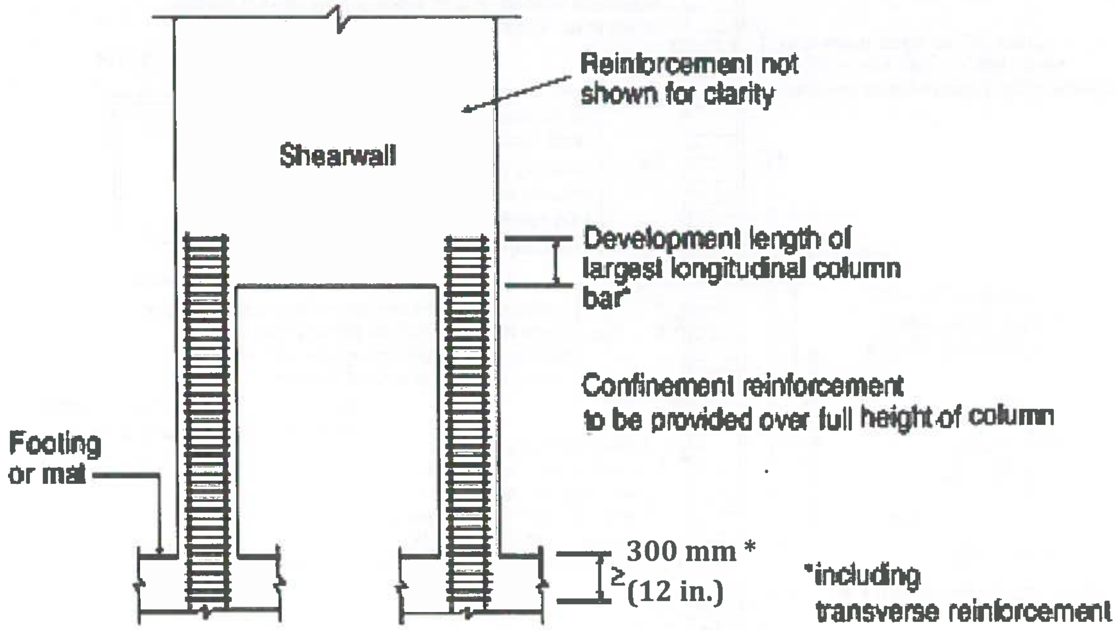


Figure 2.27 Column supporting discontinuous stiff members [*After Ref.* 2.6]

2.12.2.9 Mechanical Splices in Special Moment Frames

Mechanical splices shall be classified as either Type 1 or Type 2 mechanical splices.

- Type 1 mechanical splices shall develop in tension or compression at least 125 percent of the specified yield strength f_y of the reinforcing bar.
- Type 2 mechanical splices shall conform the provision of Type 1 mechanical splices and shall develop the specified tensile strength of the spliced bar.

During an earthquake, the tensile stresses in the reinforcement may approach the tensile strength of the reinforcement as the structure undergoes inelastic deformations. Thus, Type 2 mechanical splices can be used at any location in a member.

The locations of Type 1 mechanical splices are restricted since the tensile stresses in the reinforcement in yielding regions of the member can exceed the strength requirements for Type 1.Consequently, Type 1 mechanical splices are not permitted within a distance equal to twice the member depth from the face of the column or beam or from sections where yielding of the reinforcement is likely to occur due to inelastic lateral displacements.
[ACI 318-11, § 21.1.6]

2.12.2.10 Welded Splices in Special Moment Frames

- Welded splices in reinforcement resisting earthquake-induced forces shall develop at least 125 percent of the specified yield strength f_y of the reinforcing bar and shall not be used within a distance equal to twice the member depth from the column or beam face for special moment frames or from sections where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacements and the distance between the spliced in 600mm(24in) or more along the longitudinal axis of the reinforcement.

- Welding of stirrups, ties, inserts, or other similar elements to longitudinal reinforcement that is required by design shall not be permitted.
  [ACI 318-11, § 21.1.7]

2.12.2.11 Shear Strength Requirement: Frame Column

In addition to satisfying confinement requirements, the transverse reinforcement in columns must resist the maximum shear forces associated with the formation of plastic hinges in the frame. Although the provisions of section 2.12.2.4are intended to have most of the inelastic deformation occur in the beams, the hinging can occur in the column. Thus, as in the case of beams, the shear reinforcement in the columns is based on the probable moment strengths M_{pr} that can be developed at the ends of the column. The probable moment strength is to be the maximum consistent with the range of factored axial loads on the column; side sway to the right and to the left must both be considered. It is obviously conservative to use the probable moment strength corresponding to the balanced point (see Figure 2.28).

(i)Design Shear Force. The design shear force, V_e, shall be determined from consideration of the maximum forces that can be generated at the faces of the joints at each end of the member. These joint forces shall be determined using the maximum probable moment strengths, M_{pr}, at each end of the member associated with the range of factored axial loads, P_u, acting on the member. The member shears need not exceed those determined from joint strengths based on M_{pr} of the transverse members framing into the joint. In no case V_e shall be less than the factored shear determined by analysis of the structure.
[BNBC 2015, § 8.3.8.1(b)] [ACI 318-11, § 21.6.5.1]

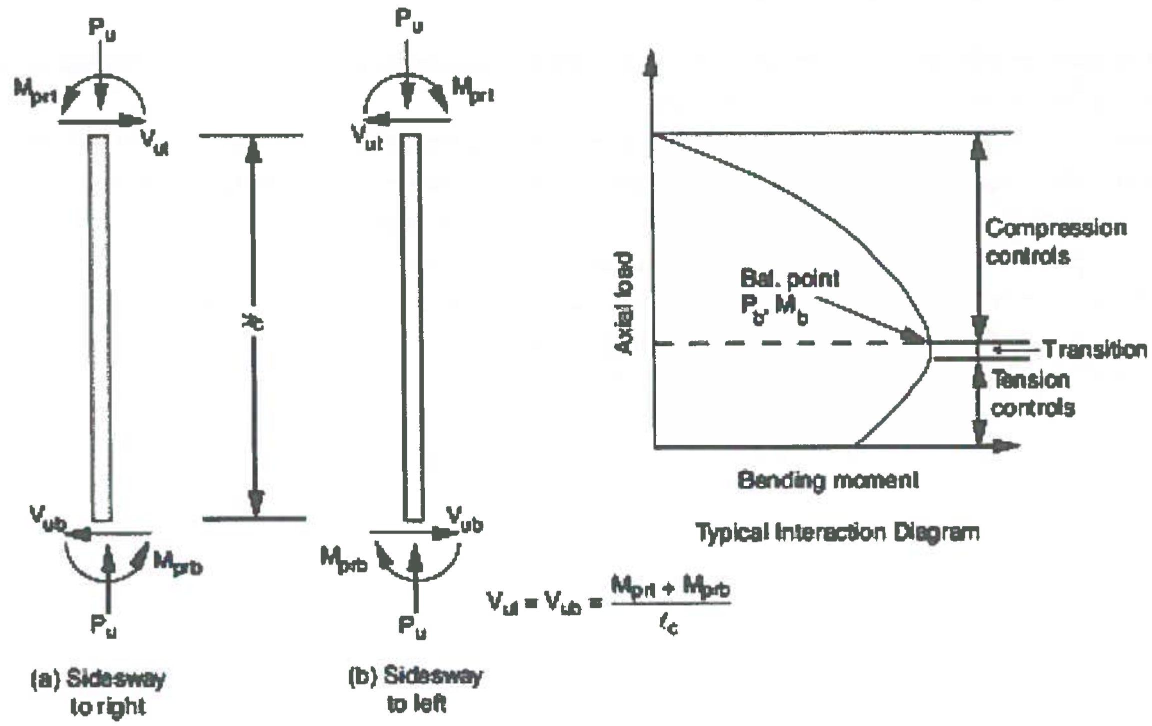


Figure 2.28 Loading Case for Design of Shear Reinforcement of Column in Special Moment Frame[*After Ref.* 2.6]

(ii)Transverse Reinforcement to Resist Shear. Transverse reinforcement over the lengths ℓ_o, shall be proportioned to resist shear assuming V_c= 0 when both (a) and (b) occur:

(a) The earthquake-induced shear force, calculated in accordance with *section 2.12.2.11(i)*, represents one-half or more of the maximum required shear strength within ℓ_o;

(b) The factored axial compressive force, P_u, including earthquake effects is less than $A_g f_c'/20$.

[BNBC 2015, § 8.3.8.2] [ACI 318-11, § 21.6.5.2]

2.12.3 Joints of Special Moment Frames

This 'Joints' segment contains the requirements for joints of special moment frames. For intermediate and ordinary cast-in place frames, the beam-column joints do not require the special design and detailing requirements as for special moment frames.

2.12.3.1 General Requirements: Joints

a) Forces in longitudinal beam reinforcement at the faces of joints of reinforced concrete frames shall be determined for a stress of $1.25f_y$ in the reinforcement.
b) Beam longitudinal reinforcement terminated in a column shall be extended to the far face of the confined column core and anchored in tension according to section 2.12.3.4.
c) Where longitudinal beam reinforcement extends through a joint, the column dimension parallel to the beam reinforcement shall be $\geq$ 20 × the diameter of the largest longitudinal bar for normal weight concrete. For lightweight aggregate concrete, this dimension shall be $\geq$ 26 × the bar diameter.

[BNBC 2015, § 8.3.7.1] [ACI 318-11, § 21.7.2]

2.12.3.2 Transverse Reinforcement: Joints

a) The transverse hoop reinforcement shall be provided according to section 2.12.2.8, unless structural members confine the joint as specified below.
b) Where members frame into all four sides of a joint and each member width is at least 3/4 the column width, the transverse reinforcement, the spacing of the transverse reinforcement may be increased to 150 mm (6 in.) within the overall depth of the shallowest framing member.
c) As required by section 2.12.2.8, transverse reinforcement shall be provided through the joint to provide confinement for longitudinal beam reinforcement outside the column core if such confinement is not provided by a beam framing into the joint.

[BNBC 2015, § 8.3.7.2] [ACI 318-11, § 21.7.3]

These provisions are shown in Figure2.29 and Figure 2.30

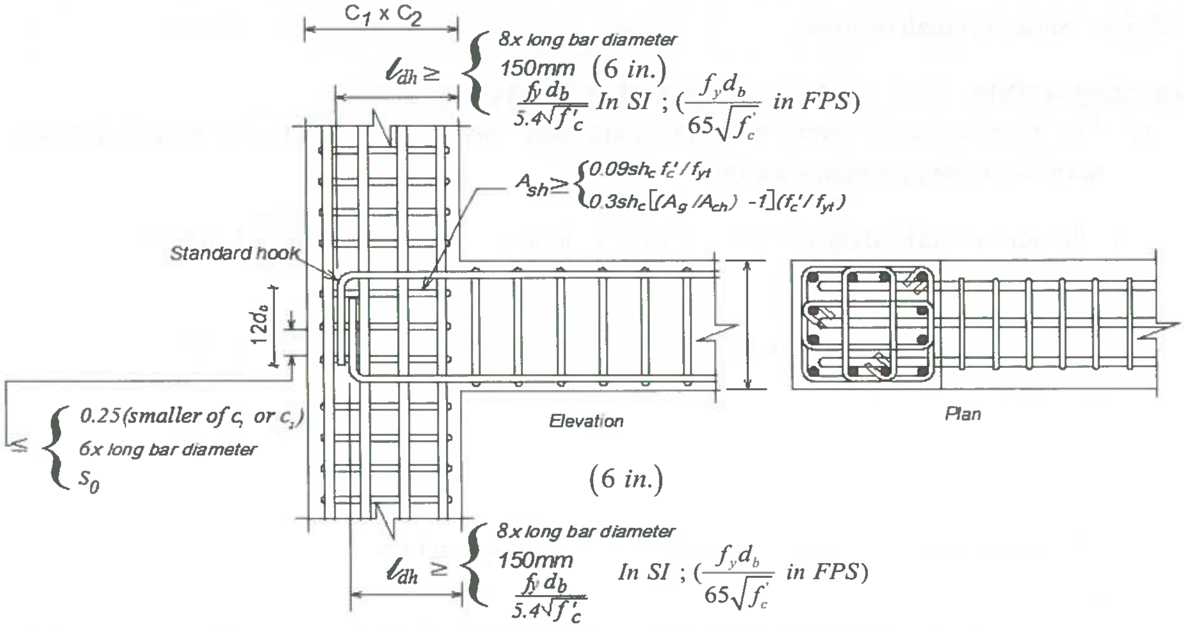


Figure 2.29 Requirements and Transverse Reinforcement Requirements for Joints not confined by Structural Member[BNBC 2015, Figure 6.8.15]

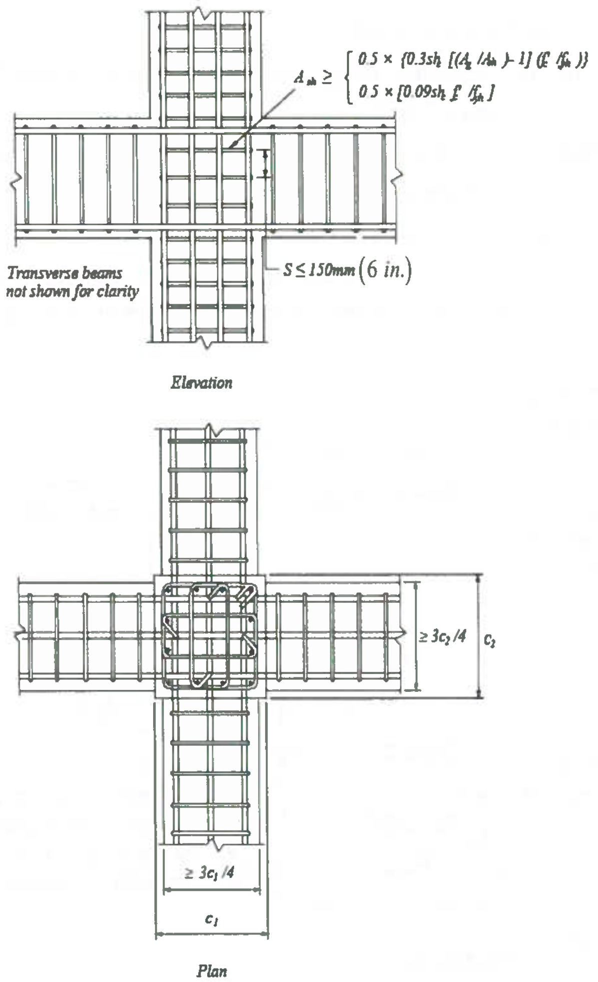


Figure 2.30 Transverse Reinforcement Requirements for Joints Confined by Structural member[BNBC 2015, Figure 6.8.16]

2.12.3.3 Shear Strength of Joint

According to BNBC 2015, § 8.3.7.3 (ACI 318-11, § 21.7.4),

a) The nominal shear strength of the joint shall not exceed the forces specified below for normal-weight aggregate concrete.

I. For joints confined on all four $1.7\sqrt{f_c'}A_j$ in SI ; $20\sqrt{f_c'}A_i$ in FPS

II. For joints confined on three faces or on two opposite faces $1.2\sqrt{f_c'}A_j$ in SI ;

$15\sqrt{f_c'}A_i$ in FPS

III. For other joints $1.0\sqrt{f_c'}A_j$ in SI ; $12\sqrt{f_c'}A_i$ in FPS

Where,

A_j = effective cross-sectional area within a joint computed from joint depth times effective joint width. The joint depth shall be the overall depth of the column. Effective joint width shall be the overall width of the column, except where a beam frames into a wider column; effective joint width shall not exceed the smaller of:

- Beam width plus the joint depth
- Twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side.(see Figure 2.31)

b) A member that frames into a face is considered to provide confinement at the joint if at least 3/4 of the face of the joint is covered by the framing member. A joint is considered to be confined if such confining members frame into all faces of the joint.

c) In determining shear forces in the joints, forces in the longitudinal beam reinforcement at the joint face shall be calculated by assuming that the stress in the flexural tensile reinforcement is $1.25f_y$ (see Figure 2.32).

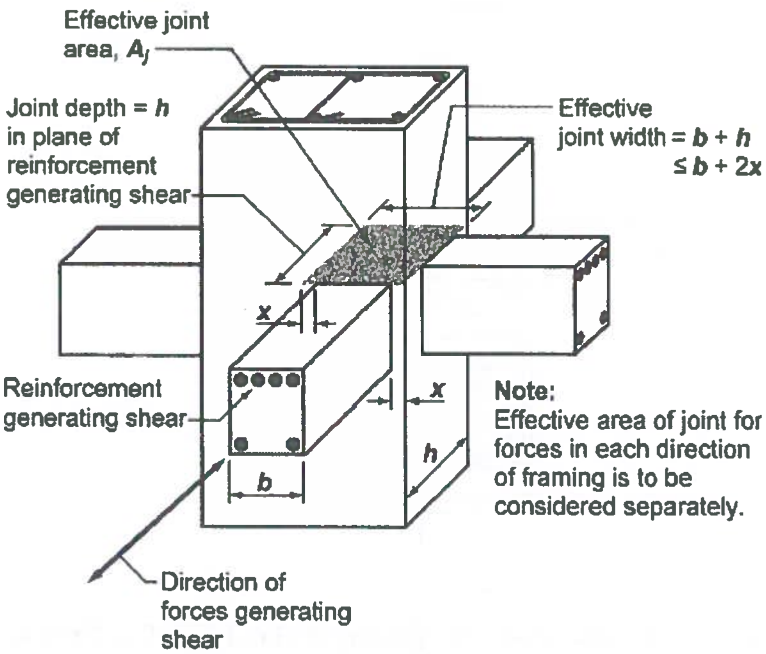


Figure 2.31 Effective Joint Area [*After Ref.* 2.4]

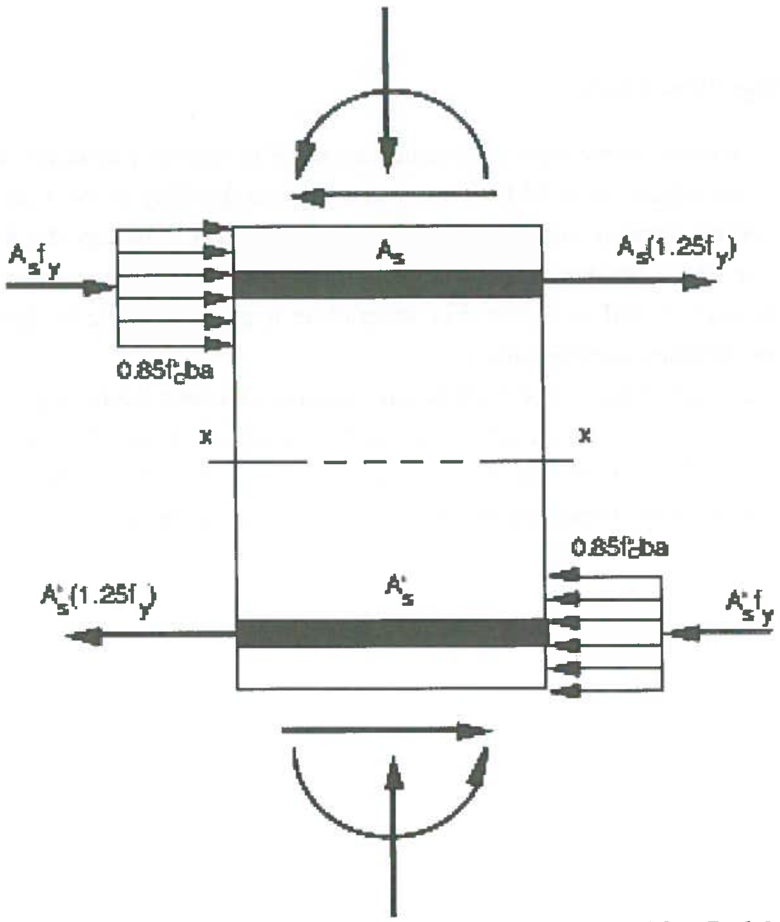


Figure 2.32 Horizontal shear in Beam-Column Joint [*After Ref.* 2.6]

2.12.3.4 Development Length of Bars in Tension

- For normal weight concrete, the development length ℓ_{dh} for a bar with a standard 90-degree hook shall not be less than the largest of :
  - 8 × diameter of the largest bar(d_b).
  - 150 mm (6 in.)
  - $f_yd_b/(5.4\sqrt{f_c'})$ in SI ; $(f_yd_b/(65\sqrt{f_c'})$ in FPS)

  For light-weight concrete, ℓ_{dh} for a bar with a standard 90^0 hook shall not be less than (i) 10 d_b, (ii) 190 mm, and (iii) 1.25 times the length required by Eq. $f_yd_b/(5.4\sqrt{f_c'})$ in SI ; $(f_yd_b/(65\sqrt{f_c'})$ in FPS). The 90^0 hook shall be located within the confined core of a column or a boundary element.

- For bar sizes 10 mm φ through 36 mm φ, the development length ℓ_{dh} for a straight bar shall not be less than the larger of (a) &(b):

(a) $2.5\ell_{dh}$ if the depth of the concrete cast in one lift beneath the bar ≤ 300mm (12 in.)
(b) $3.25\ell_{dh}$ if the depth of the concrete cast in one lift beneath the bar > 300mm(12 in.)

- Straight bars terminated at a joint shall pass through the confined core of a column or boundary element. Any portion of the straight embedment length not within the confined core shall be increased by a factor of 1.6.

  [BNBC 2015, § 8.3.7.4][ACI 318-11, § 21.7.5]

2.12.3.5 SMF Design Flow Chart

According to code provisions sometimes it becomes essential to design a structure with Special Moment Frame (SMF). Since the objective of SMF is to ensure desired ductility to the frame, that's why design procedure need to follow intrinsic rules. After getting architectural drawings the designer has to check those irregularities for which structural analysis is not essential. Some plan or vertical irregularities may be solved through discussion with architect. For some other irregularities the designer has to analyze the structure assuming preliminary member sizes.

Usually in a SMF the designer have to design beam, column, and joint according to special guideline of code. Beam and column is to be designed in such a proportion so that the principle of strong column-weak beam is attained. At the same time requirements for joint shear must be ensured. Otherwise beam and column size shall be proportioned again. A flow chart for design of Special Moment Frame is given below.

SMF Design Flow Chart

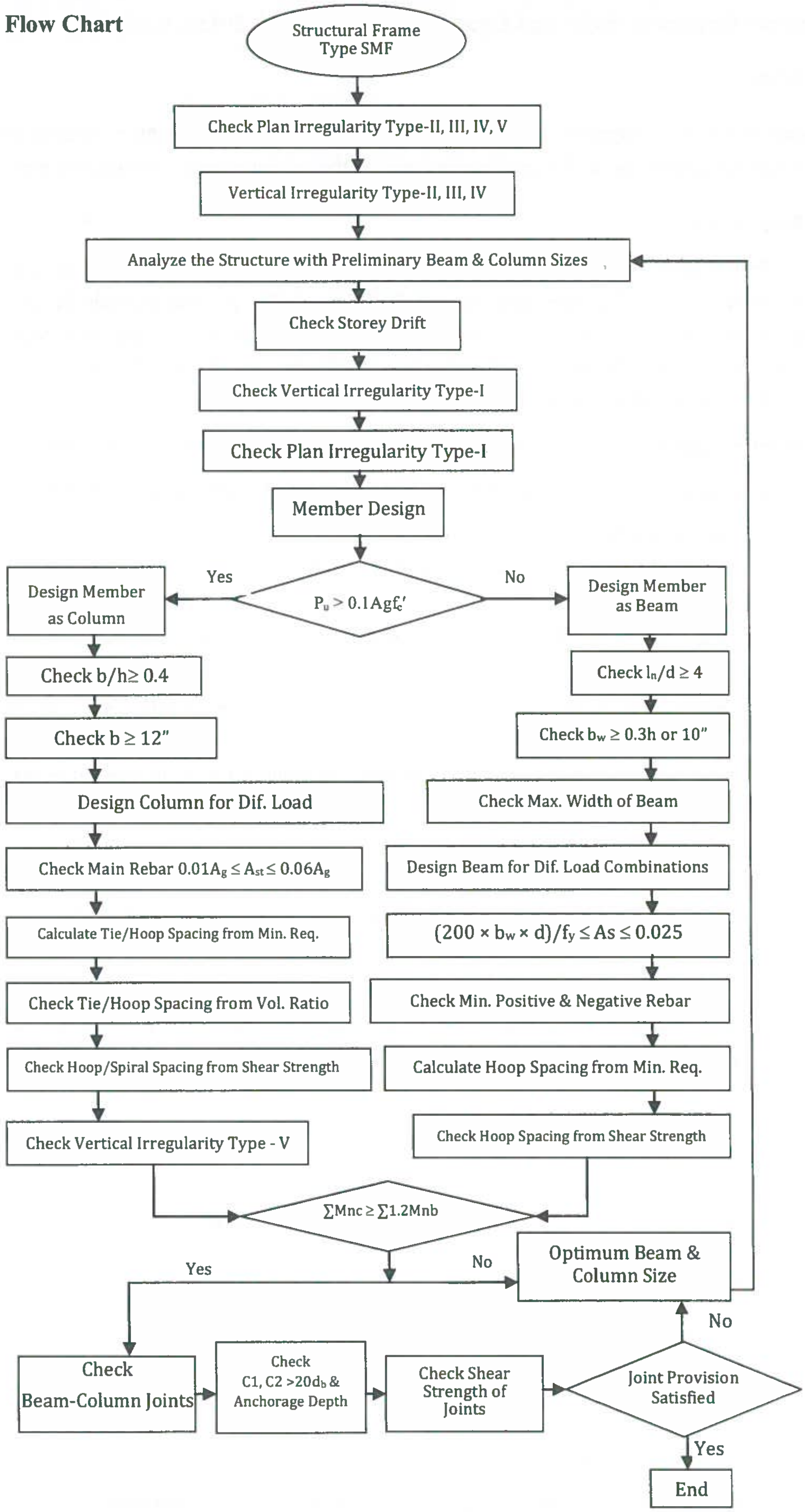


Figure 2.33 Flow chart for design of Special Moment Frame (SMF)

2.12.4 Special Structural Walls And Coupling Beams [BNBC 2015 § 8.3.6]

2.12.4.1 Scope

Requirements of sec 2.12.4 apply to special structural walls and all components of special structural walls including coupling beams and wall piers forming part of the seismic-force-resisting system.

2.12.4.2 Reinforcement

(a) The distributed web reinforcement ratios, ρ_l and ρ_t, for structural walls shall not be less than 0.0025, except that if V_u does not exceed $0.083A_{cv}\lambda\sqrt{f_c'}$, ρ_l and ρ_t shall be permitted to be reduced to the values required as specified below. Reinforcement spacing each way in structural walls shall not exceed 450 mm. Reinforcement contributing to V_n shall be continuous and shall be distributed across the shear plane.

(i) Minimum ratio of vertical reinforcement area to gross concrete area, ρ_l, shall be:

Deformed bar not larger than 16 mm diameter with f_y not less than 420 MPa: 0.0012

Other deformed bars: 0.0015

Welded wire reinforcement not larger than ASTM MW200 or MD 200: 0.0012

(ii) Minimum ratio of horizontal reinforcement area to gross concrete area, ρ_t shall be:

Deformed bar not larger than 16 mm diameter with f_y not less than 420 MPa: 0.0020

Other deformed bars: 0.0025

Welded wire reinforcement not larger than ASTM MW200 or MD 200: 0.0020

(b) At least two curtains of reinforcement shall be used in a wall if V_u exceeds $0.17A_{cv}\lambda\sqrt{f_c'}$.

(c) Reinforcement in structural walls shall be developed or spliced for f_y in tension in accordance with BNBC 2015 Sec 8.2, except:

(i) The effective depth of the member shall be permitted to be taken as $0.8l_w$ for walls where, reinforcement extended beyond the point at which it is no longer required to resist flexure for a distance equal to d or $12d_b$, whichever is greater, except at supports of simple spans and at free end of cantilevers.

(ii) The requirements of BNBC 2015 Sections 8.2.8, 8.2.9, and 8.2.10 need not be satisfied.

At locations where yielding of longitudinal reinforcement is likely to occur as a result of lateral displacements, development lengths of longitudinal reinforcement shall be 1.25 times the values calculated for f_y in tension.

2.12.4.3 Design Forces

V_u shall be obtained from the lateral load analysis in accordance with the factored load combinations.

2.12.4.4 Shear Strength

(a) V_n of structural walls shall not exceed

$$V_n = A_{cv}\left(\alpha_c\lambda\sqrt{f_c'} + \rho_t f_y\right) \qquad (2.32) \qquad \text{[BNBC 2015 6.8.9]}$$

Where, the coefficient α_c is 0.25 for $h_w/l_w \leq 1.5$, is 0.17for $h_w/l_w \geq 2.0$, and varies linearly between 0.25 and 0.17 for h_w/l_w between 1.5 and 2.0.

(b) In Sec 8.3.6.4(a), the value of ratio h_w/l_w used for determining V_n for segments of a wall shall be the larger of the ratios for the entire wall and the segment of wall considered.

(c) Walls shall have distributed shear reinforcement providing resistance in two orthogonal directions in the plane of the wall. If h_w/l_w does not exceed 2.0, reinforcement ratio ρ_l shall not be less than reinforcement ratio ρ_t.

(d) For all vertical wall segments resisting a common lateral force, combined V_n shall not be taken larger than $0.66A_{cv}\sqrt{f_c'}$, where, A_{cv} is the gross combined area of all vertical wall segments. For any one of the individual vertical wall segments, V_n shall not be taken larger than $0.83A_{cw}\sqrt{f_c'}$, where A_{cw} is the area of concrete section of the individual vertical wall segment considered.

(e) For horizontal wall segments as shown in Figure 6.8.10, including coupling beams, V_n shall not be taken larger than $0.83A_{cw}\sqrt{f_c'}$, where A_{cw} is the area of concrete section of a horizontal wall segment or coupling beam.

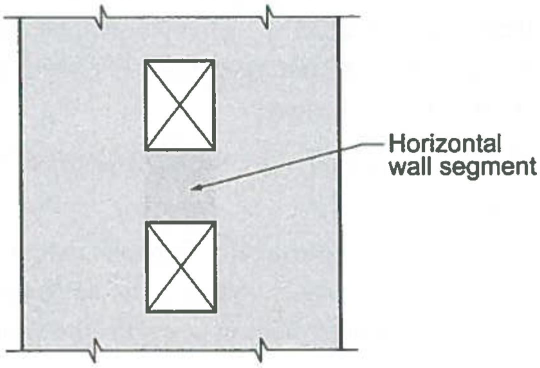


Figure 2.34 Wall with openings

2.12.4.5 Design for Flexure and Axial Loads

(a) Structural walls and portions of such walls subject to combined flexural and axial loads shall be designed in accordance with BNBC 2015 Sections 6.3.2 and 6.3.3 except that Sec 6.3.3.7 and the nonlinear strain requirements of Sec 6.3.2.2 shall not apply. Concrete and developed longitudinal reinforcement within effective flange widths, boundary elements, and the wall web shall be considered effective. The effects of openings shall be considered.

(b) Unless a more detailed analysis is performed, effective flange widths of flanged sections shall extend from the face of the web a distance equal to the smaller of one-half the distance to an adjacent wall web and 25 percent of the total wall height.

2.12.4.6 Boundary Elements of Special Structural Walls

(a) The need for special boundary elements at the edges of structural walls shall be evaluated in accordance with Sec 2.12.4.6(b) or (c). The requirements of Sec 2.12.4.6(d) and (e) also shall be satisfied.

(b) This section applies to walls or wall piers that are effectively continuous from the base of structure to top of wall and designed to have a single critical section for flexure and axial loads. Walls not satisfying these requirements shall be designed by Sec 2.12.4.6(c).

(i) Compression zones shall be reinforced with special boundary elements where

$$c \geq \frac{l_w}{600(\delta_u/h_w)} \qquad (2.3.3) \qquad [\text{BNBC 2015 6.8.10}]$$

c in Eq. 2.3.3 corresponds to the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with the design displacement δ_u. Ratio $\frac{\delta_u}{h_w}$ in Eq. 2.3.3 shall not be taken less than 0.007;

(ii) Where special boundary elements are required by b (i), the special boundary element reinforcement shall extend vertically from the critical section a distance not less than the larger of l_w or $\frac{M_u}{4V_u}$.

(c) Structural walls not designed to the provisions of (b) shall have special boundary elements at boundaries and edges around openings of structural walls where the maximum extreme fiber compressive stress, corresponding to load combinations including earthquake effects, E exceeds $0.2f_c^{'}$. The special boundary element shall be permitted to be discontinued where the calculated compressive stress is less than $0.15f_c^{'}$. Stresses shall be calculated for the factored forces using a linearly elastic model and gross section properties. For walls with flanges, an effective flange width as defined in Sec 2.12.4.5(b) shall be used.

(d) Where special boundary elements are required by Sec 2.12.4.6(b) or (c), following (i) to (v) shall be satisfied as shown in Figure 2.35:

(i) The boundary element shall extend horizontally from the extreme compression fiber a distance not less than the larger of$c - 0.1l_w$ and $\frac{c}{2}$, where c is the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with δ_u;

(ii) In flanged sections, the boundary element shall include the effective flange width in compression and shall extend at least 300 mm into the web;

(iii) The boundary element transverse reinforcement shall satisfy the requirements of Sec 2.12.2.8 as shown in Figure 2.26, except Eq. 2.30 need not be satisfied and the transverse reinforcement spacing limit of 2.12.2.8e) shall be one-third of the least dimension of the boundary element;

(iv) The boundary element transverse reinforcement at the wall base shall extend into the support at least l_d according to Sec 2.12.4.2(c), of the largest longitudinal reinforcement in the special boundary element unless the special boundary element terminates on a footing, mat, or pile cap, where special boundary element transverse reinforcement shall extend at least 300 mm into the footing, mat, or pile cap;

(v) Horizontal reinforcement in the wall web shall extend to within 150 mm of the end of the wall. Reinforcement shall be anchored to develop f_y in tension within the confined core of the boundary element using standard hooks or heads. Where the confined boundary element has sufficient length to develop the horizontal web reinforcement, and $\frac{A_v f_y}{s}$ of the web reinforcement is not greater than $\frac{A_{sh} f_{yt}}{s}$ of the boundary element transverse reinforcement parallel to the web reinforcement, it shall be permitted to terminate the web reinforcement without a standard hook or head.

(e) Where special boundary elements are not required by Sec 2.12.4.6(b) or (c), (i) and (ii) shall be satisfied as shown in Figure 2.36:

(i) If the longitudinal reinforcement ratio at the wall boundary is greater than$\frac{2.8}{f_y}$, boundary transverse reinforcement shall satisfy Sec 2.12.2.8.b), Sec 2.12.2.8.e) as shown in Figure 2.26 and Sec 2.12.4.6.(d).(i). The maximum longitudinal spacing of transverse reinforcement in the boundary shall not exceed 200 mm;

(ii) Except when V_u in the plane of the wall is less than $0.083A_{cv}\lambda\sqrt{f_c^{'}}$, horizontal reinforcement terminating at the edges of structural walls without boundary elements shall have a standard hook engaging the edge reinforcement or the edge reinforcement shall be enclosed in U-stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement.

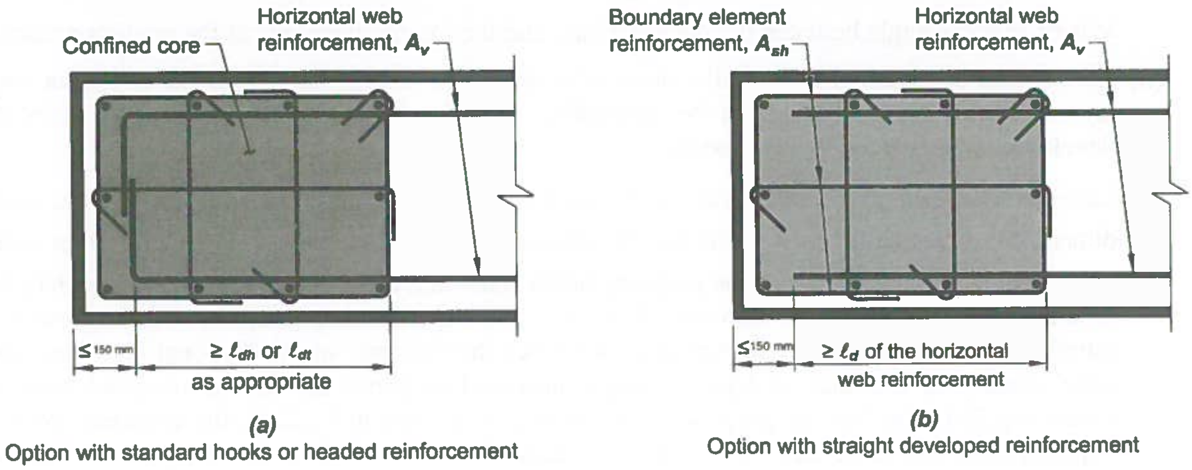


Figure 2.35 Development of wall horizontal reinforcement in confined boundary element

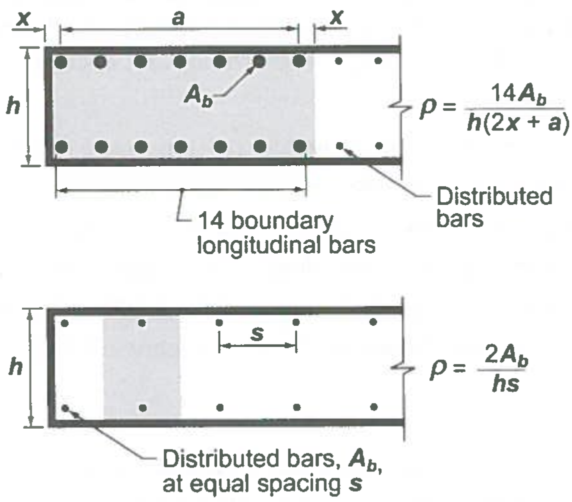


Figure 2.36 Longitudinal reinforcement ratios for typical wall boundary conditions.

2.12.4.7 Coupling Beams

(a) Coupling beams with $\frac{l_n}{h} > 4$ shall satisfy the requirements of Sec 8.12.3. The provisions of Sec 8.12.3(b) and (c) need not be satisfied if it can be shown by analysis that the beam has adequate lateral stability.

(b) Coupling beams with $\frac{l_n}{h} < 2$ and with V_u exceeding $0.33A_{cw}\lambda\sqrt{f_c'}$, shall be reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan, unless it can be shown that loss of stiffness and strength of the coupling beams will not impair the vertical load-carrying ability of the structure, the egress from the structure, or the integrity of nonstructural components and their connections to the structure.

(c) Coupling beams not governed by Sec 2.12.4.7(a) or (b) shall be permitted to be reinforced either with two intersecting groups of diagonally placed bars symmetrical about the mid span or according to Sections 2.12.2.8, 2.12.2.11, 2.12.3.4.

(d) Coupling beams reinforced with two intersecting groups of diagonally placed bars symmetrical about the mid span shall satisfy (i), (ii), and either (iii) or (iv). Requirements of BNBC 2015 Sec 6.4.5 Chapter 6 shall not apply.

(i) V_n shall be determined by

$$V_n = 2A_{vd} f_y \sin\alpha \leq 0.83 A_{cw} \sqrt{f_c'} \quad (2.34) \quad \text{[BNBC 2015 6.8.11]}$$

Where, α is the angle between the diagonal bars and the longitudinal axis of the coupling beam.

(ii) Each group of diagonal bars shall consist of a minimum of four bars provided in two or more layers. The diagonal bars shall be embedded into the wall not less than 1.25 times the development length for f_y in tension.

(iii) Each group of diagonal bars shall be enclosed by transverse reinforcement having out-to-out dimensions not smaller than $\frac{b_w}{2}$ in the direction parallel to b_w and $\frac{b_w}{5}$ along the other sides, where b_w is the web width of the coupling beam. The transverse reinforcement shall satisfy Sec 2.12.2.3 as shown in Figure 2.26 and shall have spacing measured parallel to the diagonal bars satisfying Sec 2.12.2.3 and not exceeding six times the diameter of the diagonal bars, and shall have spacing of crossties or legs of hoops measured perpendicular to the diagonal bars not exceeding 350 mm. For the purpose of computing A_g for use in Eq.2.30, the concrete cover as required in BNBC 2015 Sec 8.1.7 shall be assumed on all four sides of each group of diagonal bars. The transverse reinforcement, or its alternatively configured transverse reinforcement satisfying the spacing and volume ratio requirements of the transverse reinforcement along the diagonals, shall continue through the intersection of the diagonal bars. Additional longitudinal and transverse reinforcement shall be distributed around the beam perimeter with total area in each direction not less than $0.002 b_w s$ and spacing not exceeding 300 mm as shown in Figure 2.37(a).

(iv) Transverse reinforcement shall be provided for the entire beam cross section satisfying Sec 2.12.2.3 as shown in Figure 2.26, with longitudinal spacing not exceeding the smaller of 150 mm and six times the diameter of the diagonal bars, and with spacing of crossties or legs of hoops both vertically and horizontally in the plane of the beam cross section not exceeding 200 mm. Each crosstie and each hoop leg shall engage a longitudinal bar of equal or larger diameter. It shall be permitted to configure hoops as shown in Figure 2.37(b).

2.12.4.8 Wall Piers

(a) Wall piers shall satisfy the special moment frame requirements for columns of Sec 2.12.2.7 with joint faces taken as the top and bottom of the clear height of the wall pier. Alternatively, wall piers with $\frac{l_w}{b_w} > 2.5$ shall satisfy (i) to (vi) below:

(i) Design shear force shall be determined in accordance with Sec 2.12.2.11 and BNBC 2015 Section 8.3.8.1 with joint faces taken as the top and bottom of the clear height of the wall pier. Where the Code includes provisions to account for over strength of the seismic-force-resisting system, the design shear force need not exceed Ω_o times the factored shear determined by analysis of the structure for earthquake effects.

(ii) V_n and distributed shear reinforcement shall satisfy Sec 2.12.4.

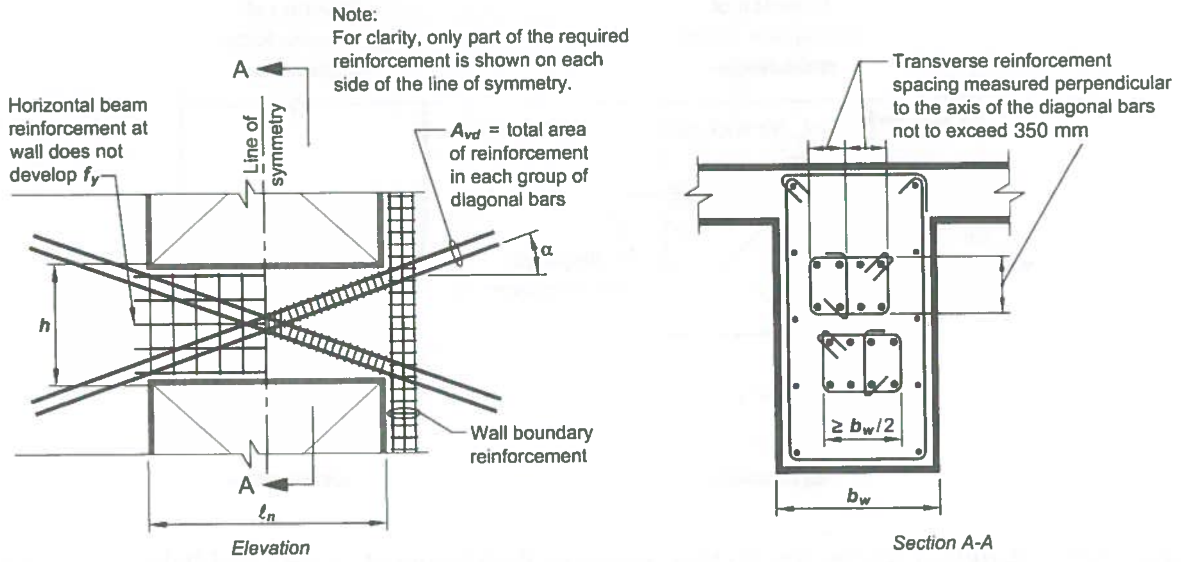
(iii) Transverse reinforcement shall be in the form of hoops except it shall be permitted to use single-leg horizontal reinforcement parallel to l_w, where only one curtain of distributed shear reinforcement is provided. Single-leg horizontal reinforcement shall have 180° bends at each end that engage wall pier boundary longitudinal reinforcement.

(iv) Vertical spacing of transverse reinforcement shall not exceed 150 mm.

(v) Transverse reinforcement shall extend at least 300 mm above and below the clear height of wall pier.

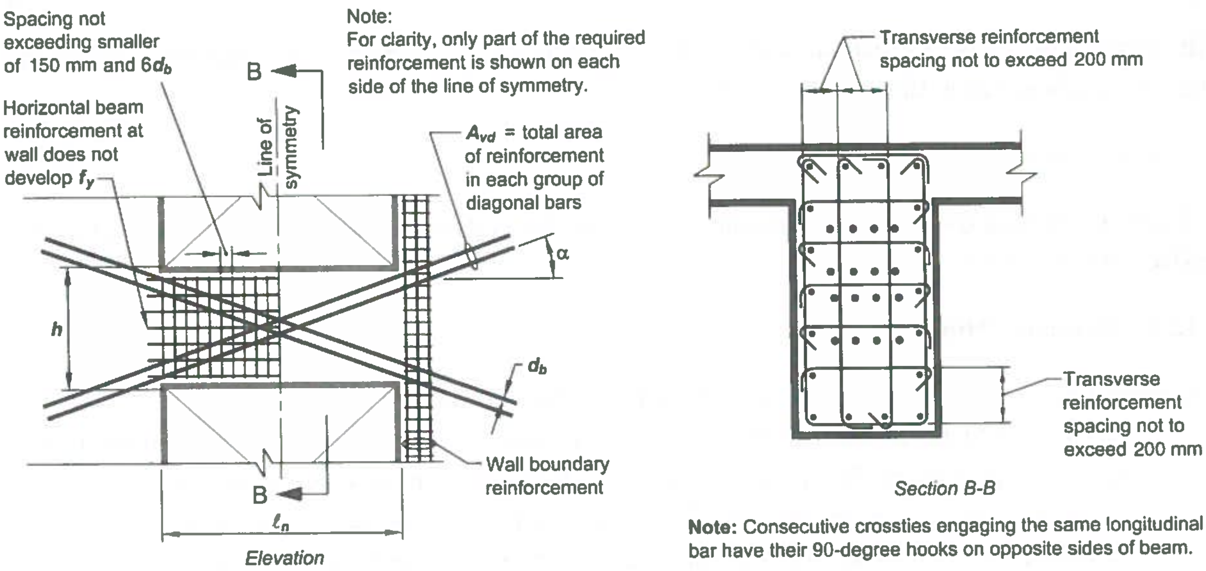
(vi) Special boundary elements shall be provided if required by Sec 2.12.4.6(c).

(b) For wall piers at the edge of a wall, horizontal reinforcement shall be provided in adjacent wall segments above and below the wall pier and be proportioned to transfer the design shear force from the wall pier into the adjacent wall segments as shown in Figure 2.38.



(a) Confinement of individual diagonals.

Note: For clarity in the elevation view, only part of the total required reinforcement is shown on each side of the line of symmetry.



(b) Full confinement of diagonally reinforced concrete beam section.

Figure 2.37 Coupling beams with diagonally oriented reinforcement. Wall Boundary reinforcement shown on one side only for clarity.

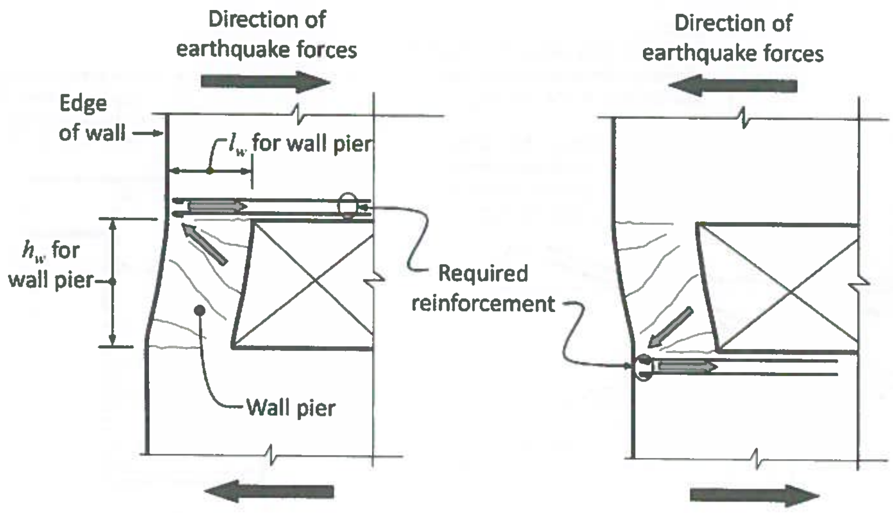


Figure 2.38 Required Horizontal Reinforcement in Wall Segments Above and Below Wall Piers At The Edge of a Wall.

2.12.4.9 Construction Joints

All construction joints in structural walls shall conform to BNBC 2015 sec 5.16.4 and contact surfaces shall be roughened as in BNBC 2015 sec 6.4.5.9.

2.12.4.10 Discontinuous Walls

Columns supporting discontinuous structural walls shall be reinforced in accordance with sec 2.12.2.8 and BNBC 2015 8.3.5.4(e).

2.12.5 Ordinary Moment Frames

The provisions described below apply only to ordinary moment frames.

a) Beam in ordinary moment frames shall have at least two longitudinal bars continuous along the top and bottom faces. This provision applies to beam-column moment frames only.

b) Columns having clear height less than or equal to five times the column dimension C_1 must be designed for shear in accordance with the reinforcement requirements for shear in intermediate moment frame.

[ACI 318-11, § 21. 2]

2.12.6 Intermediate Moment Frames

For structures assigned to SDC C, structural frames proportioned to resist forces induced by earth quake motions shall not only satisfy requirements of *section 2.12.6* but also satisfy the requirement of Chapter 6 (Part 6) of BNBC 2015. [BNBC 2015, § 8.3.10.1]

2.12.6.1 General Requirements: Frame Beams

If factored axial compressive force, $P_u \leq A_g f_c'/10$ then the frame member is considered as beam. [BNBC 2015, § 8.3.10.2] [ACI 318-11, § 21.3.2]

2.12.6.2 Shear Strength

Transverse Reinforcement for Beams & Columns resisting earthquake effect, ***E*** must also be proportioned to resist the design shear forces. Factored shear force shall not be less than the smaller of the following two criteria:

a) The factored shear force determined from the nominal moment strength of the member at each restrained end of the clear span and the gravity load on it.
b) V_u obtained from design load combinations that include E, with E assumed to be twice that prescribed by the general building code for earthquake-resistant design. For example, the load combination defined by Eq. $V_u = 1.2D + 1.0E + 1.0L + 0.2S$ would be $U = 1.2D + 2.0E + 1.0L + 0.2S$

[BNBC 2015, § 8.3.10.3] [ACI 318-11, § 21.3.3]

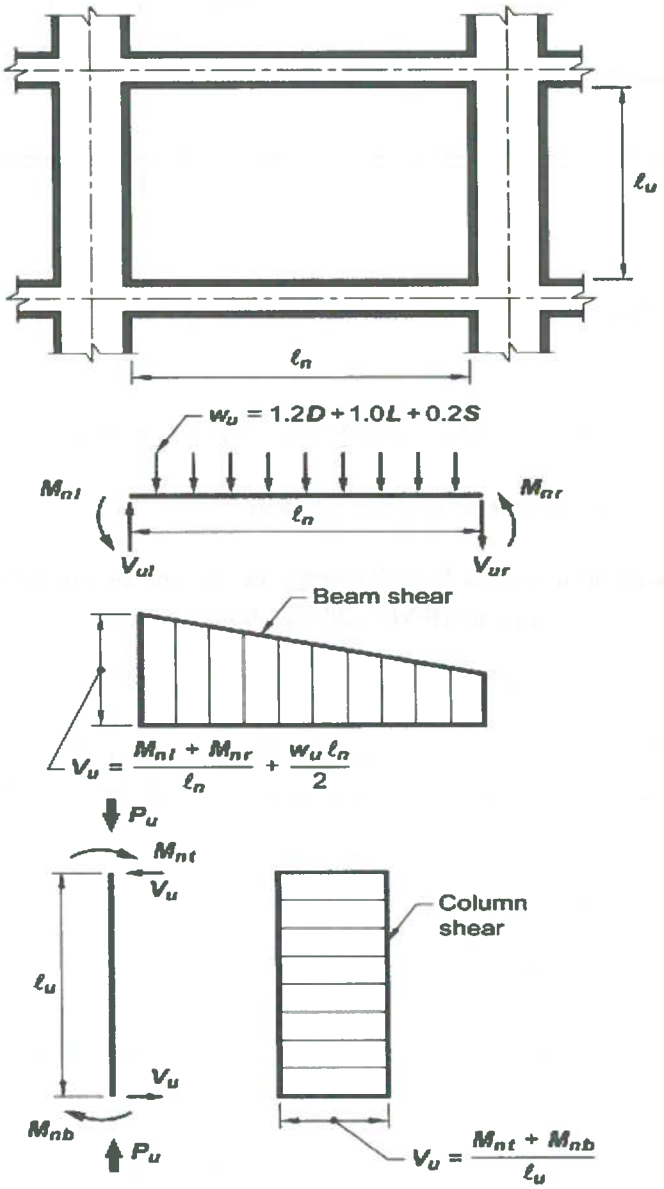


Figure 2.39 Design shear for intermediate moment frame[*After Ref.* 2.4]

2.12.6.3 Flexural Reinforcement Requirements: Frame Beams

a) Positive moment strength at joint face ≥ 1/3 negative moment strength at that face of the joint.(see Figure. 2.35)
b) Negative or positive moment strength at any section along the length of the beam ≥1/5 maximum moment strength provided at the face of either joint.

[BNBC 2015, Part 6 § 8.3.10.4(a)] [ACI 318-11, § 21.3.4.1]

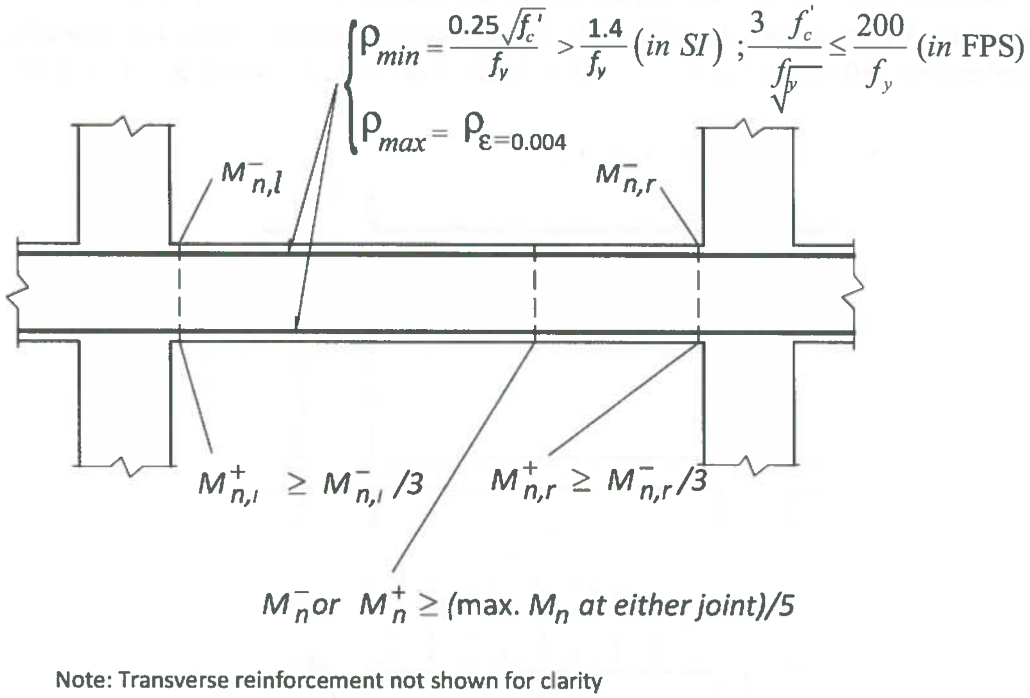


Figure 2.40 Flexural Reinforcement Requirements for Frame Beams of Intermediate Moment Frame (BNBC 2015, Figure 6.8.17)

2.12.6.4 Transverse Reinforcement Requirement: Frame Beams

Hoops are required over a length equal to twice the member depth from the face of the supporting member. Where hoops are required, the spacing shall not exceed (see Figure 2.41):

a) d/4
b) 8 × diameter of smallest longitudinal bar
c) 24 × diameter of hoop bars
d) 300mm(12in)

- The first hoop shall be located not more than 2 in. from the face of the supporting member.
- Where stirrups are used, the spacing shall be not more than *d*/2throughout the length of the beam.

[BNBC 2015, § 8.3.10.4(b) & 8.3.10.4(c)] [ACI 318-11, § 21.3.4.2 & 21.3.4.3]

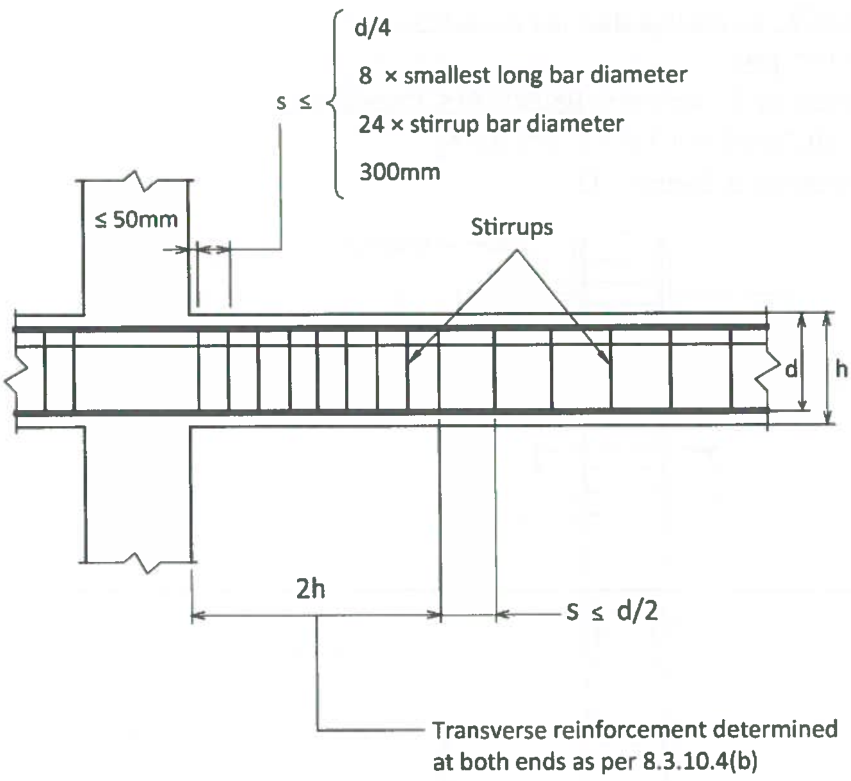


Figure 2.41 Transverse Reinforcement Requirements for Intermediate Moment Frame: Beams [BNBC 2015, Figure 6.8.18]

2.12.6.5 General Requirements: Frame Columns

If factored axial compressive force, $P_u > A_g f_c'/10$ then the frame member is considered as column. [BNBC 2015, § 8.3.10.2] [ACI 318-11, § 21.3.4.2 & 21.3.2]

2.12.6.6 Transverse Reinforcement: Frame Columns

- At both ends of the column, tie shall be provided at spacing s_o over a length l_o measured from the joint face. Spacing s_o shall not exceed the smallest of (a), (b), (c) & (d):
  - *a)* 8 × diameter of smallest longitudinal bar
  - *b)* 24 × hoop bar diameter
  - *c)* 1/2 of smallest cross-sectional dimension of the column
  - *d)* 300 mm (12 in.)

  [BNBC 2015, § 8.3.10.5(a)] [ACI 318-11, § 21.3.5.2]

- Length l_o shall not be less than the largest of (a), (b) & (c):
  - a) 1/6 of the clear span of the column
  - b) depth of the column
  - c) 450 mm (18 in.)

  [BNBC 2015, § 8.3.10.5(a)] [ACI 318-11, § 21.3.5.2]

- The first tie shall be located not more than $s_o/2$ from the joint face

[BNBC 2015, § 8.3.10.5(b)] [ACI 318-11, § 21.3.5.3]

- Outside the length l_o, tie spacing shall not exceed $2s_o$

[BNBC 2015, § 8.3.10.5 (d)]

- Joint reinforcement shall conform to BNBC 2015, Section 6.4.9

[BNBC 2015, § 8.3.10.5 (c)] [ACI 318-11, § 21.3.5.5]

These provisions are shown in Figure 2.42

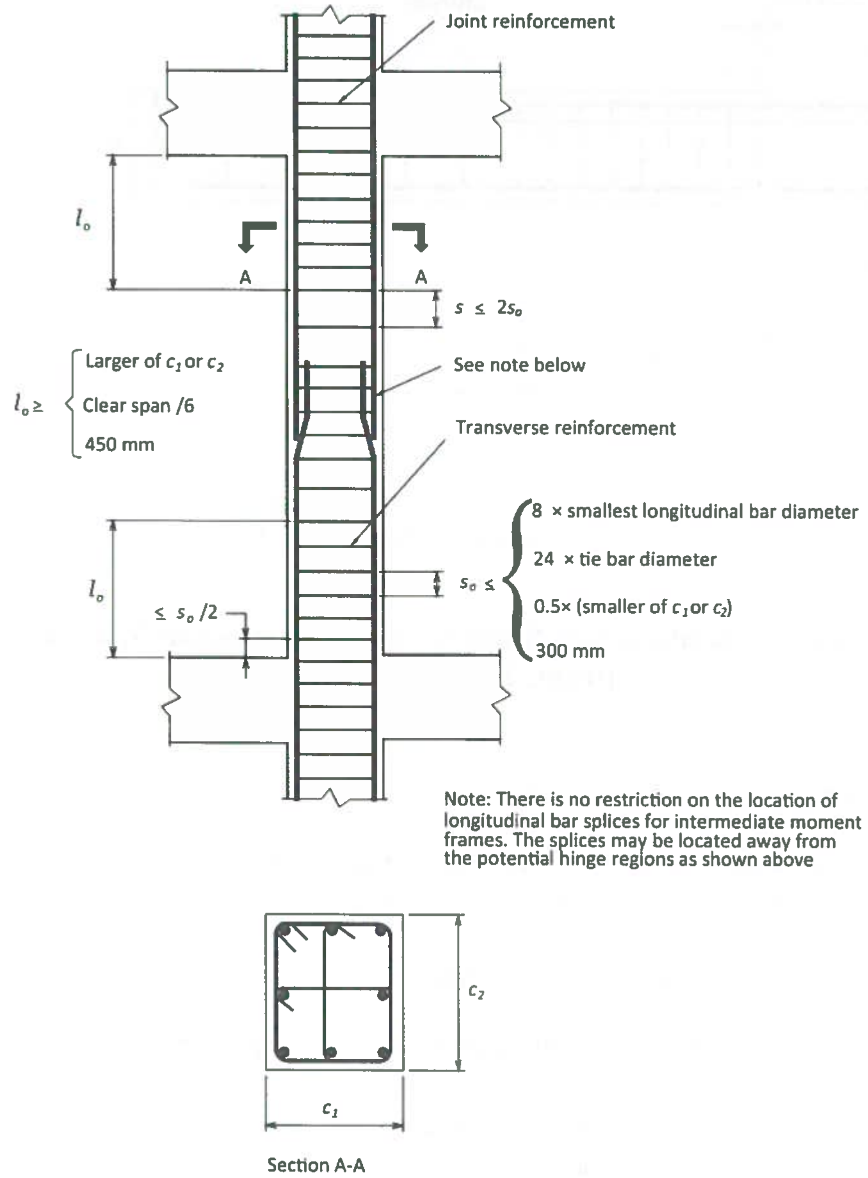


Figure 2.42 Transverse Reinforcement requirements for Frame Columns[BNBC 2015, Figure 6.8.19]

2.12.6.7 Two-way Slabs without Beams

a) Factored slab moment at support including earthquake effects, E shall be determined for load combinations given in *section 2.11.3*. Reinforcement provided to resist M_{slab} shall be placed within the column strip defined in BNBC 2015, §6.5.2.1 (ACI 318-11, § 13.2.1)(see Figure. 2.44).

b) A fraction of the unbalanced moment given by $\gamma_f M_u$ shall be considered to be transferred by flexure within an effective slab width between lines that are one and one-half slab or drop panel thickness

1.5*h* outside opposite faces of the column or capital (BNBC 2015, § 6.5.5.3.2) (ACI 318-11, § 13.5.3.2), where M_u is the factored moment to be transferred and

$$\gamma_f = \frac{1}{1+(2/3)\sqrt{b_1/b_2}} \tag{2.34}$$

c) The fractional part of the column strip moment, $\gamma_f M_{slab}$ shall be resisted by reinforcement placed within the effective width (Figure 2.43) specified in BNBC 2015, § 6.5.5.3.2(ACI 318-11, § 13.5.3.2). Effective slab width for exterior and corner connections shall not extend beyond the column face a distance greater than c_t measured perpendicular to the slab span (Figure 2.42).

d) Not less than one-half of the total reinforcement in the column strip at the support shall be placed within the effective slab width (Figure 2.43) specified in BNBC 2015,§6.5.5.3.2(ACI 318-11, § 13.5.3.2).

e) Not less than one-quarter of the top steel at the support in the column strip shall be continuous throughout the span (Figure 2.44).

f) Continuous bottom reinforcement in the column strip shall be not less than one-third of the top reinforcement at the support in the column strip (Figure 2.45).

g) Not less than one-half of all bottom reinforcement at mid span shall be continuous and shall develop its yield strength f_y at the face of support (Figure 2.47).

h) At discontinuous edges of the slab all top and bottom reinforcement at the support shall be developed at the face of the support (Figure 2.45 & 2.46).

[BNBC 2015, § 8.3.10.6] [ACI 318-11, § 21.3.6]

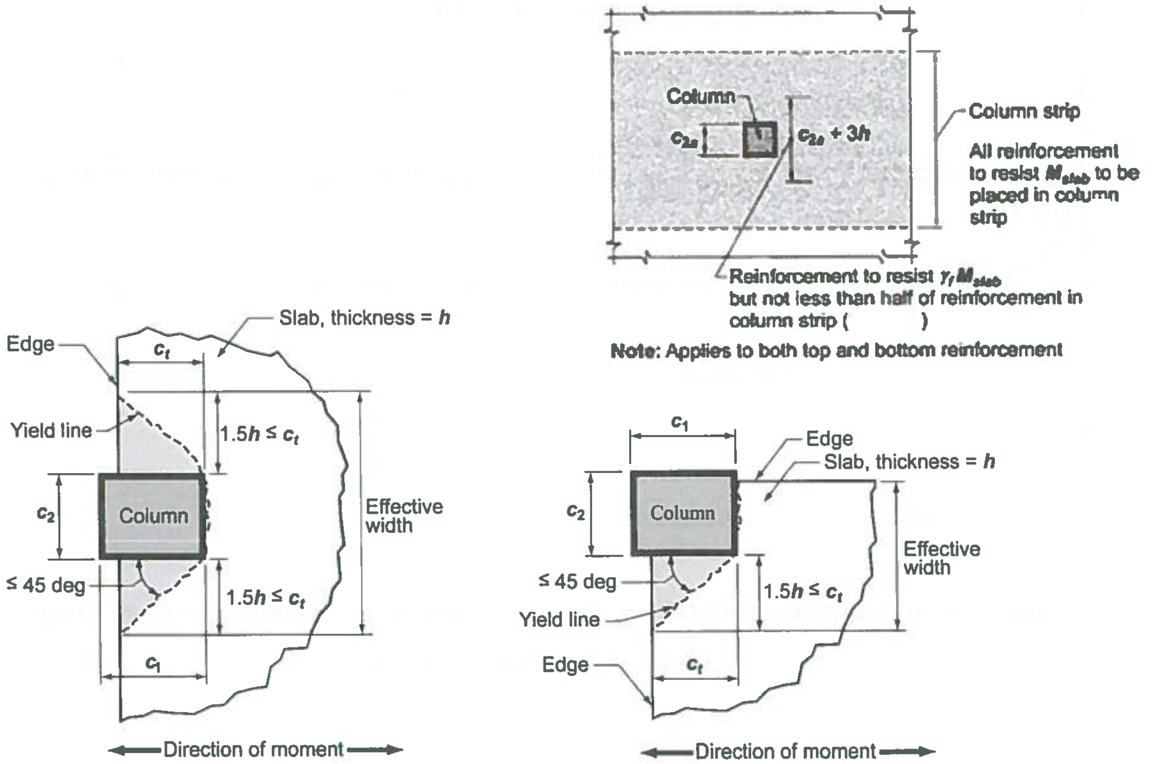


Figure 2.43 EffectiveWidth for Reinforcement Placement in Edge and Corner Connections (After Ref. 2.4 ACI 2011)

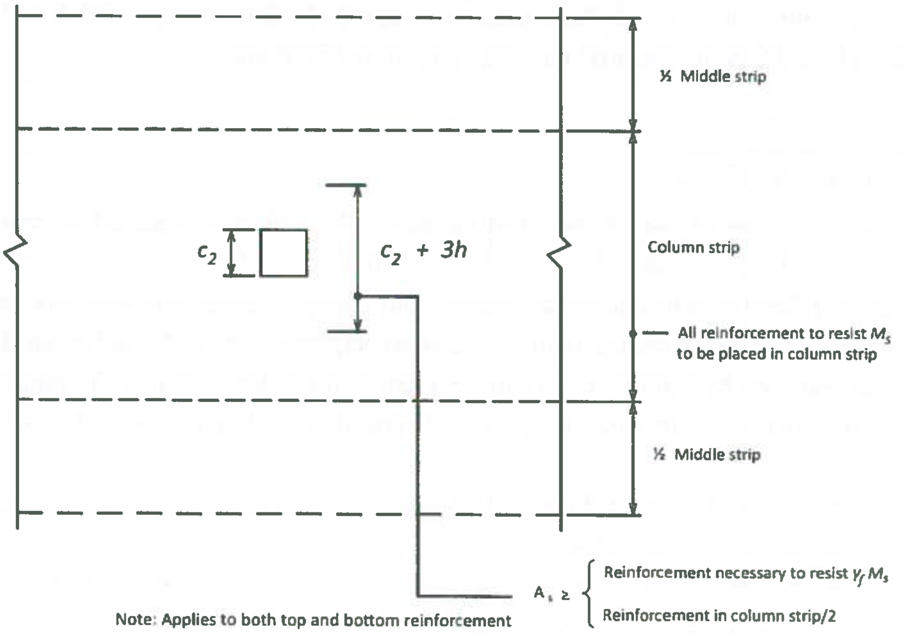


Figure 2.44 Location of Reinforcement in Slabs (BNBC 2015, Figure 6.8.20)

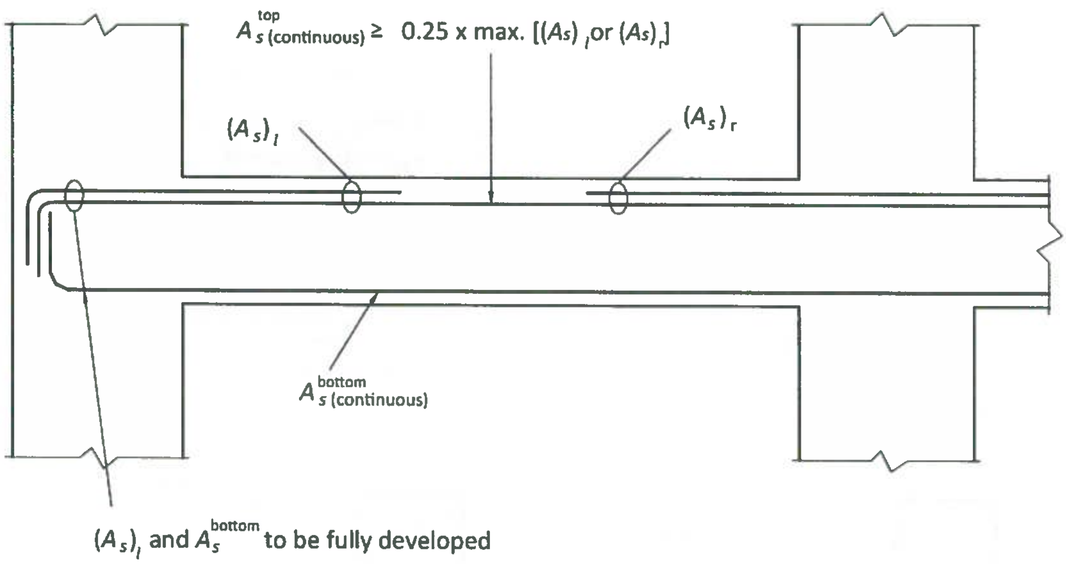


Figure 2.45 Reinforcement Details in Two-way Slabs without beams: Column Strip (BNBC 2015, Figure 6.8.21)

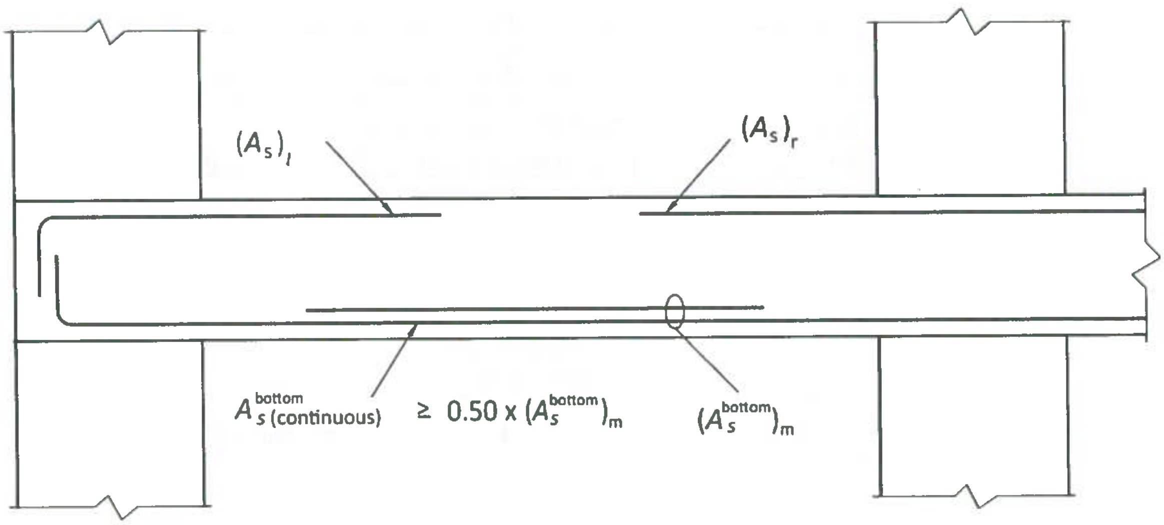
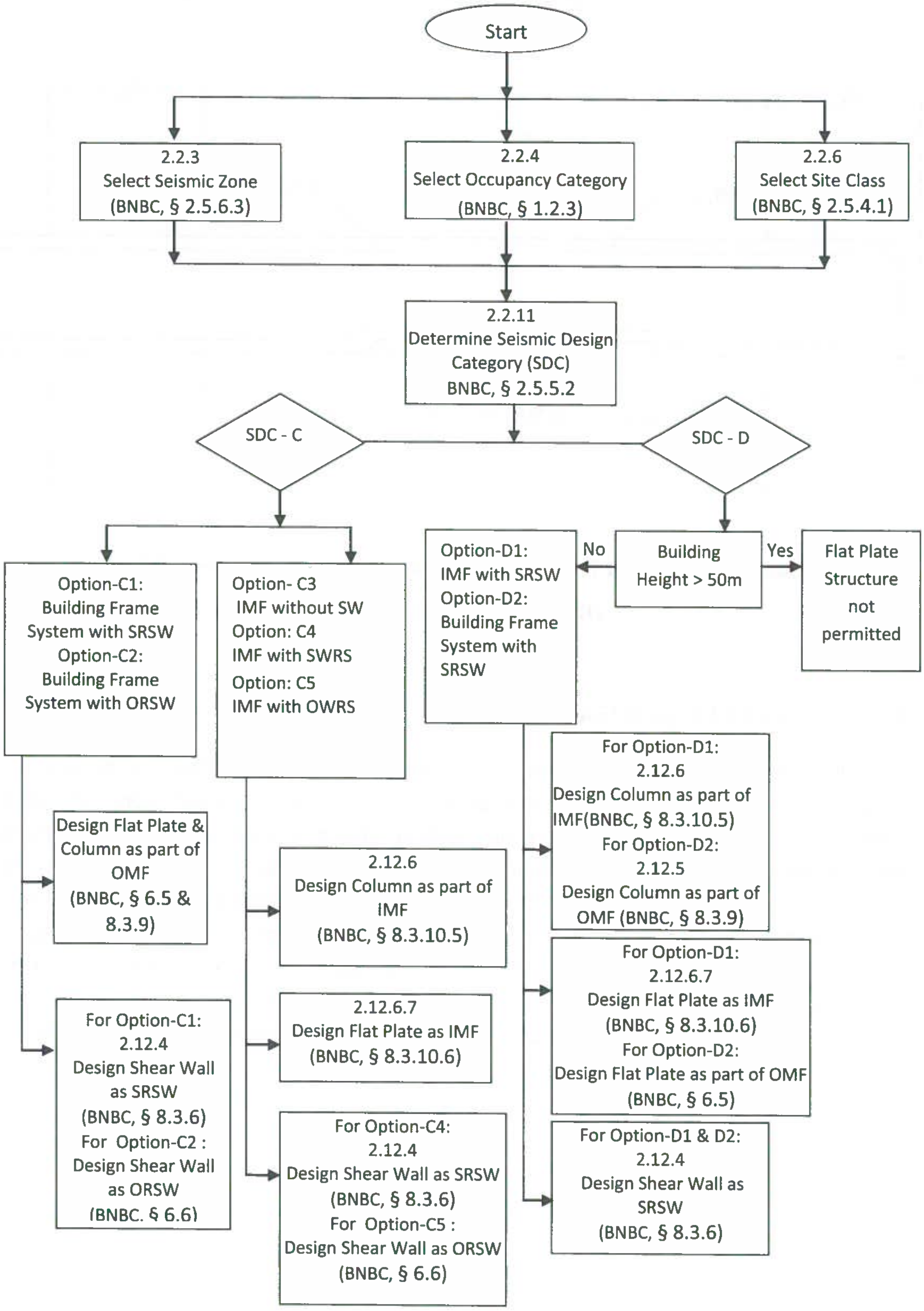


Figure 2.46 Reinforcement Details in Two-way Slabs without beams: Middle Strip (BNBC 2015, Figure 6.8.22)

2.12.6.8 Flow Chart for Design of Flat Plate Structure

Now-a-days flat plate structure is very common in Bangladesh. But most of them are being constructed in non-engineered way. When earthquake is a great concern, there are limited scopes in selection of lateral load resisting system. Flat plate structure without specially detailed shear wall performed as Intermediate Moment Frame, which may be done where Seismic Design Category (SDC) C is required. Also ordinary reinforced concrete shear wall may be added in Intermediate Moment Frame to reduce deflection of the structure. But where SDC – D is essential from seismic demand, flat plate structure is permitted with special reinforced concrete shear wall with intermediate moment frame and building height is limited to 50 m. When shear wall is a part of lateral load resisting system, moment frame shall be capable of resisting at least 25% of prescribed seismic force. Following flow chart may be used for design of a flat plate structure.

Flow Chart for Design of Flat Plate Structure



Note:

1. SRSW = Special Reinforced Concrete Shear Wall
2. ORSW = Ordinary Reinforced Concrete Shear Wall

Figure 2.47 Flow Chart for design of Flat Plate Structure

2.13 REQUIREMENT FOR MEMBERS NOT DESIGNATED AS PART OF THE SEISMIC - FORCE - RESISTING - SYSTEM [BNBC 2015 § 8.3.12]

2.13.1 Scope

(a) Requirements of sec 2.13 apply to frame members not designated as part of the seismic-force-resisting system in structures assigned to SDC D.

Frame members assumed not to contribute to lateral resistance, except two-way slabs without beams, shall be detailed according to Sec 2.13.2 or Sec 2.13.3 depending on the magnitude of moments induced in those members when subjected to the design displacement δ_u. If effects of δ_u are not explicitly checked, it shall be permitted to apply the requirements of Sec 2.13.3. For two-way slabs without beams, slab-column connections shall meet the requirements of Sec 2.13.5.

2.13.2 Induced Moment and Shear do not Exceed Design Capacities

Where the induced moments and shears under design displacements, δ_u, combined with the factored gravity moments and shears do not exceed the design moment and shear strength of the frame member, the conditions of Sections 2.13.2(a), 2.13.2(b), and 2.13.1.2(c) shall be satisfied. The gravity load combinations of ($1.2D + 1.0L + 0.2S$) or $0.9D$, whichever is critical, shall be used. The load factor on the live load, L, shall be permitted to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and all areas where L is greater than 4.8kN/m^2.

(a) Members with factored gravity axial forces not exceeding $\frac{A_g f_c'}{10}$ shall satisfy BNBC 2015 Sec 8.3.4.2(a).Stirrups shall be spaced not more than $\frac{d}{2}$ throughout the length of the member.

(b) Members with factored gravity axial forces exceeding $\frac{A_g f_c'}{10}$ shall satisfy BNBC 2015 Sections 8.3.5.3(a) and 8.3.5.4. The maximum longitudinal spacing of ties shall be s_o for the full member length. Spacing s_o shall not exceed the smaller of six diameters of the smallest longitudinal bar enclosed and 150 mm.

(c) Members with factored gravity axial forces exceeding $0.35P_o$ shall satisfy Sec 2.13.2(b). The amount of transverse reinforcement provided shall be one-half of that required by BNBC 2015 Sec 8.3.5.4(a) but shall not be spaced greater than s_o for the full member length.

2.13.3 Induced Moment or Shear Exceeds Design Capacities

If the induced moment or shear under design displacements, δ_u exceeds ϕM_n or ϕV_n of the frame ember, or if induced moments are not calculated, the conditions of Sec 2.13.3(a), (b) and (c) shall be satisfied.

(a) Materials shall satisfy BNBC 2015 § 8.3.3.3 and 8.3.3.4. Welded splices shall satisfy BNBC 2015 § 8.3.3.5.

(b) Members with factored gravity axial forces not exceeding $\frac{A_g f_c'}{10}$ shall satisfy BNBC 2015 Sections 8.3.4.2and 8.3.8. Stirrups shall be spaced at not more than $\frac{d}{2}$ throughout the length of the member.

(c) Members with factored gravity axial forces exceeding $\frac{A_g f_c'}{10}$ shall satisfy BNBC 2015 Sections 8.3.5.3, 8.3.5.4, 8.3.7.1 and 8.3.8.

2.13.4 Two-Way Slabs without Beams

For slab-column connections of two-way slabs without beams, slab shear reinforcement satisfying the requirements of BNBC 2015 Sections 6.4.10.3 and 6.4.10.5 and providing V_s not less than

$0.29\sqrt{f_c'}\, b_o d$ shall extend atleast four times the slab thickness from the face of thesupport, unless either (i) or (ii) is satisfied:

(i) The requirements of BNBC 2015 Sec 6.4.10.7 using the design shear V_{ug} and the induced moment transferredbetween the slab and column under the design displacement;

(ii) The design story drift ratio does not exceed the larger of 0.005 and $\left[0.035 - 0.05\left(\frac{V_{ug}}{\phi V_c}\right)\right]$.

Design story drift ratio shall be taken as the larger of the design story drift ratios of the adjacent stories above and below the slab-column connection. V_c is defined in BNBC 2015 Sec 6.4.10.2. V_{ug} is the factored shear force onthe slab critical section for two-way action, calculatedfor the load combination $1.2D + 1.0L + 0.2S$.

The load factor on the live load, L, shall be permitted to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and all areas where L is greater than 4.8 kN/m^2.

2.14 SEISMIC FORCE FOR NON-STRUCTURAL COMPONENTS

2.14.1 General

Non-structural components are permanently installed building items and BNBC 2015 gives the provisions for these in BNBC 2015 Part 6, § 2.5.18. To ensure life safety and preserve the functionality of essential facilities, these components should be designed to resist the lateral seismic forces. Seismic design for non-structural components also should be done with great importance.

Examples of architectural components are un-braced parapet walls on roof, interior partition walls, interior and exterior non bearing walls, retaining walls and penthouses. Mechanical/electrical components include water tanks and vessels with support systems; electrical, mechanical and plumbing equipment; and emergency communication equipment.
It is essential to consider the following facts in Bangladesh:
The out-of-plane mode of failure of infill walls has been widely observed in reinforced concrete frame structure after severe earthquakes (*See D-14 in Appendix-D of the Manual*). One of its countermeasures is shown at Sec. B-5 in Appendix-B of the Manual.

2.14.2 Seismic Design Force

Table. 6.2.22 or Table. 6.2.23 in BNBC 2015 Part 6, given a list of amplification factors (α_c) and response reduction factor (R_c). The seismic design force, F_c applied in the horizontal direction can be calculated by Equation 2.35:

$$F_c = \alpha_c\, a_h\, W_c\, I_c\,(1+2z/h)/R_c \qquad (2.35)$$

Where,
$0.75\, a_h W_c I_c \le F_c \le 1.5\, a_h\, W_c\, I_c$
α_c= components amplification factor which varies from 1.0 to 2.5
(See BNBC 2015, Tab. 6.2.22 or Tab. 6.2.23)
a_h = expected horizontal peak ground acceleration (in g) for design = 2/3×z×s
W_c = weight of component
R_c= component response reduction factor which varies from 1.0 to 12.0
(See BNBC 2015, Tab. 6.2.22 or Tab. 6.2.23)
I_c= component importance factor (See BNBC 2015 Part 6, §2.5.15.1)

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

h = roof height of structure above the base

The force F_c shall be independently applied in at least two orthogonal horizontal directions in combination with service loads associated with the component. In addition, the component shall also be designed for a concurrent vertical force of $\pm 0.5\ a_h W_c$.

The force F_c is used to design the members and connections that transfer the force into the seismic-resisting systems. (See BNBC 2015 Part 6, §2.5.15.3)

Where non-seismic loads on non-structural components exceed F_c such loads shall govern the strength design, but seismic detailing requirement and limitation shall apply. Drift, movement and other standards also apply.

2.14.3 A water tank as an example

A water tank laterally braced below the center of mass is assumed to be set on the roof of the structure built in Dhaka. In this case, seismic lateral static force is assumed to act at lateral center of gravity of the water tank and the frame is modeled by pinned support at base on roof.

According to Table 6.2.22 in BNBC 2015 Part 6, α_c and R_c are 2.5 and 3.0, respectively.

The seismic design force F_c is

$$F_c = \alpha_c\ a_h\ W_c\ I_c\ (1+2z/h)/\ R_c$$
$$= 2.5 \text{ x } 0.15 \times W_c \text{x } 1.0\ (1+2h_{roof}/h_{roof})/\ 3$$
$$= 0.38\ W_c$$

$$F_c \min = 0.75 a_h \times W_c \times I_c$$
$$= 0.75 \times 0.15 \times W_c \times I_c$$
$$= 0.11$$

Where,

$a_h = 2/3 \times 0.2 \text{ x } 1.15 \fallingdotseq 0.15$

$I_c = 1.0$

Such a component is popular in Japan, and the seismic co-efficient *"k"* is multiplied by component weight W_c.

The seismic design force F_c is

$F_c = 1.0\ W_c$

The ratio 1 to 0.38 is approximately 2.53 times that is between 2.5 and 3.0. This ratio is the same to the earthquake intensity ratio (1/2.5~1/3.0) assumed when seismic intensity is compared between Bangladesh and Japan. (See Figure 2.8 and Figure 2.9 in Chapter 2 of Part-II of the Manual).

2.15 SEISMIC DESIGN REQUIREMENT FOR NON-BUILDING STRUCTURES

Non-building structures include all self-supporting structures that carry gravity loads and that may be required to resist the effects of earthquake. Chimney, self-supported overhead water/fluid tank, silo, trussed tower, storage tank, cooling tower, monument etc. are the example of Non-building Structures. Non-building structures supported by the earth or supported by other structures shall be designed and detailed to resist the minimum lateral forces.

New BNBC 2015 (Ref. 2.1) has referred "Chapter 15 : Seismic Design Requirements for Non-Building Structures, Minimum Design Loads for building & Other Structures, ASCE Standard ASCE/SEI 7-05"(Ref. 2.3) for calculation of seismic design forces on non-building structures.

REFERENCES

2.1 Bangladesh National Building Code 2014

2.2 Bungale S. Taranath Ph.D., S.E (2005) "Wind and Earthquake Resistant Buildings: *Structural Analysis and Design", Marcel Dekker., New York, pp. 365.*

2.3 ASCE / SEI 7-05, "Minimum Design Loads for Buildings and Other Structures", ASCE, Reston, VA, 2005

2.4 (ACI 318-11)Building code requirements for structural concrete and commentary, American Concrete Institute, Farmington Hills, MI.

2.5 S.K. Ghosh and QiangShen (2008) "Seismic and wind design of concrete buildings", Portland Cement Association

2.6 PCA Notes 318-08 Building code requirements for structural concrete with design application, Portland Cement Association

CHAPTER 3. EXAMPLES BASED ON BNBC 2015

3.1 DESIGN OF A 6-STORIED ACADEMIC BUILDING

3.1.1 Building Information

- Building Location: Dhaka
- No of Storey: 6 stories
- Building Plinth Area: 690 m^2 (7431 ft^2) ; Total Area: 4140 m^2 (44586 ft^2)
- Total Height of The Building: 24.70 m (81 ft)
- Use of The Building: Institutional
- Pile foundation connected by grade beam
- Type of Structure: Moment Resisting Frame System without Shear wall.

The columns have constant, cross sections throughout the height of the building and the bases of the lowest storey are assumed fixed. The plan and elevation of the considered building are shown in the Figure 3.1, 3.2 and Figure 3.3.

Design example shown both for SI unit & FPS unit. Since professionals are familiar with FPS system, so analysis and design have been done for FPS unit and corresponding SI unit is shown.

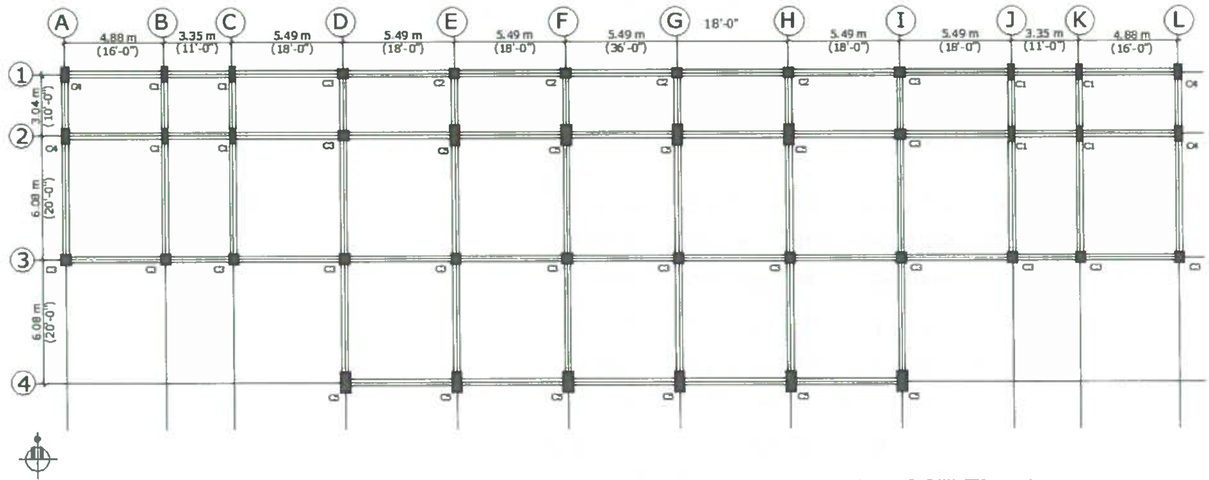


Figure 3.1 Plan of The Considered Building (Ground, 1st and 2nd Floor)

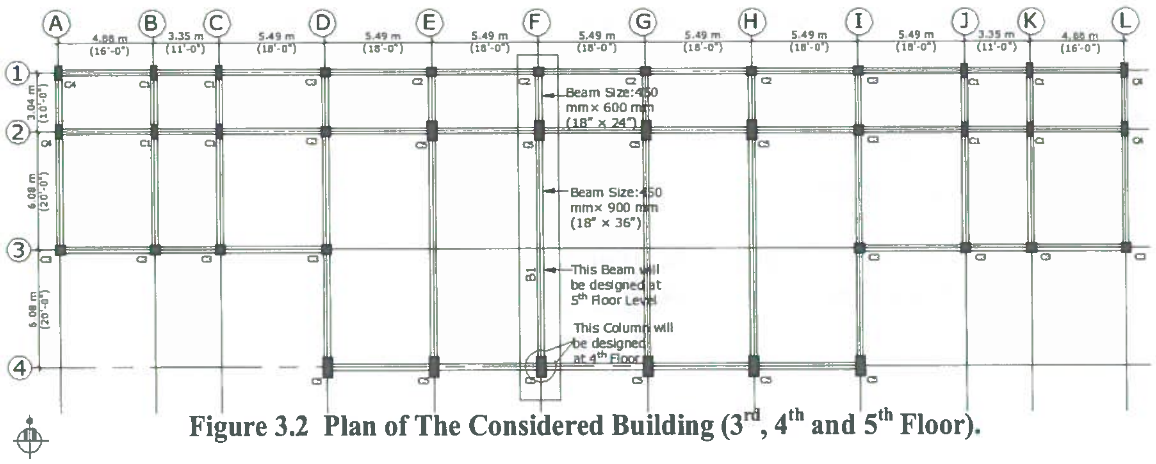


Figure 3.2 Plan of The Considered Building (3rd, 4th and 5th Floor).

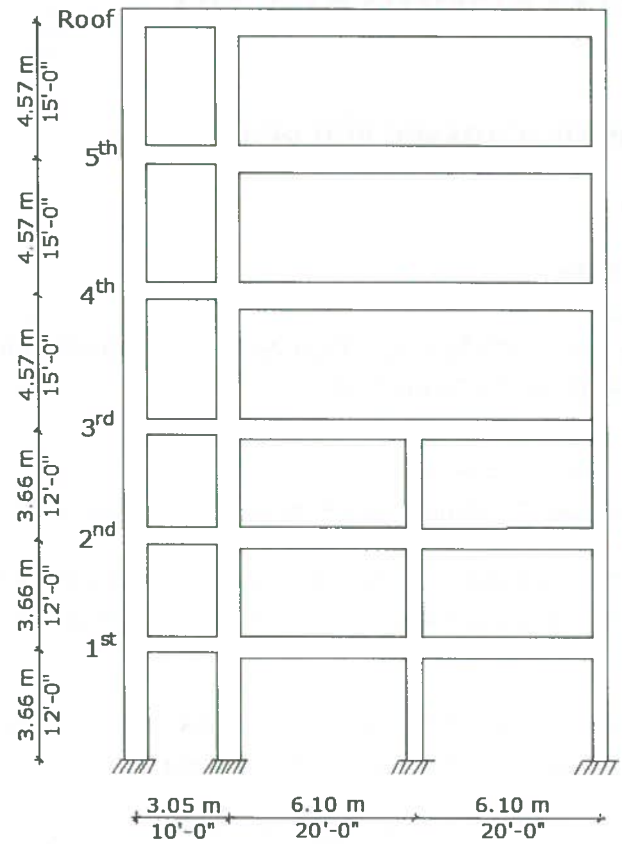


Figure 3.3 Elevation of The Considered Building (along Short Direction) [Grid F]

3.1.2 Member Dimensions

Columns: C1: 300 mm × 760 mm (12″× 30″)
C2: 500 mm × 500 mm (20″ × 20″)
C3: 500 mm × 500 mm (20″ × 20″)
C4: 380 mm × 760 mm (15″ × 30″)
C5: 500 mm × 1000 mm (20″ × 40″)
C6: 500 mm × 760 mm (20″ × 30″)
Beams: 450 mm × 900 mm (18″ × 36″) and
450 mm × 600 mm (18″ ×24″)

The stiffness properties of the members were input assuming cracked sections. The following cracked section properties were used:

- Beams: $I_{eff} = 0.5I_g$
- Columns: $I_{eff} = 0.7I_g$
- Shear Walls: $I_{eff} = 0.35I_g$

Where I_g is the gross moment of inertia of the section.
[BNBC 2015, § 6.3.10.4.1] [ACI 318-11, § 10.10.4.1]

3.1.3 Design Data

- Material Properties

Compressive strength of concrete: $f_c' = 25$ N/mm^2 (3,625 psi)

Yield strength of steel: $f_y = 400$ N/mm^2 (58,000 psi)

- Service Loads

  Live Loads: roof = 1 kN/m^2 (20 psf)

  Floor = 3 kN/m^2 (60 psf) average live load considered. (BNBC 2015, § 2.3.5)

3.1.4 Seismic Design Data

- Zone Coefficient, Z = 0.20 (BNBC 2015, Figure 6.2.24 & Table 6.2.15)
- Occupancy Category: III (BNBC 2015, Table 6.1.1)
- Importance factor, $I = 1.25$ (BNBC 2015, Table 6.2.17)
- Site Class based on Soil Investigation data SC (BNBC 2015, Table 6.2.13)
- Seismic Design Category C (BNBC 2015, Table 6.2.18)

3.1.5 Selection of Structural System

According to BNBC 2015 Table 6.2.19 for Seismic Design Category C, this building can be designed as intermediate reinforced concrete moment frames. But code allows it can be designed as special reinforced concrete moment frame. In this case, this building is designed as special reinforced concrete moment frame. For this structural system, response reduction factor, $R = 8$ and the deflection amplification factor, $C_d = 5.5$ which are given in BNBC 2015 Table 6.2.19.

3.1.6 Design Base Shear

The seismic design base shear force in a given direction is determined by BNBC 2015 Eq. 6.2.37:

$$V = S_a W \qquad (3.1)$$

Where, S_a is the Design spectral acceleration and W is the total dead load of the structure including partition walls, and applicable portions of other imposed loads as indicated in BNBC 2015, § 2.5.7.3.Total dead load of the building including partition walls = 49600 kN (11150 kip) and 25% of Live Load = 5783 kN (1300 kip).

In SI Units	In FPS Units
W = (49600 + 5783) kN= 55383 kN	W = (11150 + 1300) kip = 12450 kip

The building period is approximated by BNBC 2015 Equation 6.2.38

In SI unit	In FPS unit
Building height $h_n = 24.70$ m	Building height $h_n = 81$ ft.
$C_t = 0.0466$ (BNBC 2015 Table 6.2.20)	$C_t = 0.016$ (BNBC 2015 Table 6.2.20)
$m = 0.9$ (BNBC 2015 Table 6.2.20)	$m = 0.9$ (BNBC 2015 Table 6.2.20)
$T = C_t (h_n)^m$	$T = C_t (h_n)^m$
$= 0.0466 \times (24.7)^{0.9}$	$= 0.016 \times (81)^{0.9}$
$= 0.84$ sec.	$= 0.84$ sec.

For soil type SC, $S = 1.15, T_B = 0.2$ sec., $T_C = 0.6$ sec., $T_D = 2$ sec. (BNBC 2015 Table 6.2.16)

The normalized acceleration response spectrum C_s is determined by BNBC 2015 Equation 6.2.35c:

$$C_s = 2.5S\eta\left(\frac{T_C}{T}\right) \text{ for } T_C \leq T \leq T_D$$

Where, $S = 1.15$ (BNBC 2015 Table 6.2.16)

$\eta = 1$ for 5% viscous damping

$T_c = 0.6$ sec. (BNBC 2015 Table 6.2.16)

$T = 0.84$ sec.

$$C_s = 2.5 \times 1.15 \times 1 \times \left(\frac{0.6}{0.84}\right)$$

$= 2.05$

The design spectral acceleration S_a is calculated by BNBC 2015 Equation 6.2.34:

$$S_a = \frac{2}{3}\frac{ZIC_s}{R}$$

Seismic zone coefficient $Z = 0.2$
Structural importance factor, $I = 1.25$
Response reduction factor, $R = 8$

$$\therefore\ S_a = \frac{2}{3} \times \frac{0.2 \times 1.25}{8} \times 2.05$$

$= 0.043 \geq 0.67ZI\beta = 0.67 \times 0.2 \times 1.25 \times 0.15 = 0.03$[$\beta = 0.15$, BNBC 2015, § 2.5.4.3]

So, from equation (3.1)

In S_I Units	In FPS Units
$V = S_aW$	$V = S_aW$
$= 0.043 \times 55383$	$= 0.043 \times 12450$
$= 2382$ kN	$= 535$ kip

3.1.7 Vertical Distribution of Lateral Forces

The base shear V, shall be considered as the sum of lateral forces F_x induced at different floor levels. Part of base shear force induced at level x is calculated by BNBC 2015 Equation 6.2.41

$$F_x = V\frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad (3.2)$$

In SI Units

For $T = 0.84$ sec., $k = 1.17$

(BNBC 2015, § 2.5.7.4)

Base shear, $V = 2382$ kN

$$\sum_{i=1}^{n} w_i h_i^k$$

$= 9061 \times 3.66^{1.17} + 9061 \times 7.32^{1.17} + 10449 \times 10.97^{1.17} + 9804 \times 15.55^{1.17} + 10983 \times 20.12^{1.17} + 6027 \times 24.69^{1.17}$

$= 1174562$

Now Equation 3.2

$F_x = (2382/1174562) \times w_x h_x^{1.17}$

$= 0.002 \times w_x h_x^{1.17}$ (3.3)

In FPS Units

For $T = 0.84$ sec., $k = 1.17$

(BNBC 2015, § 2.5.7.4)

Base shear, $V = 535$ Kip

$$\sum_{i=1}^{n} w_i h_i^k$$

$= 2037 \times 12^{1.17} + 2037 \times 24^{1.17} + 2349 \times 36^{1.17} + 2204 \times 51^{1.17} + 2469 \times 66^{1.17} + 1355 \times 81^{1.17}$

$= 1059892$

Now Equation 3.2

$F_x = (535/1059892) \times w_x h_x^{1.17}$

$= 0.0005 \times w_x h_x^{1.17}$ (3.3)

From equation (3.3)

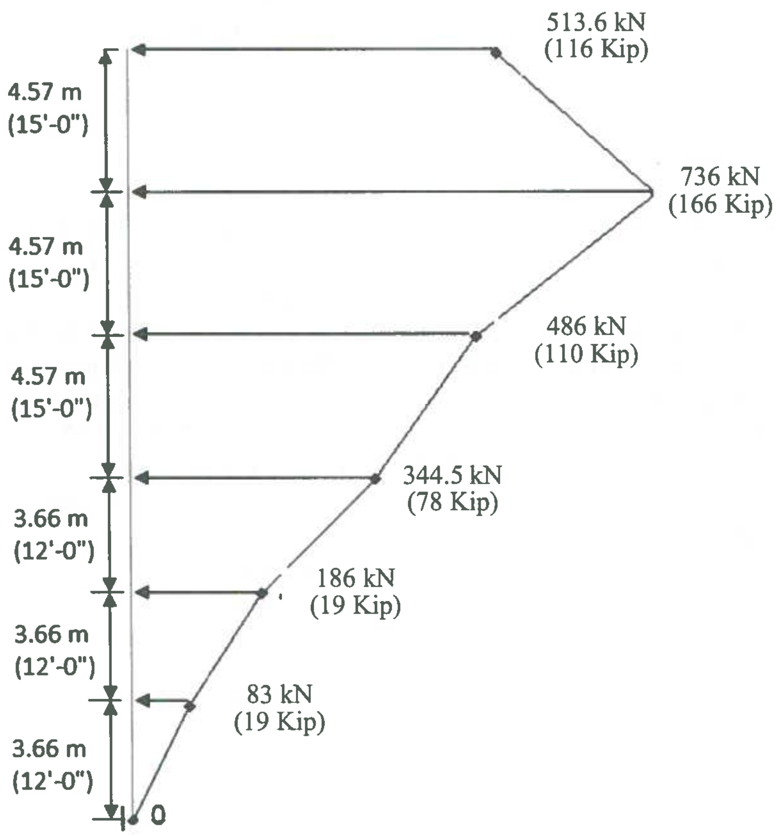


Figure 3.4 Base Shear Force at Different Floor Levels According to BNBC 2015

To compare the base shear of BNBC 2015 with that of BNBC 1993, total base shear in the same direction is determined by BNBC 1993 as follows:

Part of base shear induced at level x,

$$F_x = \frac{(V - F_t) w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$

The total design base shear in a given direction is determined by BNBC 1993 Eq. 2.5.1 (Part 6)

$$V = \frac{ZIC}{R} W$$

In SI Units

Seismic zone coefficient, $Z = 0.15$

Structure importance coefficient, $I = 1.0$

Response modification coefficient, $R = 12$

The total seismic dead load, $W = 49600$ kN

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

(BNBC1993 Eq. 2.5.2)

Site Coefficient, $S = 1.35$ [Average value of S2 and S3 is considered. Because soil profile is judged in between two.]

$T = C_t(h_n)^{3/4}$(BNBC1993 Eq. 2.5.3)

$C_t = 0.073$

$h_n = 24.7$ m

$T = C_t(h_n)^{3/4}$

$= 0.073 \times (24.7)^{3/4}$

$= 0.81$ sec.

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

$=1.25\times1.35/(0.81)^{2/3}$

$=1.94 < 2.75$

So, Base Shear, $V = (ZIC/R)W$

$=0.15\times1.0\times1.94\times49600/12$

$= 1203$ kN

For structural design this base shear shall be multiplied by a overload factor 1.4025 (i.e. 1.1×1.7×0.75)[according to load combination: 0.75(1.4DL+1.7LL+1.1×1.7 E)]

So, Design base shear = 1.4025 × 961= 1687 kN

$F_t = 0.07\, TV$ since T>0.7sec

$= 0.07 \times 0.81 \times 1687$

$= 96$ kN

In FPS Units

Seismic zone coefficient, $Z = 0.15$

Structure importance coefficient, $I = 1.0$

Response modification coefficient, $R = 12$

The total seismic dead load, $W = 11150$ Kip

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

(BNBC1993 Eq. 2.5.2)

Site Coefficient, $S = 1.35$ [Average value of S2 and S3 is considered. Because soil profile is judged in between two.]

$T = C_t(h_n)^{3/4}$(BNBC1993 Eq. 2.5.3)

$C_t = 0.03$

$h_n = 81$ ft

$T = C_t(h_n)^{3/4}$

$= 0.03 \times (81)^{3/4}$

$= 0.81$ sec.

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

$=1.25\times1.35/(0.81)^{2/3}$

$=1.94 < 2.75$

So, Base Shear, $V = (ZIC/R)W$

$=0.15\times1.0\times1.94\times11150/12$

$= 270$ Kip

For structural design this base shear shall be multiplied by a overload factor 1.4025 (i.e. 1.1×1.7×0.75)[according to load combination: 0.75(1.4DL+1.7LL+1.1×1.7 E)]

So, Design base shear = 1.4025 × 270= 379 kip

$F_t = 0.07TV$ since T>0.7sec

$= 0.07\times 0.81\times379$

$= 21.5$ kip

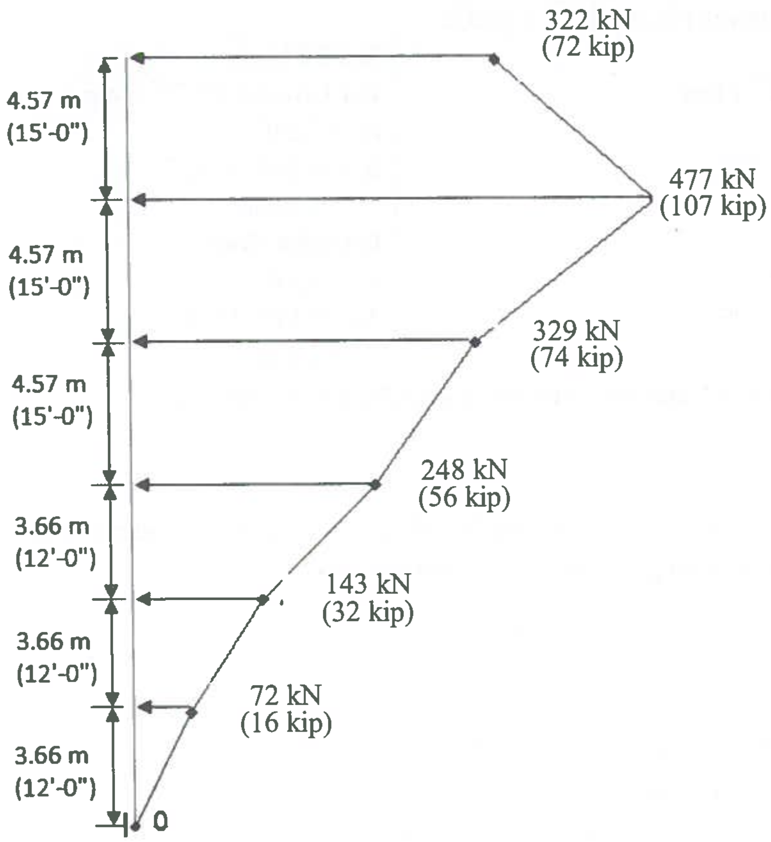


Figure 3.5 Base Shear Force at Different Floor Levels According to BNBC 1993

3.1.8 Storey Drift

The displacements δ_{xe} obtained from the three dimensional static, elastic analysis using the design seismic forces in the N-S direction are summarized in Table 3.1. The table also contains design earthquake displacement δ_x computed by BNBC 2015 Eq. 6.2.45:

$\delta_x = C_d \delta_{xe}/I$

Deflection amplification factor, $C_d = 5.5$ (BNBC 2015 Table 6.2.19)
Importance Factor, $I = 1.25$

Table 3.1 Lateral Displacement and Inter-storey Drifts due to Seismic Forces in N-S Direction

Storey	δ_{xe}	$\delta_x = C_d \delta_{xe}/I$	$\Delta = \delta_x - \delta_{x-1}$
6	31.2 mm (1.23 in.)	137.4 mm (5.41in.)	5.4 mm (0.61 in.)
5	27.7 mm (1.09 in.)	121.9 mm (4.80 in.)	24.6 mm (0.97 in.)
4	22.1 mm (0.87 in.)	97.3 mm (3.83 in.)	31.2 mm (1.23 in.)
3	15.0 mm (0.59 in.)	66.0 mm (2.60 in.)	23.6 mm (0.93 in.)
2	8.9 mm (0.38 in.)	42.4 mm (1.67 in.)	23.4 mm (0.92 in.)
1	4.3 mm (0.17 in)	19.0 mm (0.75 in)	9.0 mm (0.75 in)

The design storey drift at storey x shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the storey under consideration:

$\Delta = \delta_x - \delta_{x-1}$ (BNBC 2015 Eq. 6.2.46)

The design storey drift must not exceed the allowable storey drift Δ_a from BNBC 2015 Table 6.2.21. For Occupancy Category III, $\Delta_a = 0.015\ h_{sx}$

Where, h_{sx} is the the storey height below Level x.

In SI Units

For Ground, 1st, 2nd Floor

$h_{sx} = 3.66$ m

$\Delta_a = 0.015 \times 3.66 \times 1000$

$= 55$ mm

For other floors

Here, $h_{sx} = 4.57$ m

$\Delta_a = 0.015 \times 4.57 \times 1000$

$= 69$ mm

In FPS Units

For Ground, 1st, 2nd Floor

$h_{sx} = 12$ ft

$\Delta_a = 0.015 \times 12 \times 12$

$= 2.16$ in.

For other floors

$h_{sx} = 15$ ft

$\Delta_a = 0.015 \times 15 \times 12$

$= 2.7$ in.

It is evident from Table 3.1 that for all the stories, drifts are less than Δ_a.

3.1.9 P-Δ Effect

According to BNBC 2015, § 2.5.7.9, the P-delta effects need not be considered if the stability coefficient (θ) determined by the following equation is not more than 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \quad \text{(BNBC 2015 Eq. 6.2.48)}$$

Where,

P_x= the total vertical design load at and above level x

Δ= the design storey drift occurring simultaneously with V_x

V_x= the storey shear force acting between levels x and $x - 1$

h_{sx} = the storey height below level x

C_d= the deflection amplification factor

The stability coefficient (θ) shall not exceed θ_{max} determined as follows:

$$\theta_{\max} = \frac{0.5}{\beta C_d} \leq 0.25 \quad \text{(BNBC 2015 Eq. 6.2.49)}$$

Where, β = ratio of shear demand to shear capacity for the storey between levels x and $x - 1$. Conservatively $\beta = 1.0$.

In the N-S direction,

In SI Units

Level	h_{sx} (m)	P_x (kN)	V_x (kN)	Δ(mm)	θ
6	4.57	6027	514	15.4	0.007
5	4.57	17010	1250	24.6	0.013
4	4.57	26814	1736	31.2	0.02
3	3.66	37263	2080	23.6	0.02
2	3.66	46324	2266	23.4	0.02
1	3.66	55385	2349	9.0	0.02

In FPS Units

Level	h_{sx} (ft.)	P_x (kips)	V_x (kips)	Δ(in.)	θ
6	15	1355	116	0.61	0.007
5	15	3824	282	0.97	0.013
4	15	6028	392	1.23	0.02
3	12	8377	470	0.93	0.02
2	12	10414	512	0.92	0.02
1	12	12451	530	0.75	0.02

It is clear that P-delta effects need not be considered at any of the floor levels.

However, If $0.10 < \theta \leq \theta_{max}$, the incremental factor related to P-delta effects on displacement and member forces shall be increased by rational analysis or multiplied by a factor $1.0/(1 - \theta)$. (BNBC 2015, § 2.5.7.9)

Where θ is greater than θ_{max}, the structure is potentially unstable and shall be redesigned.

Where the P-delta effect is included in an automated analysis, BNBC 2015 Eq. 6.2.49 shall still be satisfied, however, the value of θ computed from BNBC 2015 Eq. 6.2.48 using the results of the P-delta analysis is permitted to be divided by $(1 + \theta)$ before checking BNBC 2015 Eq. 6.2.49.

3.1.10 Soft Storey

According to BNBC 2015, § 2.5.5.3.2, a soft storey is one in which the lateral stiffness is less than 70% of that in the storey above or less than 80% of the average lateral stiffness of the three stories above irregularity. An extreme soft storey is defined where its lateral stiffness is less than 60% of that in the storey above or less than 70% of the average lateral stiffness of the three stories above. This irregularity type is needed to be checked for building only assigned to SDC D.

However, this requirement will be omitted when no storey drift ratio under design lateral seismic force is greater than 130 percent of the storey drift ratio of the next storey above. Using the data in Table 3.1, check if the fourth storey satisfies the storey drift ratio requirement.

In SI Units	In FPS Units
$\left(\dfrac{\delta_{e5}-\delta_{e4}}{h_5}\right)=\left(\dfrac{24.6}{3.66\times1000}\right)=0.0068$	At 4th floor, $\left(\dfrac{\delta_{e5}-\delta_{e4}}{h_5}\right)=\left(\dfrac{1.23}{15\times12}\right)=0.0068$
$1.3\left(\dfrac{\delta_{e4}-\delta_{e3}}{h_4}\right)=1.3\left(\dfrac{23.6}{3.66\times1000}\right)=0.007$	At 3rd floor, $1.3\left(\dfrac{\delta_{e4}-\delta_{e3}}{h_4}\right)=1.3\left(\dfrac{0.97}{15\times12}\right)=0.007$
So, $\left(\dfrac{\delta_{e5}-\delta_{e4}}{h_5}\right)<1.3\dfrac{\delta_{e4}-\delta_{e3}}{h_4}$	So, $\left(\dfrac{\delta_{e5}-\delta_{e4}}{h_5}\right)<1.3\dfrac{\delta_{e4}-\delta_{e3}}{h_4}$

Thus, check for soft storey irregularity is not necessary. If soft storey exist then, designer will follow the provisions of BNBC 2015, § 2.5.17.

3.1.11 Load Combinations

Basic strength design load combinations are given in BNBC 2015, § 2.7.3.1

$U = 1.4D$ (3.4)

$U = 1.2D + 1.6L$ (3.5)

$U = 1.2D + 1.0E + 1.0L$ (3.6)

$U = 0.9D + 1.0E$ (3.7)

According to BNBC 2015 § 2.5.13, these load combination given below need to be considered

$U = (1.2\,D + E_v) + 1.0E + 1.0L$ (3.8)

$U = (0.9D - E_v) + 1.0E$ (3.9)

$U = (1.2\,D + E_v) + 1.0L + 1.0E\,(X) \pm 0.3E\,(Y)$ (3.10)

$U = (1.2\,D + E_v) + 1.0L + 1.0E\,(Y) \pm 0.3E\,(X)$ (3.11)

$U = (0.9D - E_v) + 1.0E\,(X) \pm 0.3E\,(Y)$ (3.12)

$U = (0.9D - E_v) + 1.0E\,(Y) \pm 0.3E\,(X)$ (3.13)

Where, D, L, E are the effects due to dead, live & seismic loads, respectively & the vertical seismic load effect E_v is calculated by BNBC 2015 Eq. 6.2.56:

$$E_v = 0.5(a_h)\,D$$

a_h = expected horizontal peak ground acceleration for design = $(2/3)ZS$

$= (2/3) \times 0.2 \times 1.15$

$= 0.15$

$E_v = 0.5(a_h)\,D$

$E_v = 0.5 \times 0.15 \times \mathrm{D}$

$E_v = 0.075D$

Substituting $E_v = 0.075D$ into the Equations 3.8, 3.9, 3.10, 3.11, 3.12 & 3.13

$U = 1.275D + 1.0E + 1.0L$ (3.14)

$U = 0.825D + 1.0E$ (3.15)

$U = 1.275D + 1.0L + 1.0E\,(X) \pm 0.3E\,(Y)$ (3.16)

$U = 1.275D + 1.0L + 1.0E\,(Y) \pm 0.3E\,(X)$ (3.17)

$U = 0.825D + 1.0E\,(X) \pm 0.3E\,(Y)$ (3.18)

$U = 0.825D + 1.0E\,(Y) \pm 0.3E\,(X)$ (3.19)

3.1.12 Beam Design

Beam F4 - F2 at 5th Floor Level

Negative Moment at 12.19 m (40′- 0″) span = 990 kN-m (730 k-ft) (Earthquake load governs)
Positive Moment at 12.19 m (40′- 0″) span = 678 kN-m (500 k-ft) (Gravity load governs)
Shear force at column face = 489 kN (110 kip) (Gravity load governs)

Check limitations on section dimensions as per BNBC 2015 8.3.4.1(ACI 318-11, § 21.5.1)

In SI Units	In FPS Units
Beam Size: 450 mm × 900 mm, Column Size: 500 mm× 1000 mm ▪ The factored axial compressive force on beams, which is negligible, is less than $A_g f_c'/10$. (O.K.) ▪ l_n/d = [(12.19×1000) – 1000]/825.5 = 13.54 > 4 (O.K.) ▪ width/depth = 450/900 = 0.5 > 0.3 (O.K.) width = 450 mm > 250 mm (O.K.)	Beam Size: 18 in. × 36 in, Column Size: 20 in. × 40 in. ▪ The factored axial compressive force on beams, which is negligible, is less than $A_g f_c'/10$. (O.K.) ▪ l_n/d = [(40×12) – 40]/32.5 = 13.54 > 4 (O.K.) ▪ width/depth = 18/36 = 0.5 > 0.3 (O.K.) width = 18″ > 10″ (O.K.)

Determination of flexural reinforcement:

Table 3.2 Required Reinforcement for Beam at 5th Floor Level (In SI unit)

Location	M_u (kN-m)	A_s (mm^2)	Reinforcement provided	φM_n^{**} (kN-m)
Exterior negative	-956	3415	7 – d25	958
Positive	678	2239	5 – d25	726
Interior negative	-990	3548	7 – d25 +1 – d20	1034

For 450 mm × 900 mm section

Maximum A_s = 0.025 × 450 × 825.5 = 9287 mm^2 BNBC 2015 § 8.3.4.2 (ACI 318-11, § 21.5.2.1)

Minimum $A_s = 0.25\sqrt{25} \times 450 \times 812.5/400 = 1143\,\text{mm}^2$ BNBC 2015 § 8.3.4.2 & BNBC 2015 Figure 6.8.2 (ACI 318-11, § 21.5.2.1) $= \dfrac{1.4}{400} \times 450 \times 812.5 = 1280\,\text{mm}^2$ (governs)

** Does not include slab reinforcement.

Table 3.3 Required Reinforcement for Beam on 5th Floor (In FPS Unit)

Location	M_u (k-ft)	A_s (in^2)	Reinforcement provided	φM_n^{**} (k-ft)
Exterior negative	-705	5.3	7 – d25	707
Positive	500	3.47	5 – d25	536
Interior negative	-730	5.5	7 – d25 + 1– d20	763

For 18″ × 36″ section

*Maximum $A_s = 0.025\times18\times32.5 = 14.63$ in^2 BNBC 2015 § 8.3.4.2 (ACI 318-11, § 21.5.2.1)

Minimum $A_s = 3\sqrt{3625}\times18\times32.5/58000 = 1.82$ in^2 BNBC 2015 § 8.3.4.2 & BNBC 2015 Figure 6.8.2

(ACI 318-11, § 21.5.2.1) $= \dfrac{200}{58000}\times18\times32.5 = 2.02$ in^2 (governs)

** Does not include slab reinforcement.

The flexural moment strength φM_n at each section is given in Table 3.2 & 3.3. In BNBC 2015 § 8.3.4.2 (ACI 318-11, § 21.5.2.2), it is stated that

- positive moment strength at joint face shall be not less than one-half the negative moment strength provided at that face of the joint. At exterior & interior negative location, this provision is satisfied.
- Again neither the negative nor the positive moment strength at any section along member length shall be less than one-fourth the maximum moment strength provided at face of either joint. Providing 2-d25 at any section will satisfy this requirement.
- However, to satisfy the minimum reinforcement requirement [Minimum A_s = 1268 mm^2 (1.95in^2)], a minimum of 3- d25 must be provided at any section. This also automatically satisfies the requirement that 2 bars will be continuous at both the top & bottom of any section.

When reinforcing bars extend through an interior joint, the column dimension parallel to the beam reinforcement must be at least 20 times the diameter of the largest longitudinal bar for normal weight concrete (BNBC 2015, § 8.3.7.1(d))(ACI 318-11, § 21.7.2.3). In this case, the minimum required column dimension = 20 × 25 = 500 mm (= 20 × 1 = 20 in.), which is less than the 1000 mm (40 in.) column width that is provided.

3.1.13 Shear Reinforcement Requirements

Shear requirements for beams in special moment frames are given in BNBC 2015 § 8.3.8 (ACI 318-11, § 21.5.4). Design for shear forces corresponding to end moments are obtained by assuming the stress in the tensile flexural reinforcement equal to $1.25f_y$ and a strength reduction factor φ=1.0 (probable flexural strength) plus factored tributary gravity loads.

$$M_{pr} = A_s(1.25f_y)(d - a/2)$$

Where, $$a = \frac{A_s(1.25f_y)}{0.85f_c'b}$$

In SI Units

For sidesway to the right,
At interior joint
For 7- d 25 + 1- d 20 top bar
$A_s = 3625$ mm^2
$a = A_s(1.25f_y)/0.85f_c'b$
$= 3625\times1.25\times400/(0.85\times25\times450)$
$= 190$ mm
$M_{pr} = A_s(1.25f_y)(d\text{-}a/2)$
$=3625\times1.25\times400\times(812.5\text{-}190/2)/1000/1000$
$= 1333$ kN-m

Similarly, at exterior joint the positive moment M_{pr} based on 5- d 25 bottom bars is equal to 891 kN-m.

For sidesway to the left,
At exterior joint
For 7-d25 top bar
$A_s = 3325$ mm^2
$a = A_s(1.25f_y)/0.85f_c'b$
$= 3325\times1.25\times400/(0.85\times25\times450)$
$= 190$ mm
$M_{pr} = A_s(1.25f_y)(d\text{-}a/2)$
$=3325\times1.25\times400\times(812.5\text{-}190/2)/1000/1000$
$= 1193$ kN-m

Similarly, for interior joint the positive moment M_{pr} based on 5-d25 bottom bars is equal to 891 kN-m.

Figure 3.6 shows the beam and the shear forces due to factored gravity loads plus probable moment strengths for sidesway to the right & for sidesway to the left. The equivalent factored uniform loads on the beam are determined manually or can be obtained from the structural analysis.

$w_D = 42$ kN/m
$w_L = 13.3$ kN/m
$w_u = 1.275w_D + 1.0w_L$
$=1.275\times42 + 1.0\times13.3 = 67$ kN/m

The maximum combined design shear force of 572 kN shown in fig. 3.6 is larger than the maximum shear force obtained from the structural analysis.

Shear strength is provided by both concrete (V_c) and reinforcing steel (V_s). However according to BNBC 2015 § 8.3.8.2 (ACI 318-11, § 21.5.4.2), V_c will be assumed to be zero if the earthquake induced shear force calculated in accordance with BNBC 2015 § 8.3.8.1(a)(ACI 318-11, § 21.5.4.1) is greater than or equal to 50% of the total design shear force and the factored axial compressive force including earthquake effects is less than $A_g f_c'/20$.

In this example, the beam carries negligible axial forces and the maximum earthquake induced shear force = (1333 + 891)/11.18 = 199 kN < 572/2 = 286 kN. Also check these requirements at a distance $2h = 2 \times 900 = 1800$ mm from the face of the support (plastic hinge length). The earthquake induced shear force at this location is equal to 199 kN (constant along span). The total shear force at 1800 mm = 1.8 m from the face of the support = 572 – (67 × 1.8) = 451.4 kN.

In FPS Units

For sidesway to the right,
At interior joint
For 7- d 25 + 1- d 20 top bar
$A_s = 5.8$ in.^{2}
$a = A_s(1.25f_y)/0.85f_c'b$
$= 5.8\times1.25\times58/(0.85\times3.625\times18)$
$= 7.58$ in.
$M_{pr} = A_s(1.25f_y)(d\text{-}a/2)$
$=5.8\times1.25\times58\times(32.5\text{-}7.58/2)/12$
$= 1006$ kip-ft.

Similarly, at exterior joint the positive moment M_{pr} based on 5-d25 bottom bars is equal to 689 kips-ft.

For sidesway to the left,
At exterior joint
For 7- d25 top bar
$A_s = 5.32$ in.^{2}
$a = A_s(1.25f_y)/0.85f_c'b$
$= 5.32\times1.25\times58/(0.85\times3.625\times18)$
$= 7$ in
$M_{pr} = A_s(1.25f_y)(d\text{-}a/2)$
$=5.32\times1.25\times58\times(32.5\text{-}7/2)/12$
$= 932$ kip-ft.

Similarly, for interior joint the positive moment M_{pr} based on 5-d25 bottom bars is equal to 689 kips-ft.

Figure 3.6 shows the beam and the shear forces due to factored gravity loads plus probable moment strengths for sidesway to the right & for sidesway to the left. The equivalent factored uniform loads on the beam are determined manually or can be obtained from the structural analysis.

$w_D = 2.86$ kips/ft
$w_L = 0.9$ kips/ft
$w_u = 1.275w_D + 1.0w_L$
$=1.275\times2.86 + 1.0\times0.9 = 4.6$ kips/ft

The maximum combined design shear force of 130.2 kips shown in fig. 3.6 is larger than the maximum shear force obtained from the structural analysis.

Shear strength is provided by both concrete (V_c) and reinforcing steel (V_s). However according to BNBC 2015 § 8.3.8.2 (ACI 318-11, § 21.5.4.2), V_c will be assumed to be zero if the earthquake induced shear force calculated in accordance with BNBC 2015 § 8.3.8.1(a)(ACI 318-11, § 21.5.4.1) is greater than or equal to 50% of the total design shear force and the factored axial compressive force including earthquake effects is less than $A_g f_c'/20$.

In this example, the beam carries negligible axial forces and the maximum earthquake induced shear force = (1006 + 689)/36.67 = 46.2 kips < 130.2/2 = 65.1 kips. Also check these requirements at a distance $2h = 2 \times 36 = 72$ in. from the face of the support (plastic hinge length). The earthquake induced shear force at this location is equal to 46.2 kips (constant along span). The total shear force at 72 in. = 6 ft. from the face of the support = 130.2 – (4.6 × 6) = 102.6

Therefore, at this location, 199 kN < 451.4/2 = 225.7 kN.
Thus, the Nominal shear strength provided by concrete V_c may be computed by BNBC 2015 Eq. 6.6.49 (ACI 318-11, Eq. 11-3):

$$V_c = 0.17\sqrt{f_c'}b_w d$$
$$= 0.17\sqrt{25} \times 450 \times 812.5/1000$$
$$= 311 \text{ kN}$$

Maximum shear force V_s is

$$\varphi V_s = V_u - \varphi V_c$$
$$\Rightarrow \quad V_s = V_u/\varphi - V_c$$
$$= 572/0.75 - 311$$
$$= 452 \text{ kN}$$
$$(V_s)_{max} = 0.68\sqrt{f_c'}b_w d$$
$$= 0.68\sqrt{25}\times450\times812.5/1000$$
$$= 1243 \text{ kN} > 452 \text{ kN (O.K.)}$$

kips.

Therefore, at this location, 46.2 kips < 102.6/2 = 51.3 kips. Thus, the Nominal shear strength provided by concrete V_c may be computed by BNBC 2015 Eq. 6.6.49(ACI 318-11, Eq. 11-3):

$$V_c = 2\sqrt{f_c'}b_w d$$
$$= 2\sqrt{3625}\times18\times32.5/1000$$
$$= 70 \text{ kip}$$

Maximum shear force V_s is

$$\varphi V_s = V_u - \varphi V_c$$
$$\Rightarrow \quad V_s = V_u/\varphi - V_c$$
$$= 130.2/0.75 - 70$$
$$= 103.6 \text{ kips}$$
$$(V_s)_{max} = 8\sqrt{f_c'}b_w d$$
$$= 8\sqrt{3625}\times18\times32.5/1000$$
$$= 281 \text{ kip} > 103.6 \text{ kips (O.K.)}$$

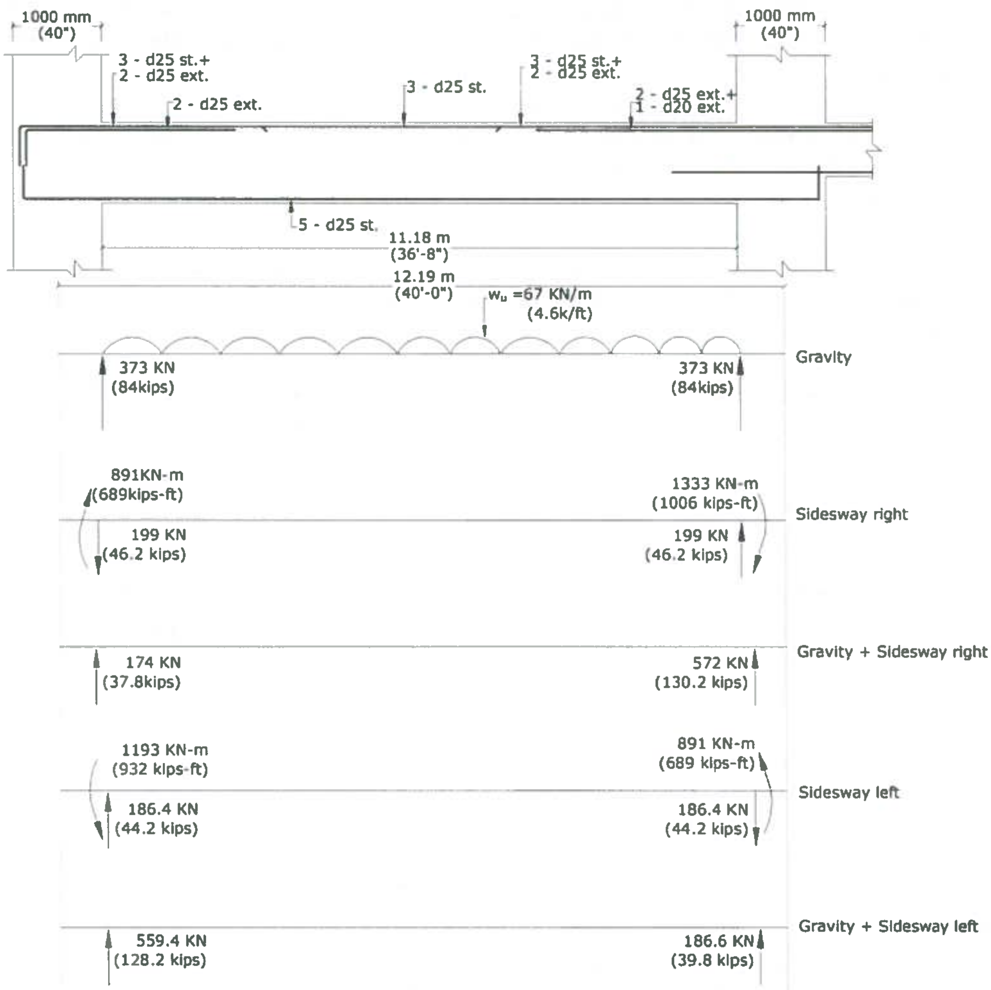


Figure 3.6 Design Shear Forces for Exterior Beam on 5th Floor

In SI Units

Required spacing of 4-leg d10 closed stirrups (hoops) for a shear force of 452 kN is determined by BNBC 2015 Equation 6.6.56(ACI 318-11, Eq. 11-15)

$s = A_v f_y d/V_s$

$= (4\times78)\times400\times812.5/(452\times1000)$

$= 224$ mm

Maximum allowable hoop spacing within a distance of $2h = 2 \times 900 = 1800$ mm from the face of the support at each end of the member is the smallest of the following (BNBC 2015 § 8.3.4.3(b)):

- $d/4 = 812.5/4 = 203$ mm
- $8 \times$ smallest longitudinal bar dia $= 8\times20$ $= 160$ mm (governs)
- $24 \times$ hoop dia $= 24\times10 = 240$ mm
- 300 mm

∴ 4-leg d10 hoop @ 150 mm c/c should be provided within $2h$ =1800 mm distance from the face of the column. The first hoop will be provided 50 mm from the face of the column.

Where hoops are not required, stirrups with seismic hooks at both ends may be used. At a distance of 1800 + 50 = 1850 mm from the face of the support :

$V_u = 572 - [67 \times (1800/1000)]$

$= 451.4$ kN

The nominal shear strength provided by concrete V_c may be computed by BNBC 2015 Equation 6.6.49(ACI 318-11, Eq. 11-3):

$$V_c = 0.17\sqrt{f_c'}\,b_w d$$
$$= 0.17\sqrt{25}\times450\times812.5/1000$$
$$= 311 \text{ kN}$$
$$\varphi V_s = V_u - \varphi V_c$$
$$\Rightarrow V_s = V_u/\varphi - V_c$$
$$= 451.4/0.75 - 311$$
$$= 291 \text{ kN}$$

Therefore the required spacing for 2-leg d10 stirrup is:

$s = A_v f_y d/V_s$ [BNBC 2015 Eq. 6.6.56]

$= (2\times78)\times400\times812.5/(291\times1000)$

$=175$ mm

$\leq A_v f_y/0.062\sqrt{f_c'}\,b_w$ [BNBC 2015, § 6.4.3.5.3] (ACI 318-11, § 11.4.6.3)

$= (2\times78)\times400/(0.062\times\sqrt{25}\times450)$

$= 447$ mm.

$\leq A_v f_y/0.35 b_w$[BNBC 2015, § 6.4.3.5.3] (ACI 318-11, § 11.4.6.3)

$= (2\times78)\times400/(0.35\times450)$

$= 396$ mm.

The maximum allowable spacing of the stirrups is $d/2 = 406.25$ mm (BNBC 2015 § 8.3.4.3(c)) (ACI 318-11, § 21.5.3.4).

here, s = 175 mm $< d/2 = 406.25$ mm

Provide 2-leg d10 @ 175 mm c/c within remaining portion of beam.

In FPS Units

Required spacing of 4-leg d10 closed stirrups (hoops) for a shear force of 103.6 kips is determined by BNBC 2015 Equation 6.6.56(ACI 318-11, Eq. 11-15)

$s = A_v f_y d/V_s$

$= (4\times0.12)\times58\times32.5/103.6$

$=8.7$ in.

Maximum allowable hoop spacing within a distance of $2h = 2\times36 = 72$ in. from the face of the support at each end of the member is the smallest of the following (BNBC 2015 § 8.3.4.3(b)):

- $d/4 = 32.5/4 = 8.125$ in.
- $8 \times$ smallest longitudinal bar dia $= 8\times0.78$ $= 6.24$ in. (governs)
- $24 \times$ hoop dia $= 24\times0.375 = 9$ in.
- 12 in.

∴ 4-leg d10 hoop @ 6 in. c/c should be provided within $2h$=72 in. distance from the face of the column. The first hoop will be provided 2 in. from the face of the column.

Where hoops are not required, stirrups with seismic hooks at both ends may be used. At a distance of 72 + 2 =74 in. from the face of the support :

$V_u = 130.2 - [4.6 \times (74/12)]$

$= 102$ kips

The nominal shear strength provided by concrete V_c may be computed by BNBC 2015 Equation 6.6.49(ACI 318-11, Eq. 11-3):

$$V_c = 2\sqrt{f_c'}\,b_w d$$
$$= 2\sqrt{3625}\times18\times32.5/1000$$
$$= 70 \text{ kip}$$
$$\varphi V_s = V_u - \varphi V_c$$
$$\Rightarrow V_s = V_u/\varphi - V_c$$
$$= 102/0.75 - 70$$
$$= 66 \text{ kips}$$

Therefore the required spacing for 2-leg d10 stirrup is:

$s = A_v f_y d/V_s$[BNBC 2015 Eq. 6.6.56 (ACI 318-11, Eq. 11-15)]

$= (2\times0.12)\times58\times32.5/66$

$=7$ in.

$\leq A_v f_y/0.75\sqrt{f_c'}\,b_w$ [BNBC 2015, § 6.4.3.5.3] (ACI 318-11, § 11.4.6.3)

$= (2\times0.12)\times58000/(0.75\times\sqrt{3625}\times18)$

$= 17$ in.

$\leq A_v f_y/50 b_w$[BNBC 2015, § 6.4.3.5.3] (ACI 318-11, § 11.4.6.3)

$= (2\times0.12)\times58000/(50\times18)$

$= 15.5$ in.

The maximum allowable spacing of the stirrups is $d/2 = 16.25$ in. (BNBC 2015 § 8.3.4.3(c)) (ACI 318-11, § 21.5.3.4).here, s = 7 in. $< d/2 = 16.25$ in.

Provide 2-leg d10 @ 7 in. c/c within remaining portion of beam.

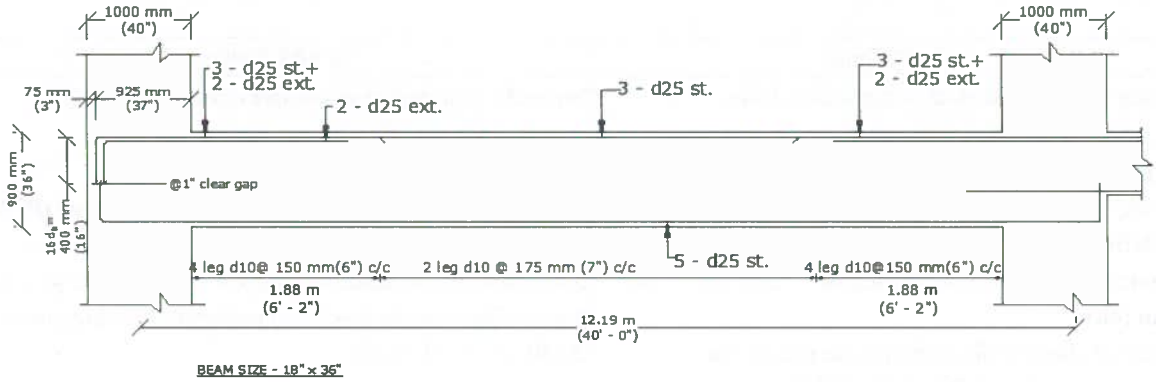


Figure 3.7 Reinforcement Detailing for Exterior Beam on 5th Floor

3.1.14 Required Length of Anchorage of Flexural Reinforcement at Exterior Column

Beam flexural reinforcement terminated in a column must be extend to the far face of the confined column core and must be anchored in tension and compression. Minimum development length l_{dh} for a bar with a standard 90^0 hook in normal weight concrete is the largest of (BNBC 2015 § 8.3.7.4) (ACI 318-11, § 21.7.5)

In SI Units

- 8×diameter of longitudinal bar = 8×25= 200mm
- 150 mm

$$l_{dh} = \frac{f_y d_b}{5.4\sqrt{f_c^{'}}} = 400\times25/(5.4\times\sqrt{25})$$

= 370 mm (governs)

Provided length = 925 mm > 370 mm (ok)

In FPS Units

- 8×diameter of longitudinal bar = 8×1 = 8 in.
- 6 in.

$$l_{dh} = \frac{f_y d_b}{65\sqrt{f_c^{'}}} = 58000\times1/(65\times\sqrt{3625})$$

= 14.8 in. (governs)

Provided length = 37 in. > 14.8 in. (ok)

3.1.15 Design of Column C5

A summary of the design axial forces, bending moments, and shear forces on column C5 in the 4th floor for gravity and seismic loads is given in Table 3.2

Table 3.4 Summary of Factored Axial Loads and Bending Moments for an Edge Column in The 4th floor for Seismic forces in Short Direction

In SI Units

Load Combination	Axial load, P_u (kN)	Bending Moment, M_u(kN-m)	Shear Force (kN)
1.4D	1140	- 360	-156
1.2 D + 1.6 L	1312	- 491	-205
1.275 D + 1.0 L + 1.0 E	1255 1188	480 - 598	-236
0.825 D +1.0 E	667 712	-369 274	-142
1.275 D + 1.0 L + 1.0 E (X / Z) + 0.3 E (Z / X)	1255 1188	480 -598	-236
0.825D + 1.0E (X/Z) + 0.3E (Z/X)	667 712	-369 274	-142

In FPS Units

Load Combination	Axial load, P_u(kips)	Bending Moment, M_u (kip-ft)	Shear Force (kips)
1.4D	256	-266	-35
1.2D + 1.6L	295	-362	-46
1.275D+1.0L+1.0E	282 267	354 -441	-53
0.825D +1.0E	150 160	-272 202	-32
1.275D + 1.0L + 1.0 E (X / Z) + 0.3 E (Z / X)	282 267	354 -441	-53
0.825 D + 1.0 E (X /Z) + 0.3 E (Z/X)	150 160	-272 202	-32

In SI Units	In FPS Units
Since the factored axial compressive force, P_u = 1312 kN > $A_g f_c'/10$ $A_g f_c'/10 = 500\times1000\times25/(10\times1000)$ = 1250 kN Thus, the following two criteria must be satisfied (BNBC 2015, § 8.3.5.1) (ACI 318-11, § 21.6.1) Shortest cross-sectional dimension = 500 mm > 300 mm (ok) Ratio of shortest dimension to perpendicular dimension = 500/1000 = 0.5 > 0.4 (ok)	Since the factored axial compressive force, P_u = 295 kips > $A_g f_c'/10$ $A_g f_c'/10 = 20\times40\times3.625/10$ = 290 kips Thus, the following two criteria must be satisfied (BNBC 2015, § 8.3.5.1) (ACI 318-11, § 21.6.1) Shortest cross-sectional dimension = 20 in. > 12 in. (ok) Ratio of shortest dimension to perpendicular dimension = 20/40 = 0.5 > 0.4 (ok)

3.1.16 Design for Axial Force and Bending

Provided longitudinal reinforcement in column is 10 – d25 + 10 – d20 (ρ_g = 1.55%) which is adequate for column C5 supporting 5th floor level. The interaction diagram for this column is shown in Figure 3.8 (for SI Unit) & Figure 3.9 (for FPS Unit). Also, the provided reinforcement ratio is within allowable range of 0.01 and 0.06 (BNBC 2015, § 8.3.5.3(a)) (ACI 318-11, § 21.6.3.1).

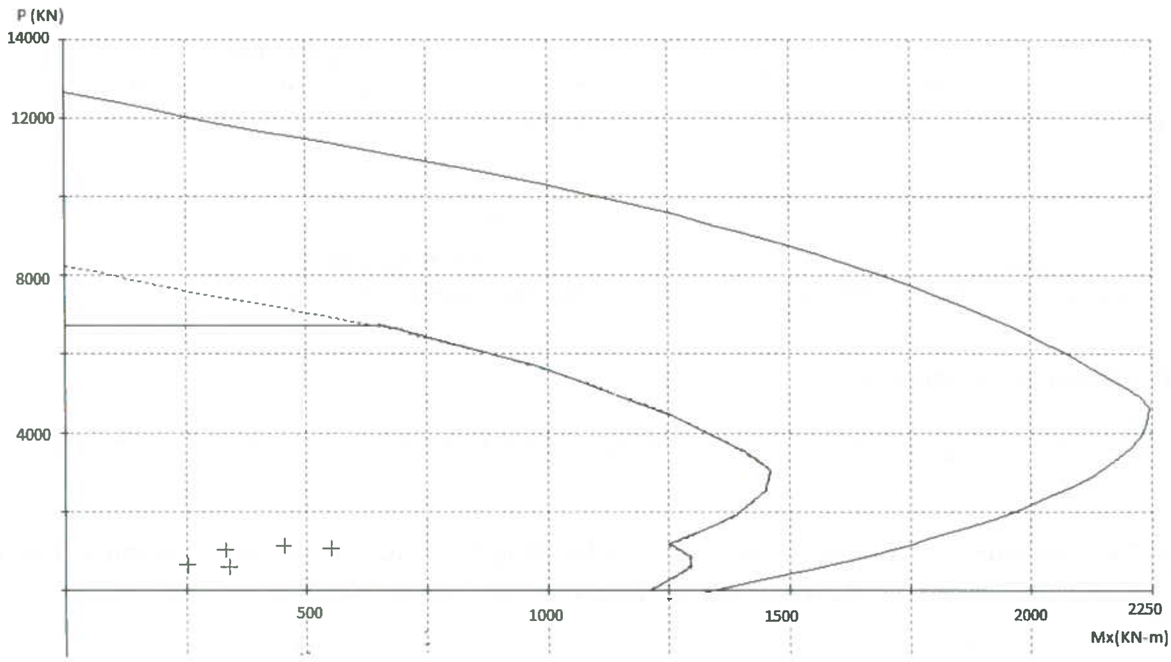


Figure 3.8 Design and Nominal Strength Interaction Diagram (In SI Unit) for Column C5 Supporting the 5th Floor Level

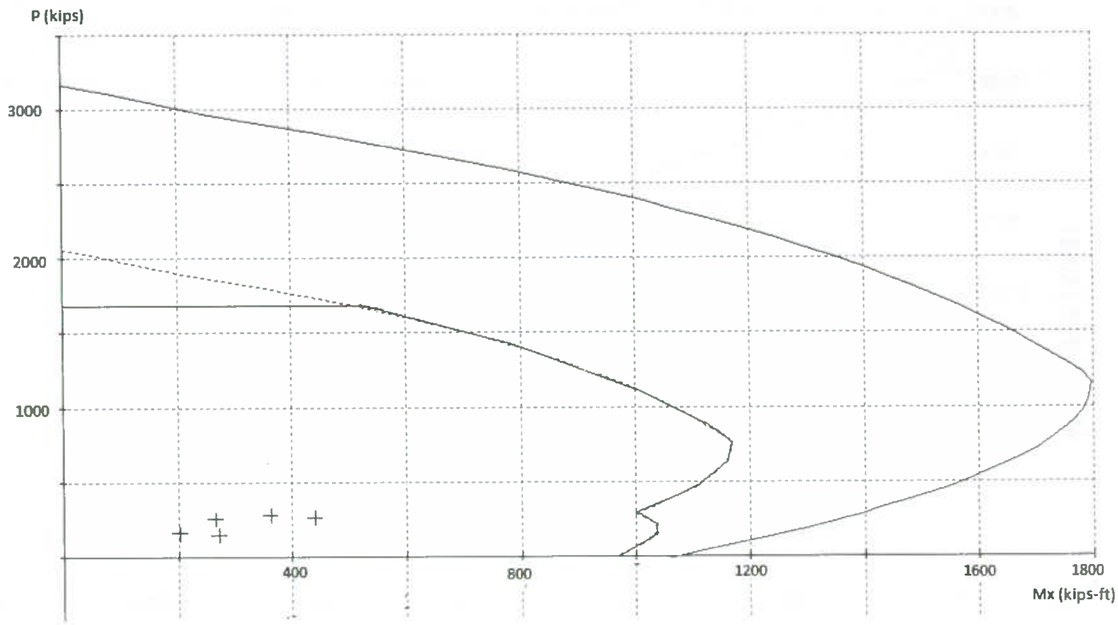


Figure 3.9 Design and Nominal Strength Interaction Diagram (In FPS Unit) for Column C5 Supporting the 5th Floor Level

3.1.17 Relative Flexural Strength of Columns and Beams

BNBC 2015, § 8.3.5.2(b) (ACI 318-11, § 21.6.2) requires that ΣM_c(columns) $\geq$ 1.2 ΣM_g(beams). The intent is to provide columns with sufficient strength so that they will not yield prior to the beams. Yielding at both ends of a column prior to the beams could result in total collapse of the structure. Only seismic load combinations need to be considered when checking the relative strengths of columns and beams.

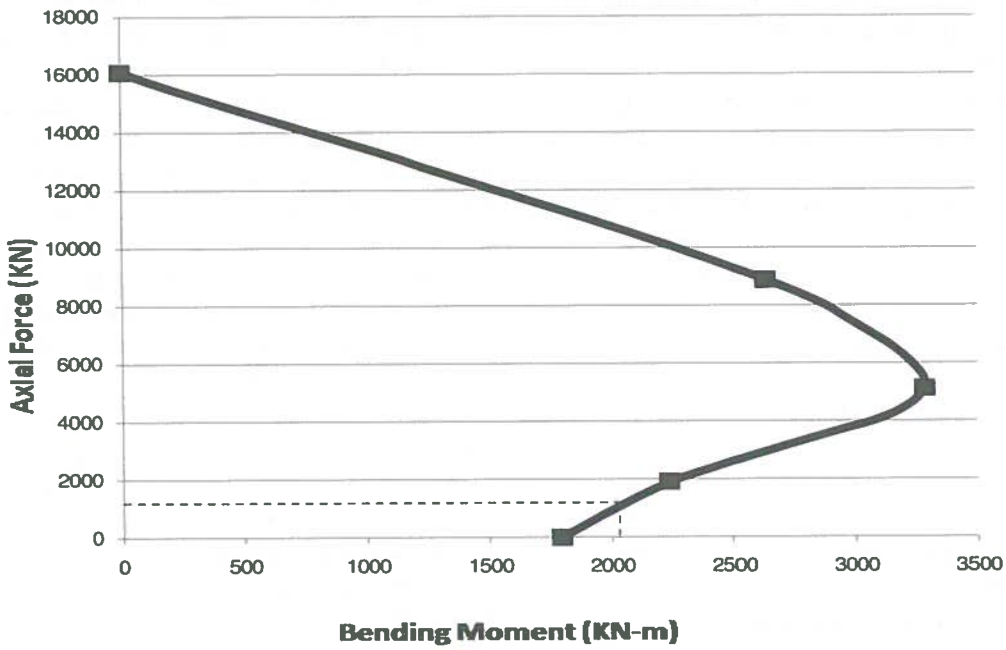


Figure 3.10 Design Strength Interaction Diagram (In SI Unit) for Column C5 with $1.25f_y = 1.25 \times 400 =$ 500 MPa and $\varphi = 1.0$

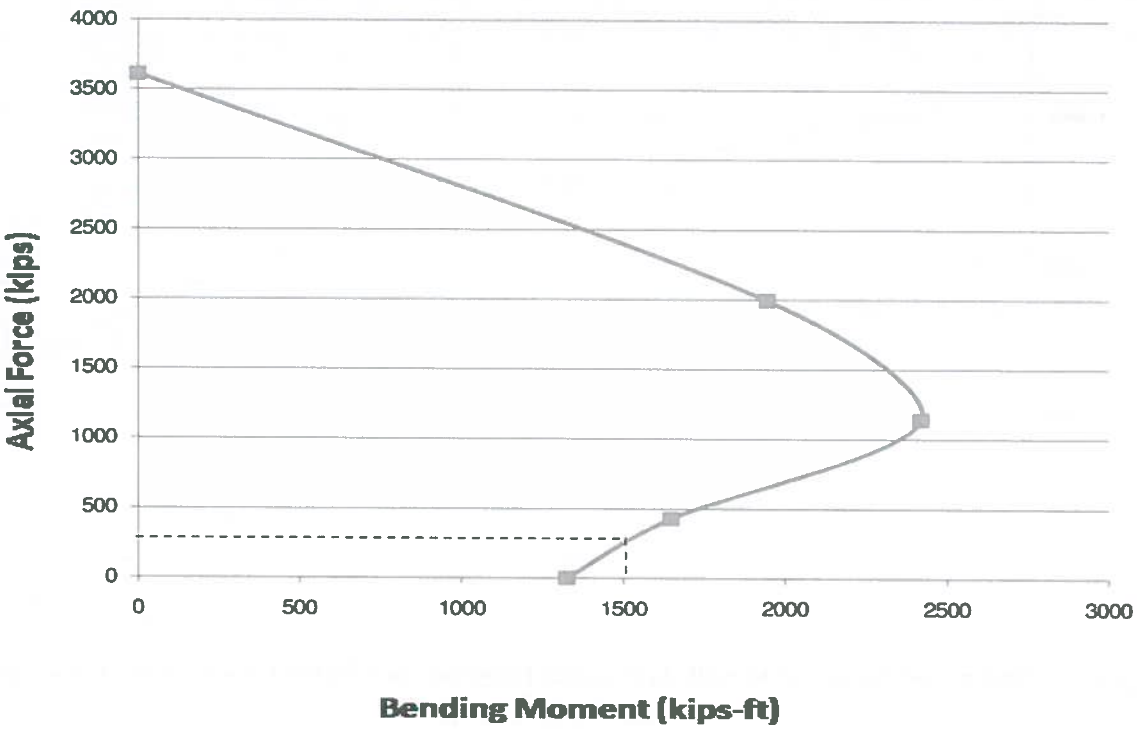


Figure 3.11 Design Strength Interaction Diagram (In FPS Unit) for Column C5 with $1.25f_y = 1.25\times58 = 72.5$ ksi and $\varphi = 1.0$

In SI Units	In FPS Units
At Interior Joint (Column line 2) Based on the reinforcement in beam F2-F4 (see Table 3.2), $M_{nb}^- = 1034/0.9 = 1149$ kN-m and $M_{nb}^+ = 726/0.9 = 807$ kN-m. Therefore, $\Sigma M_{nb} = 1149 + 807 = 1956$ kN-m Column flexural strength is determined for the factored axial force resulting in the lowest flexural strength, consistent with the direction of lateral forces considered. For the upper end of the lower column framing into the joint (i.e., the column supporting floor level 5), the minimum $M_{nc} = 1700$ kN-m, which corresponds to $P_u = 667$ kN (see fig. 3.8). Similarly, for the lower end of the upper column framing into the joint (i.e., the column supporting floor level 6), $M_{nc} = 1600$ kN-m, which corresponds to $P_u = 311$ kN. Therefore, $\Sigma M_{nc} = 3300$ kN-m. BNBC 2015 Eq. 6.8.5(ACI 318-11, Eq. 21-1): $\Sigma M_{nc} = 3300$ kN-m $> 1.2\,\Sigma M_{nb} = 1.2\times1956$ $= 2347$ kN-m (O.K.) **At Exterior Joint (Column line 4)** $M_{nb}^- = 958/0.9 = 1065$ kN-m and $M_{nb}^+ = 984/0.9 = 1093$ kN-m. $\Sigma M_{nb} = 1093$ kN-m (only moment M_{nb}^- is considered for exterior joint). $\Sigma M_{nc} = 3300$ kN-m $> 1.2\,\Sigma M_{nb}$	At Interior Joint (Column line 2) Based on the reinforcement in beam F2-F4 (see Table 3.3), $M_{nb}^- = 763/0.9 = 848$ kip-ft and $M_{nb}^+ = 536/0.9 = 596$ kip-ft. Therefore, $\Sigma M_{nb} = 848 + 596 = 1444$ kip-ft. Column flexural strength is determined for the factored axial force resulting in the lowest flexural strength, consistent with the direction of lateral forces considered. For the upper end of the lower column framing into the joint (i.e., the column supporting floor level 5), the minimum $M_{nc} = 1250$ kips-ft, which corresponds to $P_u = 150$ kips (see fig. 3.9). Similarly, for the lower end of the upper column framing into the joint (i.e., the column supporting floor level 6), $M_{nc} = 1175$ kips-ft, which corresponds to $P_u = 70$ kips. Therefore, $\Sigma M_{nc} = 2425$ kip-ft. BNBC 2015 Eq. 6.8.5(ACI 318-11, Eq. 21-1): $\Sigma M_{nc} = 2425$ kip-ft $> 1.2\,\Sigma M_{nb} = 1.2\times1444$ $= 1733$ kip-ft (O.K.) **At Exterior Joint (Column line 4)** $M_{nb}^- = 707/0.9 = 786$ kips-ft and $M_{nb}^+ = 726/0.9 = 807$ kips-ft. In this case, $\Sigma M_{nb} = 807$ kips-ft (only moment M_{nb}^- is considered for exterior joint). $\Sigma M_{nc} = 2425$ kip-ft. $\Sigma M_{nc} = 2425$ kip-ft $> 1.2\,\Sigma M_{nb}$

3.1.18 Transverse Reinforcement

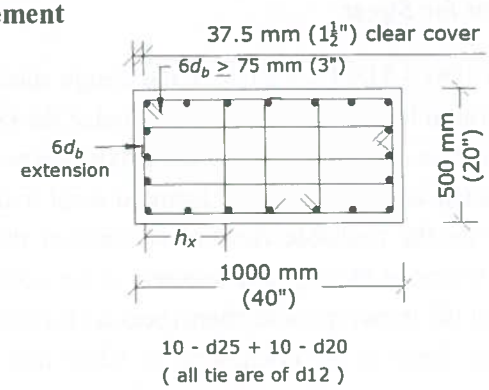


Figure 3.12 Typical Transverse Reinforcement Detailing for a Column C5 (see BNBC 2015 Figure 6.8.8)

Special transverse reinforcement for confinement is required over a distance l_o from each joint face at both column ends where l_o is equal to the largest of (BNBC 2015, § 8.3.5.4) (ACI 318-11, § 21.6.4.1)

In SI Units

- Depth of the member = 1000 mm(governs)
- Clear span/6 = [(4.57×1000 – 900]/6= 612 mm
- 450 mm

Transverse reinforcement within the distance l_o shall not be spaced greater than the smallest of (BNBC 2015, § 8.3.5.4) (ACI 318-11, § 21.6.4.3)

- Minimum member dimension/4 = 500/4= 125 mm
- 6 × diameter of smallest longitudinal bar = 6 × 20 = 120mm
- $s_x = 100 + [(350 - h_x)/3]$

(where 100 mm ≤ s_o ≤ 150 mm)

= 100 + [(350 – 329)/3]= 107 mm (governs)

where, h_x = maximum horizontal spacing of hoop or crosstie legs on all faces of the 500 mm × 1000 mm column

= [1000 – 2(37.5 + 12) – 25] (2/6) + 25 + 12 = 329 mm < 350 mm

The value of s_o shall not exceed 150 mm and need not be taken less than 100 mm.

Assuming d12 rectangular hoops with crossties around longitudinal bar @ 100 mm c/c spacing.

Minimum required cross-sectional area of rectangular hoop reinforcement A_{sh} is the largest value obtained by BNBC 2015 Eq. 6.8.7 & Eq. 6.8.8(ACI 318-11, § 21.6.4.4)

$$A_{sh} = 0.3\left(sh_c \, {f'_c}/{f_{yh}}\right)\left({A_g}/{A_{ch}} - 1\right)$$

= 0.3(100 × 913× 25/400) [(500000/393125)-1] = 465 mm²

$$A_{sh} = \frac{0.09 s h_c f'_c}{f_{yh}}$$

= 0.09 × 100 × 913 × 25/400= 514 mm² (governs)

where h_c = cross-sectional dimension of column core measured center-to-center of confinement reinforcement

= 1000 – 2[37.5 + 12/2] = 913 mm

A_g = gross area of column= 1000 × 500 = 500000 mm²

A_{ch} = cross-sectional area of member measured out-to-out of transverse reinforcement

= [1000 – (2 × 37.5)] × [500 – (2 × 37.5)] = 393125 mm²

Provided A_{sh} = 5 × 113 = 565 mm²> 514 mm² (O.K.)

In FPS Units

- Depth of the member = 40 in.(governs)
- Clear span/6 = [(15×12 – 36]/6= 24 in.
- 18 in.

Transverse reinforcement within the distance l_o shall not be spaced greater than the smallest of (BNBC 2015, § 8.3.5.4) (ACI 318-11, § 21.6.4.3)

- Minimum member dimension/4 = 20/4= 5 in.
- 6 × diameter of smallest longitudinal bar = 6 × 0.78 = 4.7 in.
- $s_x = 4 + [(14 - h_x)/3]$

(Where 4 in. ≤ s_o ≤ 6 in.)

= 4 + [(14 – 13)/3]= 4.33 in. (governs)

Where, h_x = maximum horizontal spacing of hoop or crosstie legs on all faces of the 20 in. × 40 in. column

= [40 – 2(1.5 + 0.5) – 1] (2/6) + 1 + 0.5 = 13 in. < 14 in.

The value of s_o shall not exceed 6 in. and need not be taken less than 4 in.

Assuming d12 rectangular hoops with crossties around longitudinal bar @ 4 in. c/c spacing.

Minimum required cross-sectional area of rectangular hoop reinforcement A_{sh} is the largest value obtained by BNBC 2015 Eq. 6.8.7 & Eq. 6.8.8(ACI 318-11, § 21.6.4.4)

$$A_{sh} = 0.3\left(sh_c \, {f'_c}/{f_{yh}}\right)\left({A_g}/{A_{ch}} - 1\right)$$

=0.3(4×36.8×3.625/58)[(800/629)-1]= 0.74 in²

$$A_{sh} = \frac{0.09 s h_c f'_c}{f_{yh}}$$

= 0.09 × 4 × 36.8 × 3.625/58= 0.83 in² (governs)

where h_c = cross-sectional dimension of column core measured center-to-center of confinement reinforcement

= 40 – 2[1.5 + 0.5/2] = 36.5 in.

A_g = gross area of column = 40 × 20 = 800 in²

A_{ch} = cross-sectional area of member measured out-to-out of transverse reinforcement

= [40 – (2 × 1.5)] × [20 – (2 × 1.5)]= 629 in²

Provided A_{sh} =5 × 0.175 = 0.875 in²> 0.83 in² (O.K.)

3.1.19 Transverse Reinforcement for Shear

According to BNBC 2015, § 8.3.8.1(ACI 318-11, § 21.6.5), the design shear force, V_e shall be determined from the consideration of the maximum forces that can be generated at the faces of the joints at each end of the member. These joint forces shall be determined using the maximum probable moment strengths, M_{pr}, at each end of the member associated with the range of factored axial loads, P_u, acting on the member. Shear forces are computed based on the probable flexural strength of the column at the base and the probable flexural strengths of the beams at the top. The member shear need not exceed those determined from joint strengths based on M_{pr} of the transverse members (beams) framing into the joint. It is important to note that in no case is the shear force in the column to be taken less than the factored shear force determined from analysis of the structure under the code-prescribed seismic forces. Sidesway to the right and to the left must be considered when calculating the maximum design shear forces.

In SI Units

The design strength interaction diagram for column C3 with $\varphi = 1.0$ & $f_y = 1.25 \times 400 = 500$ N/mm^2 is shown in fig. 3.10. At the base of the column the largest M_{pr} is equal to 2015 kN-m, which corresponds to an axial load equal to 1255 kN (see Table 3.4).

According to BNBC 2015, § 8.3.8.1(ACI 318-11, § 21.6.5), The shear forces at the top of the column need not exceed those determined from joint strengths based on the probable flexural strength M_{pr} of the transverse members (beam) framing into the joint.

For seismic forces in short direction, the negative probable flexural strength of the beam framing into the joint at the face of the edge column is 1193 kN-m.

Distribution of this moment to the columns is proportional to EZ/L of the columns above & below the joint.

So the moment at the top of 4th storey column is

$1193 \times [4.57/(4.57 + 4.57)] = 596.5$ kN-m

Thus design shear force is

$V_u = (596.5 + 596.5)/4.57 = 261$ kN

Which is greater than that obtained from analysis.

($V_u = 236$ kN)

Since the factored compressive force including earthquake effects, $N_u = 667$ kN $> 0.05A_gf_c' = [0.05 \times (1000 \times 500) \times 25/1000] = 625$ kN, the shear strength of the concrete may be used (BNBC 2015, § 8.3.8.2) (ACI 318-11, § 21.6.5.2). The shear capacity of the column is checked in accordance with BNBC 2015 Eq. (6.6.50) (ACI 318-11, Eq. 11-4):

$$V_c = 0.17(1 + \frac{N_u}{14A_g})\sqrt{f_c'}b_wd$$

$$= 0.17(1 + \frac{667000}{14 \times (500 \times 1000)})\sqrt{25} \times 500 \times 937.5 / 1000$$

$= 398$ kN

$\varphi V_c = 0.75 \times 427.6 = 320.7$ kN $> V_u = 261$ kN

Use d12 rectangular hoops with crossties around longitudinal bar @ 100 mm c/c spacing.

In FPS Units

The design strength interaction diagram for column C3 with $\varphi = 1.0$ & $f_y = 1.25 \times 58 = 72.5$ Ksi is shown in fig. 3.10. At the base of the column the largest M_{pr} is equal to 1500 kips-ft, which corresponds to an axial load equal to 282 kip (see Table 3.4)

According to BNBC 2015, § 8.3.8.1(ACI 318-11, § 21.6.5),

The shear forces at the top of the column need not exceed those determined from joint strengths based on the probable flexural strength M_{pr} of the transverse members (beam) framing into the joint.

For seismic forces in short direction, the negative probable flexural strength of the beam framing into the joint at the face of the edge column is 932 kips-ft.

Distribution of this moment to the columns is proportional to EZ/L of the columns above & below the joint.

So the moment at the top of 4th storey column is

$932 \times [15/(15 + 15)] = 466$ kips-ft

Thus design shear force is

$V_u = (466 + 466)/15 = 62$ kips

Which is greater than that obtained from analysis.

($V_u = 53$ kip)

Since the factored compressive force including earthquake effects, $N_u = 150$ kips (where $N_u = 150$ kips is the smallest axial force (see Table 3.2)) $> 0.05A_gf_c' = 0.05 \times (40 \times 20) \times 3.625 = 145$ kips, the shear strength of the concrete may be used (BNBC 2015, § 8.3.8.2) (ACI 318-11, § 21.6.5.2). The shear capacity of the column is checked in accordance with BNBC 2015 Eq. (6.6.50) (ACI 318-11, Eq. 11-4):

$$V_c = 2\sqrt{f_c'}bd(1 + \frac{N_u}{2000A_g})$$

$$= \frac{2\sqrt{3625} \times 20 \times 37.5}{1000} \times (1 + \frac{150000}{2000 \times (40 \times 20)}) = 99 \text{ kip}$$

$\varphi V_c = 0.75 \times 99 = 74$ kip $> V_u = 62$ kip

Use d12 rectangular hoops with crossties around longitudinal bar @ 4 in. c/c spacing.

3.1.20 Exterior Beam – Column Connection

Since an exterior joint is confined on less than four side, transverse hoop reinforcement within joint is the same as that over length l_o = 1000 mm (40 in.) (BNBC 2015, § 8.3.7.2) (ACI 318-11, § 21.7.3)

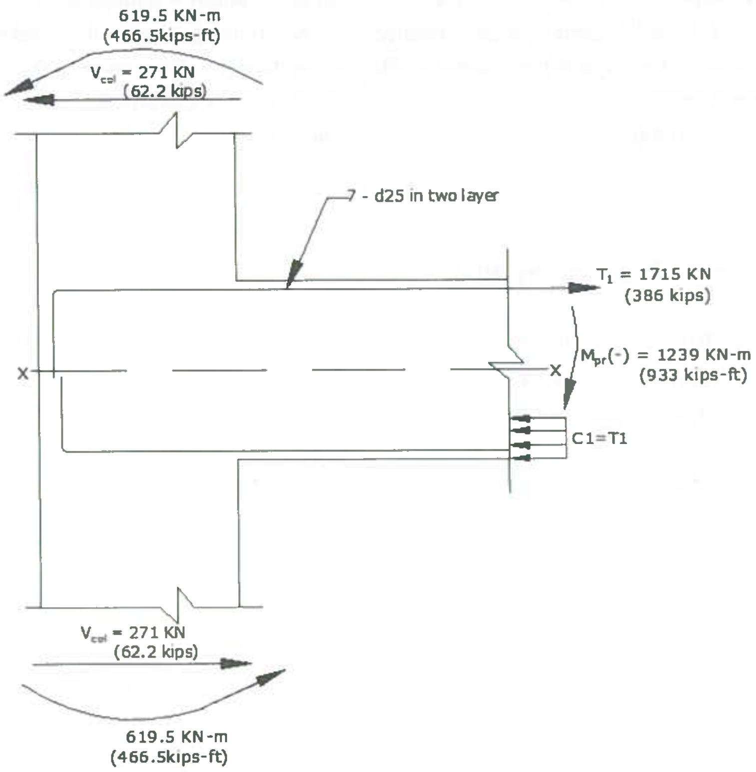


Figure 3.13 Shear & Moment of an Exterior Beam-Column Joint

To check the shear strength of the joint, the shear force acting on the joint must be calculated based on a stress of $1.25f_y$ in the flexural reinforcement.

The tensile force in the negative steel is

In SI Units	In FPS Units
$T_l = A_s(1.25f_y) = 7\times490\times1.25\times400/1000 = 1715$ kN	$T_l = A_s(1.25f_y) = 7\times0.76\times1.25\times58 = 386$ Kips
$a = 1715\times1000/(0.85\times25\times450) = 180$ mm	$a = 386/(0.85\times3.625\times18) = 7$ in.
$M^- = A_s(1.25f_y)(d - a/2) = 1715(812.5\text{-}180/2)/1000 = 1239$ kN-m	$M^- = A_s(1.25f_y)(d - a/2) = 386(32.5 - 7.0/2)/12 = 933$ kip-ft
Horizontal shear from column, V_{col} can be obtained by assuring that the beams in the adjoining floors are also deformed so that plastic hinges form at their junctions with the column, with M_{pr}(beam) = 1239 kN-m	Horizontal shear from column, V_{col} can be obtained by assuring that the beams in the adjoining floors are also deformed so that plastic hinges form at their junctions with the column, with M_{pr}(beam) = 933 ft-kips.
$V_{col} = 1239/4.57 = 271$ kN	$V_{col} = 933/15 = 62.2$ kips
Thus the net shear at section X – X of the joint is $V_u = T_l - V_{col} = 1715\text{-}271 = 1444$ kN	Thus the net shear at section X – X of the joint is $V_u = T_l - V_{col} = 386\text{-}62.2 = 323.8$ kips
For a joint not confined on four faces and all these beams' width not equal to at least 75% the column width nominal shear strength, φV_c is determined in accordance with BNBC 2015, § 8.3.7.3(ACI 318-11, § 21.7.4)	For a joint not confined on four faces and all these beams' width not equal to at least 75% the column width nominal shear strength, φV_c is determined in accordance with BNBC 2015, § 8.3.7.3(ACI 318-11, § 21.7.4)
$\phi V_c = 1.0\phi\sqrt{f_c'}A_j$	$\phi V_c = 12\phi\sqrt{f_c'}A_j$
$= 1.0\times0.85\times\sqrt{25}\times500000/1000$	$= 12\times0.85\times\sqrt{3625}\times800/1000$
= 2125 kN	= 491 kips
$\Phi V_c = 2125$ kN $> V_u = 1444$ kN (O.K.)	$\Phi V_c = 491$ kips $> V_u = 323.8$ kips (O.K.)
A_j = effective cross-sectional area within a joint computed	A_j = effective cross-sectional area within a joint computed

from joint depth times effective joint width. Joint depth shall be the overall depth of the column in the direction of analysis, which is in this case 1000 mm. The effective width of the joint is the smaller of the(a) and (b):
(a) beam width + joint depth=450 + 1000 =1450 mm
(b) beam width + twice the smaller perpendicular distance from the edge of the beam to the edge of the column = 450 + (2×25) = 500 mm (governs)
Thus, A_j = 1000 × 500 = 500000 mm^2

from joint depth times effective joint width. Joint depth shall be the overall depth of the column in the direction of analysis, which is in this case 40 inch. The effective width of the joint is the smaller of the (a) and (b):
(a) beam width + joint depth = 18 + 40=58 in.
(b) beam width + twice the smaller perpendicular distance from the edge of the beam to the edge of the column = 18 + (2×1) = 20 in. (governs)
Thus, A_j = 40 × 20 = 800 in.^{2}

3.1.21 Interior beam – column connection

The interior joint is confined on four sides by beams. But all these beams' width not equals to at least 75% the column width. Therefore, transverse hoop reinforcement within joint is the same as that over length l_o = 1000 mm (40 in.) (BNBC 2015, § 8.3.7.2) (ACI 318-11, § 21.7.3)

The interior joint at the fifth floor level is shown in Figure 3.14. The shear strength is checked in the N-S direction. To check the shear strength of the joint, the shear force acting on the joint must be calculated based on a stress of 1.25 f_y in the flexural reinforcement.

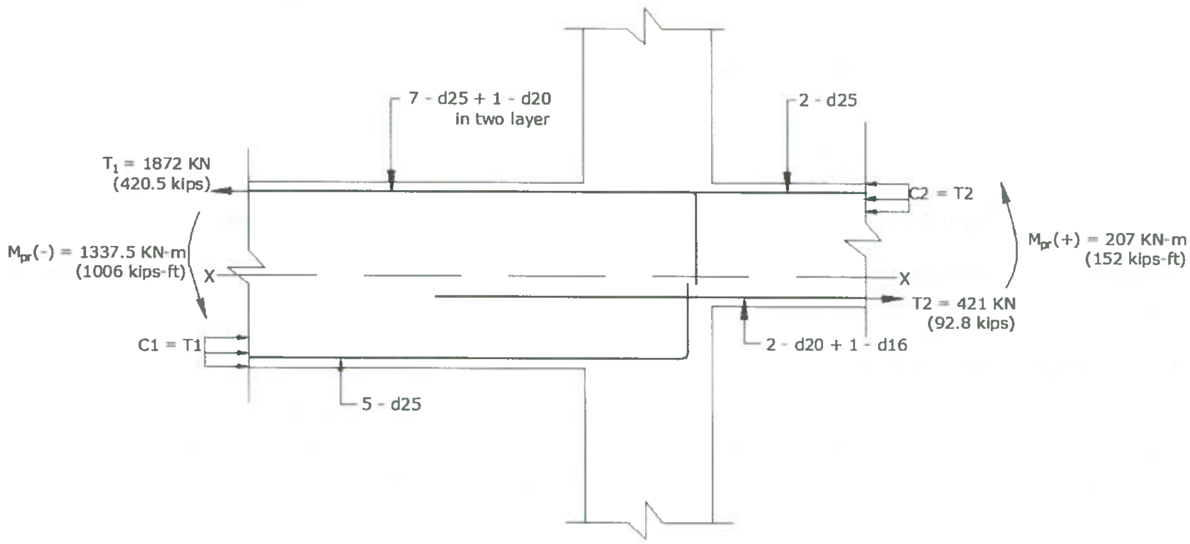


Figure 3.14 Shear & Moment of an Interior Beam-Column Joint

In SI Units

T_1(7- d25+1- d20) = $A_s(1.25f_y)$
= (7×490+1×314)×1.25×400/1000 = 1872 kN
a = 1872×1000/(0.85×25×450) = 196 mm
M$^-$ = $A_s(1.25f_y)(d- a/2)$
= 1872(812.5 -196/2)/1000 = 1337.5 kN-m
T_2(2- d20+1- d16) = $A_s(1.25f_y)$
= (842)×1.25×400/1000 = 421 kN
a = 421×1000/(0.85×25×450) = 44 mm
M$^+$ = $A_s(1.25f_y)$ $(d- a/2)$ = 421 (512.5 - 44/2)/1000 = 207 kN-m
Horizontal shear from column, V_{col} can be obtained by assuring that the beams in the adjoining floors are also deformed so that plastic hinges form at the end of the beams.
V_{col} = (1337.5 + 207)/4.57 = 338 kN

In FPS Units

T_1(7- d25+1- d20) = $A_s(1.25f_y)$
= (7×0.76+1×0.48)×1.25×58 = 420.5 Kips
a = 420.5/(0.85×3.625×18) = 7.6 in.
M$^-$ = $A_s(1.25f_y)(d- a/2)$
= 420.5(32.5- 7.6/2)/12 = 1006 kip-ft
T_2(2- d20+1- d16) = $A_s(1.25f_y)$
= 1.28×1.25×58 = 92.8 Kips
a = 92.8/(0.85×3.625×18) = 1.7 in.
M$^+$ = $A_s(1.25f_y)$ $(d- a/2)$ = 92.8 (20.5- 1.7/2)/12 = 152 kip-ft
Horizontal shear from column, V_{col} can be obtained by assuring that the beams in the adjoining floors are also deformed so that plastic hinges form at the end of the beams.
V_{col} = (1006 + 152)/15 = 77 kips

Thus the net shear at section X – X of the joint is $V_u = T_1 + T_2 - V_{col} = 1872 + 421 - 338 = 1955$ kN For a joint confined on four faces but all these beams' width not equal to at least 75% the column width, nominal shear strength φV_c is determined in accordance with BNBC 2015, § 8.3.7.3(ACI 318-11, § 21.7.4) $\phi V_c = 1.2\phi\sqrt{f_c'}A_j$ $= 1.2 \times 0.85 \times \sqrt{25} \times 500000 / 1000 = 2550$ kN $\varphi V_c = 2550$ kN $> V_u = 1955$ kN (O.K.) A_j = effective cross-sectional area within a joint computed from joint depth times effective joint width. Joint depth shall be the overall depth of the column in the direction of analysis, which is in this case 1000 mm. The effective width of the joint is the smaller of the (a) and (b): beam width + joint depth = 450 + 1000 =1450 mm beam width + twice the smaller perpendicular distance from the edge of the beam to the edge of the column = 450 + (2×25) = 500 mm (governs) Thus, $A_j = 1000 \times 500 = 500000$ mm^2	Thus the net shear at section X – X of the joint is $V_u = T_1 + T_2 - V_{col} = 420.5 + 152 - 113 = 459.5$ kips For a joint confined on four faces but all these beams' width not equal to at least 75% the column width, nominal shear strength φV_c is determined in accordance with BNBC 2015, § 8.3.7.3(ACI 318-11, § 21.7.4) $\phi V_c = 15\phi\sqrt{f_c'}A_j$ $= 15 \times 0.85 \times \sqrt{3625} \times 800 / 1000 = 614$ kips $\varphi V_c = 614$ kips $> V_u = 459.5$ kips (O.K.) A_j = effective cross-sectional area within a joint computed from joint depth times effective joint width. Joint depth shall be the overall depth of the column in the direction of analysis, which is in this case 40 in.. The effective width of the joint is the smaller of the (a) and (b): beam width + joint depth = 18 + 40= 58 in. beam width + twice the smaller perpendicular distance from the edge of the beam to the edge of the column = 18 + (2×1) = 20 in. (governs) Thus, $A_j = 40 \times 20 = 800$ in.^{2}

3.2 DESIGN OF A 12-STORIED HOSPITAL BUILDING

3.2.1 Building Information

- Building Location: Dhaka
- No of Storey: 12 (Twelve) stories
- Building Plinth Area: 1260m^2 (13600 ft^2); Total Area: 15120 m^2 (163200 ft^2)
- Total Height of The Building: 37.2 m (122ft)
- Use of The Building: Hospital
- Flat plate slab system

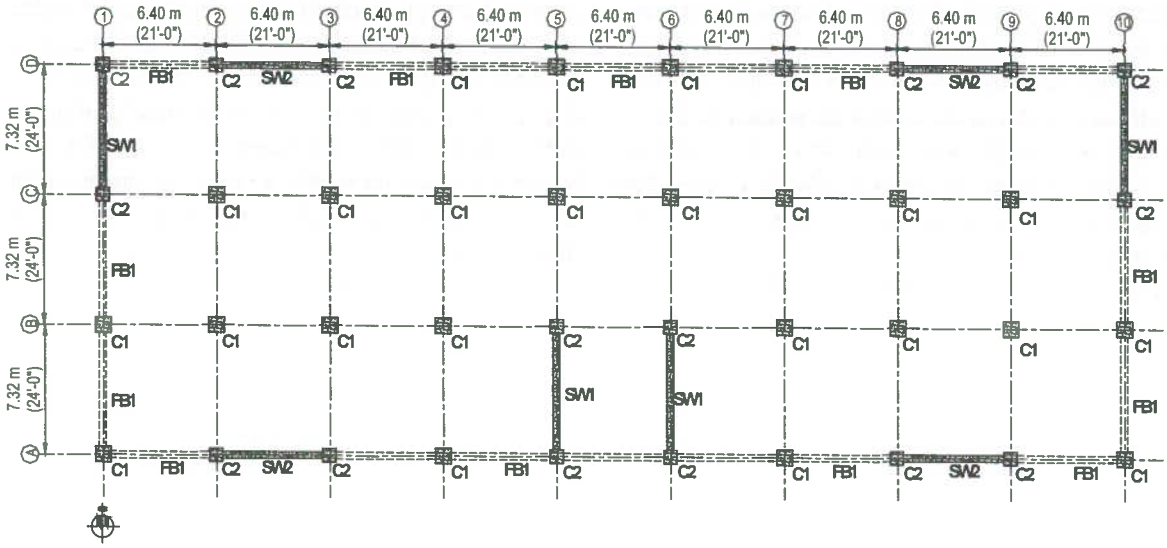


Figure 3.15 Plan of the Considered Building.

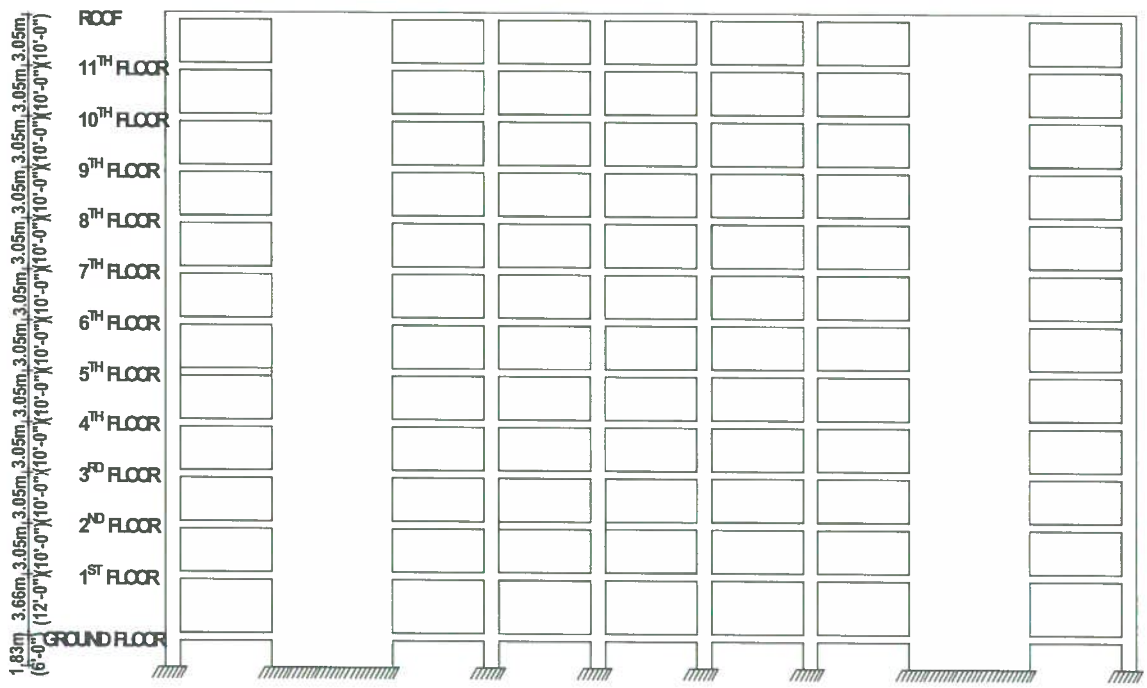


Figure 3.16 Elevation of the Considered Building.

3.2.2 Member Dimensions

Interior Columns C1: 875 mm × 875 mm (35″ × 35″) C2: 750 mm × 750 mm (30″ × 30″)
Peripheral Beams: 375 mm × 600 mm (15″ × 24″)

3.2.3 Design Data

- Material Properties

Compressive strength of concrete: $f_c' = 25$ N/mm^2 (3,625 psi)
Yield strength of steel: $f_y = 400$ N/mm^2 (58,000 psi)

- Service Loads

Live Loads: roof = 1 kN/m^2 (20 psf)
For floor of

I. Operating Rooms, Laboratories = 3.0 kN/m^2 (60 psf)
II. Patient Rooms = 2.0 kN/m^2 (40 psf)
III. Corridors above first floor = 4.0 kN/m^2 (80 psf)

For simplicity let average Live Load = 3.0 kN/m^2 (60 psf)

3.2.4 Seismic Design data:

- Zone Coefficient, Z = 0.20 (BNBC 2015, Figure 6.2.24 & Table 6.2.15)
- Occupancy Category: IV (BNBC 2015, Table 6.1.1)
- Importance factor, $I = 1.50$ (BNBC 2015, Table 6.2.17)
- Site Class based on Soil Investigation data SC (BNBC 2015, Table 6.2.13)
- Seismic Design Category D (BNBC 2015, Table 6.2.18)

Since the Occupancy Category of the building is IV, Site Class is SC and situated in Zone 2, the building is assigned to Seismic Design Category D.

3.2.5 Selection of Structural System

According to BNBC 2015, Table 6.2.19 for Seismic Design Category D, this building can be designed as Special reinforced concrete moment frame. But Because of architectural requirement for Flat Plate in this structure, there are two options: one is Building Frame System with Special Reinforced Concrete Shear Wall and another is Intermediate Moment Frame with Special Reinforced Concrete Shear Wall. In this example, Intermediate Moment Frame with Special Reinforced Concrete Shear Wall is opted. Here flat Plate will be detailed as part of Intermediate Moment Frame. There is also a height limitation for SDC-D for this type of Structure. If the height is above 50m (164 ft), flat plate structure is not permitted. The structure for this example is 37.2 m (122 ft). So, Intermediate Moment Frame with special shear wall is permitted. For this structural system, response reduction factor, $R = 6.5$ and the deflection amplification factor, $C_d = 5.0$ which are given in BNBC 2015 Table 6.2.19.

3.2.6 Method of Analysis

In case of dual system comprising Intermediate Moment Frame with special shear wall, most of lateral load shall be taken by special shear wall but the frame only shall be capable of resisting at least 25% of Earthquake Load. For Structural model one-fourth span of slab is considered for equivalent interior beam. Considering equal stiffness equivalent beam size is 300 mm × 450 mm (12″ × 18″).

Gravity Load is calculated manually by Direct Design Method. Lateral Load is determined via software. The stiffness properties of the members were input assuming cracked sections. The following cracked section properties were used:

- Slabs: $I_{eff} = 0.25I_g$
- Beams: $I_{eff} = 0.5I_g$
- Columns: $I_{eff} = 0.7I_g$
- Shear Walls: $I_{eff} = 0.35I_g$

Where I_g is the gross moment of inertia of the section.

[BNBC 2015, § 6.3.10.4.1] [ACI 318-11, § 101.10.4.1]

3.2.7 Storey Drift

The displacements δ_{xe} obtained from the three dimensional static, elastic analysis using the design seismic forces in the N-S direction are summarized in Table 3.5. The table also contains design earthquake displacement δ_x computed by BNBC 2015 Equation 6.2.45:

$\delta_x = C_d\delta_{xe}/I$

Where, Deflection amplification factor, $C_d = 5.0$ (BNBC 2015 Table 6.2.19)
Importance Factor, $I = 1.5$

Table 3.5 Lateral Displacement and Inter-Storey Drifts due to Seismic Forces in N-S Direction

Storey	δ_{xe}	$\delta_x = C_d\,\delta_{xe}/I$	$\Delta = \delta_x - \delta_{x-1}$
12	94 mm (3.76in.)	313 mm(12.52 in.)	26 mm(1.05in.)
11	86mm (3.44in.)	287 mm(11.47 in.)	27 mm(1.17in.)
10	78 mm (3.09in.)	260 mm(10.30 in.)	30 mm(1.20 in.)
9	69 mm (2.73in.)	230 mm(9.10 in.)	30 mm(1.20 in.)
8	60 mm (2.37in.)	200 mm(7.90 in.)	30 mm(1.17 in.)
7	51 mm (2.02in.)	170 mm(6.73 in.)	30 mm(1.20in.)
6	42 mm (1.66in.)	140 mm(5.53in.)	30 mm(1.16in.)
5	33 mm (1.31in.)	110 mm(4.37in.)	30 mm(1.17in.)
4	24 mm (0.96in.)	80 mm(3.20in.)	27 mm(1.07in.)
3	16 mm (0.64in.)	53 mm(2.13in.)	20 mm(0.83in.)
2	10 mm (0.39in.)	33 mm(1.30in.)	16 mm(0.67in.)
1	5 mm (0.19 in)	17 mm(0.63 in)	17 mm(0.63 in)

The design storey drift at storey x shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the storey under consideration:

$\Delta = \delta_x - \delta_{x-1}$ (BNBC 2015 Equation 6.2.46)

The design storey drift must not exceed the allowable storey drift Δ_a from BNBC 2015 Table 6.2.21. For Occupancy Category IV, $\Delta_a = 0.010h_{sx}$

Where, h_{sx} is the the storey height below Level x.

In SI Units	In FPS Units
Here, $h_{sx} = 3$ m $\Delta_a = 0.010 \times 3 \times 1000$ $= 30$ mm	Here, $h_{sx} = 10$ft $\Delta_a = 0.010 \times 10 \times 12$ $= 1.2$ in.

It is evident from Table 3.5 that for all the stories, drifts are less than Δ_a. It is also checked in E-W direction & found drifts are less than Δ_a.

3.2.8 Capable of Resisting at Least 25% of the Design Forces Without the Benefit of Shear Wall

According to BNBC 2015 Table 6.2.19, an additional safeguard is provided by requiring that moment-resisting frames be capable of resisting at least 25% of the design forces without the benefit of shear walls. Thus, the building was also analyzed using 25% of the design forces without the shear walls present.

The bending moments, shear forces, and axial forces in the frame members along column line under 100% of the seismic forces are given in Figure 3.17. The internal forces depicted in bold italic text in these figures correspond to the case when 25% of the lateral forces are applied to the frames only. In these figures (3.17) only Seismic Force (EQ) is considered.

In SI Units

12	57	***52***	56	***52***	57	***53***
11	64	***60***	64	***60***	65	***61***
10	76	***72***	76	***72***	76	***73***
9	88	***86***	87	***85***	88	***87***
8	99	***100***	98	***99***	99	***102***
7	106	***113***	106	***113***	107	***114***
6	111	***125***	111	***123***	111	***125***
5	114	***133***	114	***132***	114	***133***
4	113	***136***	113	***136***	113	***136***
3	107	***133***	107	***132***	107	***133***
2	96	***121***	96	***121***	96	***121***
1	76	***96***	76	***96***	76	***96***

Bending Moments in Slabs (m-KN)

In FPS Units

12	42	***38***	41	***38***	42	***39***
11	47	***44***	47	***44***	48	***45***
10	56	***53***	56	***53***	56	***54***
9	65	***63***	64	***63***	65	***64***
8	73	***74***	72	***73***	73	***75***
7	78	***83***	78	***83***	79	***84***
6	82	***92***	82	***91***	82	***92***
5	84	***98***	84	***97***	84	***98***
4	83	***100***	83	***100***	83	***100***
3	79	***98***	79	***97***	79	***98***
2	71	***89***	71	***89***	71	***89***
1	56	***71***	56	***71***	56	***71***

Bending Moments in Slabs (kip-ft)

Bold Italic denotes results with 25% of design base shear applied to the frames only

Figure 3.17 Results of 3-D Analysis under Seismic Forces in N-S direction for Frame 3

In SI Units

12	16	***14***	16	***15***
11	18	***16***	18	***17***
10	20	***20***	21	***20***
9	24	***23***	24	***24***
8	27	***27***	27	***28***
7	29	***31***	29	***31***
6	30	***34***	31	***34***
5	31	***36***	31	***36***
4	31	***37***	31	***37***
3	29	***36***	29	***36***
2	26	***33***	26	***33***
1	21	***26***	21	***26***

Shear Forces in Slabs (KN)

In FPS Units

12	3.5	***3.2***	3.5	***3.3***
11	4.0	***3.6***	4.0	***3.8***
10	4.6	***4.4***	4.7	***4.5***
9	5.4	***5.2***	5.4	***5.3***
8	6.0	***6.1***	6.1	***6.2***
7	6.5	***6.9***	6.5	***7.0***
6	6.8	***7.6***	6.9	***7.7***
5	7.0	***8.1***	7.0	***8.2***
4	6.9	***8.3***	6.9	***8.4***
3	6.6	***8.1***	6.6	***8.2***
2	5.9	***7.4***	5.9	***7.4***
1	4.7	***5.9***	4.7	***5.9***

Shear Forces in Slabs (kips)

Bold Italic denotes results with 25% of design base shear applied to the frames only

In SI Units

12	-50	***-52***	-113	***-104***
	-60	***-62***	-34	***-35***
11	-121	***-122***	-163	***-156***
	-75	***-87***	-37	***-50***
10	-146	***-157***	-189	***-194***
	-61	***-88***	-18	***-46***
9	-145	***-172***	-194	***-218***
	-35	***-76***	14	***-27***
8	-130	***-175***	-184	***-228***
	-7	***-56***	46	***1***
7	-110	***-167***	-167	***-226***
	23	***-24***	80	***37***
5	-85	***-148***	-145	***-213***
	52	***14***	110	***80***
6	-60	***-118***	-119	***-184***
	84	***64***	141	***132***
4	-27	***-72***	-85	***-140***
	122	***127***	179	***179***
3	16	***-4***	-38	***-69***
	179	***213***	231	***251***
2	84	***94***	37	***37***
	264	***334***	308	***344***
1	190	***237***	153	***194***
	443	***565***	466	***591***

Bending Moments in Columns (m-KN)

In FPS Units

12	-37	***-38***	-83	***-77***
	-44	***-46***	-25	***-26***
11	-89	***-90***	-120	***-115***
	-55	***-64***	-27	***-37***
10	-108	***-116***	-139	***-143***
	-45	***-65***	-13	***-34***
9	-107	***-127***	-143	***-161***
	-26	***-56***	10	***-20***
8	-96	***-129***	-136	***-168***
	-5	***-41***	34	***1***
7	-81	***-123***	-123	***-167***
	17	***-18***	59	***27***
6	-63	***-109***	-107	***-157***
	38	***10***	81	***59***
5	-44	***-87***	-88	***-136***
	62	***47***	104	***97***
4	-20	***-53***	-63	***-103***
	90	***94***	132	***132***
3	12	***-3***	-28	***-51***
	132	***157***	170	***185***
2	62	***69***	27	***27***
	195	***246***	227	***254***
1	140	***175***	113	***143***
	327	***417***	344	***436***

Bending Moments in Columns(kip-ft)

Bold Italic denotes results with 25% of design base shear applied to the frames only

Figure 3.17 Results of 3-D Analysis under Seismic Forces in N-S direction for Frame 3 (Contd.)

In SI Units

12	16	***14***	0	***0***
11	34	***30***	0	***0***
10	54	***50***	0	***0***
9	78	***73***	0	***0***
8	105	***100***	0	***0***
7	134	***131***	0	***0***
6	164	***165***	0	***0***
5	195	***201***	0	***0***
4	226	***238***	0	***0***
3	255	***274***	0	***0***
2	281	***307***	0	***0***
1	302	***333***	0	***0***

Axial Forces in Columns (KN)

In FPS Units

12	3.5	***3.2***	0	***0***
11	7.5	***6.8***	0	***0***
10	12.1	***12.2***	0	***0***
9	17.5	***16.4***	0	***0***
8	23.5	***22.5***	0	***0***
7	30.0	***29.4***	0	***0***
6	36.8	***37.0***	0	***0***
5	43.6	***45.1***	0	***0***
4	50.7	***53.4***	0	***0***
3	57.3	***61.5***	0	***0***
2	63.2	***68.9***	0	***0***
1	67.5	***74.8***	0	***0***

Axial Forces in Columns (kips)

Bold Italic denotes results with 25% of design base shear applied to the frames only

In SI Units

12	5	***5***	27	***22***
11	13	***13***	40	***36***
10	27	***22***	58	***49***
9	36	***31***	67	***62***
8	40	***40***	76	***76***
7	44	***49***	80	***85***
6	45	***53***	85	***98***
5	49	***58***	85	***102***
4	49	***67***	89	***102***
3	53	***71***	89	***102***
2	58	***80***	89	***102***
1	67	***89***	98	***107***

Shear Forces in Columns (KN)

In FPS Units

12	1	***1***	6	***5***
11	3	***3***	9	***8***
10	6	***5***	13	***11***
9	8	***7***	15	***14***
8	9	***9***	17	***17***
7	10	***11***	18	***19***
6	10	***12***	19	***22***
5	11	***13***	19	***23***
4	11	***15***	20	***23***
3	12	***16***	20	***23***
2	13	***18***	20	***23***
1	15	***20***	22	***24***

Shear Forces in Columns (kips)

Bold Italic denotes results with 25% of design base shear applied to the frames only

Figure 3.17 Results of 3-D Analysis under Seismic Forces in N-S direction for Frame 3 (Contd.)

3.2.9 Design of Shear Wall on Line 5

A summary of the design axial forces, bending moments, and shear forces at the base of the wall is given in Table 3.6

Table 3.6: Summary of Design Axial Forces, bending moments, and Shear Forces at Base of Shear Wall on Line 5

In SI Units

Load Case	Axial Force (kN)	Bending Moment, (kN-m)	Shear Force (kN)
Dead (D)	5048	0	0
Live (L)	423	0	0
Seismic (E)	± 4457	± 46949	± 2967
1.4*D*	7073	0	0
1.2*D* + 1.6 *L*	6735	0	0
1.277 D + 1.0 L +1.0E	11321	46949	2967
0.823D +1.0E	8621	46949	2967

In FPS Units

Load Case	Axial Force (Kips)	Bending Moment, (k-ft)	Shear Force (Kips)
Dead(D)	1135	0	0
Live(L)	95	0	0
Seismic(E)	± 1002	± 34628	± 667
1.4*D*	1590	0	0
1.2*D* + 1.6*L*	1514	0	0
1.275D +1.0L +1.0E	2545	34628	667
0.825D +1.0E	1938	34628	667

All load combinations are not shown here.

3.2.9.1 Design for Shear

In SI Units	In FPS Units
Reinforcement Requirements: According to BNBC 2015, § 8.3.6.2(ACI 318-11, § 21.9.2), the reinforcement ratio, for structural walls shall not be less than 0.0025 along the longitudinal and transverse directions. Reinforcement spacing each way shall not exceed 450 mm and reinforcement provided for shear strength shall be continuous and shall be distributed across the shear plane, unless the design shear force does not exceed $0.083A_{cv}\sqrt{f_c'}$, where A_{cv} is the gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered, mm^2. In such cases, the shear reinforcement may conform to BNBC 2015, § 6.6.3(ACI 318-11, § 14.3). For the wall in this example, $A_{cv} = 375 \times 4950 = 1856250$ mm^2 So, $0.083A_{cv}\sqrt{f_c'} = 0.083 \times 1856250 \times \sqrt{25}/1000$ $= 771$ kN $< V_u = 2967$ kN Therefore, the minimum reinforcement ratio is 0.0025 and the maximum spacing is 450 mm (BNBC 2015, § 8.3.6.2) (ACI 318-11, § 21.9.2) At least two layers of reinforcement shall be used in a wall if the in-plane factored shear force assigned to the wall exceeds $0.17A_{cv}\sqrt{f_c'} = 0.17 \times 1856250 \times \sqrt{25}/1000 = 1578$ kN. Inthis case, two layers of reinforcement are required, since 1578 kN < 2967 kN. The minimum required reinforcement in each direction per metre of wall is equal to $0.0025 \times 375 \times 1000 = 938$ mm^2. Assumingd12 bars in two layers, required spacing s is $s = 2 \times 113 \times 1000/938 = 241$ mm $<$ 450 mm Provide 2 layers of d12 spaced at 225 mm.	**Reinforcement Requirements:** According to BNBC 2015, § 8.3.6.2(ACI 318-11, § 21.9.2), the reinforcement ratio, for structural walls shall not be less than 0.0025 along the longitudinal and transverse directions. Reinforcement spacing each way shall not exceed 18 in. and reinforcement provided for shear strength shall be continuous and shall be distributed across the shear plane, unless the design shear force does not exceed $A_{cv}\sqrt{f_c'}$, where A_{cv} is the gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered, in^2. In such cases, the shear reinforcement may conform to BNBC 2015, § 6.6.3(ACI 318-11, § 14.3). For the wall in this example, $A_{cv} = 15 \times 198 = 2970$in.^{2} So, $A_{cv}\sqrt{f_c'} = 2970 \times \sqrt{3625}/1000$ $= 179$ kips $< V_u = 667$ kips Therefore, the minimum reinforcement ratio is 0.0025 and the maximum spacing is 18 in. (BNBC 2015, § 8.3.6.2) (ACI 318-11, § 21.9.2) At least two layers of reinforcement shall be used in a wall if the in-plane factored shear force assigned to the wall exceeds $2A_{cv}\sqrt{f_c'} = 2 \times 2970 \times \sqrt{3625}/1000 = 357$ kips. In this case, two layers of reinforcement are required, since 357 kips < 667 kips. The minimum required reinforcement in each direction per foot of wall is equal to $0.0025 \times 15 \times 12 = 0.45$in.^{2}. Assumingd12 bars in two layers, required spacing s is $s = 2 \times 0.175 \times 12/0.45 = 9.3$in. < 18in. Provide 2 layers of d12 spaced at 9 in.

Shear Strength Requirements:

In SI Units	In FPS Units
According to BNBC 2015 Equation 6.8.13 (ACI 318-11, Eq. 21-7), nominal shear strength of structural walls: $V_n = A_{cv}(0.17\sqrt{f_c'} + \rho_t f_y)$ since ratio of wall height to length $h_w/l_w = 37.2/8.21 = 4.5 > 2$ (BNBC 2015, § 8.3.8.3) (ACI 318-11, § 21.9.2) For two curtains of d12 horizontal bars spaced at 225mm $[\rho_t = 2 \times 113/(375 \times 225) = 0.0026]$ $\Phi V_n = 0.75 \times 1856250 \times (0.17 \times \sqrt{25} + 0.0026 \times 400)/1000$ $= 2631$kN $< V_u = 2967$ kN not O.K. If 175 mm spacing is used then $\rho_t = 0.0029$ & $\Phi V_n = 3021$ kN $> V_u = 2967$ kN O.K. Therefore, use 2 curtains of d12 @ 175 mm c/c in horizontal direction. Vertical Reinforcement ratio p_l for the vertical reinforcement must not be less than p_l when $h_w/l_w \leq 2.0$	According to BNBC 2015 Equation 6.8.13(ACI 318-11, Eq. 21-7), nominal shear strength of structural walls: $V_n = A_{cv}(2\sqrt{f_c'} + \rho_t f_y)$ since ratio of wall height to length $h_w/l_w = 122/26.92 = 4.5 > 2$ (BNBC 2015, § 8.3.8.3) (ACI 318-11, § 21.9.2) For two curtains of d12 horizontal bars spaced at 9 in. $[\rho_t = 2 \times 0.175/(15 \times 9) = 0.0026]$ $\Phi V_n = 0.75 \times 2970 \times (2 \times \sqrt{3625} + 0.0026 \times 58000)/1000$ $= 604$ kips $< V_u = 667$ kips not O.K. If 7 in. spacing is used then $\rho_t = 0.0033$ & $\Phi V_n = 695$ kips $> V_u = 667$ kips O.K. Therefore, use 2 curtains of d12 @ 7 in. c/c in horizontal direction. Vertical Reinforcement ratio p_l for the vertical reinforcement must not be less than p_l when $h_w/l_w \leq 2.0$

[BNBC 2015, § 8.3.8.3(e)] [ACI 318-11, § 21.9.4.3].	[BNBC 2015, § 8.3.8.3(e)] [ACI 318-11, § 21.9.4.3].
Since $h_w/l_w > 2$, use minimum reinforcement ratio of 0.0025. p_l refers to horizontal reinforcement ratio & p_t refers to vertical reinforcement ratio.	Since $h_w/l_w > 2$, use minimum reinforcement ratio of 0.0025. p_l refers to horizontal reinforcement ratio & p_t refers to vertical reinforcement ratio.
Use two curtains of d12 spaced 175 mm c/c in the vertical direction (p_l = 0.0033 > 0.0025)	Use two curtains of d12 spaced 7 in. c/c in the vertical direction (p_l = 0.0033 > 0.0025)

Design for Axial Force & Bending

The interaction diagram of the wall is given in Figure 3.18. As seen from the figure, the wall is adequate for the load combinations in Table 3.6.

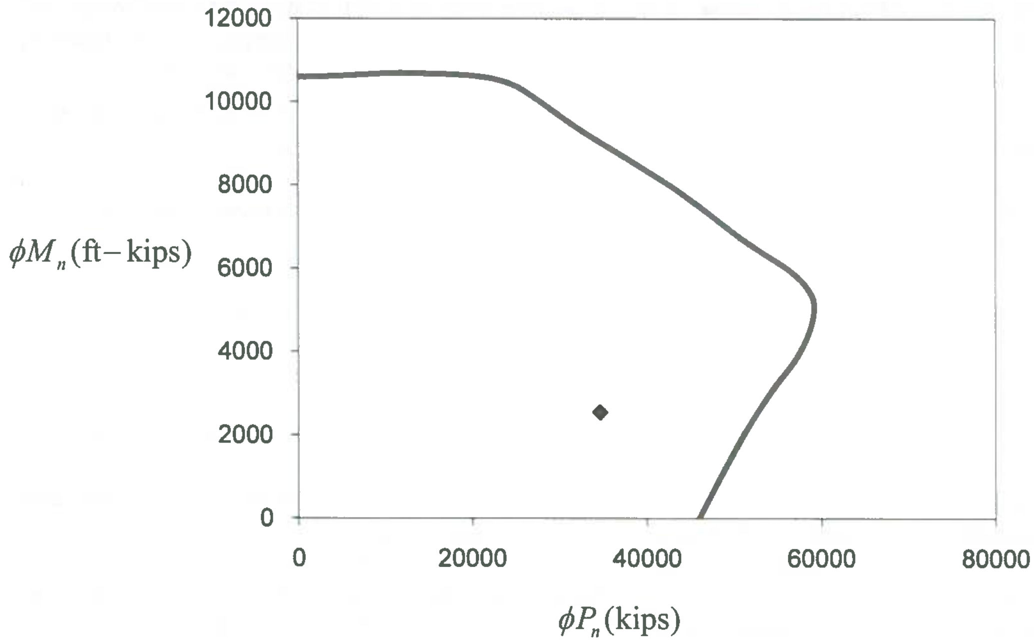


Figure 3.18 Design Strength Interaction Diagrams for the Shear Wall.

Special Boundary Elements

The need for special boundary elements at the edges of structural walls is evaluated in accordance with BNBC 2015, § 8.3.6.6(b) (ACI 318-11, § 21.9.6.2) or 8.3.6.6(c) (ACI 318-11, § 21.9.6.3). The displacement based approach in BNBC 2015, § 8.3.6.6(b) (ACI 318-11, § 21.9.6.2) is utilized in this example. In this method, the wall is displaced at the top an amount equal to the expected design displacement; special boundary elements are required to confine the concrete when the strain in the extreme compression fiber of the wall exceeds a critical value. This method is applicable to walls or wall piers that are essentially continuous in cross-section over the entire height & designed to have one critical section for flexure & axial loads.

Compression zones are to be reinforced with special boundary elements where according to BNBC 2015, Equation 6.8.10 (ACI 318-11, Equation 21-8)

In SI Units	In FPS Units
$c \geq \dfrac{\ell_w}{600(\delta_u / h_w)}$ (3.20) Ratio δ_u / h_w in Equation shall not be taken less than 0.007.	$c \geq \dfrac{\ell_w}{600(\delta_u / h_w)}$ (3.20) Ratio δ_u / h_w in Equation shall not be taken less than 0.007.

Where c = distance from extreme compression fiber to the neutral axis as per BNBC 2015, § 6.3.2.7 (ACI 318-11, § 10.2.7) calculated for the factored axial force & nominal moment strength, consistent with the design displacement δ_u, resulting in the largest neutral axis depth

ℓ_w = length of entire wall or segment of wall considered in the direction of shear force

δ_u = design displacement

= total lateral displacement expected for the design-basis earthquake as specified by the governing code

h_w = height of entire wall or of a segment of wall considered

The lower limit on the quantity δ_u / h_w is specified to require moderate wall deformation capacity for stiff buildings.

In SI Units	In FPS Units
In this example, l_w = 8.195 m = 8195 mm, δ_u is equal to δ_x from Table 3.5, which is 313 mm at the top of the wall & δ_u/h_w = 0.008 > 0.007.	In this example, l_w = 26.5ft = 328 in., δ_u is equal to δ_x from Table 3.5, which is 12.52in. at the top of the wall & δ_u/h_w = 0.008 > 0.007.
Therefore, special boundary elements are required if *c* is greater than or equal to 8195/ (600 × 0.008) = 1707 mm.	Therefore, special boundary elements are required if *c* is greater than or equal to 318/ (600×0.008) = 66.3 in.
The distance *c* to be in Equation (3.20) is the largest neutral axis depth calculated for the factored axial force & nominal moment strength consistent with the design displacement δ_u.	The distance *c* to be in Equation (3.20) is the largest neutral axis depth calculated for the factored axial force & nominal moment strength consistent with the design displacement δ_u.
From a strain compatibility analysis, the largest *c* is equal to 1345mm corresponding to a factored axial load of 11321 kN & nominal moment strength of 46949 kN-m, which is not greater than 1707 mm, Therefore special boundary elements are not required. <u>But for academic interest boundary columns are included in this example.</u>	From a strain compatibility analysis, the largest *c* is equal to 53 in. corresponding to a factored axial load of 2545 kips & nominal moment strength of 34628 kip-ft, which is not greater than 66.3 in., Therefore special boundary elements are not required. <u>But for academic interest boundary columns are included in this example.</u>
Special Boundary must extend horizontally from the extreme compression fiber a distance not less than the largest of the following (BNBC 2015, § 8.3.6.6 (d)) (ACI 318-11, § 21.9.6.4): ➢ C - 0.1 l_w = 1345 – (0.1× 8195) = 525 mm ➢ $c/2$ = 1345/2 = 672 mm (governs)	Special Boundary must extend horizontally from the extreme compression fiber a distance not less than the largest of the following (BNBC 2015, § 8.3.6.6 (d)) (ACI 318-11, § 21.9.6.4): ➢ C - 0.1 l_w = 53 – (0.1×318) = 21.2 in. ➢ $c/2$ = 53/2 = 26.5 in. (governs)
Special boundary transverse reinforcement in accordance with BNBC 2015, § 8.3.6.6(d) (ACI 318-11, § 21.9.6.4): is provided in the 750 mm boundary element at the ends of the wall.	Special boundary transverse reinforcement in accordance with BNBC 2015, § 8.3.6.6(d) (ACI 318-11, § 21.9.6.4) is provided in the 30 in. boundary element at the ends of the wall.
Vertical extent of special boundary transverse reinforcement from base of wall is the largest of [BNBC 2015, § 8.3.6.6 (b) (ii)] [ACI 318-11, § 21.9.6.2(b)] ➢ ℓ_w = 8.195 m (governs) ➢ $\dfrac{M_u}{4V_u}$ = 46949/(4×2967) = 3.96 m	Vertical extent of special boundary transverse reinforcement from base of wall is the largest of [BNBC 2015, § 8.3.6.6 (b) (ii)] [ACI 318-11, § 21.9.6.2(b)] ➢ ℓ_w = 26.5 ft. (governs) ➢ $\dfrac{M_u}{4V_u}$ = 34628/(4×667) = 12.98 ft

Special Boundary Element Transverse Reinforcement

Provisions for the amount & spacing of transverse reinforcement required in the special boundary elements are given in BNBC 2015, § 8.3.6.6(d) (ACI 318-11, § 21.9.6.4). In particular, transverse reinforcement requirements of BNBC 2015, § 8.3.5.4(a) (ACI 318-11, § 21.6.4.3) through BNBC 2015, § 8.3.5.4(c) (ACI 318-11, § 21.6.4.4) for columns in special moment frames must be satisfied except for BNBC 2015 Equation 6.8.7(ACI 318-11, Equation. 21-5).

In SI Units	In FPS Units
Assuming d12 rectangular hoops & crossties around every other longitudinal in both directions of the 750mm × 750 mm special boundary elements, the maximum allowable spacing is the smallest of the following:	Assuming d12 rectangular hoops & crossties around every other longitudinal in both directions of the 30 in. × 30 in. special boundary elements, the maximum allowable spacing is the smallest of the following:
• Minimum member dimension/4 = 750/4 = 187.5 mm • 6 × diameter of smallest longitudinal bar = 6 × 25 = 150 mm • $s_x = 100 + [(350 - h_x)/3]$ (where 100 mm ≤ s_o ≤ 150 mm) = 100 + [(350 – 243)/3] = 136 mm (governs) where, h_x = maximum horizontal spacing of hoop or crosstie legs on all faces of the 750 mm × 750 mm column = [750 – 2(37.5 + 12) – 32] (2/6) + 25 + 12 = 243mm < 350 mm O.K.	• Minimum member dimension/4 = 30/4 = 7.5 in. • 6 × diameter of smallest longitudinal bar = 6 ×1 = 6 in. • $s_x = 4 + [(14 - h_x)/3]$ (Where 4 in. ≤ s_o ≤ 6 in.) = 4 + [(14 – 9.7)/3] = 5.43 in. (governs) where, h_x = maximum horizontal spacing of hoop or crosstie legs on all faces of the 30 in. × 30 in. column = [30 – 2(1.5 + 0.5) – 1.28] (2/6) + 1 + 0.5 = 9.7 in. < 14 in. O.K.
Assuming a spacing of 125 mm, required area of transverse reinforcement is determined by BNBC 2015 Equation 6.8.8(ACI 318-11, Equation. 21-5): $A_{sh} = \frac{0.09 s h_c f_c'}{f_{yh}}$ = 0.09 × 125 × 663 × 25/400 = 466 mm^2 where h_c = cross-sectional dimension of column core measured center-to-center of confinement reinforcement = 750 – 2[37.5 + 12/2] = 663mm d12 hoops with crossties around every other longitudinal bar provides A_{sh} = 5 × 113 = 565mm^2 (O.K.)	Assuming a spacing of 5 in., required area of transverse reinforcement is determined by BNBC 2015 Equation 6.8.8(ACI 318-11, Equation. 21-5): $A_{sh} = \frac{0.09 s h_c f_c'}{f_{yh}}$ = 0.09 × 5 × 26.5 × 3.625/58 = 0.75 in^2 where h_c = cross-sectional dimension of column core measured center-to-center of confinement reinforcement = 30 – 2[1.5 + 0.5/2] = 26.5 in. d12 hoops with crossties around every other longitudinal bar provides A_{sh} = 5 × 0.18 = 0.9 in.^{2}(O.K.)

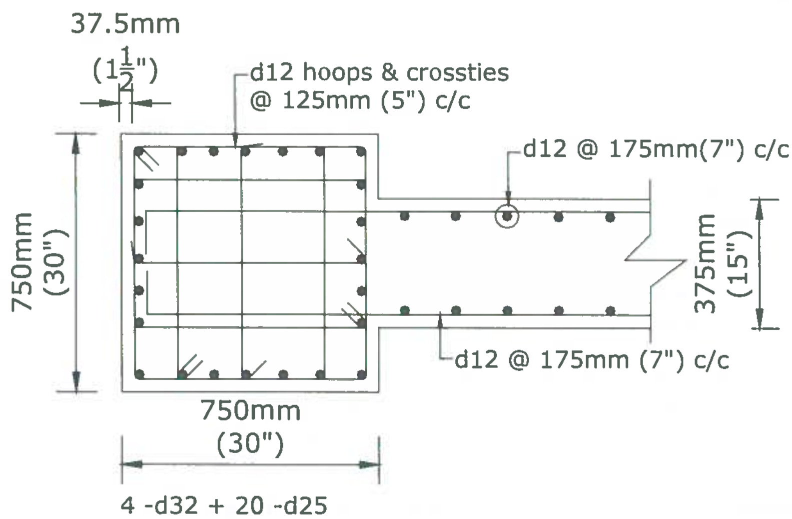


Figure 3.19 Reinforcement Details for Shear Wall Along Line 5

3.2.10 Slab Design

Design of Slab Column connection shall comply with BNBC 2015, § 8.3.12.4 (ACI 318-11, § 21.13.6). This conformance satisfies the deformation compatibility requirements of BNBC 2015, § 2.5.14.4.

Minimum thickness h for slab without edge and interior beams is determined from BNBC 2015 Table 6.6.3. For systems without drop panels & with Grade 60 reinforcement (f_y = 420 MPa (60000psi)), the minimum thickness is

In SI Units	In FPS Units
$h = l_n/30 = [(7.32\times1000) - 875]/33 = 195$ mm where l_n is the longest clear span length. Therefore, a 212.5 mm slab thickness will satisfy the requirement.	$h = l_n/30 = [(24\times12) - 35]$ v/33 = 7.7 in. where l_n is the longest clear span length. Therefore, a 8.5 in. slab thickness will satisfy the requirement.

When two-way slabs are supported directly on columns, shear near the columns is critically important, especially at exterior slab-column connections without spandrel beams. For flat plates, slab thickness will almost always be governed by two-way shear rather than serviceability requirements. Shear strength requirements are checked for both one-way & two-way shear forces due to gravity loads to determine if the 212.5 mm (8.5 in.) thick slab is adequate or not. It is worthwhile to perform these preliminary calculations at this stage so that if any adjustment in slab thickness is required, it can be made before additional calculations are performed.

Live loads on the slab are reduced in accordance with BNBC 2015 § 2.3.13

Subject to the limitation of BNBC 2015, § 2.3.13.2 through 2.3.13.5, members are permitted to be designed for a reduced live load in accordance to BNBC 2015 Equation 6.2.1, if value of $K_{LL}A_T$ is 37.16 m^2 (400 ft^2) or more.

In SI Units	In FPS Units
$L = L_o(0.25 + 4.57/\sqrt{(K_{LL}A_T)}$ Where, L = Reduced live load Unreduced live load, L_o = 2.87 kN/m^2 Live load element factor K_{LL} = 4 (BNBC 2015 Table 2.3.5) Tributary area, A_T = 7.32m × 6.40 m = 46.85 m^2 $L = L_o(0.25 + 4.57/\sqrt{(K_{LL}A_T)}$ = 2.87 (0.25 + 4.57/√(4 × 46.85)) = 1.68 kN/m^2 The design loads are: Factored slab wt. = 1.2×(212.5/1000)× 23.6 = 6.02 kN/m^2 Factored superimposed dead load = 1.2 × 2.63 = 3.16 kN/m^2 Factored live load = 1.6 × 1.68 = 2.69 kN/m^2 Total factored load w_u = 6.02 + 3.16 + 2.69 = 11.87kN/m^2	$L = L_o(0.25 + 4.57/\sqrt{(K_{LL}A_T)}$ Where, L = Reduced live load Unreduced live load, L_o = 60 psf Live load element factor K_{LL} = 4 (BNBC 2015 Table 2.3.5) Tributary area, A_T = 24 ft × 21 ft = 504 ft^2 $L = L_o(0.25 + 15/\sqrt{(K_{LL}A_T)})$ = 60 (0.25 + 15/√(4×504)) = 35 psf The design loads are: Factored slab weight = 1.2 × (8.5/12) × 150 = 128psf Factored superimposed dead load = 1.2 × 55 = 66 psf Factored live load = 1.6 × 35 = 56psf Total factored load w_u = 128 + 66 + 56 = 250psf

One-way shear:

In SI Units	In FPS Units
Investigation of one-way shear is made at a distance d = 212.5 – 31.25 = 181.25 mm from the face of the support in either direction, & is shown in Figure 3.20 at the section that gives the largest shear force (BNBC 2015, § 6.4.10.1.1) (ACI 318-11, § 11.11.1.1)	Investigation of one-way shear is made at a distance d = 8.5 – 1.25 = 7.25 in. from the face of the support in either direction, & is shown in Figure 3.20 at the section that gives the largest shear force (BNBC 2015, § 6.4.10.1.1) (ACI 318-11, § 11.11.1.1)
Factored shear force at critical section is: $V_u = 11.87 \times (3.66 - 212.5/1000 - 181.25/1000) \times 6.40$ $= 248$ kN Shear design strength is computed in accordance with	Factored shear force at critical section is: $V_u = 0.25 \times (12 - 17.5/12 - 7.25/12) \times 21$ $= 52$ kips Shear design strength is computed in accordance with
BNBC 2015 Equation 6.6.49(ACI 318-11, § 11-3) $\varphi V_c = \varphi\, 0.17\sqrt{f_c'}\, b_w d$ $= 0.75 \times 0.17\sqrt{25} \times (6.40 \times 1000) \times 181.25/1000$ $= 739$ kN $> V_u$ O.K.	BNBC 2015 Equation 6.6.49(ACI 318-11, § 11-3) $\varphi V_c = \varphi 2\sqrt{f_c'}\, b_w d$ $= 0.75 \times 2\sqrt{3625} \times (21 \times 12) \times 7.25/1000$ $= 165$ kips $> V_u$ O.K.

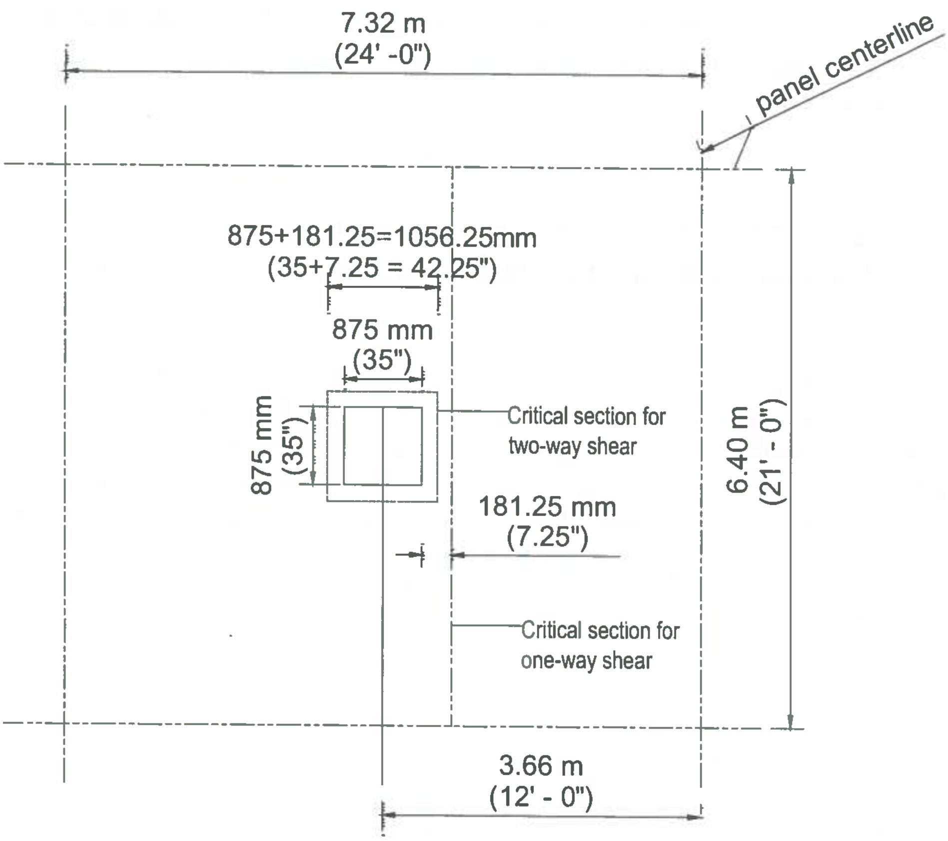


Figure 3.20 Critical Section for One-Way & Two-Way Shear at an Interior Column

Two-way shear:

In SI Units	In FPS Units
Investigation of two-way shear is made at a distance $d/2$ from the face of the support, as shown in Figure 3.20 for an interior column (BNBC 2015, § 6.4.10.1.2) (ACI 318-11, § 11.11.1.2). Maximum factored shear force at this location is: $V_u = w_u (l_1 l_2 - b_1 b_2)$ $= 11.87\times [(7.32\times6.40) - (1.05 \times 1.05)]$ $= 543$ kN	Investigation of two-way shear is made at a distance $d/2$ from the face of the support, as shown in Figure 3.20 for an interior column (BNBC 2015, § 6.4.10.1.2) (ACI 318-11, § 11.11.1.2). Maximum factored shear force at this location is: $V_u = w_u (l_1 l_2 - b_1 b_2)$ $= 0.25 \times [(24 \times 21) - (42.25 \times 42.25)/144]$ $= 123$ kips
Design shear strength is the smallest value determined in accordance with BNBC 2015 Equations 6.6.72 (ACI 318-11, Equation 11-31) through 6.6.74 (ACI 318-11, Equation 11-33). For square columns, Equation 6.6.74 (ACI 318-11, Equation 11-33) governs:	Design shear strength is the smallest value determined in accordance with BNBC 2015 Equations 6.6.72 (ACI 318-11, Equation 11-31) through 6.6.74 (ACI 318-11, Equation 11-33). For square columns, Equation 6.6.74 (ACI 318-11, Equation. 11-33) governs:
$\varphi\, V_c = \varphi 0.33\sqrt{f_c'}\, b_o d$ $= 0.75\times 0.33\sqrt{25} \times (4\times1056.25) \times 181.25/1000$ $= 948$ kN $> V_u$ O.K.	$\varphi\, V_c = \varphi 4\sqrt{f_c'}\, b_o d$ $= 0.75 \times 4\sqrt{3625}\times (4\times 42.25) \times 7.25/1000$ $= 221$ kips $> V_u$ O.K.
It is important to note that the check for shear strength is only preliminary at this stage. The shear stress needs to be checked for gravity loads and combined gravity and lateral loads at both an interior and exterior slab-column connection. A more refined check for shear strength is made at a later stage.	It is important to note that the check for shear strength is only preliminary at this stage. The shear stress needs to be checked for gravity loads and combined gravity and lateral loads at both an interior and exterior slab-column connection. A more refined check for shear strength is made at a later stage.

Design for Flexure:

In lieu of other analysis methods, the Direct Design Method in BNBC 2015, § 6.5.6 (ACI 318-11, § 13.6) is utilized in this example to determine the bending moments in the slab due to gravity loads. This method can be used only if certain geometric & loading criteria are met. All of the criteria are met in this example, as indicated below:

- There shall be a minimum of 3 consecutive spans in each direction.
  This structure has 3 spans in one direction, 9 in other direction. O.K.
- Panels shall be rectangular with a center-to-center longer span to shorter span length ratio not greater than 2.

In SI Units	In FPS Units
In this structure longer span/shorter span $= 7.32/6.40 = 1.14 < 2$ O.K.	In this structure longer span/shorter span $= 24/21 = 1.14 < 2$ O.K.

- Successive center-to-center span lengths in each direction shall not differ by more than one-third the longer span.
  In this structure in each direction, the span lengths are equal. O.K.
- Columns may be offset a maximum of 10% of the span in the direction of offset from either axis between centerlines of successive columns.
  No column offsets are present. O.K.
- Loads must be uniformly distributed gravity loads only and the live load must be less than or equal to 2 times the dead load.

In SI Units	In FPS Units
In this structure uniform live load/dead load = 1.68/7.65 = 0.22 < 2 O.K.	In this structure uniform live load/dead load = 35/162 = 0.22 < 2 O.K.

The total static service load & live load moments for spans in an interior design strip in the E-W direction are computed in accordance with BNBC 2015 Equation 6.6.84 (ACI 318-11, Equation 13-4)

$$M_o = \frac{q_u \ell_2 \ell_n^{\ 2}}{8}$$

Where, l_n is the length of clear span in direction that moments are being determined & l_2 = average of adjacent transverse spans

In SI Units	In FPS Units
Here, $l_n = 7.32 - (875/1000) = 6.45$ m $l_2 = 6.40$m $(M_o)_D = 7.65 \times 6.40 \times 6.45^2/8$ $= 255$ kN-m $(M_o)_L = 1.68 \times 6.40 \times 6.45^2/8$ $= 56$ kN-m	Here, $l_n = 24 - (35/12) = 21.08$ ft $l_2 = 21$ ft $(M_o)_D = 0.162 \times 21 \times 21.08^2/8$ $= 189$ kip-ft $(M_o)_L = 0.035 \times 21 \times 21.08^2/8$ $= 41$kip-ft

These moments are divided into positive & negative moments & then into column strip & middle strip moments according to the distribution factors in BNBC 2015 § 6.5.6.3 (ACI 318-11, § 13.6.3), 6.5.6.4 (ACI 318-11, § 13.6.4) & 6.5.6.6 (ACI 318-11, § 13.6.6). The distribution factors for a flat plate system are given in Table 3.7, where negative moments are at the face of the supports. A summary of the service gravity load bending moments in the column strip & middle strip of an end span and an interior span is given in Table 3.8.

Table 3.7 Design Moments for Flat Plate supported Directly on Columns

Slab Moments	**End Span**			**Interior Span**	
	Exterior Negative	Positive	Interior Negative	Positive	Interior Negative
Total Moment	$0.26M_o$	$0.52M_o$	$0.70M_o$	$0.35M_o$	$0.65M_o$
Column Strip	$0.26M_o$	$0.31M_o$	$0.53M_o$	$0.21M_o$	$0.49M_o$
Middle Strip	0	$0.21M_o$	$0.17M_o$	$0.14M_o$	$0.16M_o$

Table 3.8 a Service Dead & Live Load Bending Moments (kN-m) in N-S Interior Design Strip (In SI unit)

Location		**Moment**	M_D (kN-m)	M_L (kN-m)
End Span				
Column Strip	Exterior Negative	$0.26M_o$	-66.3	-14.6
	Positive	$0.31M_o$	79	17.4
	Interior Negative	$0.53M_o$	-135	-29.7
Middle Strip	Exterior Negative	0	0	0
	Positive	$0.21M_o$	53.6	11.8
	Interior Negative	$0.17M_o$	-43.4	-9.52
Interior Span				
Column Strip	Positive	$0.21M_o$	53.6	11.8
	Interior Negative	$0.49M_o$	-125	-27.4
Middle Strip	Positive	$0.14M_o$	35.7	7.8
	Interior Negative	$0.16M_o$	-40.8	-9.0

Table 3.8b Service Dead & Live Load Bending Moments (kip-ft) in N-S Interior Design Strip (In FPS unit)

Location		Moment	M_D (kip-ft)	M_L (kip-ft)
End Span				
Column Strip	Exterior Negative	$0.26M_o$	-49	-10.7
	Positive	$0.31M_o$	58.6	12.7
	Interior Negative	$0.53M_o$	-100	-21.7
Middle Strip	Exterior Negative	0	0	0
	Positive	$0.21M_o$	39.7	8.6
	Interior Negative	$0.17M_o$	-32	-7.0
Interior Span				
Column Strip	Positive	$0.21M_o$	39.7	8.6
	Interior Negative	$0.49M_o$	-92.6	-20
Middle Strip	Positive	$0.14M_o$	26.5	5.7
	Interior Negative	$0.16M_o$	-30.2	-6.6

The required flexural reinforcement is given in Table-3.10. The provided areas of steel are greater than the minimum required (BNBC 2015, § 6.5.3.1) (ACI 318-11, § 13.3.1). Also the provided spacing is less than the maximum allowed as per BNBC 2015, § 6.5.3.2 (ACI 318-11, § 13.3.2).

The amount of slab reinforcement needs to be checked at the end support and the first interior support to ensure that the moment transfer requirements of BNBC 2015, § 6.5.5.3 (ACI 318-11, § 13.5.3) are satisfied.

Table 3.9a Summary of Slab Design Bending Moments (kN-m) at Floor Level 4 (In SI unit)

Load Case	Location	End Span		Interior Span	
		Column Strip	Middle Strip	Column Strip	Middle Strip
Dead (*D*)	Ext. neg.	- 66.3	0		
	Positive	79	53.6	53.6	35.7
	Int. neg.	-135	- 43.4	-125	- 40.8
Live (*L*)	Ext. neg.	-14.6	0		
	Positive	17.4	11.8	11.8	7.8
	Int. neg.	- 29.7	- 9.5	- 27.4	- 9.0
Seismic Load (E)	Ext. neg.	± 137			
	Positive				
	Int. neg.	± 137		± 137	
Load Combination					
1.4*D*	Ext. neg.	- 92.8	0.0		
	Positive	110.6	75	75	50
	Int. neg.	-189	- 60.8	-175	-57
1.2*D* + 1.6*L*	Ext. neg.	-103	0.0		
	Positive	122.6	83.2	83.2	55.3
	Int. neg.	- 209.5	- 67.3	-194	- 63.4
1.275D + 1.0L + 1.0E	Ext. neg.	-236	0.0		
	Positive	118	80	80	53
	Int. neg.	- 339	- 64	- 324	- 61
0.825D +1.0E	Ext. neg.	-192	0.0		
	Positive	65	44.2	44.2	30
	Int. neg.	248	- 35	- 240	-33

Table 3.9b Summary of Slab Design Bending Moments (kip-ft) at Floor Level 4 (In FPS unit)

Load Case	Location	End Span		Interior Span	
		Column Strip	Middle Strip	Column Strip	Middle Strip
Dead (D)	Ext. neg.	- 49	0		
	Positive	58.6	39.7	39.7	26.5
	Int. neg.	-100	- 32	- 92.6	- 30.2
Live (L)	Ext. neg.	-10.7	0		
	Positive	12.7	8.6	8.6	5.7
	Int. neg.	-21.7	-7.0	-20	- 6.6
Seismic Load (E)	Ext. neg.	± 101			
	Positive				
	Int. neg.	± 101		± 101	
Load Combination					
1.4 D	Ext. neg.	- 68.6	0.0		
	Positive	82	55.6	55.6	37
	Int. neg.	-140	- 44.8	-129.7	- 42.3
1.2D + 1.6L	Ext. neg.	-76	0.0		
	Positive	90.6	61.4	61.4	41
	Int. neg.	-154.7	- 49.6	-143	- 46.8
1.275D + 1.0L + 1.0E	Ext. neg.	-174	0.0		
	Positive	87	59	59	40
	Int. neg.	- 250	- 48	- 239	- 45
0.825D +1.0E	Ext. neg.	-141	0.0		
	Positive	48	33	33	22
	Int. neg.	-183.5	-26	-177	-25

Since this example is done for academic purpose, all the load combinations are not shown here for simplicity.

Table 3.10a Required Slab Reinforcement at Floor Level 4 (In SI Units)

Location			M_u (kN-m)	b (mm)	A_s* (mm^2)	Reinforcement*	Reinforcement after modification
End Span	Column strip	Ext. neg.	-236	3200	3675	19 – d16	20 – d16
		Positive	122.6	3200	1850	10 – d16	10 – d16
		Int. neg.	-339	3200	5457	28 - d16	28 - d16
	Middle strip	Ext. neg.	0.0	3200	1224	7 - d16	7 - d16
		Positive	83.2	3200	1224	7 - d16	7 - d16
		Int. neg.	-67.3	3200	1224	7 - d16	7 - d16
Interior Span	Column strip	Positive	83.2	3200	1224	7 - d16	7 - d16
		Negative	-324	3200	5191	27 - d16	27 - d16
	Middle strip	Positive	55.3	3200	1224	7 - d16	7 - d16
		Negative	-63.4	3200	1224	7 - d16	7 - d16

* Minimum $A_s = 0.0018bh = 0.0018 \times (3.2 \times 1000) \times 212.5 = 1224$ mm^2 (BNBC 2015, § 6.5.3.1) (ACI 318-11, § 13.3.1)
Maximum spacing = $2h = 2\times212.5 = 425$ mm, for $b = 3200$ mm, 3200/450 =7.1 spaces, say 8 bars
** Final reinforcement after modifications described in later sections

Table 3.10b Required Slab Reinforcement at Floor Level 4 (In FPS Units)

Location			M_u (kip-ft)	b (in.)	A_s*(in.^{2})	Reinforcement*	Reinforcement after modification
End Span	Column strip	Ext. neg.	-174	126	5.88	19 – d16	20 – d16
		Positive	90.6	126	2.87	10 – d16	10 – d16
		Int. neg.	-250	126	8.73	28 - d16	28 - d16
	Middle strip	Ext. neg.	0.0	126	1.93	7 - d16	7 - d16
		Positive	61.4	126	1.93	7 - d16	7 - d16
		Int. neg.	- 49.6	126	1.93	7 - d16	7 - d16
Interior Span	Column strip	Positive	61.4	126	1.93	7 - d16	7 - d16
		Negative	-239	126	8.3	27 - d16	27 - d16
	Middle strip	Positive	41	126	1.93	7 - d16	7 - d16
		Negative	-46.8	126	1.93	7 - d16	7 - d16

* Minimum $A_s = 0.0018bh = 0.0018 \times (10.5 \times 12) \times 8.5 = 1.93$ in.^{2} (BNBC 2015, § 6.5.3.1) (ACI 318-11, § 13.3.1)
Maximum spacing = $2h = 2\times8.5 = 17$ in., for $b = 126$ in., 126/17 =7.4 spaces, say 8 bars
** Final reinforcement after modifications described in later sections

End support – additional flexural reinforcement required for moment transfer:

In SI Units	In FPS Units
The total unbalanced moment at this slab-column connection is equal to 236 kN-m (see Table 3.9).	The total unbalanced moment at this slab-column connection is equal to 174kip-ft (see Table 3.9).
A fraction of this moment $\gamma_f M_u$ must be transferred over an effective width equal to $c_2 + 3h = 875 + (3 \times 212.5) = 1512$ mm (BNBC 2015, § 6.5.5.3.2) (ACI 318-11, § 13.5.3.2)	A fraction of this moment $\gamma_f M_u$ must be transferred over an effective width equal to $c_2+3h = 35 + (3 \times 8.5) = 60$in. (BNBC 2015, § 6.5.5.3.2) (ACI 318-11, § 13.5.3.2)
The fraction of unbalanced moment transferred by flexure is calculated in accordance with BNBC 2015 Equation 6.6.81 (ACI 318-11, Equation 13-1)	The fraction of unbalanced moment transferred by flexure is calculated in accordance with BNBC 2015 Equation 6.6.81(ACI 318-11, Equation 13-1)
$\gamma_f = 1/[1+ (2/3)\sqrt{(b_1/b_2)}]$ $=1/[1+(2/3)\sqrt{((875+212.5)/(875+212.5))}]$ $= 0.61$	$\gamma_f = 1/[1+ (2/3)\sqrt{(b_1/b_2)}]$ $= 1/[1+(2/3)\sqrt{((35+7.25/2)/(35+7.25))}]$ $= 0.61$
For edge column bending perpendicular to the edge, the value of γ_f may be increased up to 25% provided that $V_u \le 0.4\varphi V_c$ (BNBC 2015, § 6.5.5.3.3) (ACI 318-11, § 13.5.3.3). No adjustment is made in this example.	For edge column bending perpendicular to the edge, the value of γ_f may be increased up to 25% provided that $V_u \le 0.4\varphi V_c$ (BNBC 2015, § 6.5.5.3.3) (ACI 318-11, § 13.5.3.3). No adjustment is made in this example.
Unbalanced moment transfer by flexure $=\gamma_f M_u = 0.61 \times 236 = 144$ kN-m. The required area of steel to resist this moment in the 1512 mm wide strip is $A_s = 2248$ mm^2, which is equivalent to 12 – d16.	Unbalanced moment transfer by flexure $=\gamma_f M_u = 0.61 \times 174 = 106$ kip-ft. The required area of steel to resist this moment in the 60 in. wide strip is $A_s = 3.6$ in.^{2}, which is equivalent to 12 – d16.
Provide the 12 -d16 bars by concentrating 12 of the column strip bars (20 – d16) within the 1512 mm width over the column. For symmetry, add another bar in the column strip and check bar spacing: ▪ For 12 - d16 within 1512 mm width: 1512/12 = 126 mm < 425 mm O.K. ▪ For 8 - d16 within (3200 - 1512) mm = 1688 mm width: 1688/8 = 211 mm < 425 mm O.K.	Provide the 12 -d16 bars by concentrating 12 of the column strip bars (20 – d16) within the 60 in. width over the column. For symmetry, add another bar in the column strip and check bar spacing: ▪ For 12- d16 within 60 in. width: 60/12 = 5 in. < 17 in. O.K. ▪ For 8 - d16 within (126 - 60) in. = 66 in. width: 66/8 = 8.25 in. < 17 in. O.K.

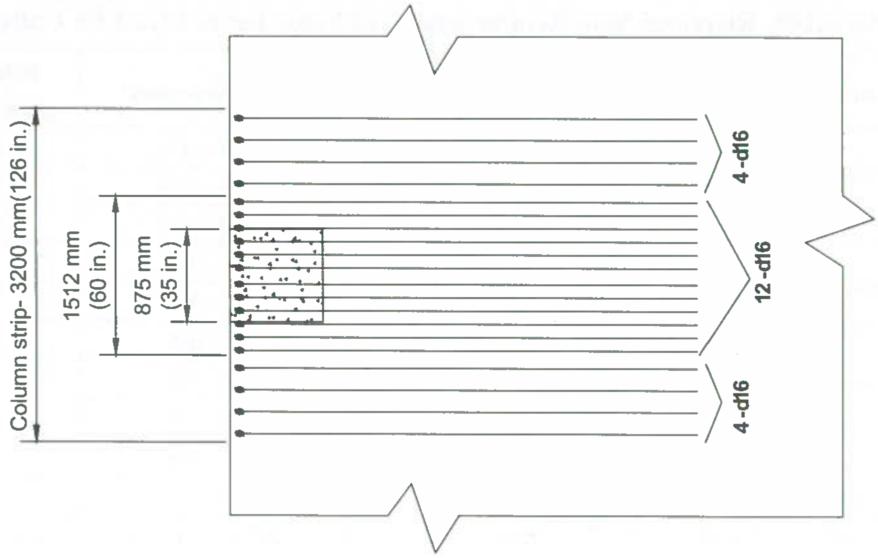


Figure 3.21 Reinforcement Detail at Exterior Column

First interior support – additional flexural reinforcement required for moment transfer:

In SI Units	In FPS Units
The total factored moment at first interior support is 339 - 336 = 15 kN-m (see Table 3.9. At an interior column, $\gamma_f = 0.6$ & $\gamma_f M_u = 0.6 \times 15 = 9$ kN-m.	The total factored moment at first interior support is 250 - 239 = 11 kip-ft (see Table 3.9. At an interior column, $\gamma_f = 0.6$ & $\gamma_f M_u = 0.6 \times 11 = 6.6$ kip-ft.
Required area of steel to resist this moment in the 1512 mm wide strip is A_s = 125 mm^2, which is less than 1(one) d16. It is clear that reinforcement provided in the column strip at the first interior column (27 – d16, see Table 3.10) can satisfy this requirement.	Required area of steel to resist this moment in the 60 in. wide strip is A_s = 0.2 in.^{2}, which is less than 1(one) d16. It is clear that reinforcement provided in the column strip at the first interior column (27 – d16, see Table 3.10) can satisfy this requirement.

Design for Shear:

In general, the total shear stress is the sum of the direct shear stress plus the shear stress due to the fraction of unbalanced moment transferred by eccentricity of shear (BNBC 2015, § 6.4.10.7.1) (ACI 318-11, § 11.11.7.1). Assuming shear stress from moment transfer by eccentricity of shear varies linearly about the centroid of the section, maximum factored shear stress v_u at the face of the critical section due to direct shear V_u & the unbalanced moment transferred by eccentricity of shear $\gamma_v M_u$ is:

$$v_u = \frac{V_u}{A_c} + \frac{\gamma_v M_u c}{J}$$

A_c = area of concrete section resisting shear transfer = $b_o d$

b_o = perimeter of critical section

γ_v = fraction of unbalanced moment transferred by eccentricity of shear

$$= 1 - \gamma_f = 1 - \frac{1}{1 + (2/3)\sqrt{b_1/b_2}}$$ [BNBC 2015 Equation 6.6.77, 6.6.81]

[(ACI 318-11, Equation 11-37, 13-1]

M_u = factored unbalanced moment at slab-column connection

J = property of critical section analogous to polar moment of inertia of segments forming A_c

c = distance from centroidal axis of critical section to perimeter of critical section in direction of analysis

Shear strength is checked at the slab-column connections at the end support and the first interior support for gravity forces & for gravity plus lateral forces.

End support – check for shear strength:

In SI Units

$$V_u = q_u\left(A_t - b_1 b_2\right) - \frac{M_1 - M_2}{\ell_n}$$

= 11.87(26 - 0.966 × 1.056) - [(275-102) / (7.32-875/1000)]
= 270 kN

A_t = tributary area of column
= (7.32/2 + 437.5/1000) × 6.40
= 26 m^2

b_1 = length of critical section perimeter in direction of analysis
= 875 + 181/2 = 966 mm (see Figure 3.21)

b_2 = length of critical section perimeter perpendicular to direction of analysis
= 875 + 181 = 1056 mm

M_1 = total negative design strip moment at interior support determined from Direct Design Method (BNBC 2015, § 6.5.6.3.3) (ACI 318-11, § 13.6.3.3)
= 0.70 M_o = 0.70 × 393 = 275 kN-m

M_1 = total negative design strip moment at exterior support determined from Direct Design Method (BNBC 2015, § 6.5.6.3.3) (ACI 318-11, § 13.6.3.3)
= 0.26 M_o = 0.26 × 393 = 102 kN-m

M_o = total factored static moment in span determined by BNBC 2015 Equation 6.6.84 (ACI 318-11, Equation 13-4)

$$M_o = \frac{q_u \ell_2 \ell_n^2}{8}$$

= 11.87×6.40×6.43^2/8
= 393 kN-m

where q_u =1.2 × 7.65 + 1.6 ×1.68 = 11.87kN/m^2

The section properties of the critical section are determined as follows;

A_c = (2b_1 + b_2) d = 541575mm^2

$$\frac{J}{c} = \frac{2b_1^2 d(b_1 + 2b_2) + d^3(2b_1 + b_2)}{6b_1}$$

= 18.2×10^{7}mm^3

According to BNBC 2015, § 6.5.6.3.6 (ACI 318-11, § 13.6.3.6), gravity load moment to be transferred between the slab & edge column must be set equal to 0.30M_o = 118kN-m. Also, γ_v =1- 0.61 = 0.39.

Therefore, the combined factored shear stress at the face of the critical section due to gravity loads is:

v_u = 270000/541575 + 0.39 × 118 × 1000 × 1000/(18.2 × 10^7)
= 0.75N/mm^2

In FPS Units

$$V_u = q_u\left(A_t - b_1 b_2\right) - \frac{M_1 - M_2}{\ell_n}$$

= 0.25 [283 - 38.6 × 42.25/144] - [(204-76)/(24-35/12)]
= 62 kips

A_t = tributary area of column
= (24/2 + 17.5/12)×21
= 283 ft^2

b_1 = length of critical section perimeter in direction of analysis
= 35 + 7.25/2 = 38.6 in. (see Figure 3.21)

b_2 = length of critical section perimeter perpendicular to direction of analysis
= 35 + 7.25 = 42.3 in.

M_1 = total negative design strip moment at interior support determined from Direct Design Method (BNBC 2015, § 6.5.6.3.3) (ACI 318-11, § 13.6.3.3)
= 0.70 M_o = 0.70 × 291.6 = 204kip-ft

M_1 = total negative design strip moment at exterior support determined from Direct Design Method (BNBC 2015, § 6.5.6.3.3) (ACI 318-11, § 13.6.3.3)
= 0.26 M_o = 0.26 × 291.6 = 76 kip-ft

M_o = total factored static moment in span determined by BNBC 2015 Equation 6.6.84 (ACI 318-11, Equation 13-4)

$$M_o = \frac{q_u \ell_2 \ell_n^2}{8}$$

= 0.25×21×21.08^2/8
= 291.6 kip-ft

where q_u = 1.2 × 0.162 + 1.6 × 0.035 = 0.25 psf

The section properties of the critical section are determined as follows;

A_c = (2b_1 + b_2) d = 866 in.^{2}

$$\frac{J}{c} = \frac{2b_1^2 d(b_1 + 2b_2) + d^3(2b_1 + b_2)}{6b_1}$$

= 11631 in.^{3}

According to BNBC 2015, § 6.5.6.3.6 (ACI 318-11, § 13.6.3.6), gravity load moment to be transferred between the slab & edge column must be set equal to 0.30M_o = 87.5 kip-ft. Also, γ_v =1- 0.61 = 0.39.

Therefore, the combined factored shear stress at the face of the critical section due to gravity loads is:

v_u = 62000/866 + 0.39 × 87.5 × 12 × 1000/11631
= 107 psi

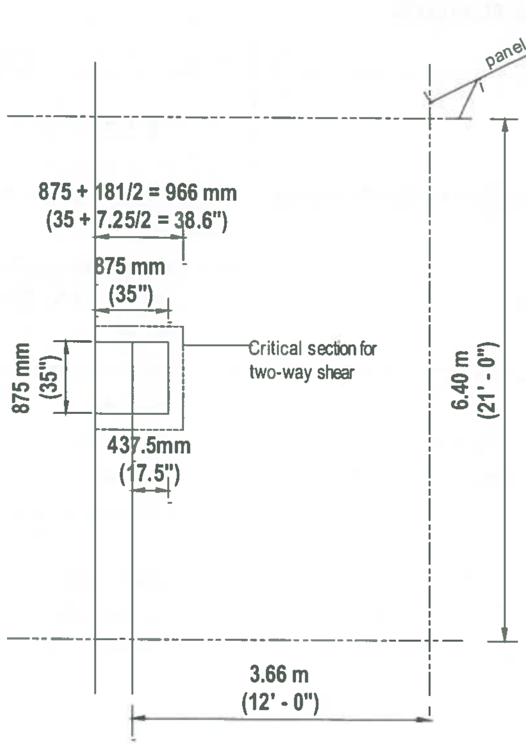


Figure 3.22 Critical Section for Two-way Shear at Exterior Column

Design shear strength of non-prestressed slabs is the smallest value determined in accordance with BNBC 2015 Equations 6.6.72 (ACI 318-11, Eq. 11-31), 6.6.73 (ACI 318-11, Equation 11-32), 6.6.74 (ACI 318-11, Equation 11-33). For square columns, Equation 6.6.74 (ACI 318-11, Equation 11-33) governs:

In SI Units

$\varphi\, v_c = \varphi V_c / b_o d = 0.33\, \varphi\, \sqrt{f_c'} = 0.33 \times 0.75 \times \sqrt{25}$

$= 1.24\text{N/mm}^2 > 0.75\text{N/mm}^2$ O.K.

In addition to gravity load case, shear strength must be checked for combined gravity & lateral loads. Direct shear forces on the critical section are as follows:

$$V_D = w_D(A_t - b_1 b_2) - \frac{M_{1D} - M_{2D}}{\ell_n}$$

$= 7.65(26 - 0.966 \times 1.056) - [(0.7 \times 253) - (0.26 \times 253)] / (7.32 - 875/1000)$

$= 174$ kN

$(M_o)_D = 7.65 \times 6.40 \times 6.43^2/8$ [BNBC 2015 Equation 6.6.84] [ACI 318-11, Eq. 13-4]

$= 253$kN-m

$$V_L = w_L(A_t - b_1 b_2) - \frac{M_{1L} - M_{2L}}{\ell_n}$$

$= 1.68(26 - 0.966 \times 1.056) - [(0.7 \times 56) - (0.26 \times 56)] / (7.32 - 875/1000)$

$= 38$ kN

$(M_o)_L = 1.68 \times 6.40 \times 6.43^2/8 = 56$ kN-m

$V_E = (137+137)/6.43 = 42.6$ kN

Therefore total factored shear force at exterior column is

$V_u = 1.275 V_D + 1.0 V_L + 1.0 V_E$

$= (1.275 \times 174) + (1.0 \times 38) + (1.0 \times 42.6)$

$= 303$ kN

In FPS Units

$\varphi\, v_c = \varphi V_c / b_o d = \varphi 4\sqrt{f_c'} = 0.75 \times 4 \times \sqrt{3625}$

$= 181\text{psi} > 107\text{psi}$ O.K.

In addition to gravity load case, shear strength must be checked for combined gravity & lateral loads. Direct shear forces on the critical section are as follows:

$$V_D = w_D(A_t - b_1 b_2) - \frac{M_{1D} - M_{2D}}{\ell_n}$$

$= 0.162(283 - 38.6 \times 42.3/144) - [(0.7 \times 189) - (0.26 \times 189)]/(24 - 35/12)$

$= 40$ kips

$(M_o)_D = 0.162 \times 21 \times 21.08^2/8$ [BNBC 2015 Equation 6.6.84] [ACI 318-11, Eq. 13-4]

$= 189$ kip-ft

$$V_L = w_L(A_t - b_1 b_2) - \frac{M_{1L} - M_{2L}}{\ell_n}$$

$= 0.035(283 - 38.6 \times 42.3/144) - [(0.7 \times 41) - (0.26 \times 41)]/(24 - 35/12)$

$= 8.7$ kips

$(M_o)_L = 0.035 \times 21 \times 21.08^2/8 = 41$ kip-ft

$V_E = (101+101)/21.08 = 9.6$ kips

Therefore total factored shear force at exterior column is

$V_u = 1.275 V_D + 1.0 V_L + 1.0 V_E$

$= (1.275 \times 40) + (1.0 \times 8.7) + (1.0 \times 9.6)$

$= 69$ kips

When lateral load is considered, shear stress computations can be based on the actual unbalanced moment, rather than on the provisions of BNBC 2015, § 6.5.6.3.6 (ACI 318-11, § 13.6.3.6), which as shown above, requires the unbalanced moment to be 0.30 M_o. the actual unbalanced moment at exterior slab column connection is 236 kN-m (see Table 3.9). The combined shear stress is v_u = 303000/541575 + 0.39 ×236×1000×1000/(18.2×10^7) = 1.07N/mm^2 <1.24N/mm^2 O.K.	When lateral load is considered, shear stress computations can be based on the actual unbalanced moment, rather than on the provisions of BNBC 2015, § 6.5.6.3.6 (ACI 318-11, § 13.6.3.6), which as shown above, requires the unbalanced moment to be 0.30 M_o. the actual unbalanced moment at exterior slab column connection is 174 kip-ft (see Table 3.9). The combined shear stress is v_u = 69000/866 + 0.39 ×174×12×1000/11631 = 150 psi < 181 psi O.K.

First interior support – check for shear strength:

At this location, the factored shear force V_u due to gravity load is:

In SI Units

$V_u = w_u(A_t - b_1b_2) + (M_1\text{-}M_2)/l_n$

$= 11.87(47 - 1.056^2) + (275\text{-}102)/(7.32\text{-}875/1000)]$

$= 541$ kN

$A_t = 7.32 \times 6.40 = 47\ \text{m}^2$

$b_1 = b_2 = 875{+}181 = 1056$ mm

$M_1 = 0.70\ M_o = 0.70 \times 393 = 275$ kN-m (see Table 3.7)

$M_2 = 0.26\ M_o = 0.26{\times}393 = 102$kN-m (see Table-3.7)

The section properties of the critical section are determined as follows:

$A_c = (2b_1 + b_2)d = 573408\text{mm}^2$

$$\frac{J}{c} = \frac{b_1 d(b_1 + 3b_2) + d^3}{3}$$

$= 27.1 \times 10^7\ \text{mm}^3$

The difference between the slab moments acting on opposite faces of the interior support needs to be transferred by shear to the first interior column. From Table 3.7, the exterior moment at the face of the support is 0.70 M_o = 0.70 × 393 = 275 kN-m, & the interior moment at the face of the support is 0.65 M_o = 0.65 × 393 = 256 kip-ft.

Therefore the unbalanced moment = 275 – 256 =19 kN-m. The combined shear stress is

$v_u = 541000/573408 + 0.4{\times}19 \times 1000 \times 1000/(27.1 \times 10^7)$

$= 0.97\text{N/mm}^2 < 1.24\text{N/mm}^2$ O.K.

Shear strength must also be checked for combined gravity & lateral loads. Direct shear forces on the critical section are as follows:

$$V_D = w_D(A_t - b_1b_2) - \frac{M_{1D} - M_{2D}}{\ell_n}$$

$= 7.65(47 - 1.056{\times}1.056) - [(0.7{\times}253) - (0.26 \times 253)]/(7.32\text{-}875/1000)$

$= 342$ kN

$$V_L = w_L(A_t - b_1b_2) - \frac{M_{1L} - M_{2L}}{\ell_n}$$

$= 1.68(47 - 1.056{\times}1.056) - [(0.7{\times}56) - (0.26{\times}56)]/(7.32\text{-}875/1000)$

$= 75$ kN

$V_E = (137{+}137)/6.43 = 42.6$ kN

Therefore total factored shear force at exterior column is

$V_u = 1.275V_D + 1.0V_L + 1.0V_E$

$= (1.275{\times}342) + (1.0{\times}75) + (1.0{\times}42.6)$

$= 554$ kN

In FPS Units

$V_u = w_u(A_t - b_1b_2) + (M_1\text{-}M_2)/l_n$

$= 0.25(504\text{-}42.25^2/144) + (204\text{-}76)/21.08$

$= 129$ kips

$A_t = 24 \times 21 = 504\ \text{ft}^2$

$b_1 = b_2 = 35{+}7.25 = 42.25$ in.

$M_1 = 0.70\ M_o = 0.70 \times 291.6 = 204$ kip-ft (see Table 3.7)

$M_2 = 0.26\ M_o = 0.26{\times}291.6 = 76$ kip-ft (see Table-3.7)

The section properties of the critical section are determined as follows:

$A_c = (2b_1 + b_2)d = 919\ \text{in.}^2$

$$\frac{J}{c} = \frac{b_1 d(b_1 + 3b_2) + d^3}{3}$$

$= 17383\ \text{in.}^3$

The difference between the slab moments acting on opposite faces of the interior support needs to be transferred by shear to the first interior column. From Table 3.7, the exterior moment at the face of the support is 0.70 M_o = 0.70 × 291.6 = 204 kip-ft, & the interior moment at the face of the support is 0.65 M_o = 0.65 ×291.6 = 190 kip-ft.

Therefore the unbalanced moment = 204 – 190 =14 kip-ft. The combined shear stress is

$v_u = 129000/919 + 0.4{\times}14{\times}12{\times}1000/17383$

$= 144$ psi < 181 psi O.K.

Shear strength must also be checked for combined gravity & lateral loads. Direct shear forces on the critical section are as follows:

$$V_D = w_D(A_t - b_1b_2) - \frac{M_{1D} - M_{2D}}{\ell_n}$$

$= 0.162\ (504 - 42.3 \times 42.3/144) - [(0.7{\times}189) - (0.26{\times}189)]/(24\text{-}35/12)$

$= 76$ kips

$$V_L = w_L(A_t - b_1b_2) - \frac{M_{1L} - M_{2L}}{\ell_n}$$

$= 0.035(504 - 42.3 \times 42.3/144) - [(0.7{\times}41) - (0.26{\times}41)]/(24\text{-}35/12)$

$= 16.4$ kips

$V_E = (101{+}101)/21.08 = 9.6$ kips

Therefore total factored shear force at exterior column is

$V_u = 1.275V_D + 1.0V_L + 1.0V_E$

$= (1.275{\times}76) + (1.0{\times}16.4) + (1.0{\times}9.6)$

$= 123$ kips

The unbalanced moment at the first interior support due to gravity loads is the difference between moments acting on the two sides of the support, while the unbalanced moment due to seismic forces is the sum of the moments acting on the two sides of the support:

In SI Units	In FPS Units
$M_u = 1.275\ [(0.7\times 253) - (0.65 \times 253)] + 1.0$ $[(0.7\times56) - (0.65\times56)] + (137+137)$ $= 293$ kN-m	$M_u = 1.275\ [(0.7\times189) - (0.65\times189)] + 1.0\ [(0.7\times 41)$ $- (0.65\times41)] + (101+101)$ $= 216$ kip-ft
The combined shear stress is:	The combined shear stress is:
$v_u = 554000/573408 + 0.4 \times 293 \times 1000 \times 1000/(27.1 \times 10^7)$ $= 1.39\text{N/mm}^2 < 1.24\ \text{N/mm}^2$ not O.K.	$v_u = 123000/919 + 0.4 \times 216 \times 12 \times 1000/17383$ $= 193 > 181$ psi not O.K.
If Slab thickness is increased to 225 mm then	If Slab thickness is increased to 9 in. then
$A_c = 613800\text{mm}^2$	$A_c = 982.3\ \text{in.}^2$
$\dfrac{J}{c} = 29.05\times10^7\ \text{mm}^3$	$\dfrac{J}{c} = 18601\ \text{in.}^3$
Then $v_u = \text{N/mm}^2 < \text{nN/mm}^2$ O.K.	Then $v_u = 180.96 < 181$ psi O.K.

Reinforcement Details

Slab reinforcement must conform to the requirements of BNBC 2015, § 6.5.3 (ACI 318-11, § 13.3). The provisions in BNBC 2015, § 6.5.3.8 must also be satisfied for slabs without beams; included are requirements for structural integrity (BNBC 2015, § 6.5.3.8.5) (ACI 318-11, § 13.3.8). Reinforcement should be done according to ACI 318-11, Figure R21.3.6.3- Arrangement of reinforcement in slabs)

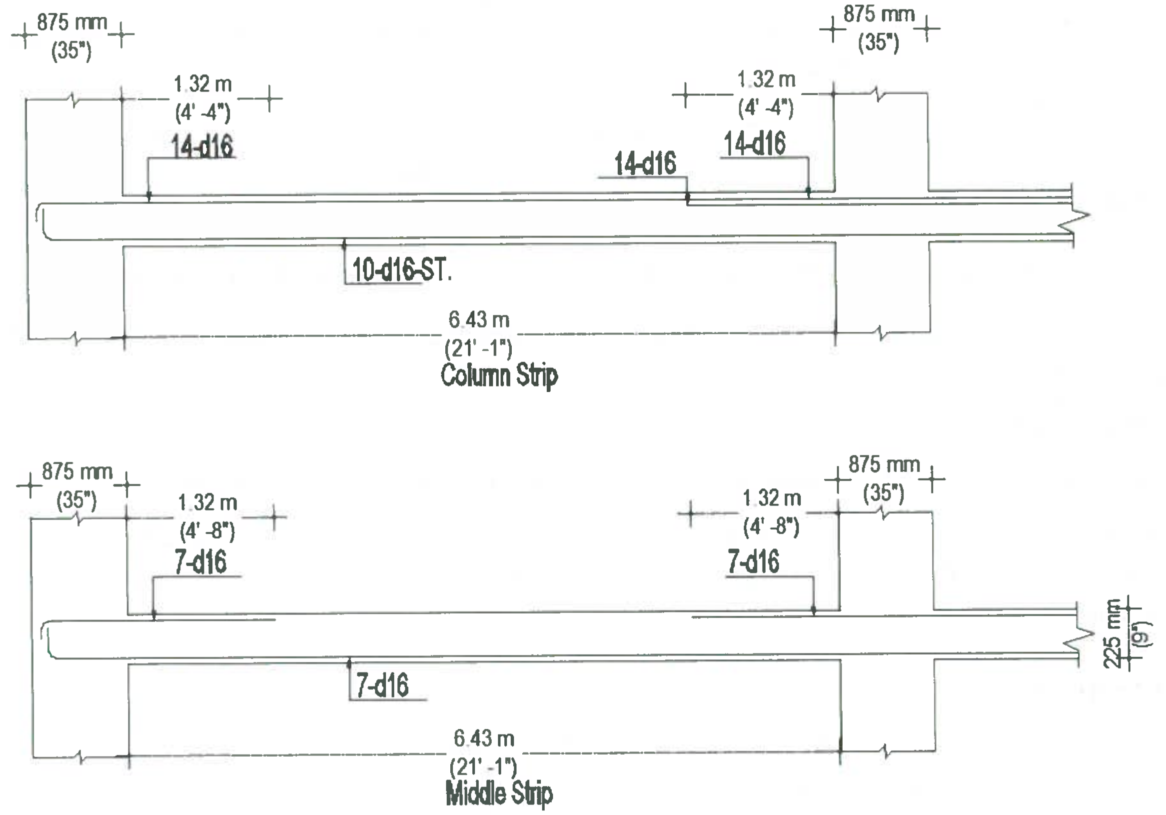


Figure 3.23 Reinforcement Details for Exterior Design Strip

In frames where two-way slabs act as primary members resisting lateral loads, length of reinforcement shall be determined by analysis but shall not be less than those prescribed in BNBC 2015 Figure 6.6.20 (BNBC 2015, § 6.5.3.8.4) [ACI 318-11, § 13.3.8.4].

Slab-Column Connections of two-way slabs without beams shall comply with BNBC 2015, § 8.3.12.4 (ACI 318-11, § 21.13.6). This conformance will satisfy the deformation compatibility requirements of BNBC 2015, § 2.5.14.4. The requirements of this section may be satisfied lieu of those shown above.

According to BNBC 2015, § 8.3.12.4 (ACI 318-11, § 21.13.6), reinforcement is not required to resist punching shear at slab-column connections where

$$\frac{\Delta}{h} < \max\left[0.005, 0.035 - 0.05\left(\frac{V_u}{\phi V_c}\right)\right]$$

Where Δ = design storey drift determined in accordance with BNBC 2015, § 2.5.7.7.

h = storey height

V_u = factored punching shear from gravity loads excluding shear stress from unbalanced moment, calculated for the load combination $1.2D + 0.5L$

ϕV_c = design two-way shear strength provided by concrete, computed in accordance with BNBC 2015, § 6.4.10.2 (ACI 318-11, § 11.11.2).

This requirement will be checked at both an end support & an interior support.

End support

In SI Units	In FPS Units
The design storey drifts in the N-S direction are summarized in Table 3.5. The largest Δ/h = 30/ (3.048×1000) = 0.0097, which occurs at the 6th, 7th, 8th, 9th, 10th floors.	The design storey drifts in the N-S direction are summarized in Table 3.5. The largest Δ/h = 1.17/ (10×12) = 0.0097, which occurs at the 6th, 7th, 8th, 9th, 10th floors.
It was determined above that V_u due to gravity loads at an end support = 320 kN. Also,	It was determined above that V_u due to gravity loads at an end support = 72 kips. Also,
$\varphi V_c = \varphi 4\sqrt{f_c'}\, b_o d$	$\varphi V_c = \varphi 4\sqrt{f_c'}\, b_o d$
$= 0.75 \times 0.33\sqrt{25} \times (4 \times 1056.25) \times 181.25/1000$	$= 0.75 \times 4\sqrt{3625} \times (4 \times 42.25) \times 7.25/1000$
= 948kN	= 221 kips
Therefore,	Therefore,
0.035-0.05(320/948) = 0.019	0.035-0.05(72/221) = 0.019
Δ/h = 0.0097 < max(0.005, 0.019) = 0.019 O.K.	Δ/h = 0.0097< max(0.005, 0.019) = 0.019 O.K.

Slab shear reinforcement is not required & the deformation compatibility requirements of BNBC 2015, § 2.5.14.4 are satisfied.

Interior support

At an interior support, no slab shear reinforcement is required, since Δ/h = < 0.0019.

Similar calculations at the end & interior supports in the E-W direction show that no slab reinforcement is required. Thus, the deformation compatibility requirements of BNBC 2015, § 2.5.14.4 are satisfied in this example without providing any shear reinforcement in the slab.

REFERENCE

1. Bangladesh National Building Code 2015 (BNBC 2015)
2. American Concrete Institute (ACI 318-11)
3. Seismic and Wind Design of Concrete Buildings- S.K Ghosh, Qiang Shen -3rd Edition, Portland Cement Association, December 2008.
4. PCA Notes 318-08 Building code requirements for structural concrete with design application, Portland Cement Association.

CHAPTER 4. SEISMIC RESPONSE OF RC FRAME STRUCTURE

4.1 DESIGN AND CONSTRUCTIONAL FEATURES IMPORTANT TO SEISMIC PERFORMANCE

A number of characteristics are important to the design of buildings and structures to ensure that they will behave adequately in strong earthquakes. These include:

Strength
Stiffness
Ductility
Irregularity of Frame
Continuous Load Pass
Stable foundation

4.1.1 Strength

Strength is a measure of how well a material can resist being deformed from its original shape. Typically, metals are specified for their tensile strength, or their resistance to being pulled apart, but compressive strength is also a legitimate material property describing resistance to being squeezed. Strength is measured in units of pressure, and is typically reported in units of MPa or Newton per mm^{2}.

If a structure is to be protected against damage during a selected or specified seismic event, inelastic displacement during its dynamic response should be prevented. This means that the structure must have adequate strength to resist internal actions generated during the elastic dynamic response of the structure. Therefore, the appropriate technique for the evaluation of earthquake-induced actions is an elastic analysis, based on stiffness properties. These seismic actions, combined with those due to other loads on the structure, such as gravity, will lead, perhaps with minor modifications, to the proportioning of structural members. Thereby the designer can provide the desired strength, shown as S_i in Figure 4.1, in terms of resistance to lateral forces envisaged.

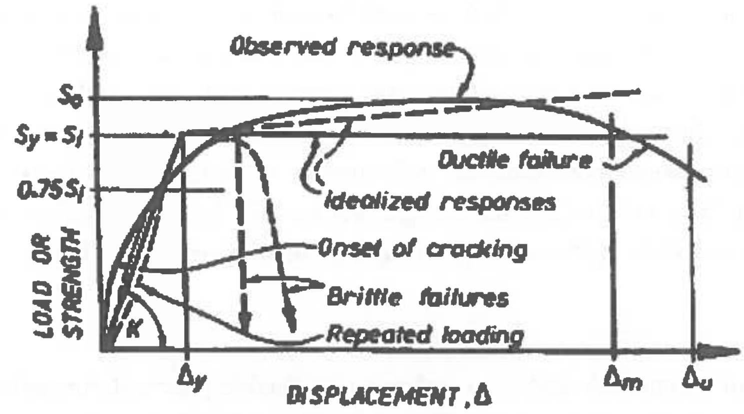


Figure 4.1 Typical Load-Displacement Relationship for a Reinforced Concrete Element
[After Ref. 4.9]

4.1.2 Stiffness

Stiffness is the rigidity of an object - the extent to which it resists deformation in response to an applied force. The stiffness, k, of a body is a measure of the resistance offered by an elastic body to deformation. For an elastic body with a single degree of freedom (for example, stretching or compression of a rod), the stiffness is defined as

$$k = \frac{F}{\delta}$$

Where,

F is the force applied on the body

δ is the displacement produced by the force along the same degree of freedom (for instance, the change in length of a stretched spring)

In the International System of Units, stiffness is typically measured in newtons per metre. In imperial units, stiffness is typically measured in pounds (lbs) per inch.

If deformations under the action of lateral forces are to be reliably quantified and subsequently controlled, designers must make a realistic estimate of the relevant property-stiffness. This quantity relates loads or forces to the ensuing structural deformations. Familiar relationships are readily established from first principles of structural mechanics, using geometric properties of members and the modulus of elasticity for the material. In reinforced concrete and masonry structures these relationships are, however, not quite as simple as an introductory text on the subject may suggest. If serviceability criteria are to be satisfied with a reasonable degree of confidence, the extent and influence of cracking in members and the contribution of concrete or masonry in tension must be considered, in conjunction with the traditionally considered aspects of section and element geometry, and material properties.

A typical nonlinear relationship between induced forces or loads and displacements, describing the response of a reinforced concrete component subjected to monotonically increasing displacements, is shown in Figure 4.1. For purposes of routine design computations, one of the two bilinear approximations may be used, where S_y defines the yield or ideal strength S; of the member. The slope of the idealized linear elastic response, $K = S_y/\Delta_y$ is used to quantify stiffness. This should be based on the effective secant stiffness' to the real load-displacement curve at a load of about $0.75S_y$, as shown in Figure 4.1, as it is effective stiffness at close to yield strength that will be of concern when estimating response for the serviceability limit state. Under cyclic loading at high "elastic" response levels, the initial curved load-displacement characteristic will modify to close to the linear relationship of the idealized response. An early task within the design process will be the checking of typical inter-story deflections (drift), using realistic stiffness values to satisfy local requirements for serviceability.

4.1.3 Ductility

Ductility is a measure of a material's ability to undergo appreciable plastic deformation before fracture; it may be expressed as percent elongation or percent area reduction from a tensile test.

An excellent example for discussing "ductility" of reinforced concrete members is a frame-beam shown in Figure 4.2. The term "frame-beam" applies to a beam that is designed as part of a lateral system. Otherwise it is simply referred to as a gravity beam. When subjected to seismic ground motions, the frame sways back and forth resulting in flexural and shear cracks in the beam. These cracks close and open alternately due to load reversal and following several cycles of loading, the beam will resemble Figure 4.2. As a result of the back and forth lateral deflections, the two ends of the beam are divided into a series of blocks of concrete held together by the reinforced cage.

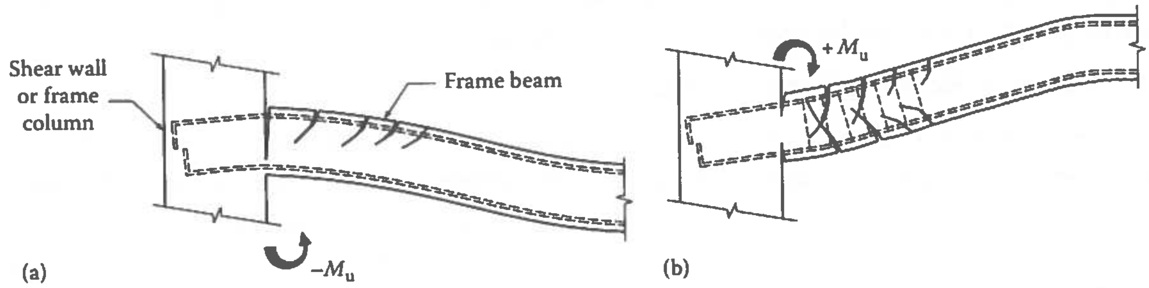


Figure 4.2 Frame-Beam Subjected to cyclic load: (a) cracks due to $-M_u$ and (b) cracks due to $+M_u$ [*After Ref. 4.9*]

If the beam cracks through, shear is transferred across the crack by the dowel action of the longitudinal reinforcement and shear friction along the crack. After the concrete outside the reinforcement crushes, the longitudinal bar will buckle unless restrained by closely spaced stirrups or hoops.

The hoop also provides confinement of the core concrete increasing it ductility. Ductility is the general term that describes the ability of the structure or its components to provide resistance in the inelastic domain of response. It includes the ability to sustain large deformations and a capacity to absorb energy by hysteretic behavior, the characteristics that are vital to a building's survival during and after a large earthquake. This capability of sustaining a high proportion of their strength that ensures survival of buildings when a major earthquake imposes large deformation is the single most important property sought by the designer of buildings located in regions of significant seismicity.

The limit to ductility, such as the displacement of Δ_u, typically corresponds to a specified limit to strength degradation. Even after attaining this limit, sometimes termed "failure," significant additional inelastic deformations may still be possible without structural collapse. Brittle failure, on the other hand, implies near-complete loss of resistance, often complete disintegration without adequate warning. For these reasons, brittle failure, which is the overwhelming cause for collapse of buildings in earthquakes, and the consequent loss of lives, must be avoided.

Ductility is defined by the ratio of the total imposed displacements Δ at any instant to that at the onset of yield Δ_y. From Figure 4.1, we have

$$\mu = \Delta / \Delta_y > 1 \qquad (4.1)$$

Ductility may also be defined in terms of strain, curvature, rotation, or deflection. An important consideration in the determination of the required seismic resistance will be that the estimated maximum ductility demand during shaking, $\mu_m = \Delta_m / \Delta_y$ does not exceed the ductility potential μ_u. In structural engineering, the roles of both stiffness and strength of members, as well as their quantification is well understood. However, quantification and utilization of the concept of ductility as a design tool are generally less well understood.

Ductility in structural members can be developed only if the constituent material itself is ductile. Concrete is an inherently brittle material. Although its tensile strength cannot be relied upon as a primary source of resistance, it is eminently suited to carry compression stresses. However, the maximum strains developed in compression are rather limited to about 0.003, unless special precautions are taken. Therefore, the primary aim of seismic detailing of concrete structures is to combine mild steel reinforcement and concrete in such way as to produce ductile members that are capable of meeting the inelastic deformation demands imposed by severe earthquakes.

4.1.3.1 Ductility of Reinforced Concrete Structure

Ductility or deformation capacity of reinforced concrete systems is provided by the ductility of its constituent materials (steel and concrete), ductility of its members (beams, columns and walls), and the overall ductility of the structural system under seismic actions. It should be noted that a ductile reinforced concrete response can be obtained only if the dominant failure mode of the structural components is flexure. Therefore brittle failure modes such as shear, diagonal tension and compression should be prevented whereas ductility in flexure should be enhanced for obtaining a ductile system response under strong seismic excitations.

4.1.3.2 Ductility of Reinforced Concrete Material

A ductile flexural member behavior can be achieved by employing materials with ductile stress-strain behavior at the critical sections where bending moments are maximum. Typical stress–strain curves for standard steel bars used in reinforced concrete construction when loaded monotonically in tension are shown in Figure 4.3. The curves exhibit an initial elastic portion, a yield plateau, a strain hardening range in which stress again increases with strain and finally, a range in which the stress drops off before fracture occurs. The length of the yield plateau is generally a function of the strength of the steel. High-strength, high-carbon types of steel generally have a much shorter yield plateau than lower strength low -carbon steel. Similarly, the cold-working of steel can cause the shortening of the yield plateau to the extent that strain hardening commences immediately after the onset of yielding. High-strength steels also have a smaller elongation before fracture than low-strength steels.

The minimum strain in the steel at fracture is also defined in steel specifications, since it is essential for the safety of the structure that the steel should be ductile. One of the two constituents of reinforced concrete is steel, which is inherently ductile enough to undergo large deformations before fracture. ASTM specifications (*Ref.4.17*) for deformed high-yield bars require an elongation, defined by the permanent extension of an 8 inch (203 mm) gauge length at the fracture of the specimen, expressed as a percentage of the gauge length, which varies with the source, grade, and bar diameter of the steel and ranges from at least 4.5 to 12%. The influence of high-strength steel on cracking and deflection of structural concrete members led to a series of studies in the past into the service behavior of such steel (*Ref. 4.18–4.22*).

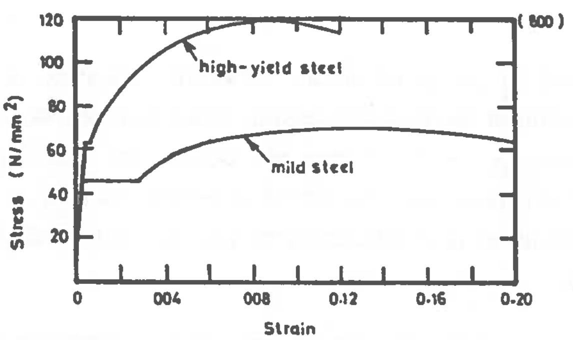


Figure 4.3 Stress-strain Relationships for Structural Steel.

The other constituent material, plain concrete does not possess such ductile uniaxial material stress-strain behavior (see curve with $\sigma_2 = 0$ in Figure 4.4). However when the conditions of stress change from uniaxial ($\sigma_2 = 0$) to tri-axial ($\sigma_2 > 0$), both stress and strain capacities of concrete enhance significantly with the increasing lateral pressure, as shown in Figure 4.4.

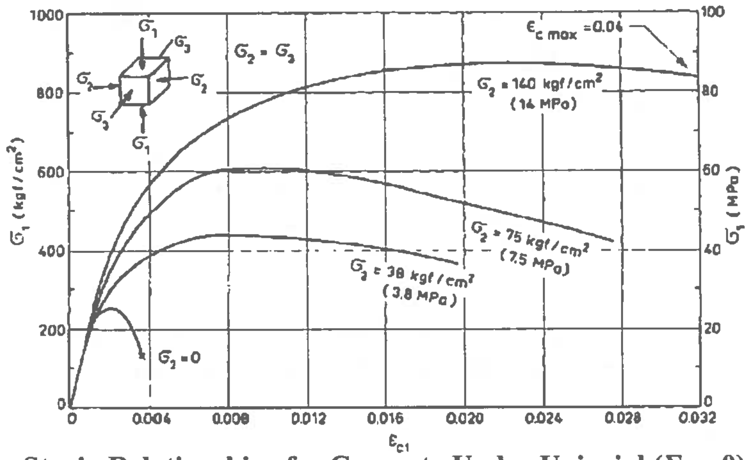


Figure 4.4 Stress-Strain Relationships for Concrete Under Uniaxial ($\Sigma_2 = 0$) and Triaxial ($\Sigma_2 > 0$) Stress.

Triaxial stress state in reinforced concrete members can be provided with confinement reinforcement. When concrete is subjected to axial stress σ1, passive lateral pressure σ2 developed by the lateral tie reinforcement (Figure 4.5 a, b and c) provides enormous increase in the strength and strain capacity of concrete. The improvement is strongly related with the tie spacing "*s*" (Figure 4.5 d, e and f).

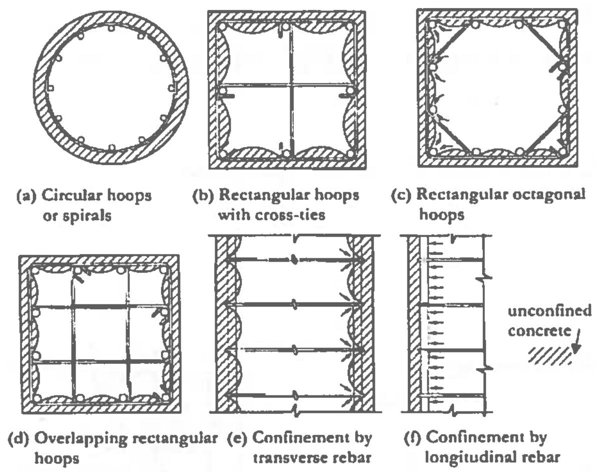


Figure 4.5 Confinement of Column Sections by Transverse Hoops and Spirals.

Strength and deformation capacities of concrete fibers in the core region of columns increase with the amount of lateral confinement reinforcement (Figure 4.6). Confinement is most effective in circular columns since lateral pressure develops uniformly in all radial directions whereas a rectangular tie is more effective at the corners as shown in Figure 4.5 c.

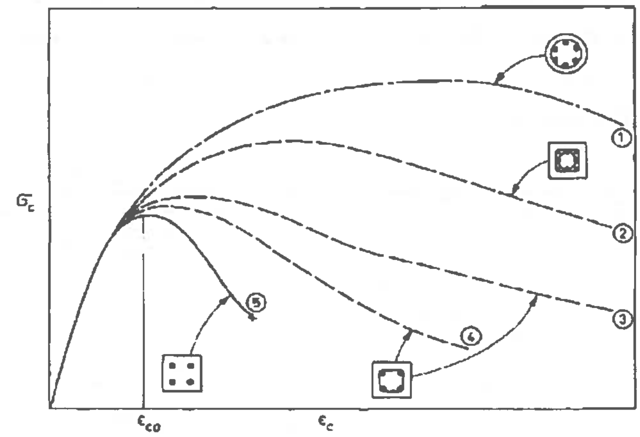


Figure 4.6 Stress-Strain Relations for Concrete In Unconfined (Curve 5) and Confined (Curves 1- 4) Reinforced Concrete Sections.

4.1.3.3 Section Ductility

The inelastic behavior of a reinforced concrete section is usually evaluated through its moment-curvature diagram (Figure 4.7). Section ductility is expressed either through strain or curvature ratios, though the latter ($\mu_u = \Phi_u / \Phi_y$) is more commonly used. The value of yield curvature Δ_y is usually associated with yield of reinforcement, whilst ultimate curvature Δ_u is normally dependent on ultimate compressive strain in the concrete.

If moment-curvature characteristic of the section is approximated by elasto-plastic relationship, the yield curvature Φ_y will not necessarily coincide with the first yield of the tensile reinforcement, which will generally occur at somewhat lower curvature Φ_y', particularly if the reinforcement is distributed around the section as would be case of the column.

From Figure 4.7 (a), the first yield curvature Φ_y' can be represented as:

$$\phi_y' = \frac{\varepsilon_y}{\left(d - c_y\right)} \tag{4.2}$$

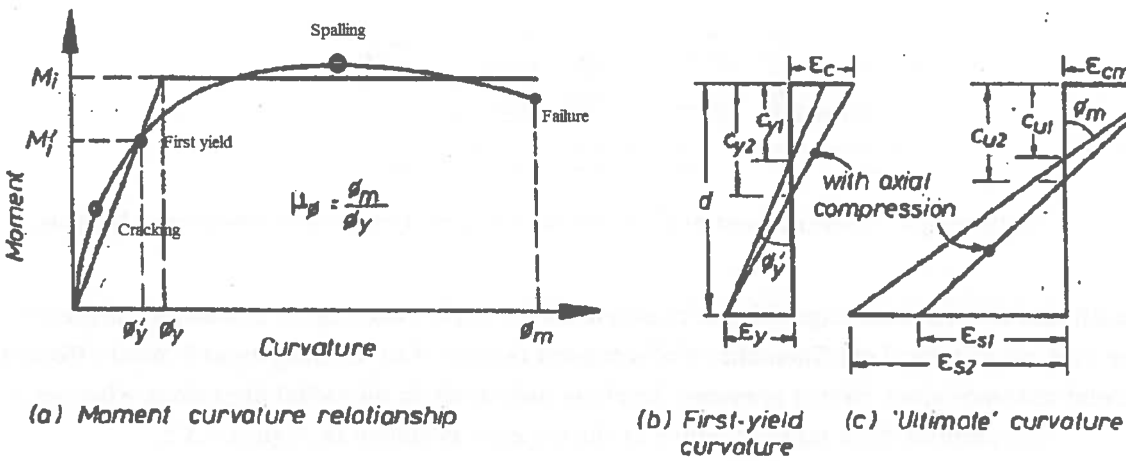


Figure 4.7 Definition of Curvature Ductility [*After Ref.4.9*]

Extrapolating linearly to the ideal moment M_i, as shown in Figure 4.7 (a), the yield curvature Φ_y is given by:

$$\phi_y = \frac{M_i}{M_i^{\prime}} \phi_y^{\prime} \tag{4.3}$$

If the section has a very high reinforcement ratio, or is subjected to high axial load, high concrete compression strain may develop before the first yield of reinforcement occurs. For such cases the yield curvature should be based on the compression strains

$$\phi_y^{\prime} = \frac{\varepsilon_c}{c_y} \tag{4.4}$$

Where ε_c is taken as 0.0015.

An acceptable approximation for beam sections is to calculate steel and concrete extreme fiber strains, and hence the curvature $\Phi_y^{'}$ based on conventional elastic section analyses at a moment of $M_i^{'} = 0.75M_i$, thus providing an equivalent yield curvature of φ_y=1.33 $\Phi_y^{'}$.

The *ultimate curvature* as it is generally termed, is normally controlled by the maximum compression strain ε_{cm} at the extreme fiber, since steel strain ductility capacity is typically high. With reference to Figure 4.7 (c), this curvature may be expressed as

$$\phi_m = \frac{\varepsilon_{cm}}{c_u} \tag{4.5}$$

For the purpose of estimating curvature, the maximum dependable concrete compression strain in the extreme fiber of unconfined beam, column, or wall sections may be assumed to be 0.004, when normal-strength concrete[$f_c^{'} \leq$ 45MPa] is used. However, for adequately confined concerete much larger compression strains may be attained and in such situations the contribution of any concrete outside a confined core, which may be subjected to compression strains in excess of 0.004, should be neglected. This generally implies spalling of the cover concrete.

(i) Factors affecting curvature ductility. Curvature ductility μ_u is mainly dependent on axial force level, compressive strength and reinforcement yield strength.

(a) Axial Force. As shown in Figure 4.7 (b) and (c), the presence of axial compression will increase the depth of the compression zone at both first yield (c_{y2}) and al ultimate (c_{u2}). By comparison with conditions without axial force (c_{y1} and c_{u1}) it is apparent that the presence of axial compression increases the yield curvature, φ_y, and decreases the ultimate curvature, φ_u .Consequently, axial compression can greatly reduce the available curvature' ductility capacity, μ_u of a section. Conversely, the presence of axial tension greatly increases the ductility capacity.

(b) Compressive strength of concrete. Increased compression strength of concrete or masonry has exactly the opposite effect to axial compression force: the neutral axis depth at yield and ultimate are both reduced, hence reducing yield curvature and increasing ultimate curvature. Thus increasing compression strength is an effective means for increasing section curvature ductility capacity.

(c) Yield strength of reinforcement. Higher values of reinforcement area and yield strain ε_y means that the yield curvature will be increased. Hence the curvature ductility ratio, $\mu_u = \varphi_u / \varphi_y$ will be less for high-strength steel.

4.1.3.4 Member Ductility

Displacement defined as $\mu_\Delta = \Delta_u / \Delta_y$ is often used to quantify member ductility. These member ductility factors provide the designer with crucial information regarding the behavior of reinforced concrete structural elements. Such information is not given by the section ratios, such as strain or curvature ductility. Yield displacement Δ_y is normally related to yield of reinforcement bars or to a pre-defined member sway (normally used in vertical members only). Ultimate displacement Δ_u may be obtained when concrete crushing failure is reached or when buckling or fracture of the reinforcement bars occurs.

A large value of strain or curvature ductility at the critical sections of a structural member will be of little use if insufficient member detailing does not allow the development of an appropriate plastic hinge length. On the contrary, rotation or displacement ductility will take into account both the section and the member properties in its values. This is illustrated Figure 4.8 for the case of a bridge pier cantilever system. The inelastic deformation Δ_p results from the spread of section inelasticity beyond the critical section, which is located at the foundation level.

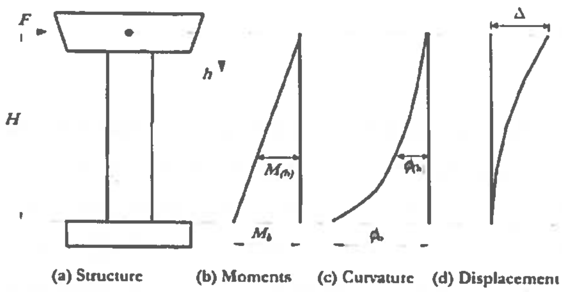


Figure 4.8 Obtaining Displacements from Curvature Distribution [*After Ref. 4.1*]

Using the nomenclature of Figure 4.8, and measuring the distance h down from the line of application of the inertia force, the moment at h, and at the base ($h = H$) will be given by:

(4.6) $$M_h = F.h; \text{ and } M_b = F.H$$

The curvatures at all heights h could then be read from the moment-curvature relationship to produce the curvature distribution ϕ_h shown in Figure 4.8(c), which could be integrated to provide the top displacement, Δ as:

(4.7) $$\Delta = \int_0^H \phi_{(h)} h.dh$$

Repeating this process for values of $0 \leq F \leq M_u/H$ would then be expected to provide-he full force-displacement response. Unfortunately this process does not produce force-displacement predictions that agree well with experimental results as number of factors such as shear deformation, anchorage deformation (strain-penetration) is ignored. Equation (4.11) implies that the curvature drops to zero immediately below the column base (or, for a beam, at the column face). In fact, strains of the tension reinforcement will only drop to zero at a depth equal to the true development length of the reinforcement. The solution to these problems is to use a simplified approach based on the concept of a *"plastic hinge"*, of length L_p over which strain and curvature are considered to be equal to the maximum value at the column base.

(i) Factors affecting member ductility. The role played by the reinforcement characteristics and axial load is determinant in the member ductility, though thorough reasons distinct from those considered at section level. Higher values of axial load increase second order effects, thus leading to earlier collapse of the member. On the contrary, higher levels of transverse reinforcement will decrease the risk of buckling of the reinforcement bars, hence increasing the ductility. If the ration of ultimate strength to yield strength of the flexural reinforcement is high, plastic deformation spreads away from the critical section as the reinforcement at the critical section strain-hardens, increasing the plastic hinge length and hence increase in ductility.

4.1.3.5 Global Ductility

A ratio between global yield and ultimate displacements at the top of the structure can also be used to quantify their ductility. In this case, instead of a moment vs. curvature (section level) or transverse force vs. displacement (member level), the ductility plot consists of total drift at the top of the structure vs. total base-shear.

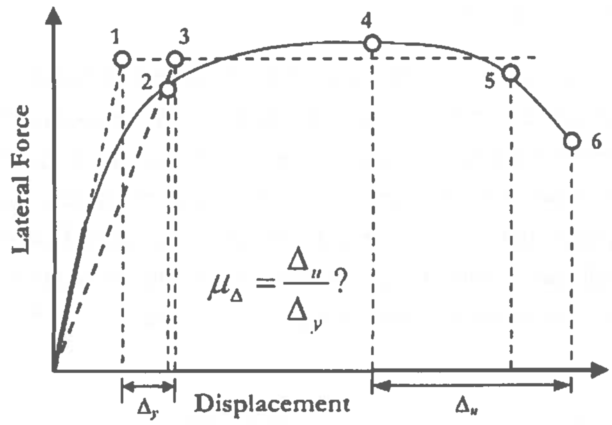


Figure 4.9 Defining the Ductility Capacity [*After Ref. 4.10*]

There has been difficulty in reaching consensus within the research community as to the appropriate definition of yield and ultimate displacements. With reference to Figure 4.9, the yield displacement has variously been defined as the intersection of the line through the origin with initial stiffness, and the nominal strength **(point 1)**, the displacement at first yield **(point 2)**, and the intersection of the line through the origin with secant stiffness through first yield, and the nominal strength **(point 3)**, amongst other possibilities. Typically, displacements at **point 3** will be *1.8 to 4* times the displacements at **point 1**. Displacement capacity, or ultimate displacement, also has had a number of definitions, including displacement at peak strength **(point 4)**, displacement corresponding to 20% or 50% (or some other percentage) degradation from peak (or nominal) strength, **(point 5)** and displacement at initial fracture of transverse reinforcement **(point 6)**, implying imminent failure. Clearly with such a wide choice of limit displacements, there has been considerable variation in the assessed displacement ductility capacity of structures.

With reference to Figure 4.9, the yield displacement is taken to be defined by point 3, and the ultimate displacement by the lesser of displacement at **point6** or **point5**, where **point5** is defined by a strength drop of 20% from the peak strength obtained.

(i) Factors affecting global ductility. Global ductility can be enhanced by judicious implementation of capacity design which ensures inelastic deformations at suitable location (in beams rather than in columns-weak beam/strong column mechanism) and through appropriate strength differentials that ensures inelastic deformation doesn't occur at undesirable locations or undesirable structural modes (shear failure). In addition to the aforementioned capacity design rules, other design features are also paramount in guaranteeing the attainment of global structural ductility. These are:

- Continuity and redundancy between members, so as to ensure a clear load path for horizontal loads and prevent brittle failures;
- Regularity of mass, stiffness and strength distribution, to avoid adverse torsional effects and soft-storey mechanisms;
- Reduced masses and sufficient stiffness, to avoid highly flexible structures which may lead to heavy non-structural damage and significant P-Δ effects.

4.1.3.6 Other Factors Concerning Ductility

Implicit in the force-reduction factor approach suggested in different building codes, is that unique ductility capacities and hence unique force-reduction factors can be assigned to different structural systems. As for example, concrete frame is assigned a force reduction of 8 in US west coast where as a value of *1.8-3.3* is applied in Japan for the identical system and materials. It has, however, become apparent over the past two decades, that is an unacceptable approximation. Ductility capacity of concrete and masonry structures depends on a wide range of factors, including axial load ratio, reinforcement ratio, and structural geometry. Foundation compliance also can significantly affect the displacement ductility capacity.

An example of the influence of structural geometry on displacement capacity is provided in Figure 4.10, which compares the ductility capacity of two bridge columns with identical cross-sections, axial *loads* and reinforcement details, but with different heights. The two columns have the same yield curvatures φ_y and ultimate curvatures φ_u and hence the same curvature ductility factor $\mu_u = \Delta_u / \Delta_y$. Yield displacements, however, may be approximated by

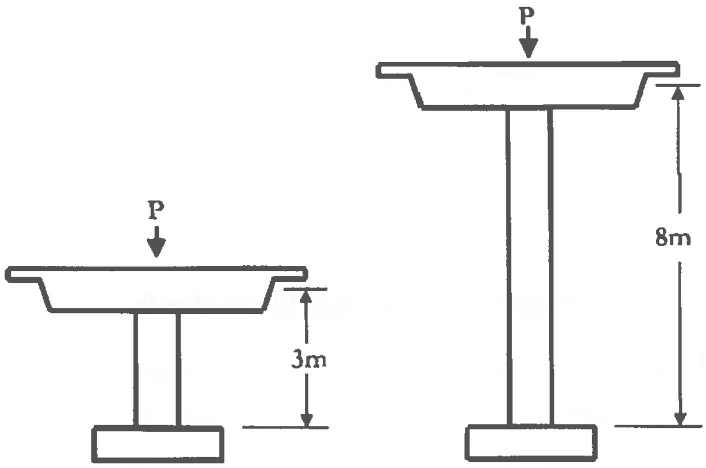
(4.8) $$\Delta_y = \frac{\phi_y H^2}{3}$$

Where H is the effective height, and the plastic displacement $\Delta_p = \Delta_u - \Delta_y$ by

(4.9) $$\Delta_p = \phi_p L_p H$$

Where, $\Delta_p = \Delta_u - \Delta_y$ is the plastic curvature capacity, and L_p is the plastic hinge length.

(4.10) $$\mu_\Delta = \frac{\Delta_u}{\Delta_y} = \frac{\Delta_p + \Delta_y}{\Delta_y} = 1 + 3\frac{\phi_p L_p}{\phi_y H}$$



(a) Squat Column, μ_Δ = 9.4 (b) Slender Column, μ_Δ = 5.1

Figure 4.10 Influence of Height on Displacement Ductility Capacity of Circular Columns (P = $0.1f_c^{/}A_g$; P_l= 0.02 and ρ_s= 0.006) [*After Ref.*4.11]

Referring to Equation (4.10) it is thus seen that the displacement ductility capacity reduces as the height increases. Using the above approach where the height-dependency of L_p is considered, it is found that the squat column of Figure 4.10 (a) has a displacement ductility capacity of μ_Δ= 9.4, while for the more slender column of Figure 4.10(b), μ_Δ = 5.1. The calculated displacement ductility capacity of the two columns differ by a factor of two, as a consequence of the plastic hinge length, and hence the plastic rotation, being only weakly dependent on the column height, while the elastic drift ratio is directly proportional to height [*Ref.4.11*]. Clearly the concept of uniform displacement ductility capacity, and hence of a constant force-reduction factor is inappropriate for the very simple class of structure.

4.1.4 Irregularity of Frame

A structure is "regular" if the distribution of its mass, strength, and stiffness is such that it will sway in a uniform manner when subjected to ground shaking – that is, the lateral movement in each storey and on each side of the structure will be about the same. Regular structures tend to dissipate the earthquake's energy uniformly throughout the structure, resulting in relatively light but well-distributed damage. In an irregular structure, however, the damage can be concentrated in one or a few locations, resulting in extreme local damage and a loss of the structure's ability to survive the shaking.

4.1.4.1 Horizontal Structural Irregularity

(i) Torsional irregularity. This condition exists when the distribution of vertical elements of the seismic-force-resisting system within a storey, including braced frames, moment frames and walls, is such that when the building is pushed to the side by wind or earthquake forces, it will tend to twist as well as deflect horizontally. Torsional irregularity is determined by evaluating the difference in lateral displacement that is calculated at opposite ends of the structure when it is subjected to a lateral force.

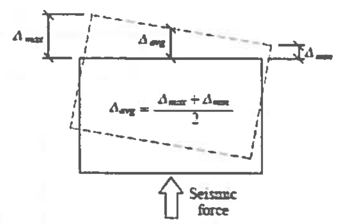


Figure 4.11 Torsional Irregularity

(ii) Extreme torsional irregularity. This is a special case of torsional irregularity in which the amount of twisting that occurs as the structure is displaced laterally becomes very large.

(iii) Re-entrant corner irregularity. A building has reentrant corner irregularity when one or more parts of the structure project beyond a reentrant corner a distance greater than 15% of the plan dimension in the given direction. Figure 4.12 shows typical plan having reentrant corner irregularity.

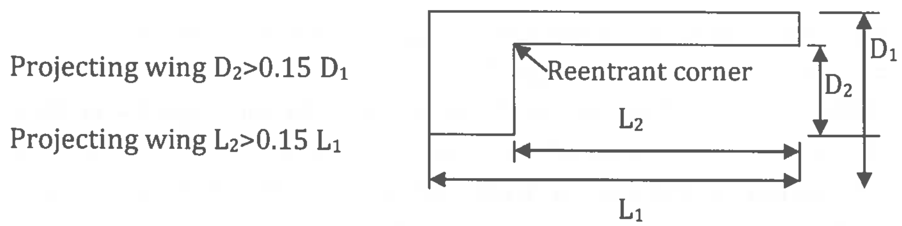


Figure 4.12 Re-entrant Corner Irregularity

(iv) Diaphragm discontinuity irregularity. This occurs when a structure's floor or roof has a large open area as can occur in buildings with large atriums. Figure 4.13 shows this type of geometry of building.

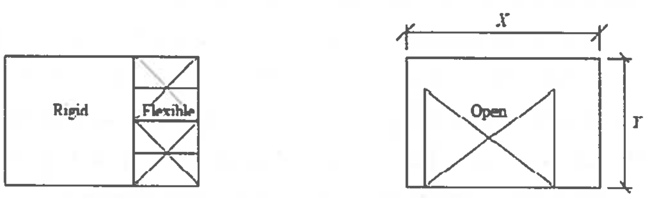


Figure 4.13 Diaphragm Discontinuity Irregularity

(v) Out-of-plane offset irregularity. This occurs when the vertical elements of the seismic-force-resisting system, such as braced frames or shear walls, are not aligned vertically from storey to storey. Out-of-plane offset irregularity is shown in Figure 4.14.

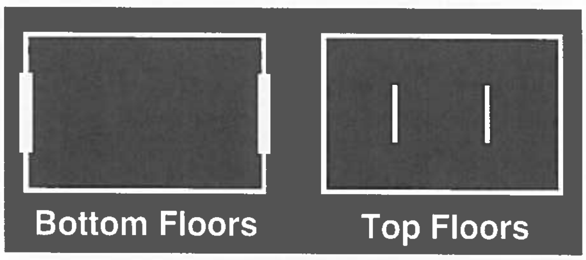


Figure 4.14 Out-of-plane Offset Irregularity

(vi) Nonparallel systems irregularity. This occurs when the structure's seismic-force-resisting does not include a series of frames or walls that are oriented at approximately 90-degree angles with each other. Figure 4.15 shows this type of geometry of building.

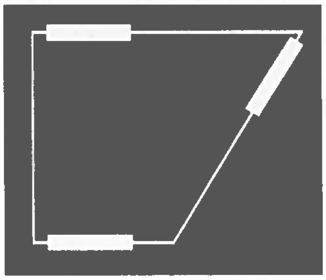


Figure 4.15 Nonparallel Systems Irregularity

4.1.4.2 Effect of horizontal irregularity

Regular rectangular plan shapes are preferable to winged, T, L or U shapes for architectural reasons. This is because winged structures and structures with re-entrant corners suffer from non-uniform ductility demand distribution, as shown in Figure 4.16. Also, extended buildings in plan are more susceptible to incoherent earthquake motion and being founded on different foundation material. Therefore, the plan aspect ratio should not be excessive; otherwise the structure may de sub-divided into parts by using seismic joints.

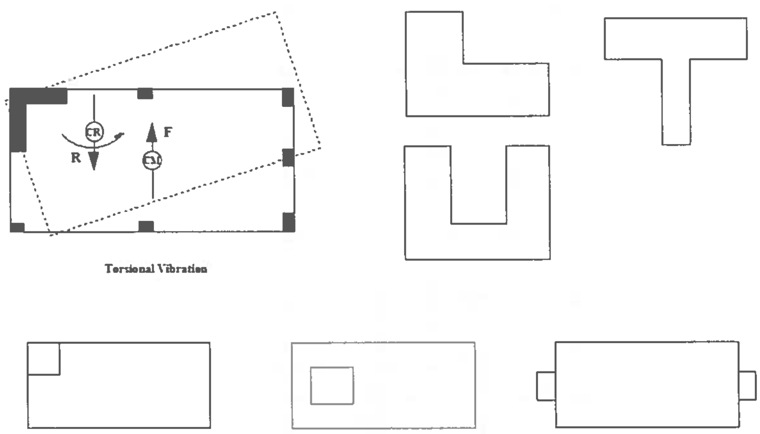


Figure 4.16 Mode of Torsional Vibration under Translational Excitation (C_R is centre of resistance; C_M is centre of mass), and Unfavorable Plan Layouts [*After Ref. 4.6*]

(i) Center of Mass, Center of Rigidity & Center of Shear Strength.

During an earthquake, acceleration-induced inertia forces will be generated at each floor level, where the mass of an entire story may be assumed to be concentrated. Hence the location of a force at a particular level will be determined by the center of the accelerated mass at that level, known as center of mass (C_M). In regular buildings, the positions of the centers of floor masses will differ very little from level to level. However, irregular mass distribution over the height of a building may result in variations in centers of masses, which will need to be evaluated.

The structural resistance is applied at the centre of stiffness of the lateral force resisting elements, known as center of rigidity (C_R). Figure 4.17 shows asymmetric plan layout of three structure. In each case three important locations are identified: centre of mass (C_M), centre of stiffness, or rigidity (C_R) and centre of shear strength (C_y). In traditional elastic analysis of torsional effects in buildings only the first two are considered, and a structure is considered to have plan eccentricity when C_M and C_R do not coincide, but it has recently become apparent that for structures responding inelastically to seismic excitation, the centre of shear strength is at least as important as the centre of rigidity [*Ref. 4.7*]

The eccentricity of the center of stiffness from the center of mass is found from:

$$e_{RX} = \frac{\sum_{i=1}^{n} K_{Zi} x_i}{\sum_{i=1}^{n} K_{Zi}}; \qquad e_{RZ} = \frac{\sum_{j=1}^{m} K_{Xj} z_j}{\sum_{j=1}^{m} K_{Xj}} \tag{4.12}$$

Where K_{Zi} and K_{Xj} are element (i.e. walls or frames) stiffness in the Z and X directions respectively, and x_i and z_j are measured from the center of mass.

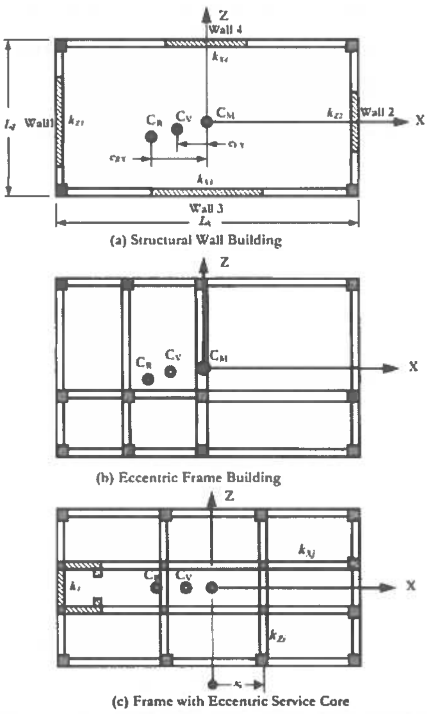


Figure 4.17 Examples of Structures Asymmetric in Plan [*After Ref. 4.1*]

The eccentricity of the centre of strength, C_y, is defined by:

$$e_{VX} = \frac{\sum_{i=1}^{n} K_{Zi} x_i}{\sum_{i=1}^{n} K_{Zi}}; \qquad e_{RZ} = \frac{\sum_{j=1}^{m} K_{Xj} z_j}{\sum_{j=1}^{m} K_{Xj}} \tag{4.13}$$

Where, V_{Zi} and V_{Xj} are the design base shear in the Z and X directions respectively, and x_i and z_j are measured from the center of mass. Note that relative, rather than absolute values may be used to establish the locations of centres of stiffness and strength in the initial stages of design.

4.1.4.3 Vertical Structural Irregularity

(i) **Stiffness irregularity.** This occurs when the stiffness of one storey is substantially less than that of the stories above. This commonly occurs at the first storey of multistory moment frame buildings when the architectural design calls for a tall lobby area. It also can occur in multistory bearing wall buildings when the first storey walls are punched with a number of large openings relative to the stories above. A soft storey is one in which the lateral stiffness is less than 70% of that in the stories above or less than 80% of the average lateral stiffness of the three storey above irregularity (Figure 4.18). An extreme soft storey is defined where its lateral stiffness is less than 60% of that in the storey above or less than 70% of the average lateral stiffness of the three stories above. Figure 4.18 shows typical stiffness soft-storey irregularity.

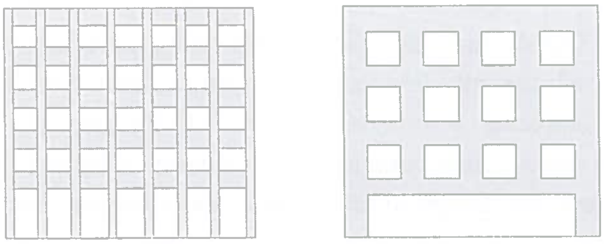


Figure 4.18 Examples of Buildings with a Soft first Storey, a Common type of Stiffness Irregularity.

(ii) **Weight/mass irregularity.** This exists when the weight of the structure at one level is substantially in excess of that at the levels immediately above or below it. This condition commonly occurs in industrial structures where heavy pieces of equipment are located at some levels. It also can occur in buildings that have levels with large mechanical rooms or storage areas. An example of Weight/Mass irregularity is shown in Figure. 4.19.

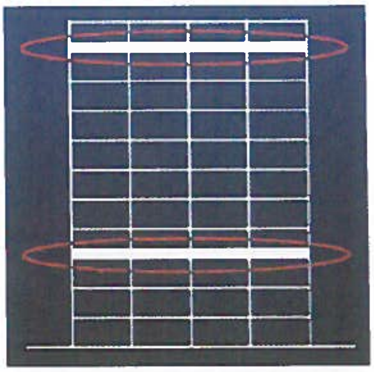


Figure 4.19 Weight/mass Irregularity

(iii) **In-plane discontinuity irregularity.** This occurs when the vertical elements of a structure's seismic-force-resisting system such as its walls or braced frames do not align vertically within a given line of framing or the frame or wall has a significant setback. An example of in plane discontinuity irregularity is shown in Figure 4.20.

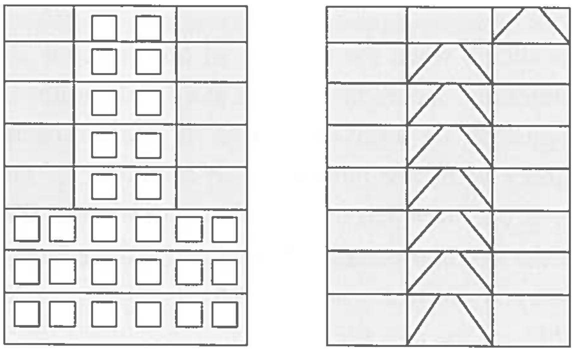


Figure 4.20 Examples of In-Plane Discontinuity Irregularities

(iv) **Weak-story irregularity.** This occurs when the strength of the walls or frames that provide lateral resistance in one storey is substantially less than that of the walls or frames in the adjacent stories. This irregularity often accompanies a soft-storey irregularity but does not always do so. A weak storey is one in which the storey lateral strength is less than 80% of that in the storey above. The storey lateral strength is the total strength of all seismic force resisting elements sharing the storey shear in the considered direction. An extreme weak storey is one where the storey lateral strength is less than 65% of that in the storey above.

4.1.4.4 Effect of Vertical Irregularities

If there is severe stiffness or mass irregularities in elevation, high demand concentrations will ensue, as indicated in Figure 4.21. As a general rule, differences of more than 20-25% in mass or stiffness between consecutive floors should be avoided. This not only infers that column dimensions should be reduced with caution, but also imposes restrictions on set-backs and linkages between adjacent buildings (such as walk ways).

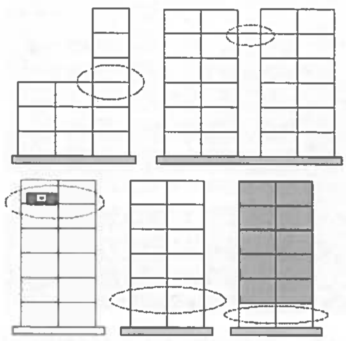


Figure 4.21 Irregularities in Elevation (Areas of Concern Indicated By Dots)

4.1.4.5 Quantification of Irregularities

Prior to the 1988 UBC, building codes published a list of irregularities defining the conditions, but provided no quantitative basis for determining the relative significance of a given irregularity. However, starting in 1988, seismic codes have attempted to quantify irregularities by establishing geometrically or by the use of building dimensions, the points at which the specific irregularity becomes an issue as to require extra analysis and design considerations over and above those of the equivalent lateral procedure.

4.1.5 Continuous Load Path

A continuous load path is critical during an earthquake or hurricane because it helps hold the house together when ground forces or high winds try to pull your home apart. A home is more likely to withstand a seismic or high wind event and stay intact when all parts of the house – roof, walls, floors and foundation – are connected together.

It is very important that all parts of a building or structure, including nonstructural components, be tied together to provide a continuous path that will transfer the inertial forces resulting from ground shaking from the point of origination to the ground. If all the components of a building or structure are not tied together in this manner, the individual pieces will move independently and can pull apart, allowing partial or total collapse to occur.

4.1.6 Stable Foundation

In addition to being able to support a structure's weight without excessive settlement, the foundation system must be able to resist earthquake-induced overturning forces and be capable of transferring large lateral forces between the structure and the ground. Foundation systems also must be capable of resisting both transient and permanent ground deformations without inducing excessively large displacements in the supported structures. On sites that are subject to liquefaction (Refer to Appendix A.4 "Liquefaction in Seismic Zone") or lateral spreading, it is important to provide vertical bearing support for the foundations beneath the liquefiable layers of soil. This often will require deep foundations with drilled shafts or driven piles. Because surface soils can undergo large lateral displacements during strong ground shaking, it is important to tie together the individual foundation elements supporting a structure so that the structure is not torn apart by the differential ground displacements. A continuous mat is an effective foundation system to resist such displacements. When individual pier or spread footing foundations are used, it is important to provide reinforced concrete grade beams between the individual foundations so that the foundations move as an integral.

4.2 DESIGN LIMIT STATES AND PERFORMANCE LEVELS

The re-examination of the fundamental precepts of seismic design has intensified in recent years, with a great number of conflicting approaches being advocated. In some cases the differences between the approaches are fundamental, while in others the differences are conceptual. A crucial catalyst for this interest has been the Vision 2000 document, [*Ref.4.14*] prepared by the Structural Engineers Association of California. The core of this document is the selection of *"seismic performance objectives"* defined as the *"coupling of expected performance level with expected levels of seismic ground motions"*. Four performance levels are defined:

i. **Level 1: Fully Operational.** Facility continues in operation with negligible damage.
ii. **Level 2: Operational**. Facility continues in operation with minor damage and minor disruption in nonessential services.
iii. **Level 3: Life Safe**. Life safety is substantially protected, damage is moderated to extensive.
iv. **Level 4: Near Collapse**. Life safety is at risk, damage is severe, structural collapse is prevented.

The relationship between the four levels of seismic excitation and the annual probabilities of exceedence of each level will differ according to local seismicity and structural importance. In California, the following levels are defined [*Ref.4.15*]:

- **EQ-I:** 87% probability in 50 years: 33% of EQ-III
- **EQ-II:** 50% probability in 50years: 50% of EQ-III
- **EQ-III:** approximately 10% probability in 50 years.
- **EQ-IV:** approximately 2% probability in 50 years: 150% of EQ-III.

The relationship between these performance levels and earthquake design levels is summarized in Figure 4.22. In Figure 4.22 the line "Basic Objective" identifies a series of performance levels for normal structures. The lines *"Essential Objective"* and *"Safety Critical Objective"* relate performance level to seismic intensity for two structural classes of increased importance, such as lifeline structures, and hospitals. As is seen in Figure 4.22 with *"Safety Critical Objective"*, operation performance must be maintained even under the EQ-IV level of seismicity.

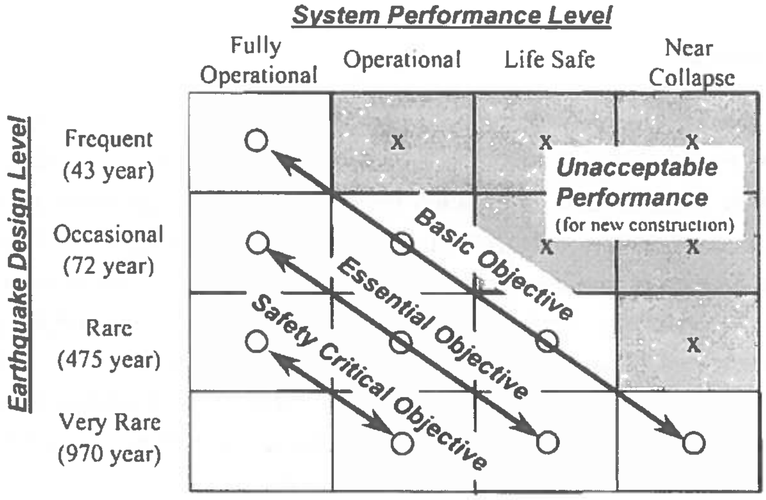


Figure 4.22 Relationship between Earthquake Design Level and Performance Level *[After Ref.4.14]*

Although the Vision 2000 approach is useful conceptually, it can be argued that it requires some modification, and that it provides an incomplete description of performance. It would appear that the gap between *"Occupancy"* and *"Life Safety"* performance levels might be too large, while that between *"Life Safety"* and *"Collapse"* might be too small. The performance levels do not include a *"damage control"* performance level, which is clearly of economic importance. For example, it has been noted that although the performance in the 1995 Kobe earthquake of reinforced concrete frame buildings designed in accordance with the weak-beam / strong-column philosophy satisfied the *"Life safe"* performance level, the cost of repairing the many locations of inelastic action, and hence of localized damage, was often excessive, and uneconomical *[Ref.4.16]*. Alternative structural systems with fewer locations of inelastic action, as might occur in structural wall buildings were more economical in terms of repair costs. The performance level implicit in most current seismic design codes is, in fact, a damage control performance level.

In order to better understand the relationship between structural responses levels and a performance levels, it is instructive to consider the different structure limit states *[Ref.4.1]*.

4.2.1 Structure Limit States

(i) **Serviceability limit state**. This corresponds to the *"fully functional"* seismic performance level of Vision 2000. No significant remedial action should be needed for a structure that responds at this limit state. With concrete and masonry structures, no spalling of cover concrete should occur, and though yield of reinforcement should be acceptable at this limit state, residual crack widths should be sufficiently small so that injection grouting is not needed. As presented in Figure 4.23, structural displacements at the serviceability limit state will generally exceed the nominal yield displacement. For masonry and concrete structures this limit state can be directly related to strain limits in the extreme compression fibres of the concrete or masonry, and in the extreme tension reinforcement.

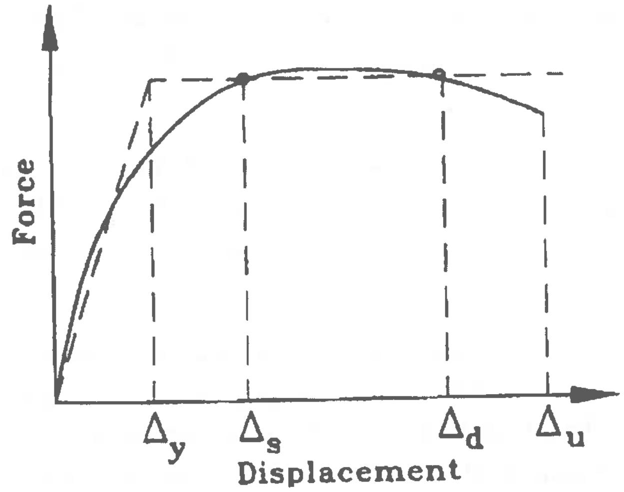


Figure 4.23 Structure Design Limit States

Ideally, non-structural elements, such as partition walls, and glazing, should be designed so that no damage will occur to them before the structure achieves the strain limits corresponding tothe serviceability limit state. Even with brittle partitions this can be achieved, by suitable detailing of the contact between them and the structure, normally involving the use of flexible jointing compounds. However, when the typical construction involves brittle lightweight masonry partitions built hard-up against the structure, significant damage to the partitions is likely at much lower displacement levels than would apply to the structure. For example, reinforced concrete or structural steel building frames are likely to be *able to sustain drifts (lateral displacements divided by height) of more than 0.012* before sustaining damage requiring repair. In such cases, the serviceability limit state is unlikely to govern design.

The frequency with which the occurrence of an earthquake corresponding to the serviceability limit state may be anticipated will depend on the importance of preserving functionality of the building. Thus, for office buildings, the serviceability limit state may be chosen to correspond to a level of shaking likely to occur, on average, once every 50 years (I.e., a 50-year-return-period earthquake). For a hospital, fire station, or telecommunications center, which require a high degree of protection to preserve functionality during an emergency, an earthquake with a much longer return period will be appropriate.

(ii) **Damage-Control Limit State.** This is not directly addressed in the Vision 2000 document, but is the basis for most current seismic design strategies. At this limit state, a certain amount of repairable damage is acceptable, but the cost should be significantly less than the cost of replacement. Damage to concrete buildings and bridges may include spalling of cover concrete requiring cover replacement, and the formation of wide residual flexural cracks requiring injection grouting to avoid later corrosion. Fracture of transverse or longitudinal reinforcement, or buckling of longitudinal reinforcement should not occur, and the core concrete in plastic hinge regions should not need replacement. This limit state is represented in Figure 4.23 by the displacement Δ_d .With well designed structures, this limit state normally corresponds to displacement ductility factors in the range $3 \leq \mu_\Delta \leq 6$.

Again, non-structural limits must be considered to keep damage to an acceptable level. This is particularly important for buildings, where the contents and services are typically worth three to five times the cost of the structure. *It is difficult to avoid excessive damage when the drift levels exceed about 0.025, and hence it is common for building design codes to specify drift limits of 0.02 to0.025*. At these levels, most buildings – particularly frame buildings - will not have reached the structural damage-control limit state.

Ground shaking of intensity likely to induce response corresponding to the damage control limit state should have a low probability of occurrence during the expected life of the building. It is expected that after an earthquake causes this or lesser intensity of ground shaking, the building can be successfully repaired and reinstated to full service.

(iii) **Survival Limit State.** It is important that a reserve of capacity exists above that corresponding to the damage-control limit state, to ensure that during the strongest ground shaking considered feasible for the site, collapse of the structure should not take place. Protection against loss of life is the prime concern here, and must be accorded high priority in the overall seismic design philosophy. Extensive damage may have to be accepted, to the extent that it may not be economically or technically feasible to repair the structure after the earthquake. In Figure 4.23 this limit state is represented by the ultimate is placement, Δ_u. The designer need to rely on structural qualities which will ensure that for the expected duration of a severe earthquake, relatively large displacement can be accommodated without significant loss of lateral force resistance, and that integrity of the structure to support gravity loads is maintained.

4.2.2 Selection of Design Limit States

The discussion in the previous sections indicates that a number of different limit states or performance levels could be considered in design. Generally only one - the damage control limit state or at most two (with the serviceability limit state as the second) will be considered, except for exceptional circumstances. Where more than one limit state is considered, the required strength to satisfy each limit will be determined, and the highest chosen for the final design. Interested readers are referred to Section 4.2.5 of Ref.4.1 for more information on strain and drift limits corresponding to different performance levels.

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CHAPTER 5. EARTHQUAKE LOAD AND ITS IMPACT ON STRUCTURE

5.1 INTRODUCTION

When an earthquake occurs, seismic waves radiate away from the source and travel rapidly through the earth's crust. These waves reach the ground surface; produce shaking that may last from seconds to minutes. The strength and duration of shaking at a particular site depends on the size and location of the earthquake and on the characteristics of the site.

Although seismic waves travel through rock over the overwhelming majority of their trip from the source of an earthquake to the ground surface, the final portion of that trip is often through soil, and the characteristics of the soil can greatly influence the nature of shaking at the ground surface. Soil deposits tend to act as *"filters"* to seismic waves by *attenuating* motion at certain frequencies and *amplifying* it at others. Since soil conditions often vary dramatically over short distances, levels of ground shaking can vary significantly within a small area [*Ref.* 5.1]

Structural damage is the leading cause of death and economic loss in many earthquakes. However, structures need not collapse to cause death and damage. Falling objects such as brick facings and parapets on the outside of a structure or heavy pictures and shelves within a structure have caused casualties in many earthquakes. Interior facilities such as piping, lighting, and storage systems can also be damaged during earthquakes.

5.2 FACTORS AFFECTING EARTHQUAKE LOAD

Structural damage due to an earthquake is not solely a function of the earthquake ground motion. The primary factors affecting the extent of damage are:

i. Earthquake characteristics, such as (a) peak ground acceleration, (b) duration of strong shaking, (c) frequency content, and (d) length of fault rupture
ii. Site characteristics, such as (a) distance between the epicenter and structure, (b) geology between the epicenter and structure, (c) soil conditions at the site, and (d) natural period of the site
iii. Structural characteristics, such as (a) natural period and damping of the structure, (b) age and construction method of the structure, and (c) seismic provisions (i.e., detailing) included in the design

5.2.1 Earthquake Characteristics

The ground motions produced by earthquakes can be quite complicated. At a given point, they can be completely described by three components of translation and three components of rotation. In practice, only the translational components are usually measured, and they are usually measured in orthogonal directions by accelerographs.

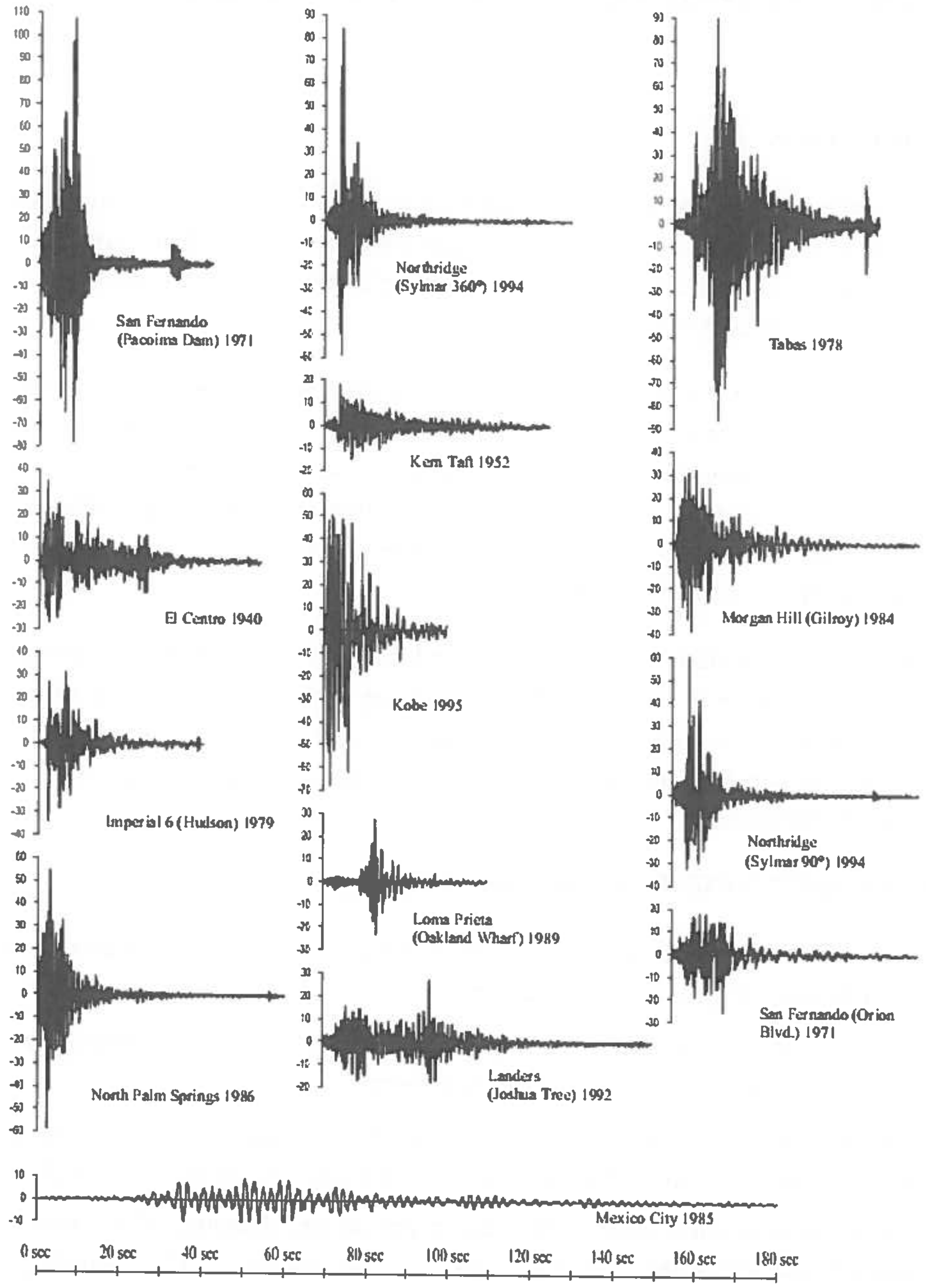


Figure 5.1 Earthquake Ground Acceleration in Epicentral Regions (All Accelerograms are Plotted to the Same Scale for Time and Acceleration) [*After Ref.* 5.2]

No two accelerograms are identical (see Figure 5.1), even when the earthquakes originate in the same part of a fault, with similar magnitudes, and the site where the accelerograms are recorded is the same. Great earthquakes extend for much longer periods of time.

Strong ground motions can be quite complicated, and their complete description involves a large amount of data. For engineering purposes, the essential characteristics of a strong ground motion can be described in much more compact form using *ground motion parameters* [*Ref.* 5.1].

From an earthquake engineering standpoint, the most important characteristics of a strong ground motion are the *amplitude, frequency content, and duration.* All of these characteristics can significantly influence earthquake damage. Consequently, knowledge of the amplitude, frequency content, or duration alone may not be sufficient to describe accurately the damage potential of a ground motion. A number of different ground motion parameters have been proposed, each of which provides information about one or more of these characteristics. In practice, it is usually necessary to use more than one of these parameters to characterize a particular ground motion adequately.

5.2.1.1 Amplitude

Commonly used amplitude parameters include *peak acceleration, peak velocity, and peak displacement.* The peak acceleration provides a good indication of the *high-frequency* component of a ground motion. The peak velocity and peak displacement describe the *amplitudes of the intermediate- and low-frequency components*, respectively.

The most commonly used measure of the amplitude of a particular ground motion is *peak horizontal acceleration* (*PHA*). The *PHA* for a given component of motion is simply the largest (absolute) value of horizontal acceleration obtained from the accelerogram of that component. Horizontal accelerations have commonly been used to describe ground motions in Building Codes because of their natural relationship to inertial forces; indeed, the largest dynamic forces induced in certain types of structures (i.e., very stiff structures) are closely related to the *PHA*.

Vertical accelerations have received less attention in earthquake engineering than horizontal accelerations, primarily because the margins of safety against gravity-induced static vertical forces in constructed works usually provide adequate resistance to dynamic forces induced by vertical accelerations during earthquakes. Ground motions with high peak accelerations are usually, but not always, more destructive than motions with lower peak accelerations. Very high peak accelerations that last for only a very short period of time may cause little damage to many types of structures.

5.2.1.2 Frequency Content

The dynamic response of compliant objects, be they buildings, bridges, slopes, or soil deposits, is very sensitive to the frequency at which they are loaded. Earthquakes produce complicated loading with components of motion that span a broad range of frequencies. The *frequency content* describes how the amplitude of a ground motion is distributed among different frequencies and is generally described through the use of different types of spectra. *Fourier spectra* and *power spectra* directly illustrate the frequency content of the motion itself. *Response spectra* reflect the influence of the ground motion on structures of different natural periods.

5.2.1.3 Duration

The *duration* of strong ground motion can have a strong influence on earthquake damage. Many physical processes, such as the degradation of stiffness and strength of certain types of structures and the buildup of pore water pressures in loose, saturated sands, are sensitive to the number of load or stress reversals that occur during an earthquake. A motion of short duration may not produce enough load reversals for damaging response to build up in a structure, even if the amplitude of the motion is high. On the other hand, a motion with moderate amplitude but long duration can produce enough load reversals to cause substantial damage. The duration of a strong ground motion is related to the time required for release of accumulated strain energy by rupture along the fault. As the length, or area, of fault rupture increases, the time required for rupture increases. As a result, the duration of strong motion increases with increasing earthquake magnitude.

5.2.2 Site Characteristics

The amplitude of earthquake ground shaking diminishes with distance from the source, and the *rate of attenuation is less for lower frequencies of motion than for higher frequencies.* So, for a site located at larger distance from the source will experience lower frequency waves with greater amplitude than the higher frequencies and will affect comparatively long period structures than the short period stiff structures [*Ref.* 5.2].

The soil at a site has a significant effect on the characteristics of the ground motion and, therefore, on the structure's response. *Especially at low amplitudes of motion and at longer periods of vibration, soft soils amplify the motion at the surface with respect to bedrock motions.* This amplification is diminished somewhat, especially at shorter periods as the amplitude of basic ground motion increases, due to yielding in the soil [*Ref.* 5.2].

5.2.3 Structural Characteristics

The response of a particular building in a given site will depend on factors such as natural frequency, damping, ductility, stability of resistance under repeated reversals of inelastic deformation, and over-strength. The natural frequency is dependent on the mass and stiffness of the building. As mentioned earlier (*Section* 5.2.1.2), the ground shaking during an earthquake contains a mixture of many sinusoidal waves of different frequencies, ranging from short to long periods. If the ground is shaken back-and-forth by earthquake waves that have short periods, then short period (T) buildings will have large response. Similarly, if the earthquake ground motion has long period waves, then long period buildings will have larger response. Thus, depending on the value of T of the buildings and on the characteristics of earthquake ground motion (i.e., the periods and amplitude of the earthquake waves), some buildings will be shaken more than the others.

All structural systems are not created equal when response to earthquake induced forces is of concern. Some structural forms are inherently more ductile than others due to careful consideration of configuration, symmetry, mass distribution, horizontal and vertical regularity and the locations, often termed as plastic hinges, where inelastic deformation may occur. Irregularities, often unavoidable, contribute to the complexity of structural behavior. When not recognized, they may result in unexpected damage and even collapse [*Ref.* 5.3].

Figure 5.2 Different Buildings Respond Differently to Same Ground Vibration [*After Ref.* 5.4].

5.3 STRUCTURAL RESPONSE TO GROUND SHAKING

The first important difference between structural response to an earthquake and response to most other loadings is that the earthquake response is *dynamic, not static*. In case of an earthquake ground shaking, the above ground portion of a structure is not subjected to any applied force. The stresses and strains within the superstructure are created entirely by its *dynamic response* to the movement of its base, the ground.

Due to nature of earthquakes' source mechanisms, displacement demand imposed on structures may range from moderate to extremely high values. Hence, adopting the same linear elastic design methodology as used for static loads (gravity and wind) results in an unnecessary "heavy" design of structural members which is neither economical nor practical. Furthermore, large values of acceleration could result from the elastic response of the structure, which could endanger lives and cause extensive nonstructural damage.

Alternatively, earthquake-resistant structures are usually designed to respond in a nonlinear fashion recognizing the fact that *strength has a lesser importance when considering seismic action*. In this way, the structure is designed for a seismic load significantly lower than the value computed from elastic analysis, relying on its ductile capacity to deform in elastically to the required deformation imposed by earthquake without significant loss of strength. This result not only in a more economical design, but also safeguards the structure for the eventuality of the seismic loads being higher than those initially predicted at the design stage.

This behavior is conceptually described in Figure 5.3 (a), where the response of an inverted pendulum is described. If the response is elastic, all the seismic energy transmitted to the structure through ground movement (area abc) is converted to kinetic energy once the movement is reversed. Hence, when the base motion ceases, the structure returns to its initial state. If, on the other hand, the structure does not possess sufficient capacity to resist the inertia force generated by the ground movement, it will respond in an inelastic fashion. Such behavior is schematically represented in Figure 5.3 (b) through an elasto-plastic approximation, where the area adef represents the total input of seismic energy. The inelastic deformation of the structure (segment de) is achieved through the development of a plastic hinge at the base of the oscillator where energy is dissipated through hysteresis. When the ground movement is reversed, only part of the energy is recovered through kinetics, thus once the base motion terminates, the structure does not fully recover to its initial state.

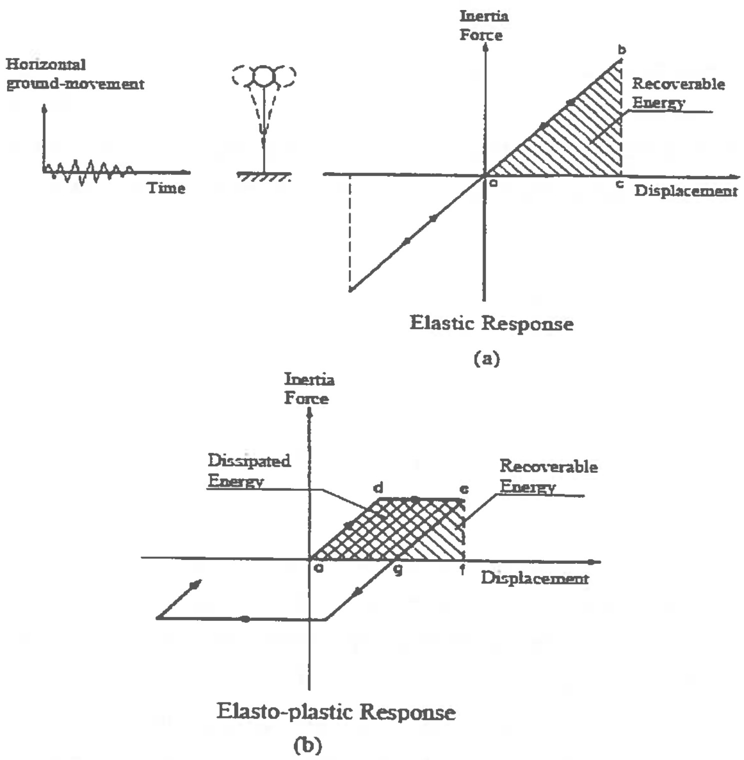


Figure 5.3 Response of a SDOF System Under Base Excitation: (A) Elastic and (B) Elasto-Plastic Behavior [*After Ref. 5.5*]

Structures with several degrees-of-freedom, such as residential buildings, will dissipate energy not only through plastic hinging in their members, but also through the development of appropriate failure mechanisms through well-established *capacity design* procedure. These should, on the one hand, maximize the energy dissipation of the whole structure and on the other hand, minimize the level of damage and risk of failure. In Figure 5.4, three different failure modes are depicted. It is clear that the column sway mechanism shown in Figure 5.4 (b), will dissipate less energy than the beam sway mechanism. For the same displacement level at the top of the structure, the member demands are much higher for the column-sway mechanism, thus the structure will fail much earlier.

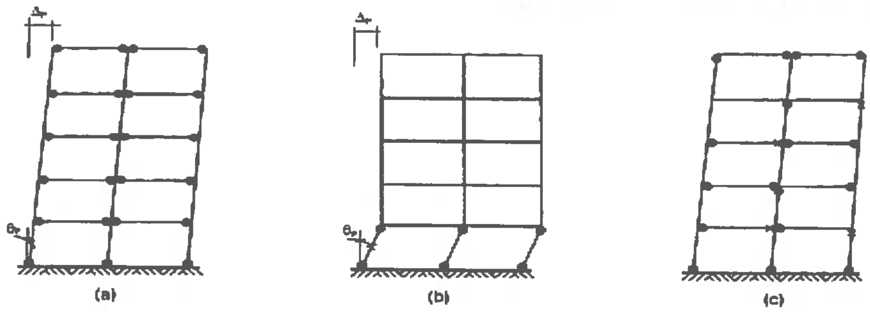


Figure 5.4 Collapse Mechanisms: (A) Beam Sway; (B) Column Sway; (C) Mixed Mode

5.3.1 Elastic Response Spectra

Now a central concept in earthquake engineering, the response spectrum provides a convenient means to summarize the peak response of all possible linear SDF systems which resembles to a simple one-story frame with mass concentrated at the roof level to a particular component of ground motion. The vibrational characteristics of such a simple oscillator may be reduced to two: the natural period and the amount of damping.

A plot of the peak value of a response quantity as a function of the natural vibration period T of the system, or a related parameter such as circular frequency ω or cyclic frequency f, is called the *response spectrum* for that quantity. Each such plot is for SDF systems having a fixed damping ratio ξ and several such plots for different values of ξ are included to cover the range of damping values encountered in actual structures. Whether the peak response is plotted against f or T is a matter of personal preference. However, use natural period is preferred rather than natural frequency because the period of vibration is a more familiar concept and one that is intuitively appealing [*Sec.* 6.5, *Ref.* 5.6].

A variety of response spectra can be defined depending on the response quantity that is plotted. Consider the following peak responses:

$$S_d(T,\xi) = \max_t |u(t,T,\xi)| \tag{5.1}$$

$$S_v(T,\xi) = \max_t |\dot{u}(t,T,\xi)| \tag{5.2}$$

$$S_a(T,\xi) = \max_t |\ddot{u}(t,T,\xi)| \tag{5.3}$$

The deformation response spectrum is a plot of S_d against T for fixed ξ. A similar plot of S_v is relative velocity response spectrum, and for S_a is acceleration response spectrumDüzce ground motion [*After Ref.* 5.7]

5.3.1.1 Construction of Elastic Response Spectrum

Consider various SDOF systems with different T, but the same ξ, subjected to a ground excitation as shown in Note that $T_1 < T_2 < T_3 < \cdots$in Figure 5.5.

We can calculate the displacement response of each SDOF system *u (t)* by direct integration. Time variations *u (t)* and *ü (t)* of 5 percent damped SDOF systems with T_1 = 0.5 s, T_2= 1.0 s and T_3= 2.0 s under certain given ground motion (say NS component of 1999 Düzce ground motion) are plotted in Figure 5.6.

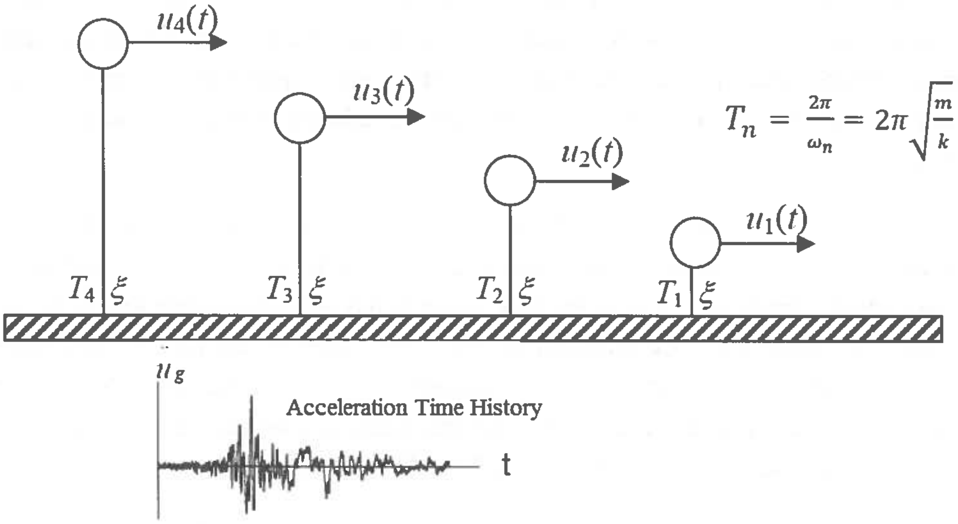


Figure 5.5 Different SDOF Systems under Earthquake Ground Excitation [*After Ref.* 5.7]

We can select the peak displacement response from each *u (t)* function, and define this value as the spectral displacement S_d, where,

$$S_d(T,\xi) = \max_t |u(t,T,\xi)| \tag{5.4}$$

Similarly, spectral acceleration can be defined as the peak value of total acceleration

$$S_a(T,\xi) = \max_t |\ddot{u}^t(t,T,\xi)| \tag{5.5}$$

Where,

$$\ddot{u}^t = \ddot{u}_g + \ddot{u} \tag{5.6}$$

$\ddot{u}_g$ is ground acceleration

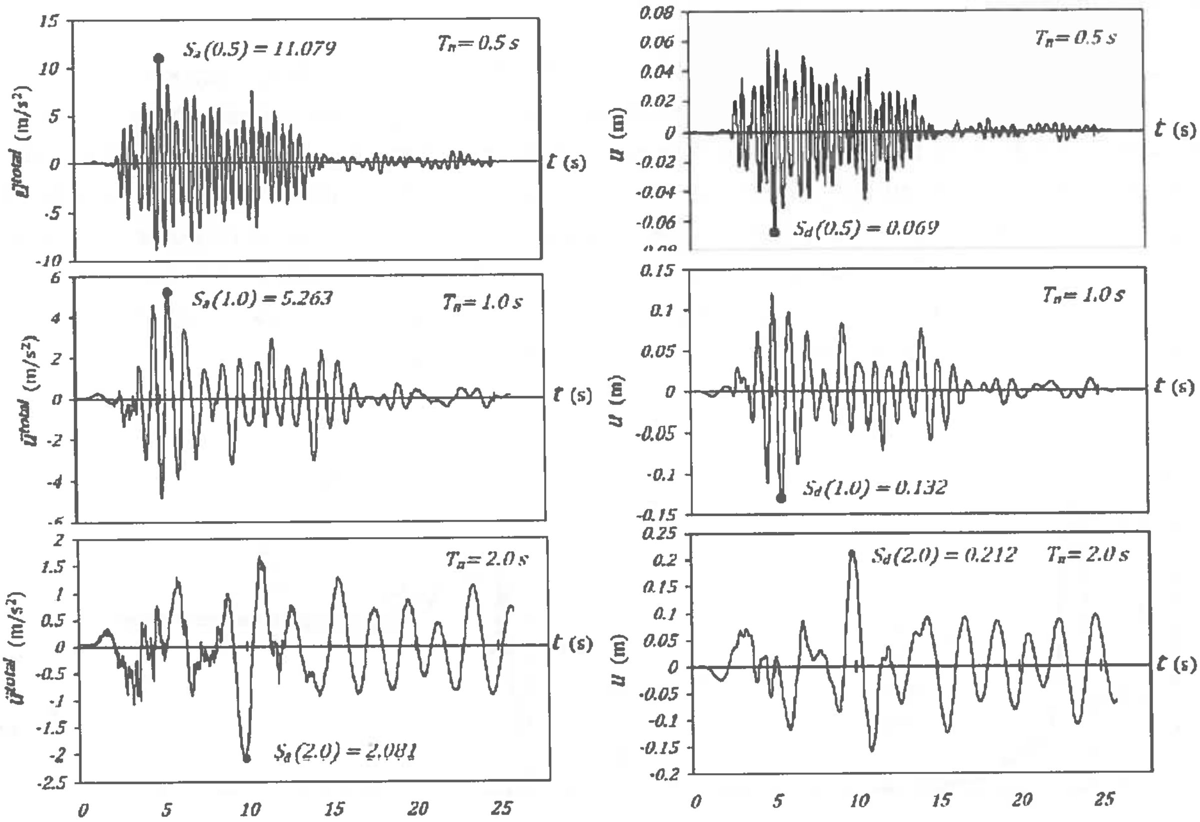


Figure 5.6 Time Variations of Displacement and Acceleration Responses of Several SDOF Systems under the NS Component of 1999 Düzce Ground Motion [*After Ref.* 5.7]

Accordingly, S_d and S_a in Eqns. (5.4) and (5.5) can be plotted as functions of T and ξ. When this process is repeated for a set of damping ratios, a family of S_a and S_d curves are obtained. The family of these curves is called the *acceleration response spectra* and *displacement response spectra* of an earthquake ground motion, respectively. Acceleration and displacement response spectra for the given ground motion are plotted in Figure 5.7. The peak values indicated in Figure 5.6 are also marked on Figure 5.7.

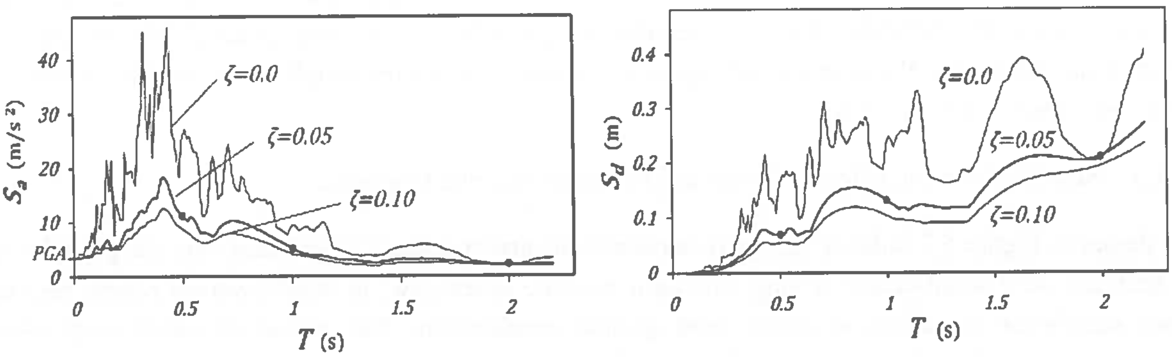


Figure 5.7 Acceleration and Displacement Response Spectra of the NS Component of 1999 Düzce Ground Motion [*After Ref.* 5.7]

5.3.1.2 Limitations

The duration effect is almost lost in the spectral information since an earthquake response spectrum is only based on the consideration of the complex time-dependent dynamic response to a single key parameter, most likely to be needed by the designer: namely peak response. This is practical for design, but it should be remembered that *survivability of a structure depends not only on peak response levels, but also on the duration of strong ground shaking and the number of cycles where response approaches the peak response level*. For example[*Sec.* 2.3.2, *Ref.* 5.3], the severe structural damage in the 1985 Mexico earthquake is at least partly attributable to the high number of cycles of response at large displacements demanded of structures by the earthquake characteristics.

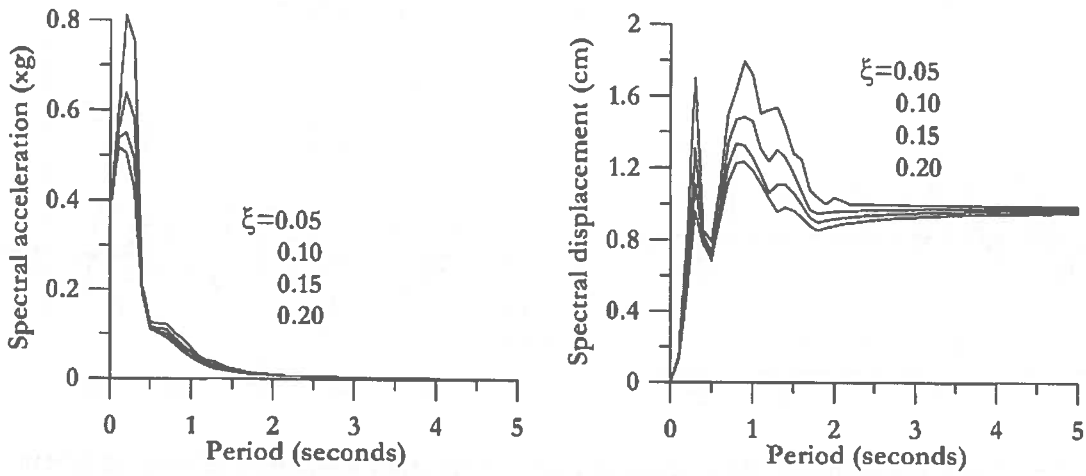


Figure 5.8 Acceleration and Displacement Response Spectra for Accelerograms of Whittier (M_w= 6.0, 1987) Earthquake [*After Ref.* 5.8]

As apparent from Figure 5.8, information from the acceleration response spectra cannot be extracted for periods of $T > 1.5$ sec since the response accelerations are so low. The displacement spectra provide much more readily accessible information for the medium to long period range, but indicate surprisingly regular displacements at periods greater than about 2 seconds. In fact this is false data, since the accelerogram was recorded by an analogue, rather than digital accelerograph, and a filter at 3 seconds was used to determine the displacement response. *Ref.* 5.9, have shown that the roll-off associated with filtering makes the response spectra unreliable for periods greater than about 2/3rds of the filter period. Thus the data in the displacement spectra of Figure 5.8 are meaningless for periods greater than about 2 sec. [*Sec.* 2.2.1, *Ref.* 5.8].

5.3.1.3 Structural Period: Effect of Structural Period on Seismic Response

The shapes in Figure 5.7 indicate that peak accelerations are irregularly distributed over the period range but decrease very significantly at long structural periods. In the low- *to middle-period range*, response shows significant amplification above peak ground acceleration. *The period at which peak elastic response occurs depends on the earthquake characteristics and the ground conditions.* Moderate earthquakes recorded on firm ground typically result in peak response for periods in the range 0.15 *to* 0.4 *s*. On soft ground, peak response may occur at much longer periods.

It can be observed from Figure 5.7 that when $T = 0$, $S_a = \ddot{u}_{g,\ max}$ (PGA) and $S_d = 0$. On the other hand, when T approaches infinity, S_a approaches zero and S_d approaches $u_{g,\,max}$ (PGD). These limiting situations can be explained with the aid of Figure 5.9.

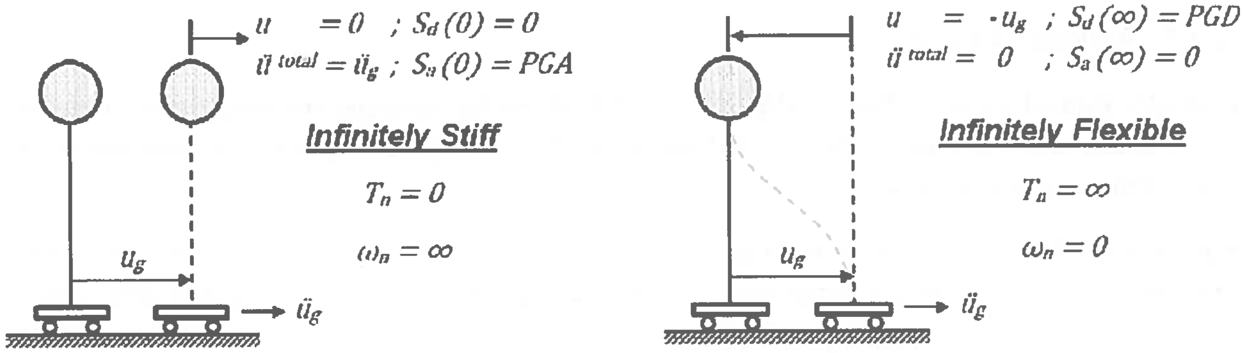


Figure 5.9 Response of Infinitely Stiff and Infinitely Flexible SDOF Systems to Ground Excitation

$T = 0$ is equivalent to $\omega_n = \infty$, i.e. the system is infinitely stiff. Then the spring does not deform ($u = 0$, hence $S_d = 0$) and the motion of the mass becomes identical to the motion of ground. Accordingly, maximum acceleration of the mass becomes identical to the acceleration of the ground which makes their maximum values equal.

T approaches infinity when ω_n approaches zero. Hence the system becomes infinitely flexible. An infinitely flexible system has no stiffness and it cannot transmit any internal lateral force from the ground to the mass above. Ground moves while the mass stays stationary during the earthquake. The total displacement of the mass is zero ($u_{total} = u+u_g = 0$). Accordingly $|u|_{\max} = |u_g|_{max}$, or $S_d = PGD$. Similarly, the total acceleration of the mass is zero ($u_{total} = u+u_g = 0$) which makes $S_a = 0$ from Eq. (5.5).

5.3.1.4 Methods for Calculating Structural Period

The determination of the natural period of vibration of an RC structure is an essential procedure in earthquake design and assessment. An improved understanding of the global demands on a structure under a design earthquake can be obtained from this single characteristic. *This property is dependent on the mass, strength and stiffness of the structure and is thus affected by many factors such as the structural regularity, the number of stories and bays and the section properties including dimensions and extent of cracking.* Cracking of RC members is a phenomenon often ignored in period calculation however it generally occurs under gravity loading and after moderate seismic action. The stiffness of RC members significantly decreases after cracking and so this stiffness reduction should be adequately modeled in analyses to determine an expected period of vibration [*Sec.* 1.1, *Ref.* 5.10].

The natural period of modern buildings can seldom, if ever, be calculated from simple vibration theory. Five other methods, listed below, can be used when knowledge of a building period is needed [*Sec.* 3-8, *Ref.* 5.11].

I. Analytical models based on finite element analysis (FEA) and other modeling techniques can be used.

II. A scale model of the building can be constructed and the natural period extrapolated from measurements on the model. (This is seldom done, however.)
III. If the building has been constructed, actual measurements can be taken.
IV. Empirical relations (such as proposed in different building codes) can be used.
V. Rayleigh's method can be used.

5.3.1.5 Rayleigh's Method

Rayleigh's method is a procedure developed by Lord Rayleigh for analyzing vibrating systems using the law of conservation of energy. Its principal use is for determining an accurate approximation of the natural frequency of a structure.

In an undamped elastic system, the maximum potential energy can be expressed in terms of the external work done by the applied forces. In terms of a generalized coordinate this expression can be written as

$$(PE)_{max} = \frac{Y}{2}\sum f_i \emptyset_i = \frac{f^* Y}{2} \tag{5.7}$$

Where

Y: generalized coordinate
f_i :lateral force applied at level i = 1 to n
$\emptyset_i$: shape function

Similarly, the maximum kinetic energy can be expressed in terms of the generalized coordinate as

$$(KE)_{max} = \frac{\omega^2 Y^2}{2}\sum_i m_i \emptyset_i^2 = \frac{\omega^2 Y^2 m^*}{2} \tag{5.8}$$

According to the principle of conservation of energy for an undamped elastic system, these two quantities must be equal to each other and to the total energy of the system. Equating Eq. 5.7 to Equation 5.8 results in the following expression for the circular frequency:

$$\omega = \sqrt{f^*/m^* Y} \tag{5.10}$$

Basically, the building period "*T*" is

$$T = \frac{2\pi}{\omega} = 2\pi\sqrt{m^* Y/f^*} \tag{5.11}$$

Multiplying the numerator and denominator of the radical by Y and using the equation below results in the expression of the fundamental period "*T*":

$$\delta(x,t) = \emptyset(x)Y(t) \tag{5.12}$$

$$T = 2\pi\sqrt{\frac{\sum_{i=1}^{n} w_i \delta_i^2}{g\sum_{i=1}^{n} f_i \delta_i}} \tag{5.13}$$

5.3.1.6 Damping

Damping reduces the response of a structure, as expected, and the reduction achieved with a given amount of damping is different in the three spectral regions. In the limit as *T* damping does not affect the response because the structural mass stays still while the ground underneath moves. The effect of damping tends to be greatest in medium period region (0.5-3 sec) of the spectrum. *In this spectral region the effect of damping depends on the ground motion characteristics*. If the ground motion is nearly harmonic over many cycles (e.g., the record from Mexico City shown in Figure 5.10), the effect of damping would be especially large for systems near "resonance" (Sec.6.2, Ref.5.6). If the ground motion is short in duration with only a few major cycles (e.g., the record from Parkfield, California, shown in Figure 5.10) the influence of damping would be small, as in the case of impulsive excitations Interested readers are referred to Chapter 5 and Chapter 6 of Ref.5.6, for further explanation on effect of damping on structural response under harmonic and impulse loading.

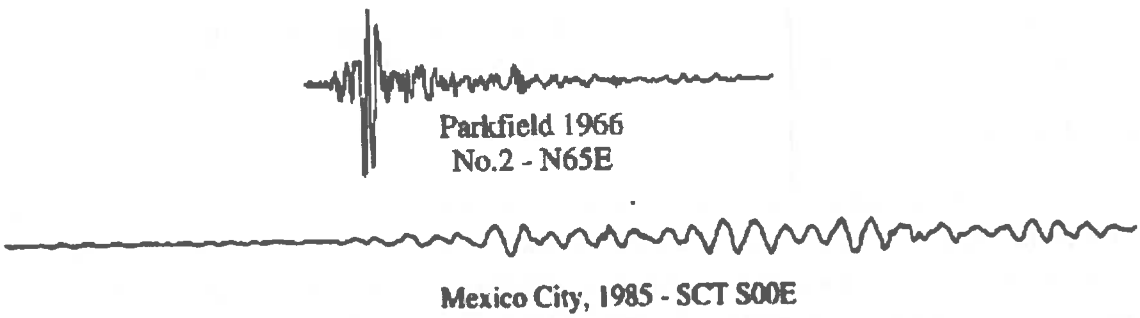


Figure 5.10 Ground Motion Records of Parkfield 1966 Earthquake & Mexico City, 1985 Earthquake

Figure 5.11 shows the peak pseudo-acceleration $PS_a(\xi)$ normalized relative to $PS_a(\xi = 0)$, plotted as a function of ξ for several T values. It is observe that the effect of damping is stronger for smaller damping values. This means that if the damping ratio is increased from 0 to 2%, the reduction in response is greater compared with the response reduction due to an increase in damping from 10% to 12%. The effect of damping in reducing the response depends on the period T of the system, but there is no clear trend from Figure 5.11. This is yet another indication of the complexity of structural response to earthquakes.

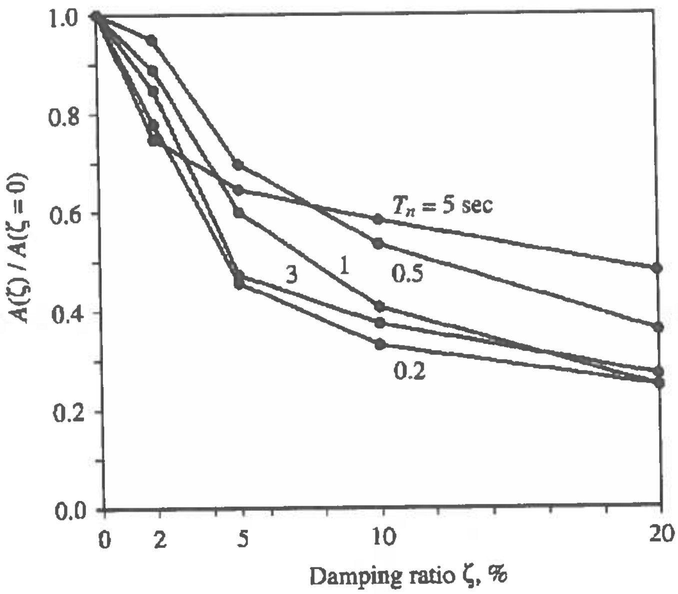


Figure 5.11 Variation of Peak Pseudo-Acceleration with Damping for Systems with T = 0.2, 0.5. 1, 3 and 5 Sec; EI Centro Ground Motion [*After Ref.* 5.6]

5.3.2 Inelastic Response Spectra

The inelastic response spectra are usually derived from the elastic response spectra. There are several well-known methods of obtaining the inelastic response spectra from the elastic response spectra, but few of them are suitable for manual calculations. Perhaps the quickest and easiest, though not necessarily the most rigorous, method is simply to *scale* the elastic curves downward by some function of the ductility factor.

$$S_{a,inelastic} = \frac{S_{a,elastic}}{\text{Force reduction factor, R}} \qquad (5.7)$$

The force reduction factor for a given value of ductility factor μ is dependent on the structural period and can be explained by three established approximation which are briefly explained below.

5.3.2.1 Equal Displacement Approximation

It has been found, from inelastic time-history analyses, that for many structures whose fundamental time period is in the range of $T > T_m$ ($T = 0.6$ - 2 seconds), (see Figure 5.12), maximum seismic displacements of elastic and inelastic systems with the same initial stiffness and mass (and hence the same elastic time periods) are very similar, as illustrated in Figure 5.13 (a). The geometry of Figure 5.13 (a) thus implies that the ductility achieved by the inelastic system is approximately equal to the force reduction factor. That is,

$$R = \mu \qquad (5.20)$$

This observation is referred to *Equal displacement approximation.*

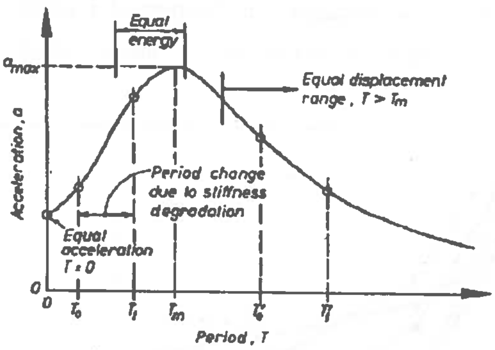


Figure 5.12 Influence of Period on Ductile Force Reduction [*After Ref.* 5.3].

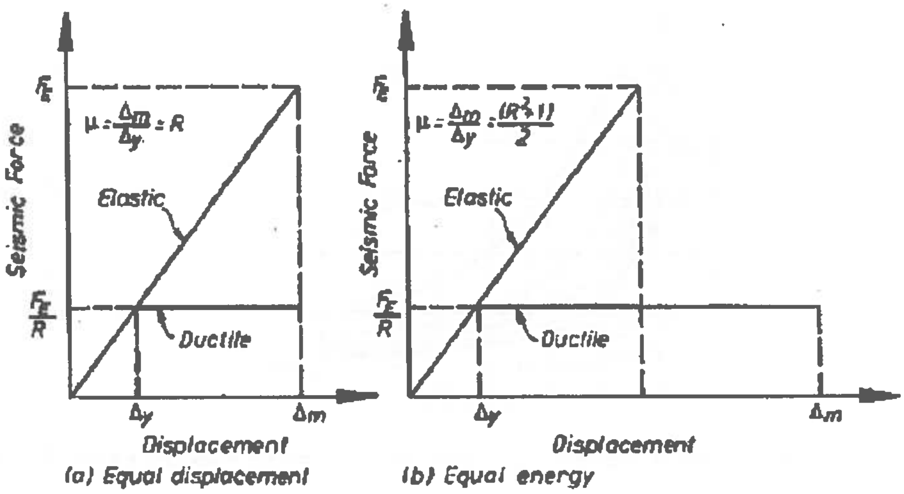


Figure 5.13 Relationship between Ductility and Force Reduction Factor [*After Ref.* 5.3].

5.3.2.2 Equal Energy Approximation

For short period structure with $T<T_m$, the *equal displacement approximation* is found to be non-conservative *i.e*, the ductility factor is greater than the force reduction factor. For structures in this period range, it is found that the peak displacement ductility factor achieved can be estimated reasonably well by equating the area under the inelastic force-deflection curve and the area under the elastic relationship with equal initial stiffness as shown in Figure 5.13 (b).Since the areas represent the total energy absorbed by the two systems under a monotonic run to maximum displacement, Δ_m this is sometimes termed the *equal-energy approximation.*

From Figure 5.13(b) the relationship between displacement ductility factor and force reduction factor can be expressed as

$$\mu=\frac{R^2+1}{2} \text{ or } R=\sqrt{2\mu-1} \tag{5.9}$$

5.3.2.3 Equal Acceleration Approximation

For very-short period structure the (say T<0.2 sec) the force reduction factor given by Eq. (5.9) has still been found non-conservative. In the limit when the period approaches $T=0$, even small force reduction factors imply very large ductility, since the structural deformations become insignificant compared with ground motion deformations. Consequently, the, structure experiences the actual ground accelerations, regardless of relative displacements, and hence ductility. The corollary of this is that very short- period structures should not be designed for force levels less than peak ground acceleration. The behavior above is theoretically consistent and may be reasonably termed the *Equal acceleration approximation.*

So, for a very short period structure,

$$R=1 \qquad \text{(regardless of } \mu) \tag{5.10}$$

So, in summary the following force reduction factors for a given value of μ are

For long period structures: $R=\mu$ (5.11)

For short period structures: $R=\sqrt{2\mu-1}$ (5.12)

For extremely short-period structures: $R=1$ (regardless of μ) (5.13)

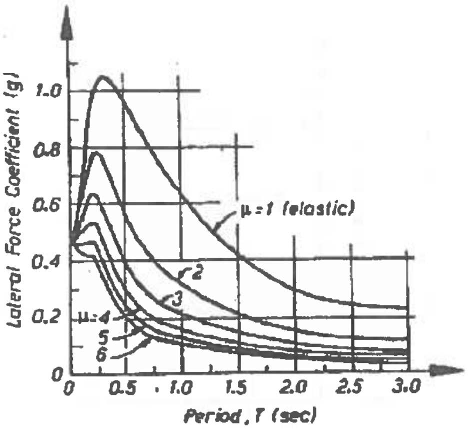
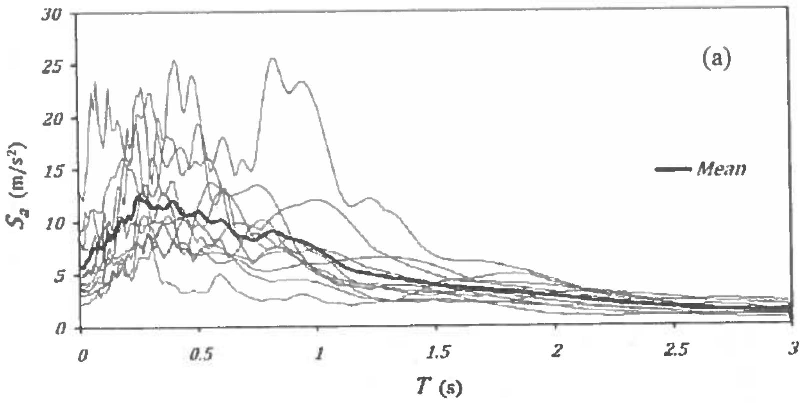


Figure 5.14 Typical Inelastic Acceleration Response Spectra [*After Ref.* 5.3].

Figure 5.14 shows a typical inelastic acceleration response spectra based on the foregoing principles.

5.3.3 Normalized Design Response Spectra

The design spectrum is intended for the design of new structures, or the seismic safety evaluation of existing structures, to resist future earthquakes. Spectral acceleration shapes for different earthquake ground motions at a given site, as shown in Figure 5.15 (a) for a suit of 10 ground motions, exhibit significant variations, each showing varying jaggedness in response which reflects the distinct nature of each ground motion. So, for design purpose the response spectrum for ground motions recorded during past earthquakes is inappropriate. Similarly, it is not possible to predict the jagged response spectrum in all its detail for a ground motion that may occur in the future. Thus the design spectrum should consist of a set of smooth curves or a series of straight lines with one curve for each level of damping.



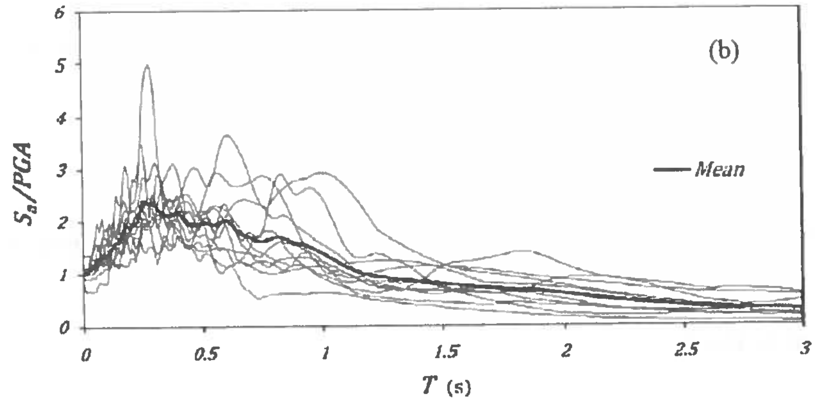


Figure 5.15 Acceleration Response Spectra of 10 Ground Motions, (b) Acceleration Response Spectra of 10 Ground Motions Normalized with Respect to their Peak Ground Accelerations.

The design spectrum should in a general sense, be representative of the characteristics of all seismic properties experienced at a specific site. The design response spectrum should be based on geologic, tectonic, seismological, and soil characteristics associated with that specific site. If none have been recorded at the site, the design spectrum should be based on ground motions recorded at other sites under similar conditions [*Sec.* 5.9, *Ref.* 5.6]

The design spectrum is based on statistical analysis of the response spectra for the ensemble of ground motions. To remove the characteristic of individual ground motion, they are usually normalized by selecting a fixed damping ratio first, usually $\xi = 0.05$, then removing the effect of with peak ground acceleration $\ddot{u}_{g,max}$ (*PGA*) by dividing $S_a(T)$ with *PGA* for all *T*. The spectral acceleration shapes normalized with respect to *PGA* in Figure 5.15 (b), display less variation compared to the non-normalized spectra in Figure 5.15(a).

It is possible to obtain statistical averages of $S_a(T)/PGA$ over period (Figure 5.15 (b)). However, this is usually done first by grouping the ground motions with respect to the soil conditions of the recording stations (sites), then obtaining the mean values of $S_a(T)/PGA$ values over *T*. This exercise was first carried out by Seed et al. [*Ref.* 5.12], which indicate the effect of local soil conditions on the shape of mean acceleration spectra (Figure 5.16). These shapes form the basis of earthquake design spectra defined for local soil conditions.

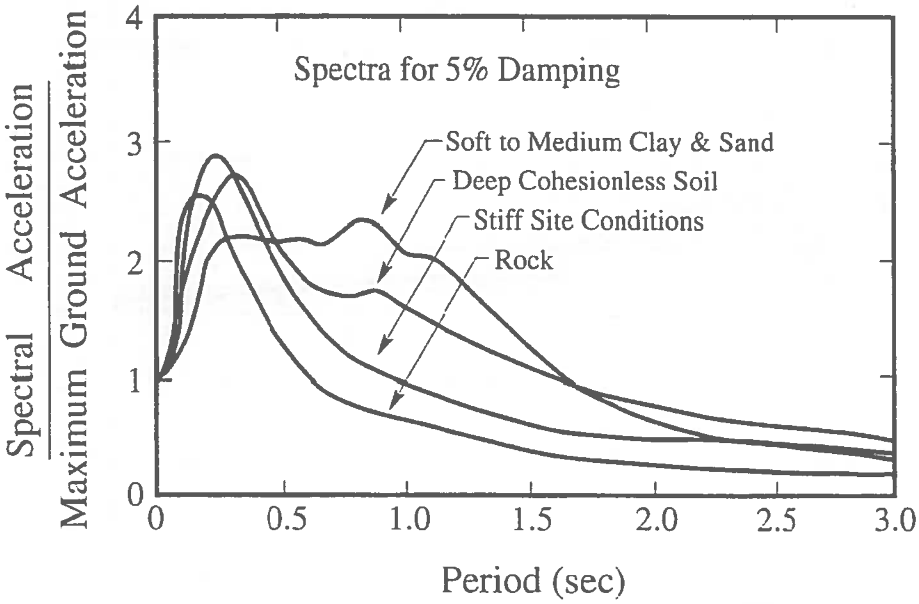


Figure 5.16 Average Normalized Response Spectra (5% damping) for Different Local Site Conditions [*After Ref.* 5.12]

5.3.4 Displacement & Drift

The term "drift" has two connotations:

i. *"Storey drift"* is the maximum lateral displacement within a storey (i.e., the displacement of one floor relative to the floor below caused by the effects of seismic loads).
ii. The *lateral displacement or deflection* due to design forces is the absolute displacement of any point in the structure relative to the base. This is not *"storey drift"* and is not to be used for drift control or stability considerations since it may give a false impression of the effects in critical stories. However, it is important when considering seismic separation requirements.

Storey drift is shown graphically in Figure 5.18. There are several reasons to control drift.

a. First, excessive movement in upper stories has strong adverse psychological and physical effects on occupants.
b. Second, it is difficult to ensure structural and architectural integrity with large amounts of drift. Excessive drift can be accompanied by large secondary bending moments and inelastic behavior known as P-Δ effects (*see section* 5.3.5)
   Buildings subjected to earthquakes need drift control to restrict damage to partitions, shaft and stair enclosures, glass, and other fragile nonstructural elements and, more importantly, to minimize differential movement demands on the seismic safety elements.

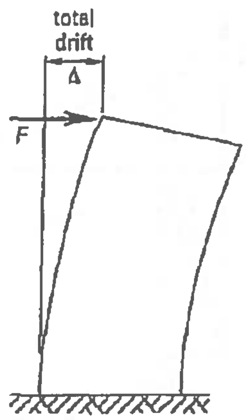


Figure 5.17 Total drift

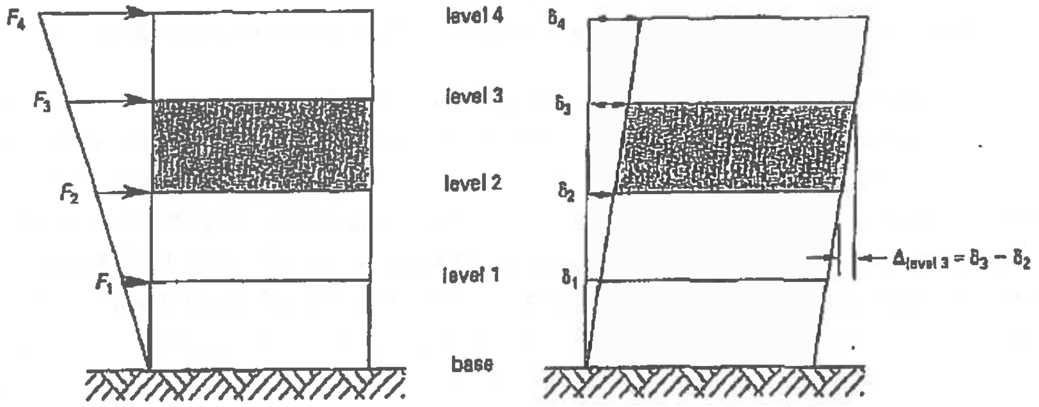


Figure 5.18 Storey drift

5.3.5 P-Δ Effect

When flexible structures are subjected to lateral forces, the resulting horizontal displacements lead to additional overturning moments because the gravity load P_g is also displaced. Thus in the simple cantilever model of Figure 5.19 (a), the total base moment is

$$M_b = F_E H + P_g \Delta \tag{5.14}$$

Therefore, in addition to the overturning moments produced by lateral force F_E, the secondary moment $P_g \Delta$ must also be resisted. This moment increment in turn will produce additional lateral displacement, and hence Δ will increase further. In very flexible structures, instability, resulting in collapse, may occur.

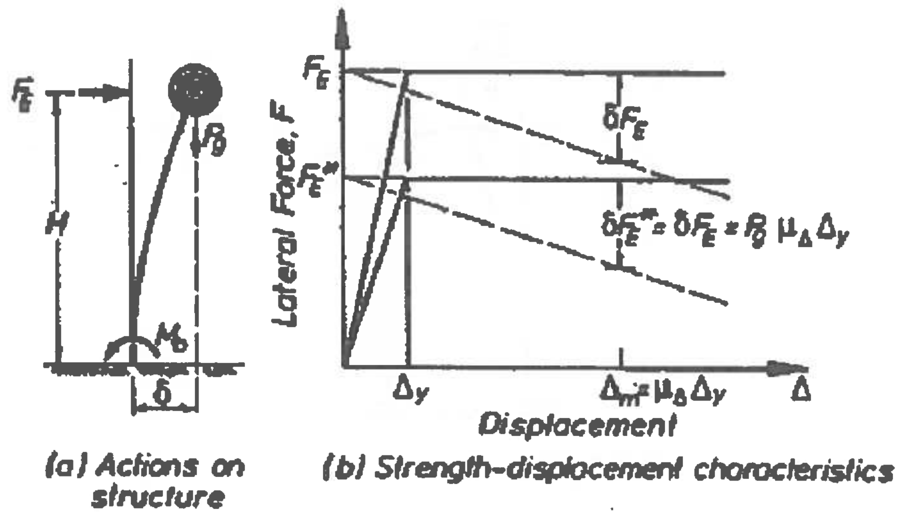


Figure 5.19 Influence of *P-Δ* effects on resistance to lateral forces [*After Ref.* 5.3].

It is necessary to recognize when assessing seismic design forces that the importance of P–Δ effects will generally be more significant for structures in regions of low-to-moderate seismicity than for structures in regions of high seismicity, where design lateral forces will be correspondingly higher. This can be explained with the help of Figure 5.19 (b) which shows the characteristic inelastic load-deformation response (here, elasto-plastic response is assumed) of two structures, one designed for a region of high seismicity with a strength under lateral forces of F_E and the other, in a less seismically active region, with a lateral strength of F_E^*. If the moment capacity M_b is developed in inelastic response, then the lateral inertia force that can be resisted reduces as the displacement increases to its maximum Δ_m, according to the relationship

$$F_E = \frac{M_b - P_g \Delta}{H} \tag{5.15}$$

Since the weight P_g of the two structural systems is presumed the same, the reduction in strength δF for a specified displacement Δ_m is the same. That is $\delta F_E = \delta F_E^* = P_g \Delta_m$. If both structures have the same yield displacement Δ_y and the same expected structural ductility μ_Δ, the reduction in strength available to resist lateral forces at maximum displacement is $\delta F_E = \delta F_E^* = P_g \mu_\Delta \Delta_y$. It is clear from Figure 5.19(b) that this reduction in strength is more significant for the weaker system, since the ratio $\delta F_E^* / F_E^* > \delta F_E / F_E$. Therefore, in most situations, particularly in regions where large seismic design forces need to be considered, P–Δ phenomena will not control the design of frames.

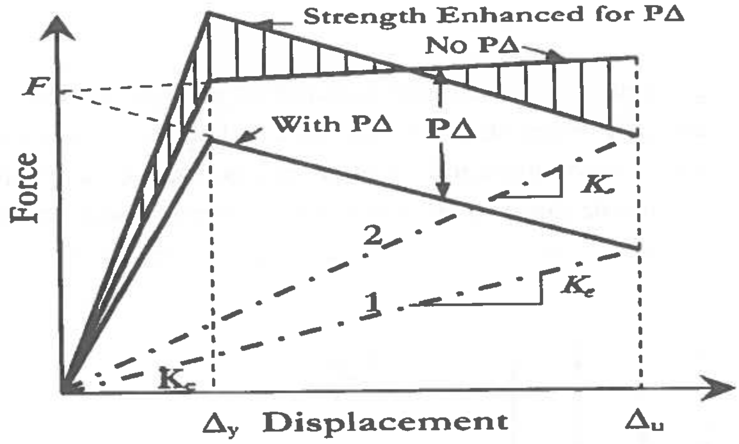


Figure 5.20 P-Δ Effect on Force-Displacement Response [*After Ref.* 5.8]

Another interesting thing to note from Figure 5.20 is that P-Δ effect not only reduces the lateral force capacity, but also modifies the entire lateral force-displacement characteristic. The effective initial stiffness is reduced, and the post-yield stiffness may become negative.

The significance of P-Δ effects is recognized in most seismic design codes, and is typically quantified by some form of "stability index", θ_Δ, which compares the magnitude of the P-Δ effect at either nominal yield, or at expected maximum displacement, to the design base moment capacity of the structure. Since the P-Δ effect is of maximum significance at the design level of seismic response, it is more rational to relate the stability index to conditions at maximum response, as recommended in [*Ref.* 5.3]:

$$\theta_\Delta = \frac{P_g \Delta_m}{M_D} \tag{5.16}$$

Two different approaches are typically adopted in different codes to account for P-Δ effect. One approach is to increase the expected design displacement to Δ_m^*:

$$\Delta_m^* = \frac{\Delta_m}{1-\theta_\Delta} = \frac{\mu_\Delta \Delta_y}{1-\theta_\Delta} \tag{5.17}$$

The alternate approach is to increase the strength in an attempt to avoid an increase in the expected design displacement. Pauly and Priesty [*Ref.* 5.3], discussing the design of reinforced concrete frame buildings, recommended that when the stability index is less than θ_Δ= 0.085, P-Δ effects may be ignored. With reference to Figure 5.20, it should be recognized that increasing strength of a frame is more effective in controlling drift than increasing stiffness. This is because the more vigorously a frame responds in the inelastic range; the less is the significance of stiffness. [*Sec.* 1.5.1.11, *Ref.* 5.13].

5.3.6 Other Important Factors

5.3.6.1 Structural Over-strength Factor, Ω_o

While some deformation-controlled members (i.e beams in beam-sway mechanism), detailed to provide ductility, are expected to deform in elastically, force-controlled members (i.e columns in beam-sway mechanism) that are designed to remain elastic would experience a significantly higher seismic force level than that predicted based on actual design seismic forces. To account for this effect, seismic codes use a seismic force amplification factor, Ω_o, such that the realistic seismic force in these force-controlled members can be conveniently calculated from the elastic design seismic forces. Ω_o is termed the *Structural Over-strength Factor*.

The seismic-force-resisting system is expected to reach significant yield for forces in excess of the design forces. The term "significant yield" is not the point where first yield occurs in any member but, rather, is defined as that level causing complete plastification of at least the most critical region of the structure (such as formation of a first plastic hinge in the structure).A concrete frame reaches significant yield when at least one of the sections of its most highly stressed component reaches its strength.

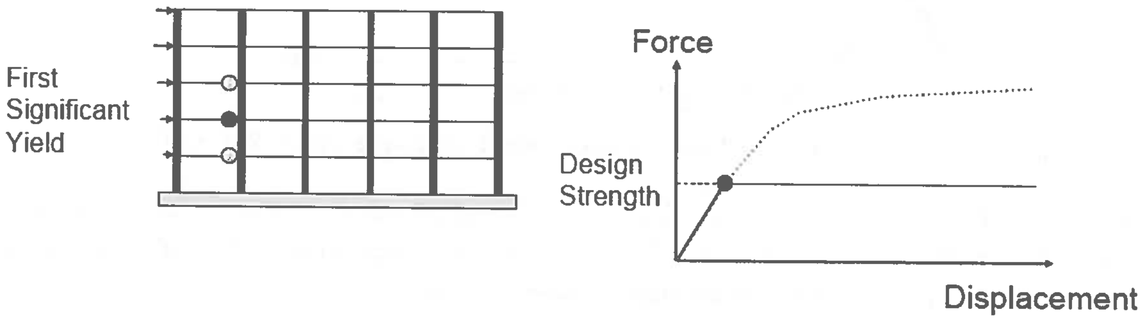
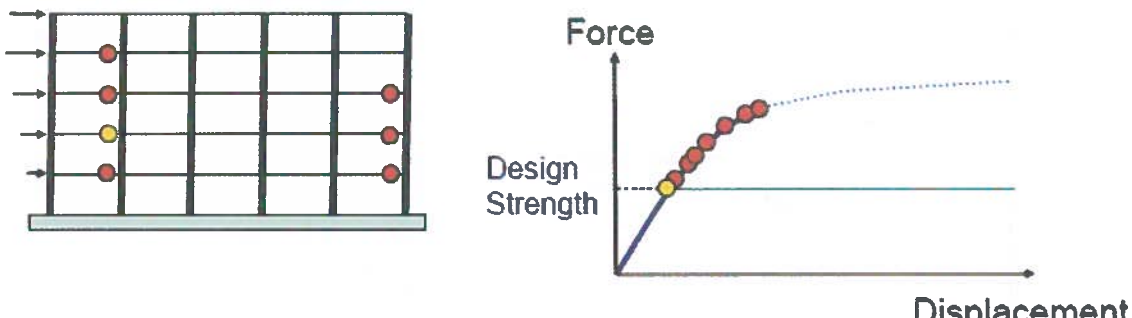
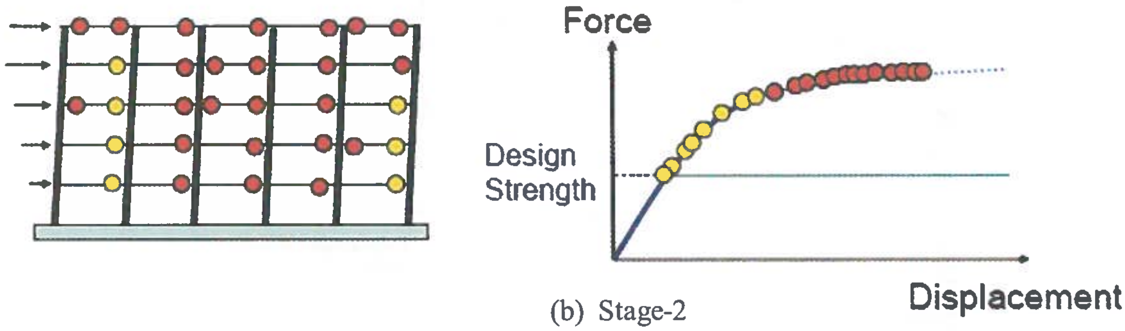


Figure 5.21 Development of Plastic Hinges in Structure at First Significant Yielding

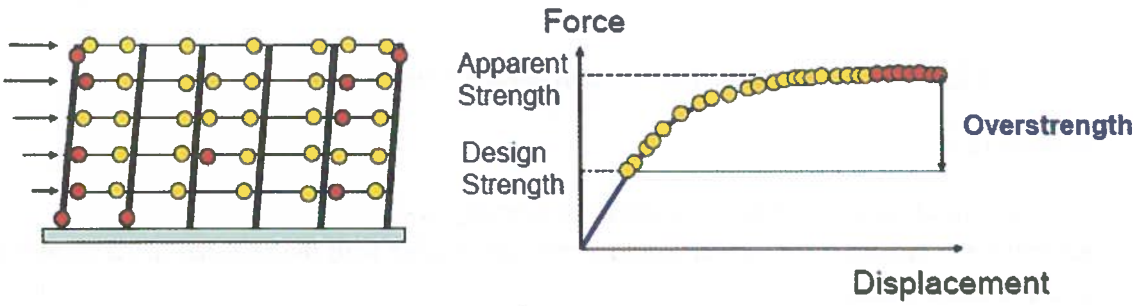
The strength requirements set at different codes contemplate that the design includes a seismic – force -resisting system with redundant characteristics wherein significant structural over - strength above the level of significant yield can be obtained by plastification at other points in the structure prior to the formation of a complete mechanism.



(a) Stage-1



(b) Stage-2



(c) Stage-3

Figure 5.22 Sequential Development of Plastic Hinges in Structure [*After Ref.* 5.14]

For example, Figure 5.23 shows the lateral load-deflection curve for a typical structure. Significant yield is the level where plastification occurs at the most heavily loaded element in the structure, shown as the lowest yield hinge on the load-deflection diagram of Figure 5.23. With increased loading, causing the formation of additional plastic hinges (see Figure. 5.23), the capacity increases (following the solid curve) until a maximum is reached. The maximum resistance developed along the curve is substantially higher than that at first significant yield, and the margin is referred to as the *over-strength capacity*. The over-strength capacity obtained by this continued inelastic action provides the reserve strength necessary for the structure to resist the extreme motions of the actual seismic forces that may be generated by the design ground motion. With reference to Figure 5.22, Structural Over-strength Factor is thus defined as:

$$\text{Structural Over-strength Factor, } \Omega o = \frac{\text{Apparent Strength}}{\text{Design Strength}}$$

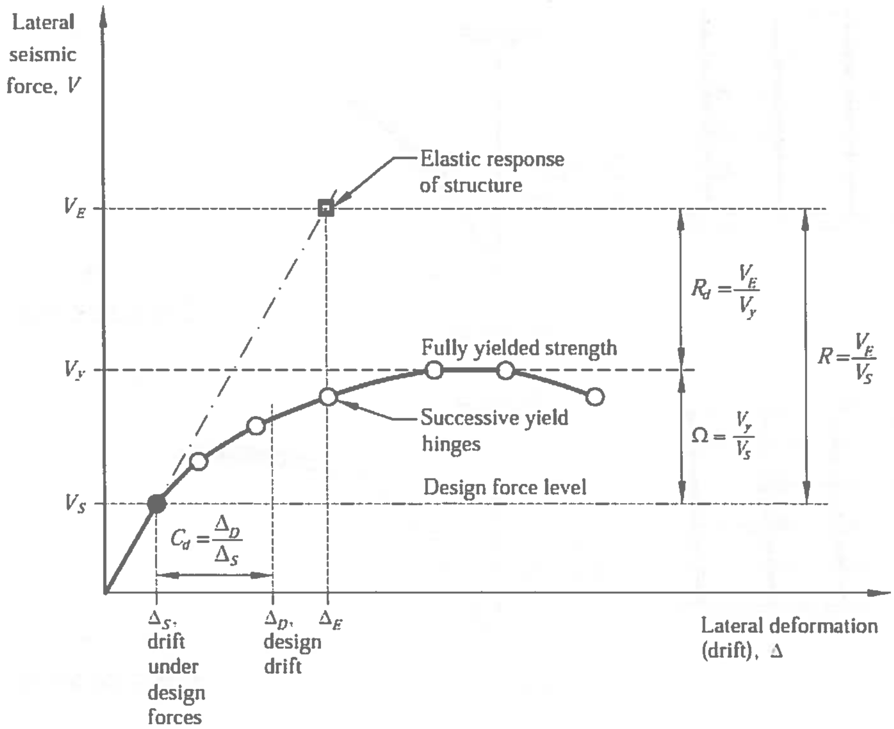


Figure 5.23 Inelastic Force-Deformation Curve [*After Ref.* 5.15]

5.3.6.2 Sources of Over-Strength

i. Development of sequential plastic hinging in a properly designed, redundant structure.

ii. Material Over-strength (i.e., actual material strengths higher than the nominal material strengths specified in the design)

iii. Member design strengths usually incorporate a strength reduction (or resistance) factor, Φ, to produce a low probability of failure under design loading.

iv. Designers themselves introduce additional over-strength by selecting sections or specifying reinforcing patterns that exceed those required by the computations.

Due to these over-strengths, it has been found that first significant yielding of structures may occur at lateral load levels that are 30 to 100 percent higher than the prescribed design seismic forces, such as for moment resisting frame. If provided with adequate ductile detailing, redundancy and regularity, full yielding of structures may occur at load levels that are two to four times the prescribed design force levels [*Sec.*4.2, *Ref.* 5.15].

Most structural systems have some components or limit states that cannot provide reliable inelastic response or energy dissipation. Such components or limit states must be protected by *capacity design* where the corresponding forces at the design seismic force level are amplified by the structural over-strength factor. This specified over-strength factors in design codes are neither an upper nor a lower bound; it is simply an approximation specified to provide a nominal degree of protection against undesirable behavior.

5.3.6.3 Response Modification Factor, *R*

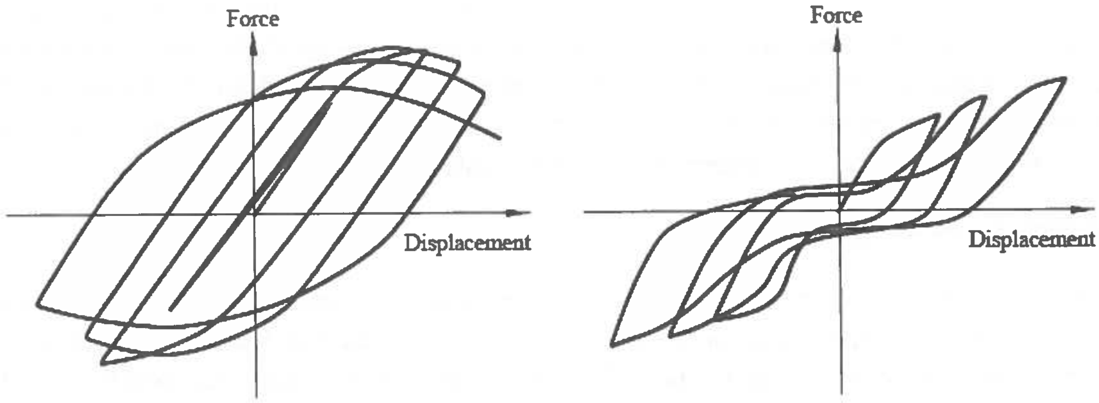
The response modification coefficient, *R,* represents the ratio of the forces that would develop under the specified ground motion if the structure had entirely linear-elastic response to the prescribed design forces (see Figure 5.23). The structure must be designed so that the level of significant yield exceeds the prescribed design force. The ratio *R*, expressed as $R = V_E/V_S$, is always larger than 1.0.

This reduction is possible for a number of reasons [*Sec. C.*12.1.1, *Ref.* 5.16].

i. As the structure begins to yield and deform in-elastically, the effective period of response of the structure lengthens which, for most structures, results in a reduction in strength demand.

ii. Furthermore, the inelastic action results in a significant amount of energy dissipation (hysteretic damping) in addition to other sources of damping present below significant yield.

The combined effect, which is also known as the ductility reduction, explains why a properly designed structure with a fully yielded strength (V_y in Figure 5.23) that is significantly lower than the elastic seismic force demand (V_E in Figure 5.23) is capable of providing satisfactory performance under design ground motion excitations.



a. Ductile hysteresis loops

b. Pinched hysteresis loops

Figure 5.24 Typical Hysteretic Curve [*After Ref.* 5.16]

The energy dissipation resulting from hysteretic behavior can be measured as the area enclosed by the force-deformation curve of the structure as it experiences several cycles of excitation. Some structures have far more energy dissipation capacity than others. The extent of energy dissipation capacity available depends largely on the amount of stiffness and strength degradation the structure undergoes as it experiences repeated cycles of inelastic deformation. Figure 5.24 shows representative load deformation curves for two simple substructures such as a beam-column assembly in a frame.

Hysteretic curve (a) in the figure is representative of the behavior of substructures that have been detailed for ductile behavior. The substructure can maintain nearly all of its strength and stiffness over several large cycles of inelastic deformation. The resulting force-deformation "loops" are quite wide and open, resulting in a large amount of energy dissipation.

Hysteretic curve (b) represents the behavior of a substructure that has not been detailed for ductile behavior. It loses stiffness rapidly under inelastic deformation, and the resulting hysteretic loops are quite pinched. Such a substructure has much less energy dissipation than that for the substructure (a) but has a greater change in response period. The structural response is determined by a combination of energy dissipation and period modification.

Figure 5.23 allows introducing the following relationships:

$$\text{Ductility Reduction Factor, } R_d = \frac{\text{Elastic Strength Demand}}{\text{Apparent Strength}}$$

$$\text{Response Modification Factor, } R = \frac{\text{Elastic Strength Demand}}{\text{Design Strength}}$$

$$= R_d \Omega_o$$

The R values used in different codes are based largely on engineering judgment of the performance of the various materials and systems in past earthquakes, the toughness (ability to dissipate energy without serious degradation) of the system, and the amount of damping typically present in the system when it undergoes inelastic response as well as research studies [*Sec.* 4.2, *Ref.* 5.15].The R factor for a specific project should be chosen and used with care. For example, lower values should be used for structures possessing a low degree of redundancy wherein all the plastic hinges required for the formation of a mechanism may be formed essentially simultaneously and at a force level close to the specified design strength. This situation can result in considerably more detrimental P-Δ effects.

5.3.6.4 Deflection Amplification Factor, *Cd*

To control drift or to check deformation capacity in some deformation-controlled members, an approach similar to the structure over-strength factor, Ω_o is also adopted. A Deflection Amplification Factor, C_d (see Figure 5.23) is introduced to predict expected maximum deformations from that produced by the design seismic forces.

A deformation or storey drift check in the force-based design procedure has been performed in either of two formats in different codes: serviceability and ultimate limit state check. Prior to the 1997 SEAOC Blue Book [*Ref.* 5.18] and *Uniform Building Code* (UBC) [*Ref.* 5.19], the serviceability drift check was intended to minimize nonstructural damage caused by more frequent minor or moderate earthquakes. A drift limit of 0.005 of the storey height is generally accepted as effective for this purpose. Originally developed by ATC 3 - 06 [*Ref.* 5.20], and later the NEHRP *Seismic Design Provisions*, a second format checks inelastic storey drift expected from the design ground motion at a value several times larger than 0.005 of the storey height. The expected inelastic drift, Δ_D, is computed by amplifying the storey drift, Δ_s, by the deflection amplification factor, C_d(see Figure 5.23). The associated drift limit is in the range of 0.015 to 0.025 of the storey height.

5.3.6.5 Redundancy Factor, ρ

Redundancy is an important characteristic of a structure, providing multiple paths of resistance (i.e., load paths). Higher redundancy in a structure implies better reliability. Inelastic action of a structure during a major seismic event can cause part of the structure to fall. For structures expected to experience severe inelastic demands, the lateral load-resisting system of the structure should be made as redundant as possible so that loads can be distributed to other lateral-farce-resisting elements. As an example, redundancy can be achieved by having a moment-resistant frame with many columns and beams, all with ductile connections or by having a dual system, such as shear walls plus a moment-resistant frame.

Redundancy provisions were first introduced into building codes and standards via the 1997 UBC [*Ref.* 5.19]. The redundancy factor, ρ, is applied as an increase in horizontal seismic forces associated with the base shear. This factor effectively reduces the response modification factor, R, based on the extent of structural redundancy inherent in the design configuration of the structure and its lateral force - resisting system.

It should be noted that the lack of redundancy exists when the failure of a component results in the failure of the entire system. Therefore, a logical way to determine the lack of redundancy is to check whether a components failure results in an unacceptable amount of the loss of storey strength or in the development of extreme torsional irregularity.

5.3.6.6 Structural Importance Factor, *I*

The purpose of this factor, *I* is to specifically improve the capability of certain types of buildings such as essential facilities and structures containing substantial quantities of hazardous materials, to function during and after design earthquakes. Table 6.2.17 of BNBC 2015 [*Ref.* 5.21, Part 6 § 2.5.5.1] provides importance factor,

I =1-1.5 different occupancy category. This factor is intended to reduce the ductility demands and result in less damage.

Although a value of *I* greater than unity has the effect of reducing the ductility expected of a structure, however, added strength due to higher design forces by itself is not sufficient to ensure superior seismic performance. Connection details that assure ductility, quality assurance procedures, and limitations on building deformation are also important to improve the functionality and safety in critical facilities and those with high-density occupancy. Consequently, the reduction in the damage potential of critical facilities is also addressed by using more conservative drift controls and by providing special design and by detailing requirements and material limitations.

5.3.6.7 Seismic Dead Load, *W*

The weight, *W* used to calculate base shears and building periods is normally the total seismic dead load of the structure. This includes the weight of the ceiling, partitions, pipes, ducts, and equipment that are normally attached. *W* does not include full design roof and live loads. The objective in summing up seismic weight *W* is to include all contributions to mass likely to be present at the time of an earthquake. However, applicable portions of other loads should be included as follows [*Ref.* 5.2, Part 6 §2.5.9.3]

i. For live load up to and including 3 kN/m^2, a minimum of 25% of the live load shall be applicable.
ii. For live load above 3 kN/m^2, a minimum of 50% of the live load shall be applicable.
iii. Total weight (100%) of permanent heavy equipment or retained liquid or any imposed load sustained in nature shall be included.

Where the probable imposed loads (mass) at the time of earthquake are more correctly assessed, the designer may go for higher percentage of live load.

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CHAPTER 6. DETAILING OF REINFORCEMENT

6.1 GENERAL

In the context of reinforced concrete and masonry structures, detailing refers to the preparation of placing drawings, reinforcing bar configurations, and bar lists that are used for fabrication and placement of reinforcement in structures. But detailing also incorporates a design process by which the designer ensures that each part of the structure can perform safely under service load conditions and also when specially selected critical regions are to accommodate large inelastic deformations. Particularly, it covers such aspects as the choice of bar sizes, the distribution of bars, curtailment and splice details of flexural reinforcement, and the size, spacing, configuration, and anchorage of transverse reinforcement, intended to provide shear strength and ductility to critical regions [*Ref* 6.1].

The essence of seismic detailing is to prevent premature shear failures in members and joints, buckling of compression bars, and crushing of concrete. It is not sufficient to have only strength capability; there must also be special details to actualize the inelastic behavior of the seismic-resisting elements to ensure that the system remains stable at deformations corresponding to maximum expected ground motion. Vertical loads must be supported even when maximum elastic deformations are exceeded. In other words, inelastic yielding is allowed in resisting seismic loads as long as yielding does not impair the vertical load capacity of the structure. [*Ref* 6.2]

6.2 DETAILS FOR DUCTILE BEHAVIOR

Ductile behavior of reinforced concrete members is based on the following principles [*Ref* 6.4].

6.2.1 Confinement for Heavily Loaded Sections

Plain concrete has relatively small usable compressive strain capacity (around 0.003), and this might limit the deformability of beams and columns of special moment frames. Strain capacity can be increased ten-fold by confining the concrete with reinforcing spirals or closed hoops. The hoops act to restrain dilation of the core concrete as it is loaded in compression, and this confining action leads to increased strength and strain capacity. In addition to providing confining effect hoops act-

- As shear reinforcement
- Prevents buckling of longitudinal reinforcement
- Prevents bond splitting failures

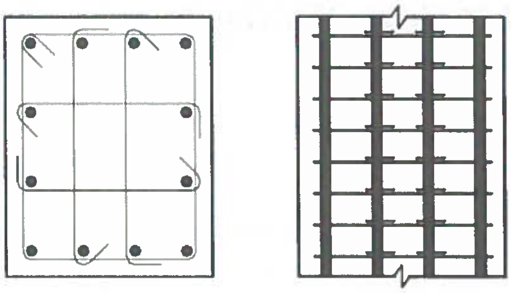

Figure 6.1 Hoops Confine Heavily Stressed Cross Sections of Columns ond Beams, with (A) Hoops Surrounding the Core and Supplementary Bars Restraining Longitudinal Bars, All of Which are (B) Closely Spaced along the Member Length.

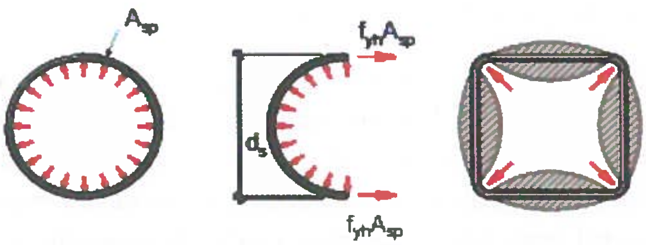

Figure 6.2 Confinement Provided by Different Hoop Shape

Hoops typically are provided at the ends of columns, as well as through beam-column joints, and at the ends of beams. Figure 6.2 shows a column hoop configuration using rectilinear and circular hoops. The hatched areas in the figures may spall. Confining steel is in tension (hoop stress effect) because, due to Poisson's effect, as the concrete is compressed in one direction, it expands in the orthogonal directions. This is shown in the center illustration. Note that hoops are not as efficient as spirals in confining concrete because the sides of the hoop can flex outward as the confined concrete expands outward.

To be effective, the hoops must enclose the entire cross section except the cover concrete, which should be as small as allowable, and must be closed by 135° hooks embedded in the core concrete; this prevents the hoops from opening if the concrete cover spalls off. Crossties should engage longitudinal reinforcement around the perimeter to improve confinement effectiveness. The hoops should be closely spaced along the longitudinal axis of the member, both to confine the concrete and restrain buckling of longitudinal reinforcement. Crossties, which typically have 90° and 135° hooks to facilitate construction, must have their 90° and 135° hooks alternated along the length of the member to improve confinement effectiveness.

6.2.2 Ample Shear Reinforcement

Shear strength degrades in members subjected to multiple inelastic deformation reversals, especially if axial loads are low. In such members ACI 318 [*Ref.* 6.3] requires that the contribution of concrete to shear resistance be ignored, that is, $V_c = 0$. Therefore, shear reinforcement is required to resist the entire shear force.

6.2.3 Bond & Anchorage

Efficient interaction of the two constituent components of reinforced concrete and masonry structures requites reliable bond between reinforcement and concrete to exist. In certain regions, particularly where inelastic and reversible strains occur, heavy demand may be imposed on stress transfer by bond. The most severe locations are beam-column joints, to be examined in Section 6.3.3.

Established recommendations, embodied in various codes, aim to ensure that reinforcement bars are adequately embedded in well-compacted concrete so that their yield strength can be developed reliably without associated deformations, such as slip or pullout, becoming excessive. Code provisions are generally such that an adequately anchored bar, when overloaded in tension, will fracture rather than pull out from its anchorage.

6.2.3.1 Development of Bar Strength

The length of a deformed bar required to develop its strength, whether it is straight (l_d) or hooked (l_{dh}), is affected by a number of principal parameters, such as concrete strength, yield strength of steel, thickness of cover concrete, and the degree of confinement afforded by transverse reinforcement or transverse compression stresses.

(i) **Development of Straight Deformed Bars in Tension:** A bar should extend beyond the section at which it may be required to develop its strength f_y, by at least a distance ld provided by Equation 12-1 of ACI318-08.

$$\ell_d = \left[\frac{3 f_y \psi_t \psi_c \psi_s}{40\lambda \sqrt{f_c'}\left(\frac{C_b + K_{tr}}{d_b}\right)}\right] d_b \tag{6.1}$$

(ii) **Development of Deformed Bars in Tension Using Standard Hooks**: In the event that the desired tensile stress in a bar cannot be developed by bond alone, it is necessary to provide special anchorage at the ends of the bars, usually by means of a 90^0 or a 180^0 hook. Hooked bar resist pullout by the combined actions of bond along the straight length of bar leading to the hook and anchorage provided by the hook. The different code provisions accounts for these combined actions and recommends a total development length, l_{dh}, as shown in Figure 6.4 and is measured from the critical section to the farthest point on the bar, parallel to the straight part of the bar.

$$\ell_{dh} = \left[\frac{0.02 f_y \psi_e}{\lambda \sqrt{f_c'}}\right] d_b \tag{6.2}$$

But shall not be less than the larger of $8d_b$ and 6 in (150 mm) for normal weight concrete and reinforcements other than epoxy-coated reinforcements ψ_e and λ shall be taken as 1.0.

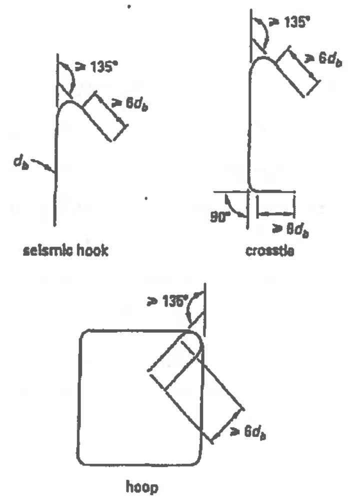

Figure 6.3 Seismic Hook, Crosstie, and Hoop [*After Ref.* 6.3, § 21.1 and 21.3.3.1]

A seismic hook is a hook on a stirrup that engages the longitudinal reinforcement and projects into the interior of the stirrup or hoop. For a seismic hook, *the bend must be not less than* 135°, *except that circular hoops shall have a bend not less than 90° and extension past the bend of at least 6d_b (but not less than 3 inch/75mm)*. A crosstie is a continuous reinforcing bar. It has *a seismic hook at one end and a hook at least 90° with an extension past the bend of at least 6d_b at the other end.* A hoop is a closed tie made up of several reinforcing elements, each consisting of seismic hooks at both ends. A hoop can also be a continuously wound tie having seismic hooks at both ends. The details of seismic hooks, crossties, and hoops are shown in Figure 6.3.

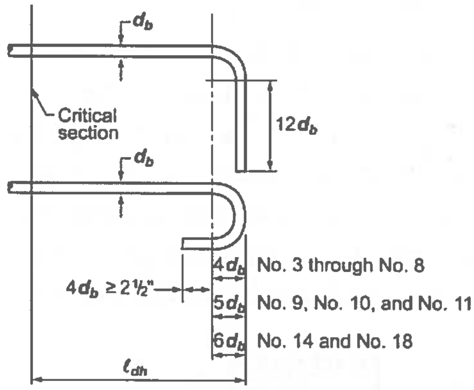

Figure 6.4 Hooked Bar Details for Development of Standard Hooks in Tension [*After Ref.* 6.4 Figure R 12.5] Lapped Splices Requirements of Lap Splices for Deformed Bars and Deformed Weirs in Tension

By necessity, reinforcing bars placed in structural member need often to be spliced. This is commonly achieved by overlapping parallel bars, as shown in Figure 6.5. Force transmission relies on bond between bars and the surrounding concrete and the response of the concrete in between adjacent bars. Therefore, the length of the splice l_s as shown in Figure 6.5 (c), is usually the same as the development length l_d described in § 6.2.3.1. However, when large steel forces are to be transmitted by bond, cracks due to splitting of the concrete can develop. Typical cracks at single or lap-spliced bars are shown in Figure 6.5 (a) and (b). To enable bar forces to be transmitted across continuous splitting cracks between lapped bars, as seen in Figure 6.5, a shear friction mechanism needs to be mobilized [*Ref. 6.5*]. To control splitting forces, particularly at the end of splices [Figure 6.5 (c)], clamping forces developed in transverse ties are required.

6.2.4 Avoidance of Anchorage or Splice Failure

Severe seismic loading can result in loss of concrete cover, which will reduce development and lap-splice strength of longitudinal reinforcement. Lap splices, if used, must be located away from sections of maximum moment (that is, away from ends of beams and columns) and must have closed hoops to confine the splice in the event of cover spalling. Bars passing through a beam-column joint can create severe bond stress demands on the joint; for this reason, ACI 318 restricts beam bar sizes. Bars anchored in exterior joints must develop yield strength (f_y) using hooks located at the far side of the joint.

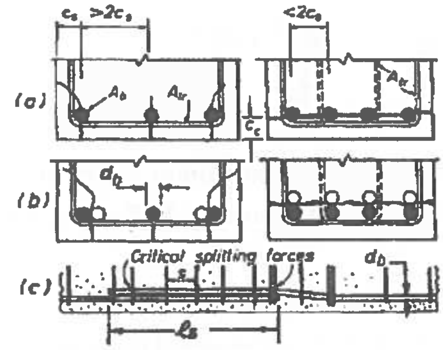

Figure 6.5 Bar Force Transmission by Shear Friction at a Lapped Splice [*After Ref.* 6.1]

6.3 Arrangement of Reinforcing Steel in SMF

6.3.1 Rebar Arrangement in Beam

6.3.1.1 Flexural Reinforcement

As explained in Figure 6.6, earthquake shaking results in tension in both faces of structural elements due to direction reversal of the shaking whereas gravity loading causes tension at fixed locations (tension at bottom face at mid span and top face near support for beam). Since concrete is very weak in tension, ACI 318, § 21.5.2.1 requires that at least two top and two bottom bars must run through the entire length of the beams as shown in Figure 6.7 to resist reversal of bending moment.
As shown in equation below, $A_{s,min}$ shall not be less than ($1.4/f_y$) $b_w d$ in SI.

$$A_{s,min} = \frac{0.25\sqrt{f_c'}}{f_y} b_w d \geq \frac{1.4}{f_y} b_w d$$

(*Ref:* BNBC-2015 Part 6 § 8.3.4.2) Similarly, steel bars are required on all faces of column too.

$$A_{s,min} = \frac{3\sqrt{f_c'}}{f_y} b_w d \geq \frac{200}{f_y} b_w d$$

(ACI-318-11: Equation 10-3 and 21.5.2.1) in FPS

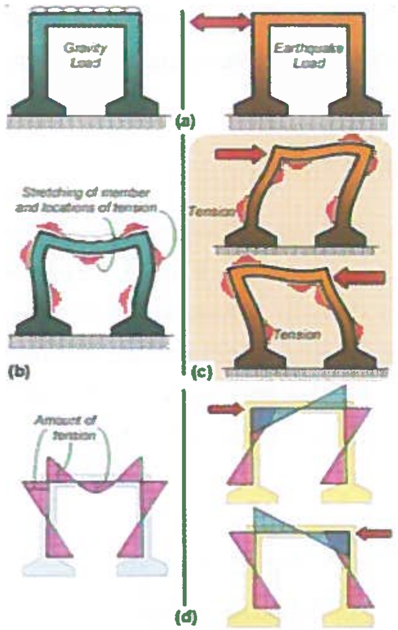

Figure 6.6 Earthquake Shaking Reverses Tension and Compression in Members-Reinforcement is Required on both Faces of Members [*After Ref.* 6.6]

6.3.1.2 Transverse Reinforcement

Beams in special moment frames [*For SDC D, E & F*] are required to have either hoops or stirrups along their entire length. Hoops fully enclose the beam cross section and are provided to confine the concrete, restrain longitudinal bar buckling, improve bond between reinforcing bars and concrete, and resist shear. Stirrups, which generally are not closed, are used where only shear resistance is required.

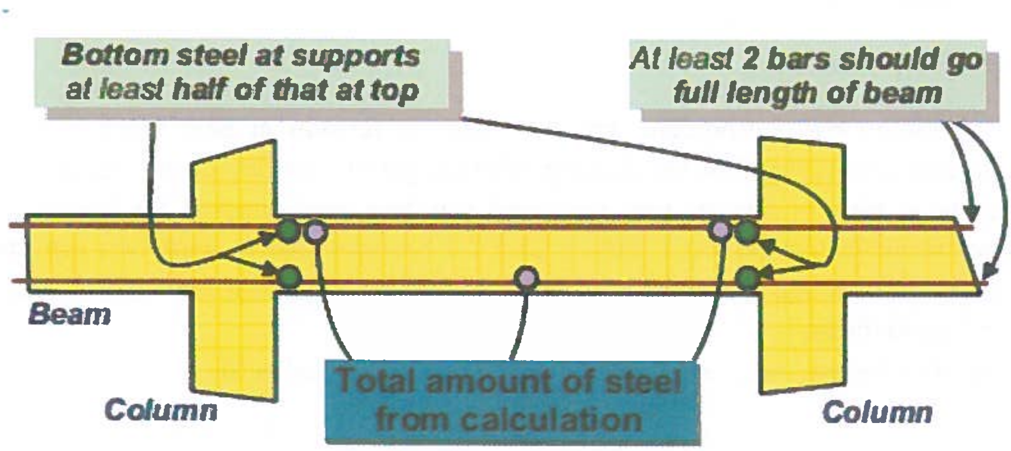

Figure 6.7 Location and Amount of Longitudinal Steel Bars in Beams- these Resist Tension due to Flexure [*After Ref.* 6.7].

Hoop reinforcement may be constructed of one or more closed hoops. Alternatively, it may be constructed of typical beam stirrups with seismic hooks at each end closed off with crossties having 135° and 90° hooks at opposite ends. Using beam stirrups with crossties rather than closed hoops is often preferred for constructability so that the top longitudinal beam reinforcement can be placed in the field, followed by installation of the crossties.

(i) **Placement of Hoops & Stirrups.** Where hoops are being provided at each end of a beam and along a reinforcement splice, there may not be much length of the beam left where stirrups are acceptable. Because of this aspect, and to prevent placement errors, it is practical to extend the hoop detail and spacing over the entire length of the beam. A quick quantity comparison should be conducted to see the difference in the amount of detailed reinforcement.

Hoops are required along the beam end zones (where flexural yielding is expected) and along lap splices, with spacing limits. Elsewhere, transverse reinforcement is required *at a spacing not to exceed d/2* and is permitted to be in the form of beam stirrups with seismic hooks. Hoops spacing details should not discussed as per ACI-318-11: 21.5.3 (BNBC-2015 Part 6 § 8.3.4.3)

6.3.1.3 Tension Lap-Splice

Beam longitudinal bar lap splices shall not be used (a) within beam-column joints, (b) within a distance from the face of a joint equal to twice the member depth, and (3) where flexural yielding is anticipated [*Ref.* 6.4, §21.5.2.3].

Lap splices are not permitted where flexural yielding is expected (*i.e*, at plastic hinge points) because such splices are not reliable when the loading repeatedly exceeds the yield strength of the reinforcement. Generally, if lap splices are used, they are placed near the mid-span of the beam. Where permitted, they are proportioned as tension lap splices. If beam longitudinal bars are lap-spliced, hoops are required along the length of the lap, and longitudinal bars around the perimeter of the cross section are required to have lateral support conforming to ACI 318, §7.10.5.3.

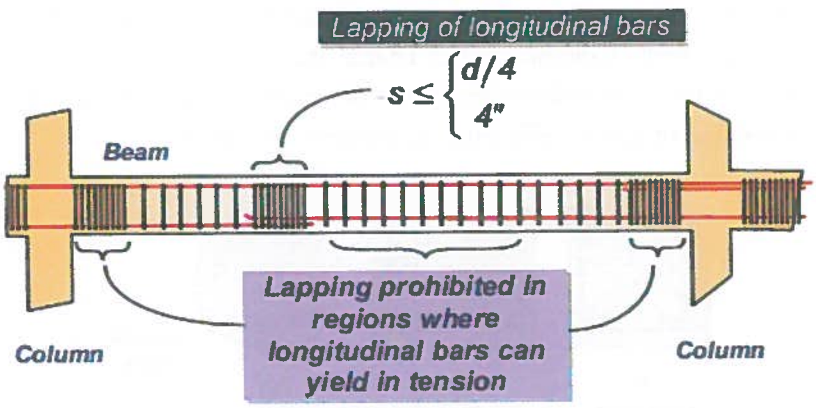

Figure 6.8 Detail of Lapping Reinforcement in Seismic Beams [*After Ref.* 6.7]

6.3.2 Rebar Arrangement in Column

Because of drift limits, slender columns cannot be used for seismic-dominated ductile frames. Therefore, it is often found that columns are large enough to resist the specified earthquake forces with steel contents in the range $0.01 < \rho < 0.06$ [*Ref.* 6.4, § 21.6.3] ACI 318, § 21.6.1 requires the minimum member dimension of column to be 300mm (12 in) and the ratio of the shortest-to-longest dimension cannot be less than 0.4.

6.3.2.1 Vertical Reinforcement in Column

In the choice of the number and size of bars to be used in columns to satisfy strength requirements, apart from aspects of economy and ease in construction, the following points should also be considered [*Ref.* 6.1].

i. ACI 318 (7.10.5.3) maximum clear spacing 150 mm (6") For efficient confinement in end regions, column bars should be reasonably closely spaced around the periphery. Bars should not be farther apart than 8 in (200 mm) center to center, or 1/3 of the cross-section dimension in the direction considered in the case of rectangular columns, or 1/3 of the diameter in the case of circular cross sections.

ii. Intermediate column bars, such as shown in Figure 6.9, also serve as vertical joint shear reinforcement. If they are omitted, additional joint shear reinforcement would need to be provided.

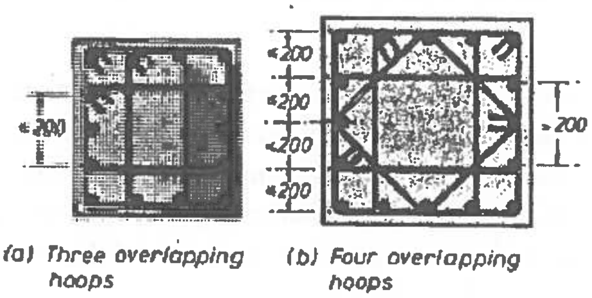

Figure 6.9 Typical Column Details Showing the Use of Overlapping Hoops [*After Ref.* 6.1]

iii. The two points above suggest a reasonably uniform spacing of bars, preferably of the same size, around the section periphery. Column design charts are prepared for this common case. Some designers are tempted to utilize bundled column reinforcement, grouped in the four corners, as shown in Figure 6.10 (a), because of greater efficiency in moment resistance.

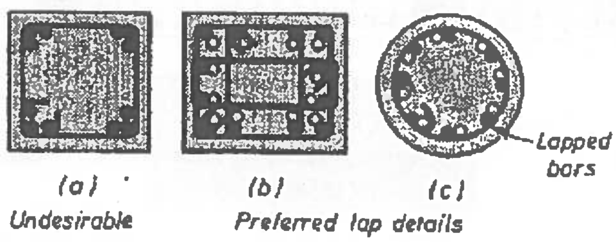

Figure 6.10 Typical Arrangement of Column Bars in Bundles and at Lapped Splices [*After Ref.* 6.1]

6.3.2.2 Transverse Reinforcement in Column (ACI 318-11: 21.6.4)

The column transverse reinforcement should initially be selected based on the confinement requirements of ACI 318 - 11: 21.6.4. For rectangular cross sections, the total cross-sectional area of rectangular hoop reinforcement is not to be less than that required by either of the following two equations, whichever gives the larger amount.

$$A_{sh} = 0.3(s\, b_c f'_c / f_{yt})\, [(A_g / A_{ch}) - 1]$$
$$A_{sh} = 0.09\, s b_c f'_c / f_{yt}$$

Both of these equations must be checked in both principal directions of the column cross section.

6.3.3 Beam-Column Joint

6.3.3.1 Beam-Column Joint Under Seismic Loading

Figure 6.11 and Figure 6.12 shows joint loads acting on the free body of a typical joint of a frame subjected to gravity and seismic load respectively. In general the moment acting on opposite beams are opposite in sense for gravity loading whereas they act in same direction (clockwise or anti-clockwise) in case of seismic loading which results in very high horizontal shear within the joint (see Figure 6.12 (b)).

Under these moments, the top bars in the beam-column joint are pulled in one direction and the bottom ones in the opposite direction (Figure 6.13 (a)). These forces are balanced by bond stress developed between concrete and steel in the joint region. If the column is not wide enough of the strength of the concrete in the joint is low, there is insufficient grip of concrete on the steel bars. In such circumstances, the bar slips inside the joint region, and beams lose their capacity to carry load.

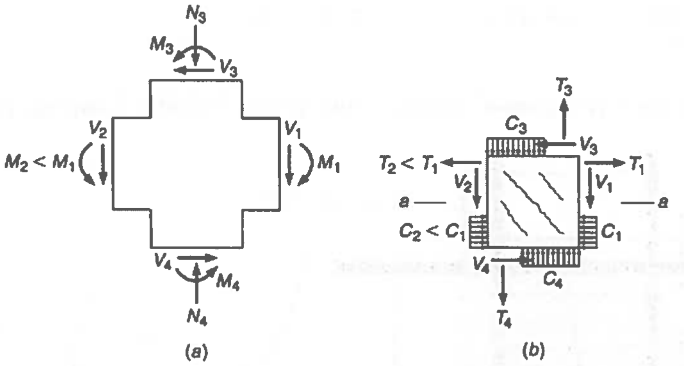

Figure 6.11 Joint Loads & Forces Resulting from Gravity Loads: (A) Forces & Moments on the Free Body of a Joint Region (B) Resulting Internal Forces [*After Ref.* 6.8]

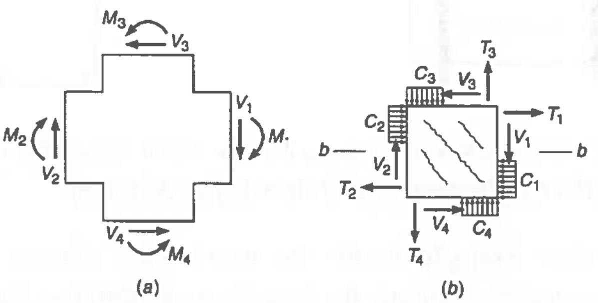

Figure 6.12 Joint Loads & Forces Resulting from Seismic Loads: (A) Forces & Moments on the Free Body of a Joint Region (B) Resulting Internal Forces [*After Ref.* 6.8]

Further, under the action of the above pull-push effect, joints undergo geometric distortion (Figure 6.13 (b)). Again, if the column cross section size insufficient, the concrete in the joint develops diagonal cracks.

6.3.3.2 Beam-Column Joint Detailing for Special Moment Frames

Detailing beam-column joints is an art requiring careful attention to several code requirements as well as construction requirements. Problems of diagonal cracking and crushing of concrete in joint region can be controlled by two means, providing large column sizes and providing closely spaced hoops around column main bars (Figure 6.14).

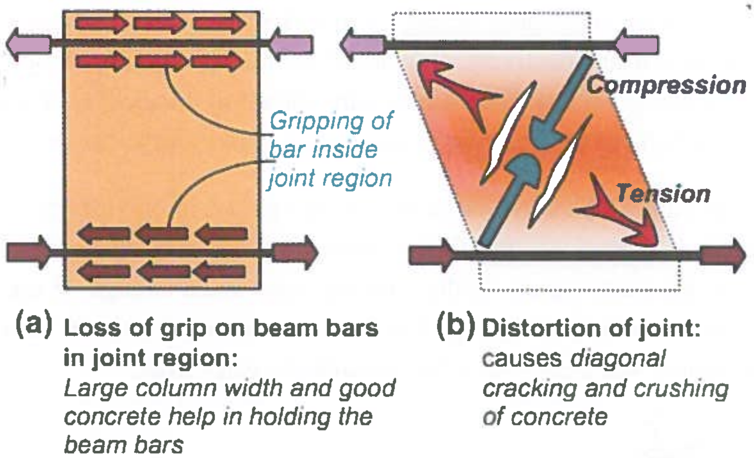

Figure 6.13 Push-Pull Effect in Beam-Column Joint under Seismic Loading [*After Ref.* 6.9]

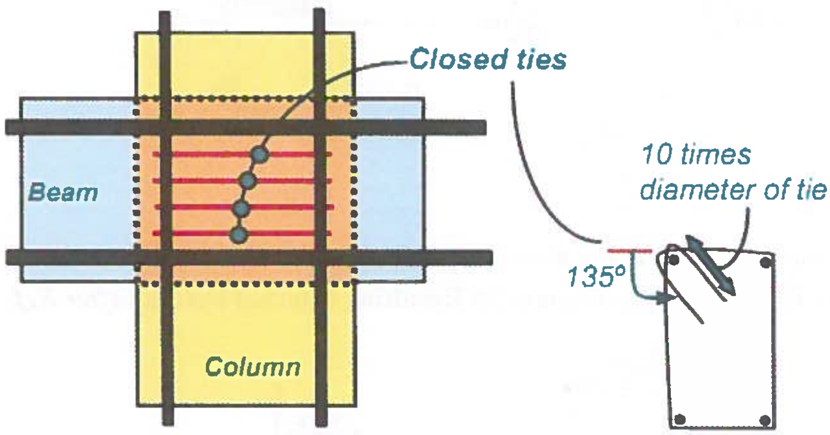

Figure 6.14 Closed Loop Steel Ties in Beam-Column Joints- Such Ties with 135° Hooks Resist the Ill Effect of Distortion of Joints [*After Ref.* 6.9]

Figure 6.15 and Figure 6.16 show example details for interior and exterior beam-column joints, respectively. Note that beam bars, possibly entering the joint from two different framing directions, must pass by each other and the column longitudinal bars. Joint hoop reinforcement is also required. Large-scale drawings or even physical mockups of beam-column joints should be prepared prior to completing the design so that adjustments can be made to improve constructability. This subject is discussed in more detail in Section 6.3.4.

(i) **Anchoring beam bars in joints**: It is important for beam and column longitudinal reinforcement to be anchored adequately so that the joint can resist the beam and column moments. Different requirements apply to interior and exterior joints.

In interior joints, beam reinforcement (both top & bottom) typically extends through the joint and is anchored in the adjacent beam span. Also, these bars must be placed within the column bars and with no bends (Figure 6.17).

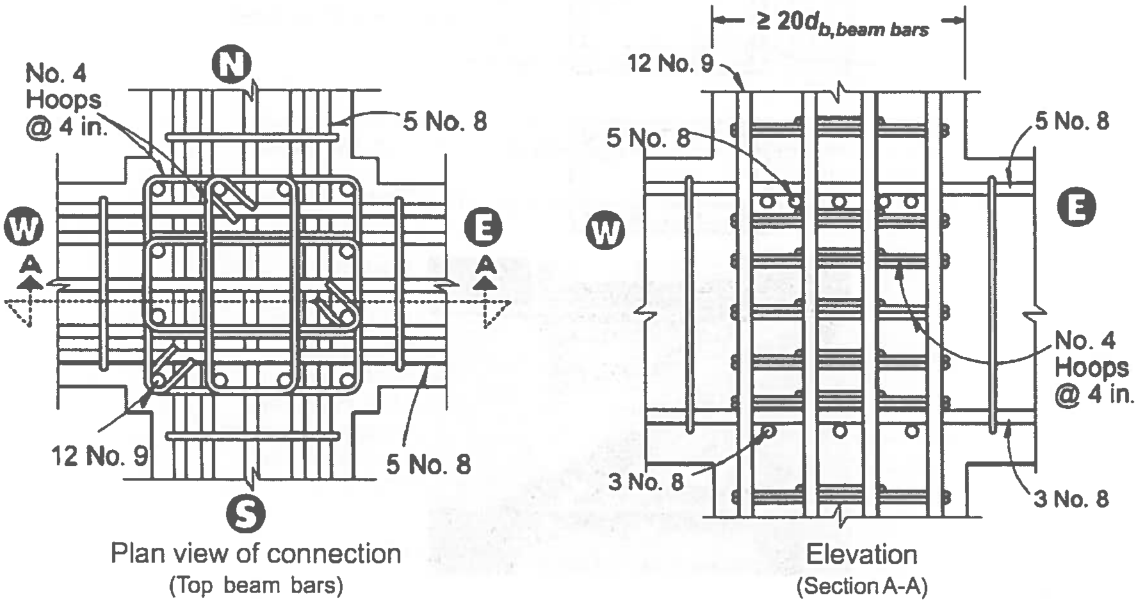

Figure 6.15 Example Interior Joint Detailing [*After Ref.* 4.3]

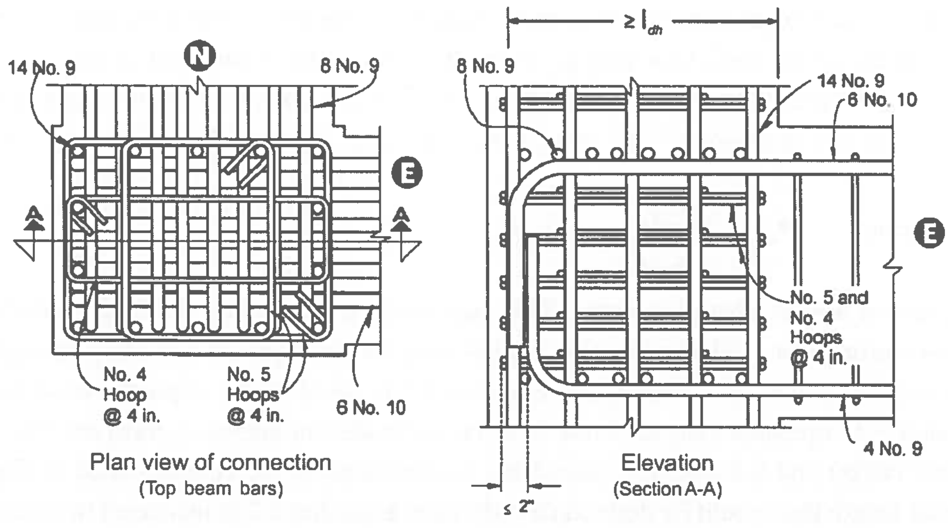

Figure 6.16 Example Exterior Joint Detailing [*After Ref.* 6.3]

ACI 318 requires that for special moment frame, the column dimension parallel to the beam longitudinal reinforcement be *at least 20 times longitudinal bar diameters* for normal weight concrete (Figure 6.15). This requirement helps improve performance of the joint by resisting slip of the beam bars through the joint. Some slip, however, will occur even with this column dimension requirement.

ACI-ASCE Committee 352 report *Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures* [*Ref. 6.10*], recommends that the beam depth be at least *20 times the diameter of the column longitudinal reinforcement* for the same reason. ACI 318 does not include this requirement.

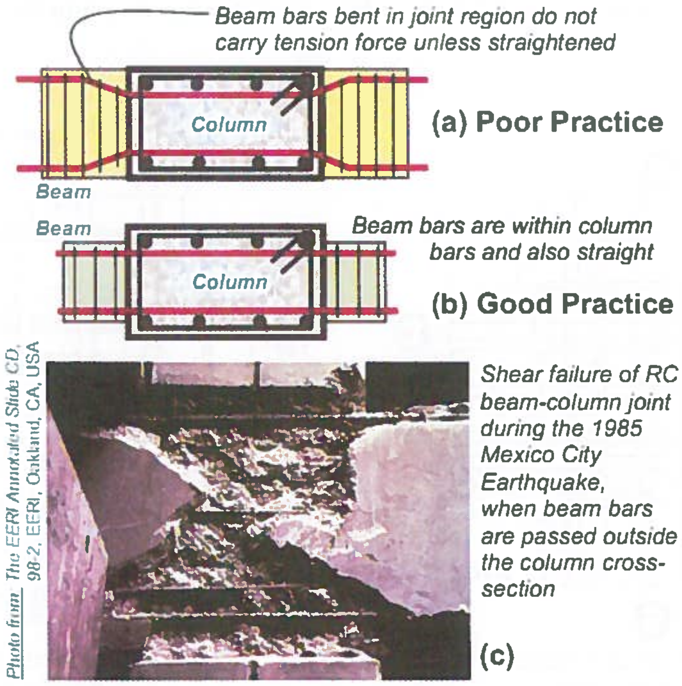

Figure 6.17 Anchorage of Beam Bars in Interior Joints [*After Ref.* 6.9]

For exterior joints, beam longitudinal reinforcement usually terminates in the joint with a standard hook (Figure 6.16). The tail of the hook must project toward the mid-depth of the joint so that a joint diagonal compression strut can be developed. The length for a standard 90° hook in normal-weight concrete *must be the largest of 8 bar diameters, 150 mm (6 inches), and the length required by the following expression*:

$$l_{dh} = \frac{f_y d_b}{65\sqrt{f_c'}} \text{ in FPS} \qquad l_{dh} = \frac{f_y d_b}{5.4\sqrt{f_c'}} \text{ In SI} \qquad (6.3)$$

The latter expression almost always governs. This expression assumes that the hook is embedded in a confined beam-column joint. The expression applies only to bar sizes 10 mm (#3) through 32 (#11). ACI 318 in its commentary R 21.7.5 states that Equation 6.3 is based on the requirement of Equation 6.2. Because Equation 6.3 stipulates that, the hook is to be embedded in confined concrete. The coefficient 0.7 (for concrete cover) and 0.8 (for ties) have been incorporated in the constant used in Equation 6.3. The development length that would be derived directly from Equation 6.2 is increased to reflect the effect of load reversal.

In addition to satisfying the length requirements of l_{dh}, hooked beam bars are required to extend to the far side of the beam-column joint (ACI 318-11,§ 21.7.2.2, BNBC-2015 Part 6 §8.3.7). This is to ensure the full depth of the joint is used to resist the joint shear generated by anchorage of the hooked bars. It is common practice to hold the hooks back 25 mm (an inch) or so from the perimeter hoops of the joint to improve concrete placement. Hooks shall not be considered effective in developing bars in compression.

(ii) **Joint Transverse Reinforcement:** Joint transverse reinforcement is provided to confine the joint core and improve anchorage of the beam and column longitudinal reinforcement. The amount of transverse hoop reinforcement in the joint is to be the same as the amount provided in the adjacent column end regions.

6.3.4 Detailing & Constructability Issues

A special moment frame relies on carefully detailed and properly placed reinforcement to ensure that it can maintain its strength through multiple cycles beyond the yield deformation. Architectural requirements often push to get the beams and columns as small as possible, resulting in beams, columns, and joints that become very congested. Early in the design process, it is important to ensure that the required reinforcement not only fits within the geometric confines of the elements, but also can be properly placed in the field.

6.3.4.1 Beam-Column Joint

The beam-column joint is the critical design region. By keeping the column and beam dimensions large, beam and column longitudinal reinforcement ratios can be kept low and beam-column joint volumes kept large so that joint shear stresses are within limits. Large joints with low reinforcement ratios also help with placement of reinforcing bars and concrete.

Providing hoops in the joint region requires some extra effort. In practice, this is achieved by preparing the cage of the reinforcement (both longitudinal bars and stirrups) of all beams at a floor level to be prepared on top of the beam formwork of that level and lowered into the cage (Figure 6.18 (a) and (b)). However, this may not always be possible; particularly when the beams are long and the entire reinforcement cage become heavy.

When the beam and column are the same width, these bars are in the same plane in the beam and the column, and they conflict at the joint. This requires bending and offsetting one set of bars, which will increase fabrication costs. Offsetting the bars can also create placement difficulties and results in bar eccentricities that may affect ultimate performance. If the beam is *at least 100 mm (4 inches) wider or narrower than the column (50 mm (2 inches) on each side)*, the bars can be detailed so that they are in different planes and thus do not need to be offset.

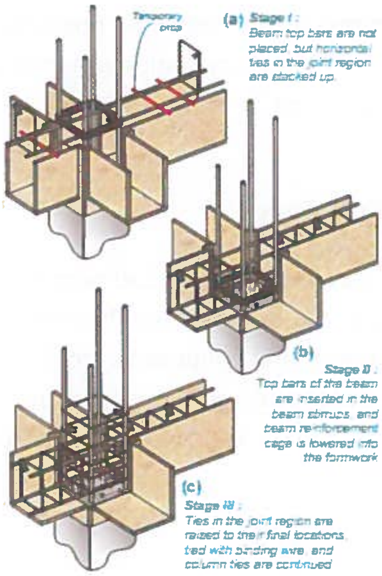

Figure 6.18 Providing Horizontal Ties in the Joints [After *Ref.* 6.9]

Figur 6.19 Beam-Column Joint with Beam Corner Bars Swept to Inside of Column Corner Bars.

Figure 6.20 Beam-Column Joint with Small Diameter Corner Bars

An option, pictured in Figure 6.19, is to gently sweep the corner beam bars to the inside of the column corner bars. This will work if the hoops are detailed as stirrups with a cap tie. Near the column, the corner beam bars will be closer together and the vertical legs of the stirrups are usually flexible enough that they can be pulled over to allow the corner bar to be placed within the 135° hooks. Corner bars will not fit tightly within the bends of the cap tie, but the hook extensions of the 135° hooks are normally long enough so they are still anchored into the core of the beam. One might consider using 135° hooks at both ends of the cap tie to improve the anchorage into the core of the beam.

Another option, pictured in Figure 6.20, is to provide a smaller, discontinuous bar to support the stirrups at the edge of the beam. This requires additional reinforcement that is not contributing to the strength of the moment frame and requires more pieces to be placed. The added reinforcement should be of small diameter so it does not create a large discontinuity in flexural strength of the beam in the potential plastic hinge region.

Making the beam wider or narrower than the column may create undesirable conditions along the exterior edge of a floor and may increase forming costs for both exterior and interior framing locations. Consideration needs to be given to the architectural condition along this exterior location. Even though different beam and column widths work well for the structure, this may create a complicated enclosure detail that is more costly.

To support the beam hoops and stirrups, some of the top bars must be made continuous with lap splices or mechanical couplers near mid-span. To meet the negative moment requirements, shorter bars passing though the column can be added to the continuous top bars.

Figure 6.21 Beam-Column Joint Having Multiple Layers of Beam Reinforcement Hooked at Back Side of Joint. Note the Upturned Beam (The Slab is Cast at the Bottom Face of the Moment Frame Beam).

Multiple layers of longitudinal bars should usually be avoided where possible, as this condition makes placement very difficult, especially when two or more layers of top bars must be hooked down into the joint at an exterior column. If more than one layer of bars is required, it may be because the beam is too small; if this is the case, enlarging the beam is recommended, if possible. This situation also occurs where lateral resistance is concentrated in a few moment frames, requiring large, heavily reinforced beams (Figure 6.21).

6.3.4.2 Beam and Column Confinement

Confinement of beams and columns is crucial to the ductile performance of a special moment frame. Usually confinement is provided by sets of hoops or hoops with crossties.

Hoops are required to have 135°hooks; crossties are permitted to have a 135° hook at one end and a 90° hook at the other end, provided the crossties are alternated end for end along the longitudinal axis of the member. The 135° hooks are essential for seismic construction; alternating 135° and 90° hooks is a compromise that improves constructability. Moreover, there is no real cost premium for 135° hooks and their performance in extreme loadings is superior to 90° hooks.

Another option besides crossties with hooks is to use headed reinforcement (that is, deformed reinforcing bars with heads attached at one or both ends to improve bar anchorage). It is important to ensure that the heads are properly engaged. Special inspection of their final placement is very important. Yet another option is to use continuously bent hoops, that is, hoops constructed from a single piece of reinforcement (Figure 6.22).Whereas these hoops can result in reinforcement cages with excellent tolerances, the pre-bent shape limits field adjustments that may be required when interferences arise.

ACI 318 permits the horizontal spacing between legs of hoops and crossties to be *as large as 350 mm (14 inches) in* columns. Confinement can be improved by reducing this spacing. It is recommended that longitudinal bars be spaced around the perimeter no more than 150 mm (6 inches) or 200 mm (8 inches) apart. According to ACI 318-11,§21.6.4.3, BNBC 8.3.5.4 (b) vertical spacing of hoop sets can be increased from 100 mm (4 inches) to 150 mm (6 inches) as horizontal spacing of crosstie legs decreases from 350 mm (14 inches) to 200 mm (8 inches). This condition is valid only if the smallest longitudinal bar of the column is 25 mm and the minimum width of the column is 600 mm (24 in) ACI-318-11: 21.6.4.3 condition (a) and (b). The extra vertical spacing can reduce the total number of hoop sets and facilitate working between hoop sets. Because a typical hoop set comprises a three-layer stack of bars (crossties in one direction, then the hoop, then the crossties in the other direction), the actual clear spacing between hoop sets can be quite small. The ties and stirrups should be kept to 12 mm (#4) or 16 mm (#5 bars). 20 mm (#6) and larger bars have very large diameter bends and are difficult to place.

Figure 6.22 Column Cage with Hoops Constructed from Single Reinforcing Bar.

Spirally reinforced columns are more ductile than columns with ties and are therefore better for extreme seismic loads. The spirals need to be stopped below the beam-column joint because it is very difficult, if not impossible, to integrate the spirals with the longitudinal beam reinforcement. Because transverse reinforcement is required to extend through the joint per ACI 318, §21.7.3, the spirals can be replaced within the joint by circular hoop reinforcement.

6.3.4.3 Bar Splices

Lap splices of longitudinal reinforcement must be positioned outside intended yielding regions, as noted in Sections 6.3.1.3 and 6.3.2.2. Considering that column and beam ends, as well as lap splice lengths, all require closely spaced hoops, it commonly becomes simpler to specify closely spaced hoops along the entire beam or column length. This is especially common for columns.

Large diameter bars require long lap splices. In columns, these must be detailed so they do not extend outside the middle half of the column length and do not extend into the length l_0 at the end of the column. If longitudinal bars are offset to accommodate the lap splice, the offset also should be outside the length l_0 (Figure 6.23).

Lap splices of the longitudinal reinforcement create a very congested area of the column as the number of vertical bars is doubled and the hoops must be tightly spaced. Splicing the vertical bars at every other floor as shown in Figure 6.24 will eliminate some of the congestion. Mechanical splices also may help reduce congestion.

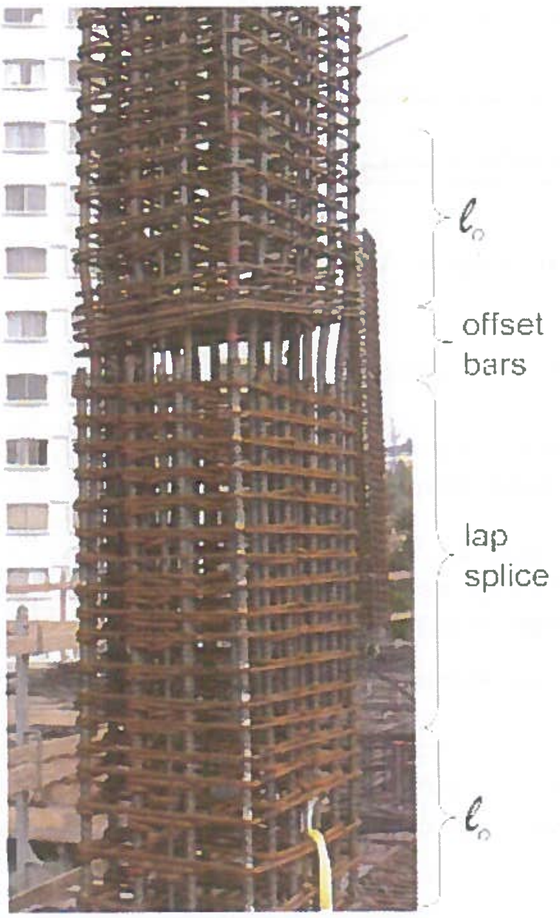

Figure 6.23 Column Cage Lap Splices are Not Permitted to Extend Outside the Middle Half of the Column Length and Should Not Extend into the Length L_0 at the Column End.

Figure 6.24 Longitudinal Column Reinforcement Spliced Every other Floor to Reduce Congestion.

6.4 Arrangement of Reinforcing Steel in Structural Wall

Special reinforced concrete structural walls are walls that have been proportioned and detailed to meet special code requirements for resisting combinations of shear, moment, and axial force that result as a building sways through multiple displacement cycles during strong earthquake ground shaking. Special proportioning and detailing requirements result in a wall capable of resisting strong earthquake shaking without unacceptable loss of stiffness or strength.

6.4.1 Wall Reinforcement

Figure 6.25 illustrates typical reinforcement for a special structural wall of rectangular cross section. As a minimum, a special structural wall must have distributed web reinforcement in both horizontal and vertical directions. In many cases, a special structural wall also will have vertical reinforcement concentrated at the wall boundaries to provide additional resistance to moment and axial force. Typically, longitudinal reinforcement is enclosed in transverse reinforcement to confine the concrete and restrain longitudinal bar buckling

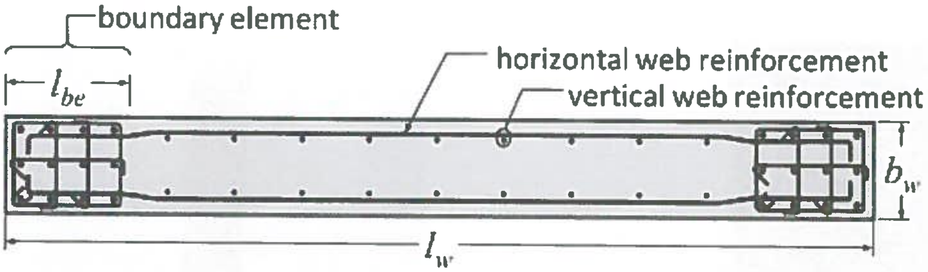

Figure 6.25 Typical Reinforcement for Rectangular Wall [*After Ref.* 6.11]

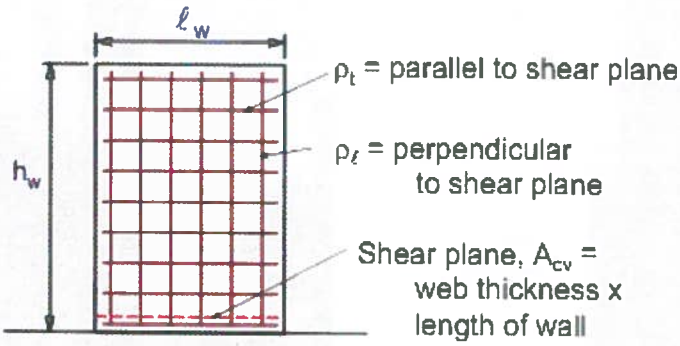

Figure 6.26 ACI 318 Notation for Dimensions and Reinforcing Ratios in Shear Walls.

The distributed web reinforcement ratios, ρ_l *for vertical reinforcement and* ρ_t *for horizontal reinforcement, must be at least 0.0025,* except that ρ_l and ρ_t are permitted to be reduced if $V_u \leq A_{cv}\lambda\sqrt{f'_c}$ .If V_u does not exceed $A_{cv}\ \lambda\sqrt{f'_c}$, minimum ratio of vertical reinforcing area to gross concrete, shall be. (ACI-318 § 21.9.2 & § 1 4.3)

a) 0.0012 for deformed bars not larger than diameter 16 mm (#5) with f_y not less than 415 N/mm^2 (60,000 psi), or
b) 0.0015 for other deformed bars, or
c) 0.0012 for welded wire reinforcement not larger than W31, or D31.

Reinforcement spacing *each way is not to exceed 450 mm (18 inches). At least two curtains (layers) of reinforcement are required if* $V_u \geq 2A_{cv}\lambda\sqrt{f'_c}$

$V_u \geq 0.166A_{cv}\lambda\sqrt{f'_c}$ in SI., reinforcement ρ_t also is to be designed for wall shear forces, as described in Section 6.4. Finally, if $h_w/l_w \leq 2.0$, ρ_l is not to be less than the provided ρ_t. ACI 318 has no requirements about whether vertical or horizontal distributed reinforcement should be in the outer layer, although lap splices of vertical reinforcement will perform better if horizontal bars are placed outside the vertical bars as shown in Figure 6.25.

Here reduced vertical and horizontal reinforcement should be described as per ACI 318-11: 14.3.2 (BNBC 2015 Part 6§ 6.6.3)

6.4.1.1 Boundary Element

A boundary element is a portion along a structural wall edge or opening that is strengthened by longitudinal and transverse reinforcement. Where combined seismic and gravity loading results in high compressive demands on the edge, ACI 318 requires a special boundary element. These have closely spaced transverse reinforcement enclosing the vertical boundary bars to increase compressive strain capacity of core concrete and to restrain longitudinal bar buckling.

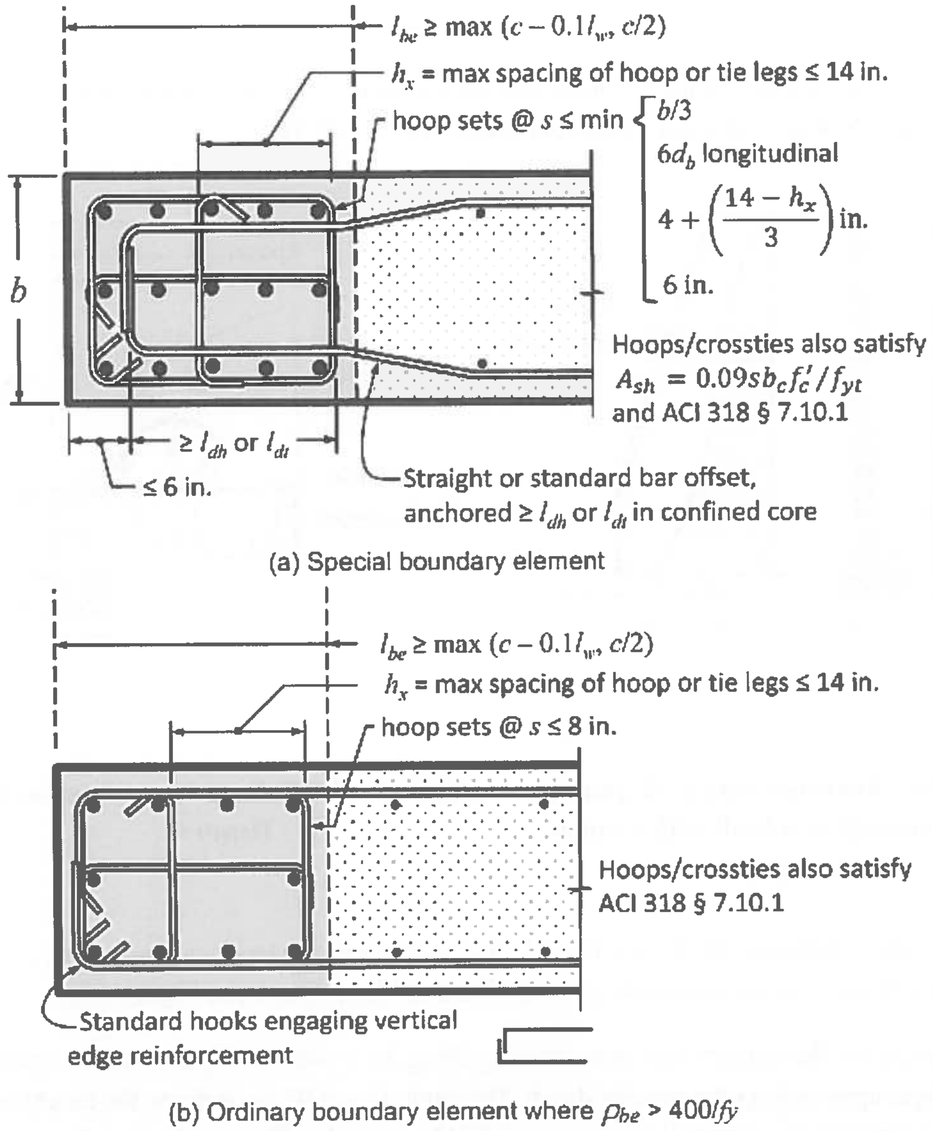

Figure 6.27 Special and Ordinary Boundary Element [*After Ref.* 6.11]

Where compressive demands are lower, special boundary elements are not required, but boundary element transverse reinforcement still is required if the longitudinal reinforcement ratio at the wall boundary, $A_{s,be}/A_{g,be}$ is greater than $400/f_y$. For clarity, these latter elements are referred to as ordinary boundary elements (a term not used in ACI 318). Figure 6.27 shows examples of special and ordinary boundary elements.

ACI 318 provides two methods for determining whether special boundary elements are required. The preferred method (ACI 318, §21.9.6.2), Method I, applies to walls or wall segments that are effectively continuous from base of structure to top of wall or segment and designed to have a single critical section for flexure and axial force, as shown in Figure 6.28. Some discontinuity over wall height is acceptable provided the wall is proportioned so that the critical section occurs where intended. To use the method, the seismic force-resisting system is first sized and then analyzed to determine the top-level design displacement δ_u and corresponding maximum value of wall axial force P_u. The flexural compression depth c corresponding to nominal moment strength $M_{n,CS}$ under axial force P_u is then calculated (Figure 6.29). If

$$c \geq \frac{l_w}{600(\delta_u / h_w)} \qquad (6.6)$$

Where h_w refers to total wall height from critical section to top of wall, then special boundary elements are required. δ_u/h_w shall not be taken less than 0.007 (ACI-318: 21.9.6.3)

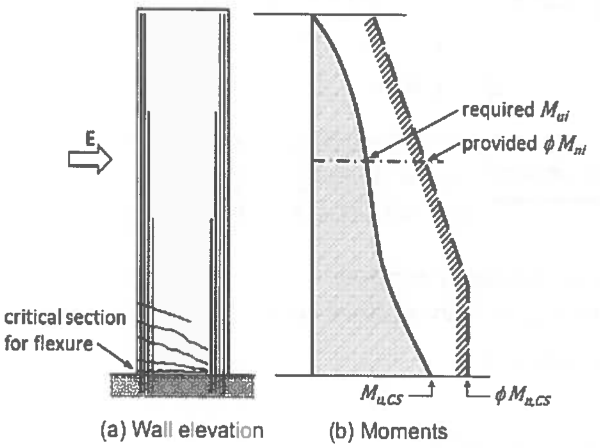

Figure 6.28 Provided Versus Required Flexural Strength in a Wall with a Single Critical Section

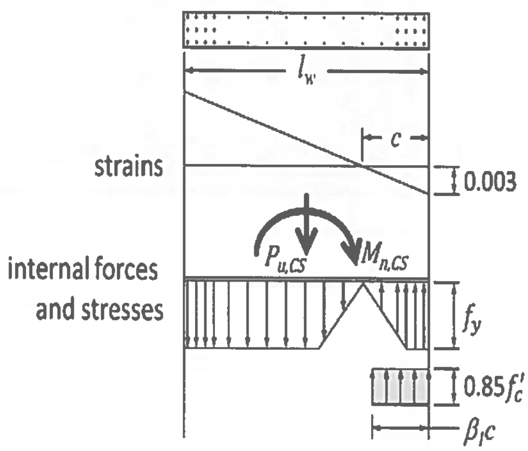

Figure 6.29 Calculation of Neutral Axis Depth C.

Where special boundary elements are required by Method I, they must extend vertically above and below the critical section a distance not less than the greater of l_w and $M_{u,CS}/4V_{u,CS}$ (see Figure 6.30).

The limit l_w is based on the expectation that cover spalling in a well-confined section typically will spread along a height approaching the section depth. The limit $M_{u,CS}/4V_{u,CS}$ defines the height above the critical section at which the moment will decrease to $0.75M_{u,CS}$, a value likely to be less than the spalling moment, assuming a straight-line moment diagram. Where the critical section occurs at or near the connection with a footing, foundation mat, pile cap, or other support, different requirements apply to the vertical extension of the special boundary element.

The second method for determining if special boundary elements are required, herein refers to as Method II, is based on nominal compressive stress (ACI 318 § 21.9.6.3). First, the seismic force-resisting system is sized and analyzed to determine axial forces and moments under design load combinations. Using a gross section model of the wall cross section, nominal stress at wall edges is calculated from $\sigma = P_u/A_g + M_{ux}/S_{gx} + M_{uy}/S_{gy}$.

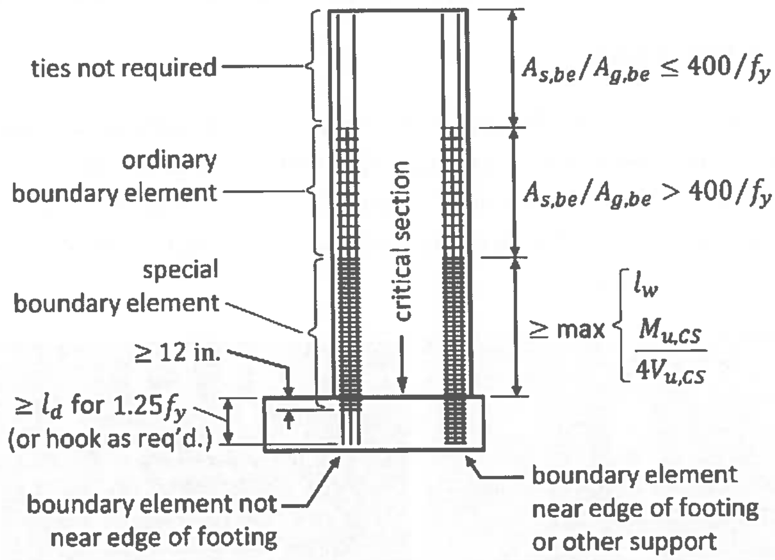

Figure 6.30 Boundary Element Extensions for Walls Designed by Method I, For Critical Section at Foundation Interface. For Ordinary and Special Boundary Element Details, See Figure 6.34 [*After Ref.* 6.11]

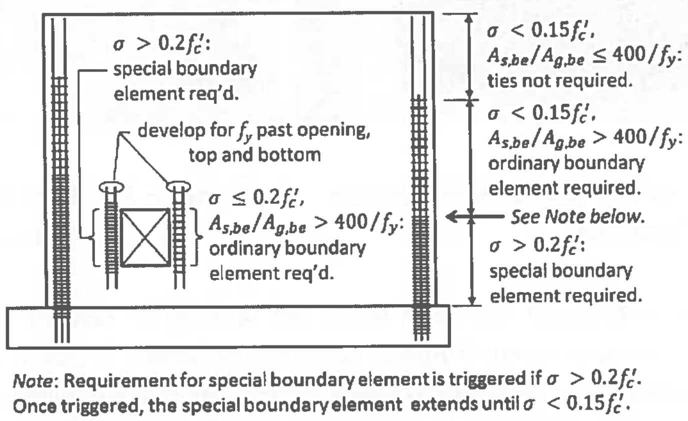

Figure 6.31 Boundary Element Requirements for Walls Designed By Method II. [*After Ref.* 6.11]

Special boundary elements are required at an edge *if nominal stress exceeds* $0.2f'_c$. If a special boundary element is required, *it must be continued vertically (upward and downward) until compressive stress drops below* $0.15f'_c$ (See Figure 6.31). Although Method II can be used for any wall, the preferred use is for irregular or discontinuous walls for which Method I does not apply.

6.4.2 Detailing & Constructability Issue

A special reinforced concrete structural wall relies on carefully detailed and properly placed reinforcement to ensure that it can maintain strength through multiple cycles of deformation beyond yield. Although a structural wall is considered a singular element, reinforcement modules within the wall are typically pre-tied and hoisted into the place as separate pieces (Figure 6.32). The pre-tied modules are spliced to create a fully interlocked reinforcement cage prior to closing the forms and casting the wall.

6.4.2.1 Boundary Element Confinement

The confinement variables typically at the designer's discretion are the confinement bar size, and the horizontal and vertical spacing of confinement hoop legs and crossties. Large diameter confining bars are desirable to reduce congestion, but bars *larger than 16 mm (# 5) are impractical* because of required space for bar bends and hook tails. For higher strength steel, there also can be a limit to what bar size is bendable with locally available equipment.

Figure 6.32 Pre-Tied Modules (Some Modules Encircled).

Figure 6.33 Boundary Confinement for Planar Wall

Horizontal spacing of confinement legs, and hence the spacing of vertical reinforcement within the boundary element, *will typically be much tighter (100 mm (4 inches) to 200 mm(8 inches)* than desired for the remainder of the wall. It is common to select vertical bar spacing within a boundary element that is a divisor of the vertical bar spacing in the unconfined portion of the wall. For example, if 300 mm (12-inch) spacing of vertical reinforcement is considered practical for the unconfined wall, the spacing of vertical bars within the boundary element should be 150 mm (6 inches) or 100 mm (4 inches). This is beneficial because as vertical boundary bars drop off at higher elevations, the remaining bars align with and can be spliced to the 12-inch grid.

Boundary element reinforcement very much resembles a *ductile column* within the structural wall. A representative boundary element at the end of a planar wall is shown in Figure 6.33. Note that *each crosstie has a 90° and a 135° hook, and these must be alternated end for end along both the length and the height.*

Some flanged walls require confinement throughout the flange, in which case confinement *must extend at least 300 mm (12 inches) into the wall web* (Figure 6.34). For very long confined boundary regions, one approach is to provide closely spaced confinement reinforcement in both directions at wall ends, with only closely spaced through-wall crossties along the middle extent of the wall. In this case, more widely spaced horizontal shear reinforcement in the web satisfying ACI 318 Eqn.21-5 can adequately confine the wall lengthwise.

Figure 6.34 Boundary Confinement for Wall Flange.

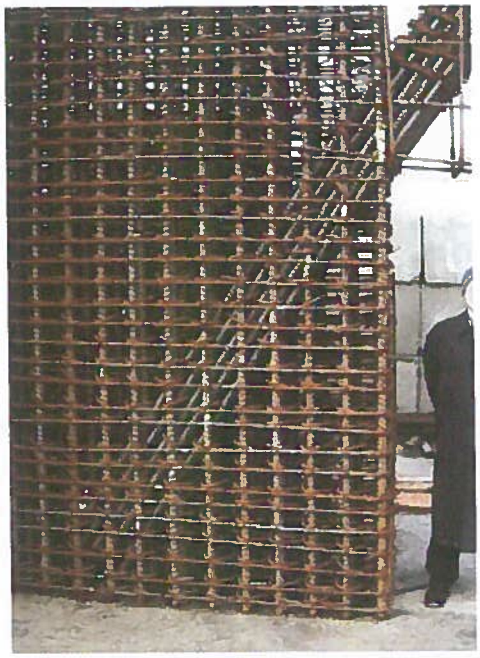

Figure 6.35 Anchorage of Diagonal Reinforcement in Heavily Reinforced Boundary Element.

Structural wall longitudinal reinforcement must extend into supporting elements and be fully developed for f_y or $1.25f_y$ in tension. See Figure 6.34 for details. Where boundary elements are provided, equivalent horizontal confinement must be extended into the support.

For structural walls on shallow foundations, this confinement *must be extended* 300 *mm (*12 *inches)* into the footing or mat. For structural walls supported by all other elements, or where the edge of the boundary element is within one-half the footing depth from an edge of the footing, the confinement must extend into the support distance equal to the development length of the largest vertical bar in the boundary. The critical subset of this category is boundary elements landing flush with the edge of a foundation or significant foundation step. This commonly occurs for structural walls that enclose elevator cores. The elevator pit dimensions commonly require a significant depression on one side of the structural wall. For this condition, it is recommended that the base of the depression be considered as the base of the structural wall. Vertical bars are therefore developed below the depression, and confinement is continued through the depth of the depression.

6.4.2.2 Bar Compatibility

The critical location for detailed consideration of bar placement is the interface of wall ends with coupling beams. The main coupling beam reinforcement must extend into the wall end a length sufficient to fully develop the bar capacity (*see Section 6.5*). Bar compatibility becomes especially challenging where diagonal bars must extend into a heavily confined section. Full-scale pre-construction mockups can help identify solutions for particularly challenging designs (Figure 6.35).

To be reliably anchored, *coupling beam longitudinal reinforcement must be placed inside the wall vertical reinforcement*. For conventionally reinforced beams, this results in side cover over beam longitudinal reinforcement around 3 inches (Figure 6.38, Section A-A). Transverse reinforcement must be detailed for this increased cover so that the corner longitudinal bars are firmly placed in stirrup and crosstie bends. This decreased available width must also be considered when verifying clear horizontal spacing between longitudinal bars, a necessary measure to facilitate concrete placement and consolidation.

6.4.2.3 Bar Splices

According to ACI 318 § 21.9.2.3, reinforcement in structural walls is required to be developed or spliced for f_y in tension in accordance with ACI 318 Chapter 12, with some noted exceptions. *At locations where yielding of longitudinal reinforcement is likely to occur as a result of lateral displacements, development and lap splice lengths of longitudinal reinforcement are required to be 1.25 times values calculated for f_y in tension.* Lap splices, mechanical splices, and welded splices are permitted, with laps splices being the most common. As for all elements of concrete construction, reinforcing bars larger than #11 may not be lap spliced in structural walls. Mechanical and welded splices are required to satisfy ACI 318 § 21.1.6 and 21.1.7.

The first splice of vertical reinforcement typically occurs immediately above the foundation, where wall longitudinal reinforcement laps with dowel bars. These dowels provide the critical mechanism of transferring tension and shear forces from the structural wall to the foundation. All vertical reinforcement must be extended into the foundation a depth sufficient to be fully developed for tension. *For constructability purposes, it is recommended that dowels with 90° hooks extend to the bottom of the foundation with free end of the hook oriented towards center of the column (ACI-318:21.12.2) where they can be tied firmly the foundation bottom reinforcement.*

For structural walls with two curtains of reinforcement, it is preferred for the vertical reinforcement to be inside the horizontal reinforcement. This arrangement improves splice strength and buckling restraint for the verticals.

Horizontal reinforcement is always treated as "top-cast" reinforcement, requiring ψ_t = 1.3 for all development and lap splice length calculations. Splice locations might not be finalized until the contractor has determined the breakdown of pre-tied segments and the overall erection sequence including formwork operability. For structural walls with re-tied segments, the horizontal reinforcement has the additional function of tying the pieces together in the final arrangement (Figure 6.32)

6.5 COUPLING BEAMS

Coupling beams often have relatively low aspect ratios and high deformation demands, requiring special details to achieve ductile performance. Coupling beam-wall connections require additional attention to avoid conflicts in reinforcing bar placement.

ACI 318, § 21.9.7 classifies coupling beams into three categories based on aspect ratio l_n/h and shear demand. As a practical matter, a fourth category for very deep beams is considered in Ref. 4.11. Figure 6.36 illustrates the design options for these categories.

i. **Coupling beams with $l_n/h \geq 4$** must satisfy proportioning and detailing requirements specified for beams of special moment frames, except certain dimensional limits are exempted. Such beams are considered too shallow for efficient use of diagonally placed reinforcement as allowed for deeper beams. Instead, flexural reinforcement is placed horizontally at top and bottom of the beam.

ii. **Coupling beams with $l_n/h < 2$ and $V_u > 4\lambda\sqrt{f'_c}A_{cw}$** are required to be reinforced with two intersecting groups of diagonally placed bars symmetrical about the mid span, unless it can be shown that loss of stiffness and strength of the coupling beams will not impair the vertical load-carrying ability of the structure, post-earthquake egress from the structure, or the integrity of nonstructural components and their connections to the structure. Implicit in the exception is the requirement for the engineer to demonstrate that the seismic force-resisting system satisfies code strength and drift requirements in the absence of the excepted coupling beams.

iii. **Other coupling beams** not falling within the limits of the preceding two bullets are permitted to be reinforced as either conventionally reinforced special moment frame beams or diagonally reinforced beams. In Figure 6.36, beams falling to the right of the dashed line likely can be designed efficiently as special moment frame beams, whereas those to the left probably are better designed with diagonal reinforcement.

iv. **Very low aspect ratio beams** are better designed using the strut-and-tie model of ACI 318 Appendix A. Design of these beams is not covered in this Manual. The darkly shaded area of Figure 6.36 defines the upper limit on beam design shear stress. The lightly shaded area indicates designs that are permitted by ACI 318 but that may have constructability problems because of reinforcement congestion.

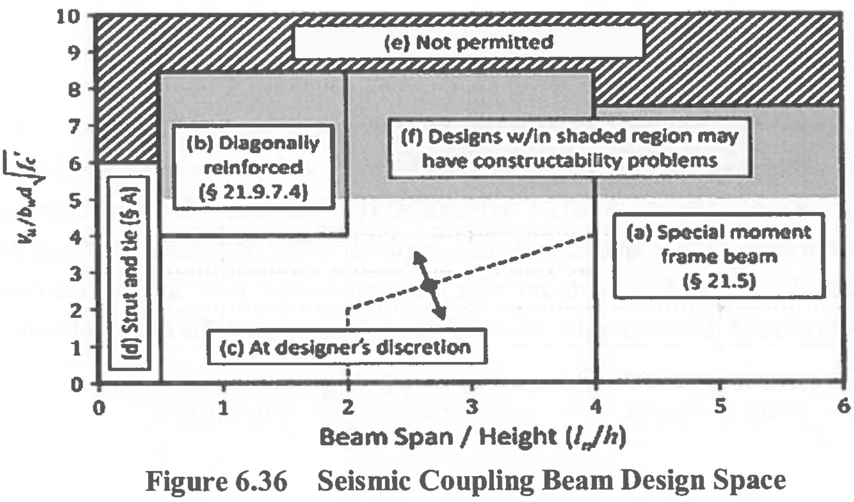

Figure 6.36 Seismic Coupling Beam Design Space

6.5.1 Coupling Beam Detailing

Beams designed as special moment frame beams (ACI 318, §21.5) must have flexural reinforcement placed horizontally at top and bottom of the beam and hoop reinforcement that confines the end regions. Figure 6.37 illustrates typical details.

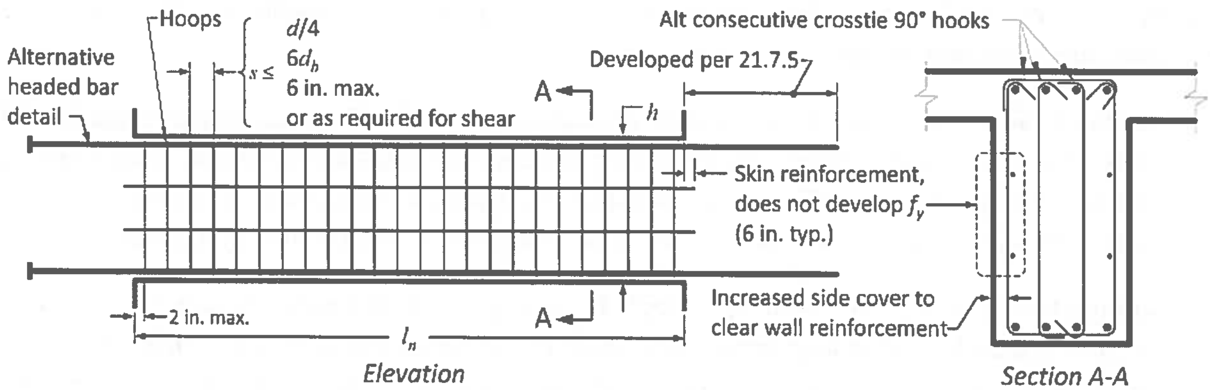

Figure 6.37 Details for Conventionally Reinforced Coupling Beams.

Because l_n/h are relatively small, longitudinal bars cannot be lapped and it may be easier to use closed hoops over the entire beam span rather than only $2h$ at each end. Skin reinforcement, if any, typically is terminated after short extension into the wall (~150 mm or 6 inches); alternatively, it can be developed into the wall in which case it contributes to beam flexural strength.

Figure 6.38 shows typical details for a coupling beam reinforced with two intersecting groups of diagonally placed bars symmetrical about the mid-span. Each group of diagonal bars consists of a minimum of four bars provided in two or more layers. *The diagonal bars are required to extend straight into the wall a distance at least* 1.25 *times the development length for f_y in tension.* A challenge is avoiding interference between the diagonal bars and the boundary element transverse and longitudinal reinforcement. If an adjacent wall opening or edge (for example, at the top of the wall) requires the diagonal bar extension to be bent, additional reinforcement is required to resist the unbalanced force resulting from the change in reinforcement direction, similar to the requirement for offset bars in columns (ACI 318 § 7.8.1.3). This detail should be avoided where practicable. *The minimum wall thickness to accommodate both wall and coupling beam reinforcement is around 350 mm (14 inches), although 375 mm to 400 mm (16 to 18 inches) is more practical.*

ACI 318 § 21.9.7.4 prescribes requirements for two reinforcement options. The first option is to confine individual diagonals using hoops and crossties such that corner and alternate diagonal bars are restrained in a hoop or crosstie corner (Figure 6.38 (a)). Confinement reinforcement along the entire diagonal length must satisfy the volumetric ratio requirements that apply at ends of special moment frame columns, assuming each diagonal as an isolated column with minimum cover over the diagonal cage. Maximum permitted hoop spacing along the diagonal is the smaller of s_o and $6\ d_b$ of the diagonal bars, where

$S_o = 100 + \left(\dfrac{350 - h_x}{3}\right)$ (mm) In SI unit $\qquad$ $S_o = 4 + \left(\dfrac{14 - h_x}{3}\right)$ (in.) In FPS

Confinement reinforcement can be difficult to place along the free lengths of the diagonals and even more difficult where the diagonals intersect or enter the wall boundaries. See Section 6.4.2 for additional discussion.

The second option is intended to ease construction difficulties commonly encountered with the first option. By this option, hoops and crossties confine the entire beam cross section (Figure 6.38 (b)). Confinement reinforcement along the entire beam length must satisfy the volumetric ratio requirements that apply at ends of special moment frame columns, with maximum spacing along the beam span not exceeding 150 mm *(6 inches) or* $6d_b$ of the diagonal bars, and with spacing of crossties or legs of hoops around the beam cross section *not exceeding 8 inches*. Although the total amount of confinement reinforcement may be greater with this second option, the increased material costs are often more than offset by reduced labor costs.

Regardless of the option selected for the diagonally reinforced beam, longitudinal and transverse reinforcement is required around the beam section (Figure 6.38). The longitudinal reinforcement, typically 12 mm (#4) or 16 mm (#5) bars, should extend only a short distance into the wall boundary so that it will not develop significant tensile stress due to beam flexure. Transverse reinforcement varies depending on the option selected for confinement reinforcement. See ACI 318 § 21.9.7.4.

Interested readers are referred to *Ref.* 4.11 for further details regarding design issues of coupling beams.

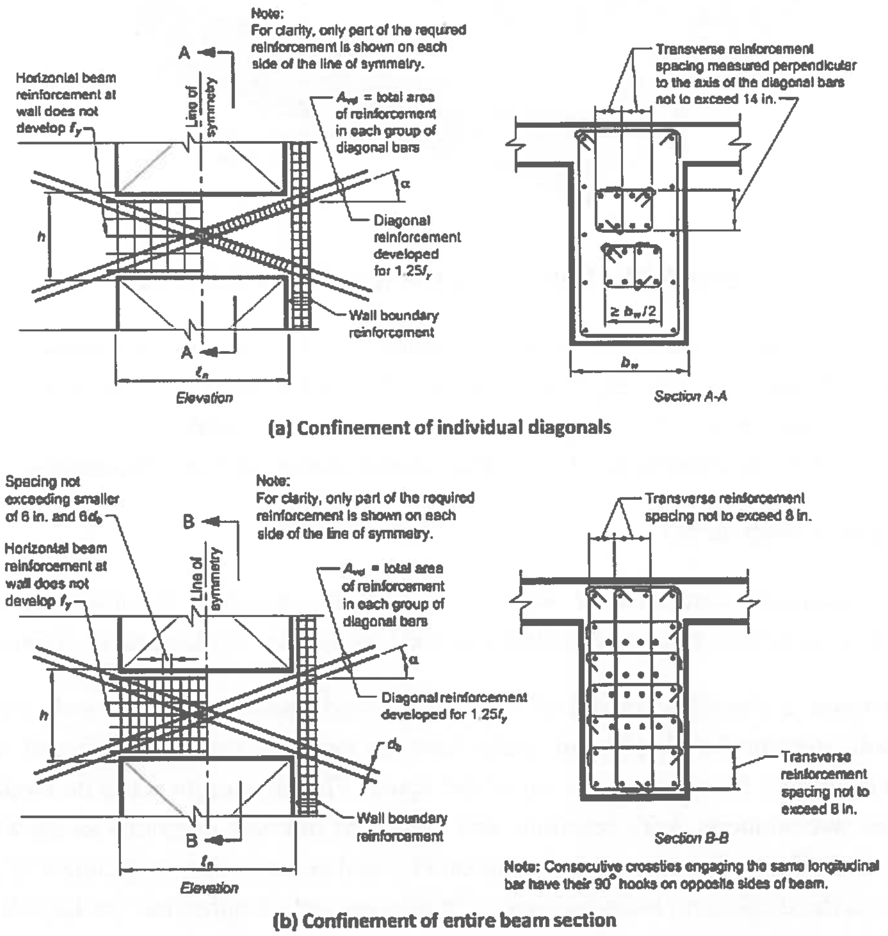

Figure 6.38 Alternative Details for Diagonally Reinforced Coupling Beams

6.6 ARRANGEMENT OF REINFORCING STEEL IN STRUCTURAL DIAPHRAGM

In building construction, diaphragms are structural elements, such as floor or roof slabs, that perform some or all of the following functions (see Figure 6.39):

i. Provide support for building elements such as walls, partitions, and cladding, and resist horizontal forces but not act as part of the vertical seismic-force-resisting system.
ii. Transfer lateral forces to the vertical seismic-force-resisting system.
iii. Interconnect various components of the seismic-force-resisting system with appropriate strength, stiffness, and toughness to permit deformation and rotation of the building as a unit.

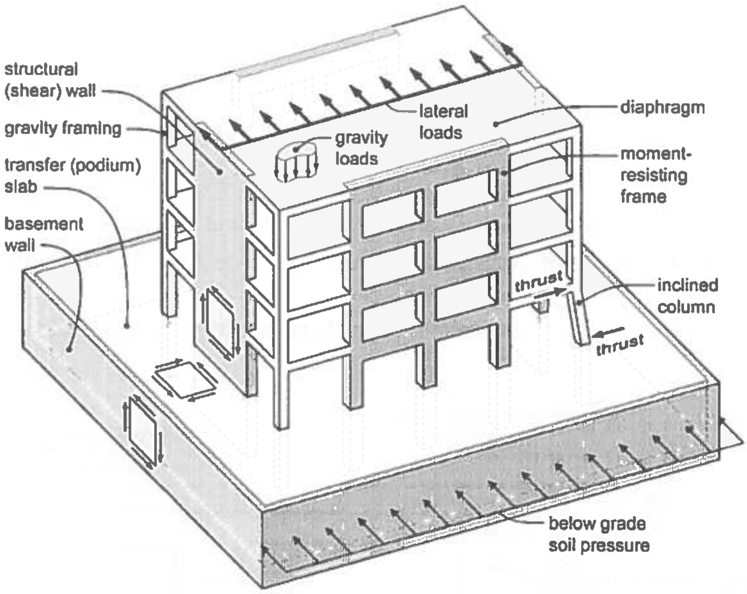

Figure 6.39 Roles of Diaphragms [*After Ref. 6.12*]

By comparison with requirements for vertical elements of the seismic-force-resisting system, code provisions for diaphragms are relatively brief. Consequently, many aspects of diaphragm design are left open to interpretation and engineering judgment. Ref. provides detail guidance to assist in understanding and application of code requirements for the design of cast-in-place concrete diaphragms.

6.6.1 Diaphragms Components

Diaphragms are commonly composed of various components, including the *diaphragm slab, chords, collectors* (also known as drag struts or distributors), and *connections to the vertical elements.*

Figure 6.39 illustrates a simplified model of how a diaphragm resists in-plane loads and identifies its parts. Here, a solid rectangular diaphragm spans between two end walls, with lateral inertial loading indicated schematically by the arrow at the top of the figure. The diaphragm could be modeled as a beam spanning between two supports, with reactions and shear and moment diagrams as shown (Figure 6.40 (b)). Bending moment M_u can be resisted by a tension (T_u) and compression (C_u) couple (Figure 6.40 (c)). The components at the diaphragm boundary acting in tension and compression are known as *the tension chord and the compression chord*, respectively.

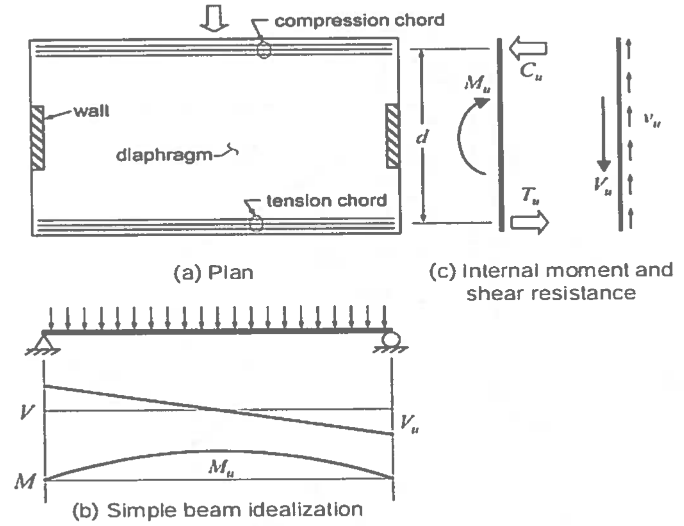

Figure 6.40 Tension and Compression Chords [*After Ref.* 6.12]

If the diaphragm moment is resisted by tension and compression chords at the boundaries of the diaphragm as shown in Figure 6.40 (a), then equilibrium requires that the diaphragm shear be distributed uniformly along the depth of the diaphragm as shown in Figure 6.40 (c).

Tension and compression elements called *collectors* are required to "collect" this shear and transmit it to the walls. A *collector* can transmit all its forces into the ends of the walls as shown on the right side of Figure 6.41 (a), or if the forces and resulting congestion are beyond practical limits, the collector can be spread into the adjacent slab as shown on the left side of Figure 6.41 (a).

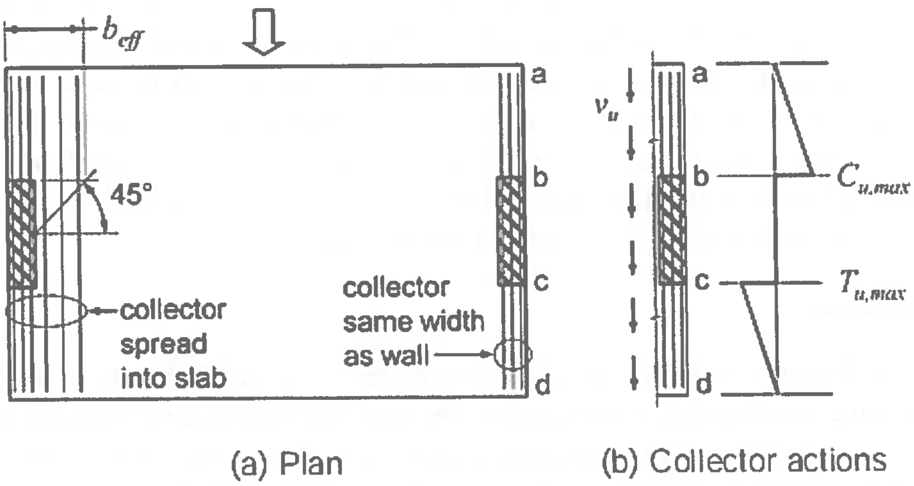

Figure 6.41 Collectors [*After Ref. 6.12*]

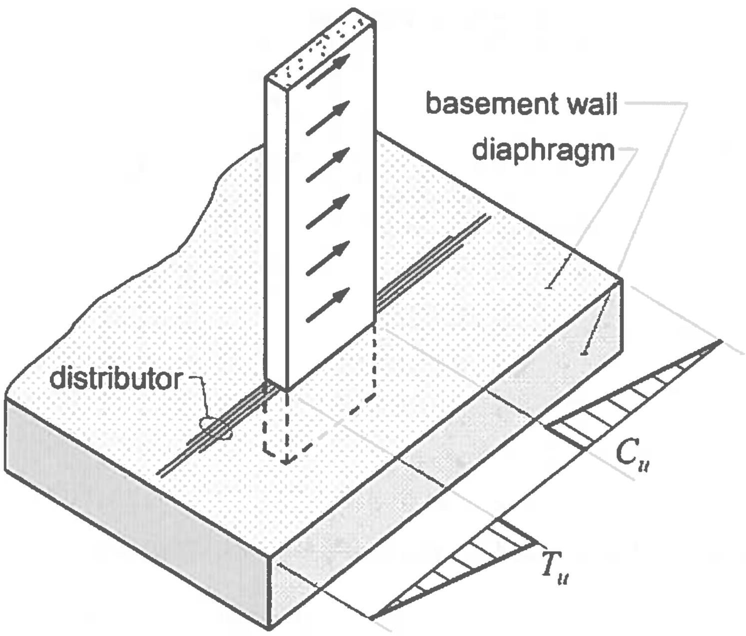

Figure 6.42 Distributor [*After Ref.* 6.12]

Figure 6.41 (b) illustrates how the tension and compression forces in the *collector* are determined for the case where the width of the collector is the same as that of the wall. Starting at a free end, the tension or compression force increases linearly as shear is transferred into the collector.

Diaphragms also transfer load among vertical elements of the seismic force-resisting system. A common example is where a wall intersects a podium slab in a building with subterranean levels. In this case, shear is transferred from the wall into the diaphragm and from there to other elements such as basement walls. This element transferring the force from the wall to the diaphragm is a *collector,* but sometimes is referred to as a *distributor*. See Figure 6.42. To differentiate between this two, a collector is an element that takes distributed load from the diaphragm and delivers it to a vertical element, whereas a distributor takes force from a vertical element and distributes it into the diaphragm.

6.6.2 Reinforcement

The minimum reinforcement ratio for structural diaphragms is the same as that required by ACI 318 §7.12 for temperature and shrinkage reinforcement. The maximum reinforcement spacing of 450mm (18 in.) is intended to control the width of inclined cracks. According to ACI 318 § 21.11.7.5, collector elements must have transverse reinforcement as specified in § 21.6.4.4 through § 21.6.4.6 when the compressive stress at any section exceeds $0.2 f_c'$.
Note that compressive stress is calculated for the factored forces using a linearly elastic model and gross section properties. The transverse reinforcement is no longer required where the compressive stress is less than $0.15 f_c'$.

6.6.2.1 Flexural reinforcement

Diaphragms typically are designed using classical flexural theory assuming plane sections remain plane even though the proportions may be more like those of a deep beam. Traditionally, flexural demands are resisted by tension and compression chords located close to opposite outer edges of the diaphragm (see Figure 6.40). The chord compression, C_u, and tension, T_u, in the chords are computed as

$$C_u = T_u = \frac{M_u}{d} \tag{6.7}$$

Using this approach, the in-plane shear stress is uniform across the depth of the diaphragm, with value V_u/td.

ACI 318 § 21.11.8 permits the use of distributed reinforcement to resist diaphragm moment. If this is done, the moment strength is calculated using the traditional approach in which strain varies linearly through the depth, with stresses appropriately corresponding to strains. With distributed reinforcement, development of moment strength may require large tensile strains and potentially unacceptable cracking near the tension edge. For this reason, ACI 318 Commentary Section R21.11.8 does not recommend eliminating all of the boundary reinforcement. A good rule of thumb is that the required flexural tension reinforcement should be located within the outermost quarter of the diaphragm depth.

If distributed reinforcement away from a diaphragm edge is used to resist flexure, the unit shear stress is not constant through the diaphragm depth but instead varies gradually through the depth and has a peak value exceeding V_u/td. The diaphragm should be designed for higher shear stresses, where they occur.

Typically the chord reinforcement is placed within the middle third of the slab or beam thickness, so as to minimize interference with slab or beam longitudinal reinforcement and reduce contributions to slab and beam flexural strength. Where chord reinforcement is positioned within a beam, the chord and the beam typically are oriented to resist orthogonal effects, such that the same reinforcing bars can resist flexure for loading in one direction and chord tension for loading in the orthogonal direction.

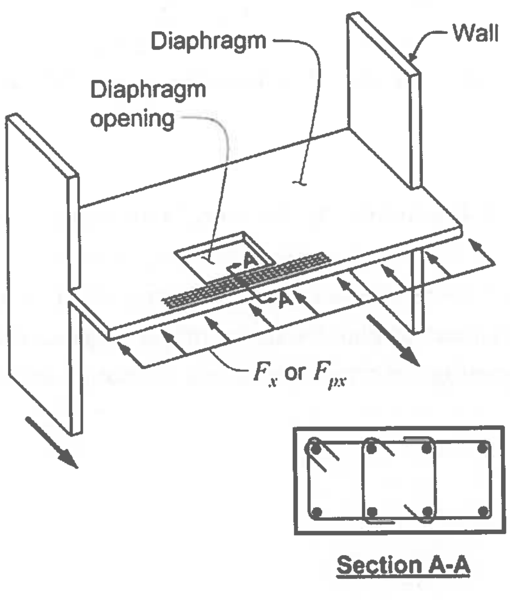

Figure 6.43 Confinement Reinforcement in Axial Struts Around Openings.

Prior to the 2008 edition of ACI 318, *compression chords were required to have confinement reinforcement if the compressive stress exceeded $0.2f'_c$.* This requirement was eliminated in the 2008 edition except for diaphragm elements subjected primarily to axial compressive forces (struts) and used to transfer diaphragm shear or flexural forces around openings or other discontinuities (Figure 6.43). If the calculated compressive stress on a strut exceeds 0.2f'c, confinement reinforcement satisfying requirements for special boundary elements of special structural walls is required (ACI 318 § 21.11.3). The required confinement reinforcement, including hoops with required seismic hook dimensions, can be difficult to fit within typical slab depths, and may require increased depth. *Where required, confinement reinforcement should be continued into the slab beyond the strut the larger of the tension development length of the longitudinal reinforcement and 12 inches.*

6.6.2.2 Shear Reinforcement

Shear reinforcement can be placed anywhere within the slab thickness within required cover limits. Some designers specify a continuous mat of bottom reinforcement that satisfies both the diaphragm shear and slab moment requirements. Where this is done, the total reinforcement area is the sum of areas required for moment and shear.

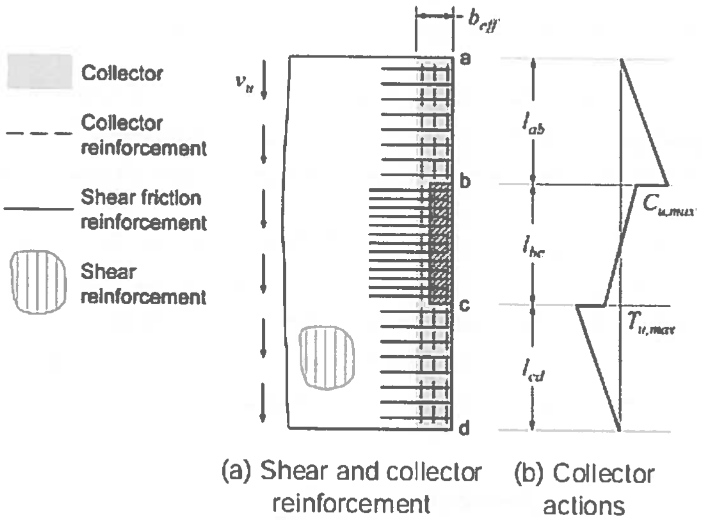

Figure 6.44 Shear Reinforcement, Collector Reinforcement, and Shear-Friction Reinforcement.

6.6.2.3 Special Cases

(i) **Openings Adjacent to Vertical Elements**: Architectural requirements sometimes dictate openings adjacent to walls that are part of the seismic force-resisting system. This can create force transfer challenges, especially at podium levels where large forces may need to be transferred. The preferred approach is to work with the architect to plan locations of openings so that they do not interfere with major force transfers. Where openings in critical locations cannot be avoided, workable solutions can sometimes be designed.

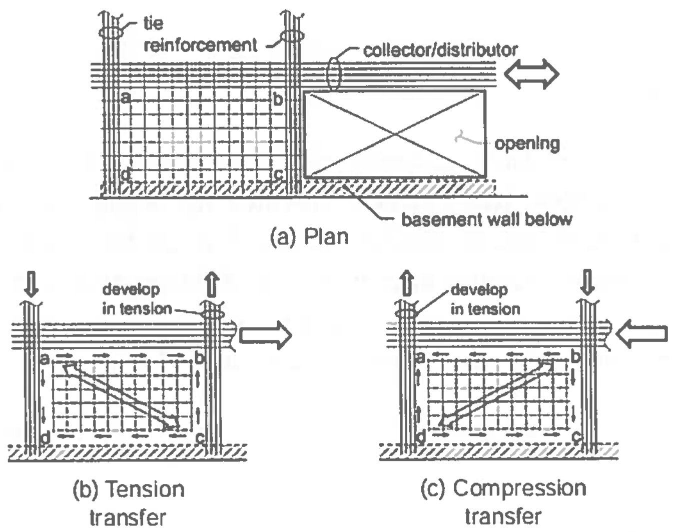

Figure 6.45 Reinforcement to Transfer Collector /Distributor Force Around an Opening

Figure 6.45 illustrates how reinforcement might be detailed in the diaphragm segment to the left of the opening. For the distributor in tension, uniform shear flow can be assumed between the distributor and diaphragm segment along *ab*, resulting in shear stresses acting on the diaphragm segment as shown in Figure 6.45 (b). Moment equilibrium of the segment requires equal shear stresses along faces *ab, bc, ad,* and *cd*. Shear reinforcement satisfying ACI 318 Equre 21-10 is required uniformly in both directions to resist this applied shear. The basement wall also must be reinforced locally along *cd* for the shear applied along that length. Edge *bc* of the diaphragm segment requires a tension tie to pick up the shear stresses along that edge; that tension tie needs to be developed into the adjacent diaphragm. A compression reaction near point *a* completes the equilibrium requirements. Similar but opposite action occurs for the collector in compression (Figure 6.45 (c)).

(ii) **Re-entrant Corners** At re-entrant corners of diaphragms such as shown in Figure 6.46, either tension chord ac can extend across the full width of the diaphragm, or the chord can follow the profile of the diaphragm around the re-entrant corner. In the latter case, the tension chord reinforcement ac needs to be developed into the diaphragm. The developed length bc transfers force to the diaphragm, creating shear in panel bcd. Chord reinforcement bd, of the same area as reinforcement bc, needs to be provided and developed into the diaphragm. Finally, tension chord ed can be designed based on the corresponding moment and effective depth. Because there is considerable moment at d, chord reinforcement de should be hooked at d and lapped with adjacent reinforcement bd.

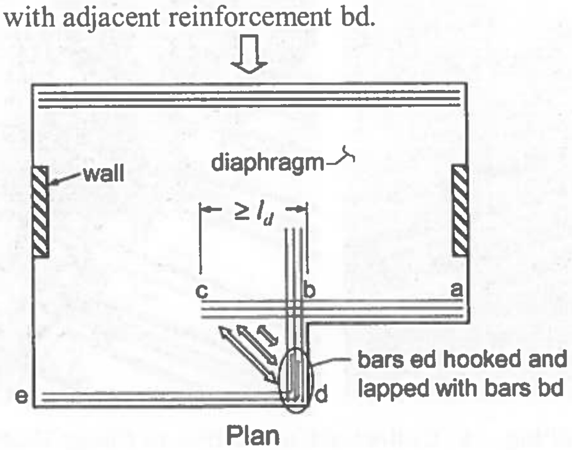

Figure 6.46 Reinforcement Associated with Re-Entrant Corners

6.6.3 Detailing & Constructability Issues

6.6.3.1 Diaphragm Reinforcement

Many concrete slabs are designed to have a continuous bottom mat of uniformly distributed reinforcement. For this reason, transverse reinforcement provided for diaphragm shear resistance is commonly incorporated into the bottom mat. In heavily reinforced diaphragms that are thick slabs, a continuous top and bottom mat of reinforcement is often provided. Designers should specify the required lap splice and development length of the reinforcement in the construction documents, as diaphragm reinforcement splice and development requirements may exceed what is otherwise required.

The text that follows is based on construction experiences, both good and bad, and is drawn from recommendation of *Ref. 4.12*.

6.6.3.2 Collector and Chord Detailing

Collector and chord reinforcement is often located in the mid-depth of the slab. In structures assigned to Seismic Design Categories D, E, and F, *ACI 318 requires center-to-center spacing at least* $3d_b$, *but not less than 1.5 in., and concrete clear cover at least* $2.5d_b$, *but not less than 50 mm (2 in.)*Otherwise, transverse reinforcement is required.

Connections of collector reinforcement to vertical elements of the seismic force-resisting system are often congested regions. In many cases, numerous large diameter bars are required to be developed into confined boundary zones of shear walls as shown in Figure 6.47 (a). Designers should study these connections in detail to ensure adequate space exists. In many cases, increased slab thickness or beams are required to accommodate reinforcement detailing at the connections.

Figure 6.47 (b) shows where a beam was created to accommodate the collector reinforcement. Designers should also consider the slab depth provided where large collectors intersect. Multiple layers of large diameter reinforcing bars can result in excessive congestion. Similarly, designers should be aware of locations where collectors intersect concrete beam longitudinal reinforcement.

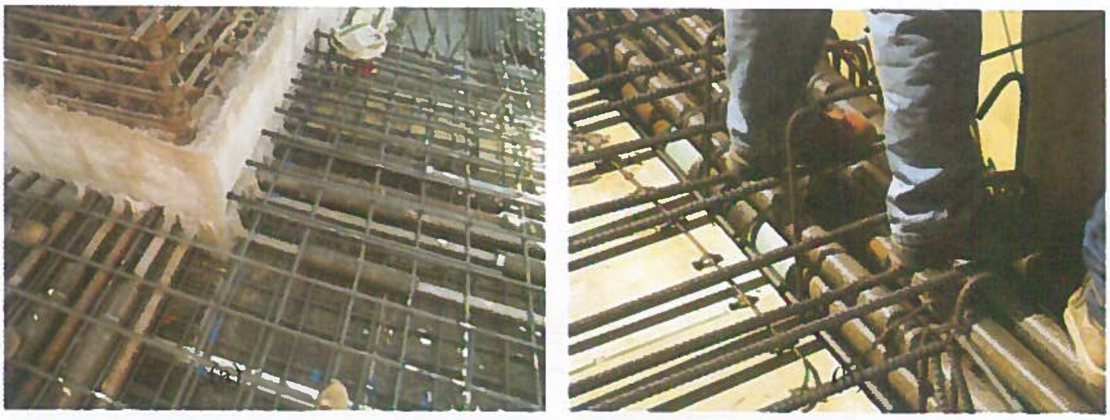

Figure 6.47 Collector Detailing, (A) Collector Connection to Shear Wall Boundary Zone; (B) Beam for Large Collector

Long collectors, such as the one shown in Figure 6.48, can accumulate strains over their length resulting in displacements that may be incompatible with modeling assumptions or deformation capacities of adjacent components. Designers can consider additional collector reinforcement to reduce the strain and associated collector elongation. Providing confinement reinforcement can also increase the ductility of the concrete locally, but will not address potential problems associated with incompatible deformations. Redesigning the force transfer system should also be considered.

Figure 6.48 A Long Collector with Confinement Reinforcement.

Where collector (or chord) reinforcement is required at a location coincident with a beam, the chord reinforcement can be placed within the beam. Beam transverse reinforcement, if properly detailed, can also serve as collector (or chord) confinement. If chord reinforcement does not fit entirely within the beam width, then the effective diaphragm depth should be based on the actual distribution and location of the chord reinforcement.

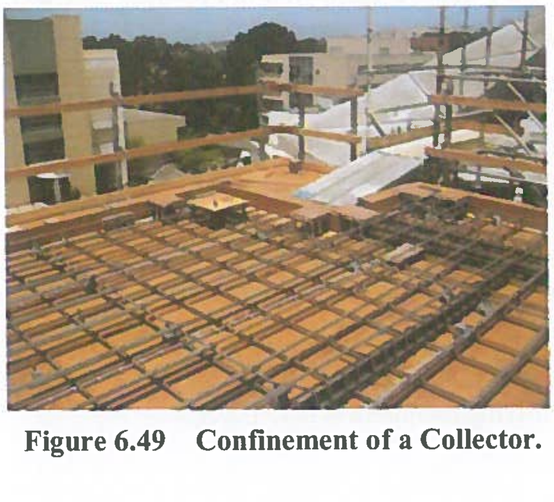

Figure 6.49 Confinement of a Collector.

6.6.3.3 Confinement

Transverse (confinement) reinforcement may be required in collectors or other elements transferring axial forces around openings or other discontinuities. The required seismic hook dimensions can sometimes make reinforcement detailing difficult in typical slab depths. If there are concerns over congestion, designers can increase the width of the collector within the slab or the thickness of the slab until the compressive stresses are low enough that confinement is not required. Alternatively, beams with sufficient dimensions can be added to facilitate the required confinement detailing. Where confinement is not required by the code, the designer still may consider adding some transverse reinforcement to improve connection toughness at critical locations; Figure 6.48 and Figure 6.49 show examples of added transverse reinforcement that is not in the form of closed hoops, yet will result in improved collector behavior.

6.7 CORNER & T JOINTS

Figure 6.50 shows some example of corner joints in many common types of reinforced concrete structure, where moments and other forces must be transmitted around these corners. A comparative study of such joints by Nilson and Losberg [*Ref.6.13*], showed that many commonly used join details will transmit only small fraction of their assumed strength. Ideally, the joint should resist a moment at least as large as the calculated failure moment of the members framing into it (i.e., the joint efficiency should be at least 100 percent). Tests have shown that, for common rebar details; joint efficiency may be as low as 30 percent.

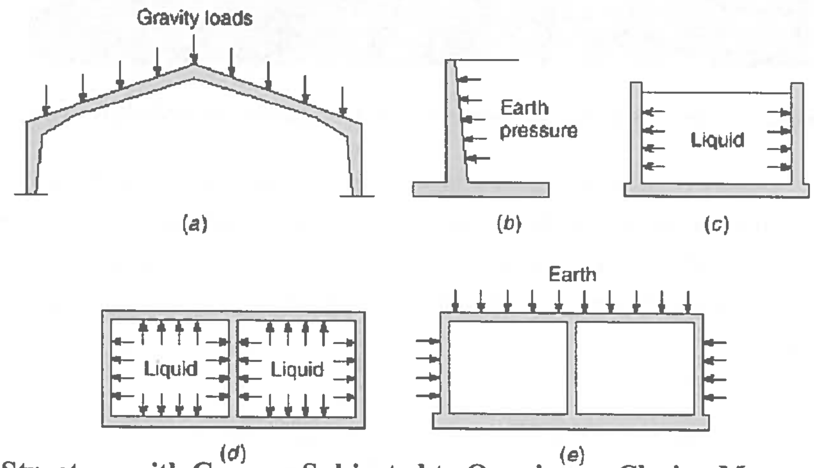

Figure 6.50 Structure with Corners Subjected to Opening or Closing Moments: (A) Gable Frame; (B) Earth-Retaining Wall; (C) Liquid Storage Tank; (D) Plan View of Multicell Liquid Storage Tank; (E) Large Box Culvert.

Corner joints may be subjected to opening moments, causing flexural tension on the inside of the joint or closing moments, causing tension on the outside. Generally, the first case is the more difficult to detail properly.

Test results for a large number of joints with alternative bar details are reported in Ref. 6.13. Comparative efficiencies for some specific details, relating to the maximum moment transmitted by the corner joint to the flexural capacity of the entering members, are summarized in Figure 6.51. In all cases the reinforcement ratio of the entering members is 0.75 percent.

As evident from Figure 6.51, the best performance results from the detail of Figure 6.51(e), the same as Figure 6.51 (d), except for the addition of a diagonal bar. This improves joint efficiency to 115 percent, so that joint is actually stronger than the design strength of the members framing into it. It is determined experimentally that *the area of the diagonal bar should be about one-half of that of the main reinforcement.*

T joints also may be subjected to bending moments, such as if only one cell of the multiple-cell liquid storage tank of Figure 6.51 (d) were filled. Tests of such joints, reported in *Ref.* 6.13, again indicate importance of proper detailing. The rebar arrangement of Figure 6.52 (a), which is sometimes seen, permits a joint efficiency of only 24 to 40 percent, but the simple rearrangement of Figure 6.52 (b) improves the efficiency to between 82 and 110 percent. In both cases, efficiency depends upon the main steel ratio in the entering members, with highest efficiency corresponding to the lowest tensile steel ratio.

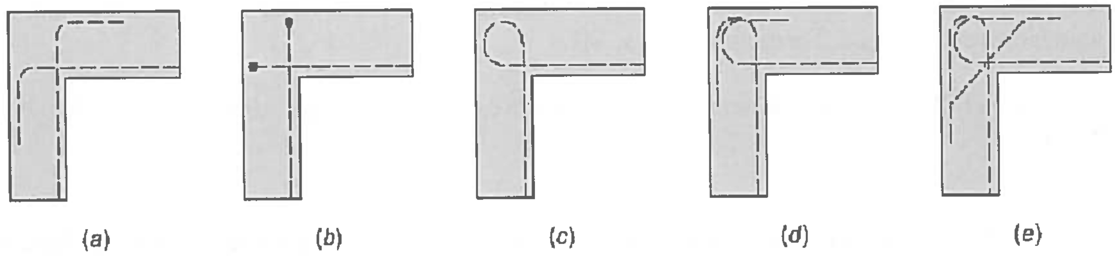

Figure 6.51 Efficiencies of Corner Joints Subjected to Opening Moments for Various Reinforcing Details: (A) 32 Percent; (B) 68 Percent; (C) 77 Percent; (D) 87 percent; (E) 115 Percent [*After Ref. 6.13*]

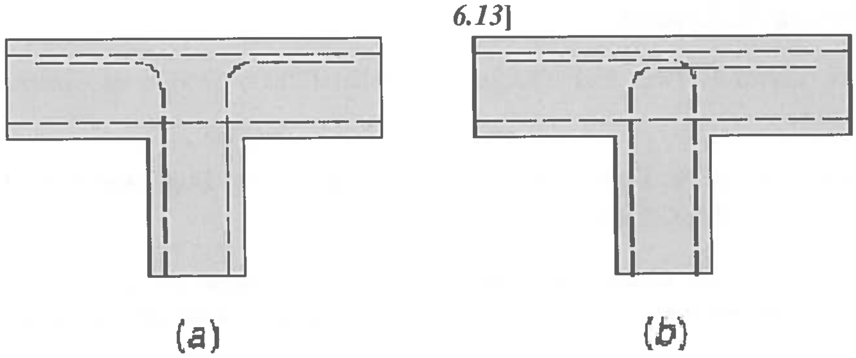

Figure 6.52 Comparative Efficiencies of T Joints Subject to Bending Moment: (A) 24 to 40 Percent Depending on Steel Ratio; (B) 82 to 110 Percent Depending on Steel Ratio [After Ref. 6.13].

Joints subjected to closing moments, with main reinforcement passing around the corner close to the outside face, cause few detailing problems because the main tension steel from the entering members can be carried around the outside of the corner. There is, however, a risk of splitting the concrete in the plane of the bend, or concrete crushing inside the bend. The efficiency of such joints can be improved by increasing the bend radius of the bar.

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APPENDIX: PART-I

APPENDIX A. FOUNDATION DESIGN IN SEISMIC ZONE

A 1 INTRODUCTION

If the soil bearing capacities within shallow depths are adequate, spread footings are used for low and medium rise buildings and mat foundations are used for high-rise buildings in Bangladesh. Cast-in-situ or precast piles are commonly used if the adequate soil bearing capacities are not available within shallow depths.

Design methods for foundations are given in Chapter 3, Chapter 6 and Chapter 7 (Part 6 of BNBC-2015), which are basically according to Chapter 15 of ACI 318-08. The severe earthquakes may cause liquefaction to occur in loose sandy soils, leading to complete loss of soil bearing capacities. The possibility of liquefaction in Bangladesh is described in BNBC-1993 Part 6 § 3.11.1.12 and BNBC-2015, Part 6 § 3.9.10.

The liquefaction is main subject to address in Appendix-A. But a typical spread footing is described along with a typical illustrative example.

A 2 TECHNICAL SUBJECTS

A 2.1 BASIC CONCEPT

The building foundations that may be subject to seismic force should be designed so as to maintain structural safety equivalent to or exceeding that of the upper structure. It is important to conduct an investigation according to the state of the site with respect to possible ground deformation such as liquefaction and landslide, and measures such as appropriate soil improvement or pile foundation method should be taken as needed.

A 2.2 EXTERNAL FORCE

The seismic lateral force for building foundations is composed of two force elements. One is the seismic lateral force "Pho" of the lowest storey for upper ground part of building that is base shear "V" in BNBC-2015. The other is the seismic lateral force "Phg" of underground part of a building.

$$Ph = Pho + Phg \qquad \text{(Eq. A.2-1)}$$

Pho is obtained according to BNBC-2015 as below.

$$V = S_a \cdot W \qquad \text{(Eq. A.2-2)}$$

On the other hand, "Phg" is not prescribed in BNBC-2015, therefore the Japanese code is introduced below as a reference.

$$P_{hg} = k \times W_g \qquad \text{(Eq. A.2-3)}$$

Where

k : seismic coefficient of underground part of a building

Wg : weight of foundation

The seismic lateral force "Phg" of underground structure is obtained by multiplying the sum of the fixed load of the foundation, the live load and weight of soil on the foundation slab by the seismic coefficient of (Eq. A.2-4).

$$k \geqq 0.1\left(1-\frac{H_f}{40}\right)Z \qquad \text{(Eq. A.2-4)}$$

Where

k : previously mentioned (k is estimated to 0.03～0.045 in Bangladesh)

H_f : depth, in meters, of each corresponding part of a building below the ground level

(to be regarded as 20 in excess thereof)

Z : zoning factor between 0.7 and 1.0 "Z" does not mean "z" in BNBC-2015.
(See Chapter 1 §1.3.2 in the Supplementary-Volume)

A 2.3 REDUCTION IN HORIZONTAL FORCE

The horizontal design force on a pile foundation is obtained by subtracting the resistant force due to the embedment effect of the foundation slab from the total horizontal force on the foundation.

This resistant force includes passive resistance of underground exterior walls as well as frictional resistance of the side of exterior walls and the foundation bed.

For a pile foundation, the horizontal force Php at the bottom of the foundation slab is usually calculated using the following equation.

$$P_{hp} = P_h \times (1-\alpha) \qquad \text{(Eq. A.2-5)}$$

$$\alpha = 1 - 0.2\frac{\sqrt{H}}{\sqrt[4]{D_f}} \qquad \text{(Eq. A.2-6)}$$

P_{hp} : horizontal force at the bottom of the foundation slab

α : allocation ratio of horizontal force at foundation slab embedment (Max. 0.7)

H : height of superstructure (m)

D_f : depth of embedment of the foundation (m)

(in principle, $D_f \geqq 2$m)

Figure A.1 shows the relationship between α and H and D_f. Eq. A.2-6 is based on a trial calculation in which frictional resistance in the side is taken into consideration assuming that the passive structure of embedment is an elastic spring. (Ref. 1-Ref.3)

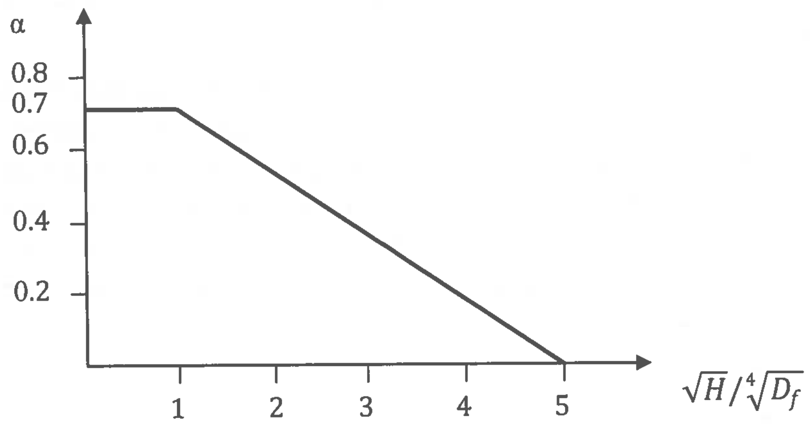


Figure A.1 Relation among α, H and D_f

Equation Eq. A.2-6 is applicable at $D_f \geqq 2$m, because it is possible that the passive structure depends on backfilling and exerts limited resistant force at a reduced D_f.

However, Eq. A.2-6 could probably be adopted even at $D_f \leqq 2$m if backfilling has been desirably obtained.

According to measured earthquake pressure during earthquakes, incremental earth pressure on underground exterior walls at the time of an earthquake is constant in the depth direction of those walls, or has a reverse triangle-like distribution in which the upper earth pressure is greater than the lower earth pressure. (Ref.4)

A 3 FOOTING DESIGN

(Ref. 1)

A 3.1 FOOTINGS SUBJECT TO ECCENTRIC LOADING

Footings are often subject to lateral loads or overturning moments, in addition to vertical loads. The lateral loads are due to earthquake or wind.

Bending moment at the bottom of a column in the lowest floor of a building, especially in the seismic zone, makes equilibrium with ground beams. When the bending moment is not high in magnitude, sometimes it is treated by spread footings. Lateral loads or overturning moments result non-uniform soil bearing pressure at one side than the other. Non-uniform soil bearing pressure can also be caused by a foundation pedestal not being located at the footing center.

If the lateral loads and overturning moments are small in proportion to the vertical loads, then the entire bottom of the footing is in compression and a P/A ± M/S type of analysis is appropriate for calculating the soil bearing pressures, where the various parameters are defined as follows:

P = The total vertical load, including any applied loads along with the weight of all of the components of the foundation, and also including the weight of the soil located directly above the footing

A = The area of the bottom of the footing

M = The total overturning moment measured at the bottom of the footing, including horizontal loads times the vertical distance from the load application location to the bottom of the footing plus any overturning moments

S = The section modulus of the bottom of the footing

If M/S exceeds P/A, then P/A – M/S results in tension, which is generally not possible at the footing and soil interface. This interface is generally only able to transmit compression, not tension. A different method of analysis is required when M/S exceeds P/A.

Following are the typical steps for calculating bearing pressures for a footing, when non-uniform bearing pressures are present. These steps are based on a footing that is rectangular in shape when measured in plan, and assumes that the lateral loads or overturning moments are parallel to one of the principal footing axes. These steps should be completed for as many load combinations as required to confirm compliance with applicable design criteria. For instance, the load combination with the maximum downward vertical load often causes the maximum bearing pressure while the load combination with the minimum downward vertical load often causes the minimum stability.

1. Determine the total vertical load, P.
2. Determine the lateral and overturning loads.
3. Calculate the total overturning moment M, measured at the bottom of the footing.
4. Determine whether P/A exceeds M/S. This can be done by calculating and comparing P/A and M/S or is typically completed by calculating the eccentricity, which equals M divided by P. If e exceeds the footing length divided by 6, then M/S exceed P/A.
5. If P/A exceeds M/S, then the maximum bearing pressure equals P/A + M/S and the minimum bearing pressure equals P/A-M/S.
6. If P/A is less than M/S, then the soil bearing pressure is as shown in Figure A.2. Such a soil bearing pressure distribution would normally be considered undesirable because it makes the footing structurally ineffective. The maximum bearing pressure, shown in the figure, is calculated as follows:

Maximum Bearing pressure = 2P/ [(B) (X)]

Where

X= 3(L/2-e) and e =M/P

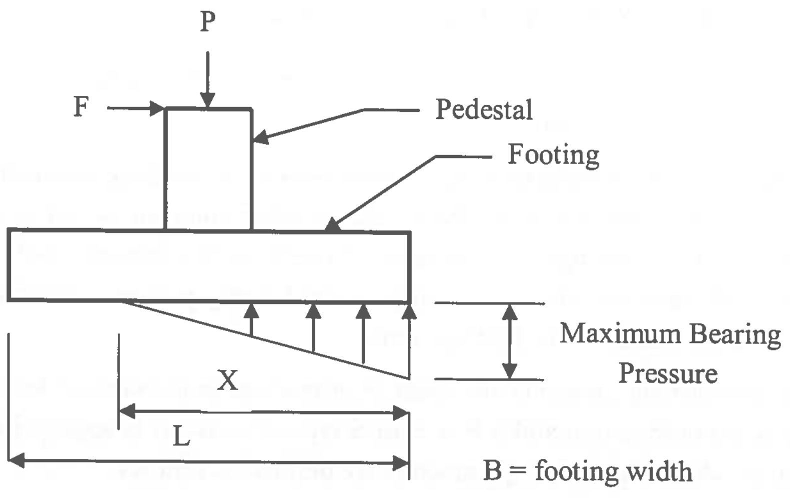


Figure A.2 Footing under Eccentric Loading

A 3.2 FOOTING DESIGN EXAMPLE OF A RECTANGULAR SPREAD FOOTING

Determine the size and reinforcing for a rectangular spread footing that supports a 16 in. square column, founded on soil.

As mentioned previously, the bending moment at column bottom on lowest floor of the buildings in seismic zone, can be made equilibrium with ground beams.

In case of this example here, the footing is designed only for vertical axial load without considering seismic lateral load, by means of that bending moment at column bottom is divided to ground beams (${}_CM_B = {}_FM_r + {}_FM_l$).

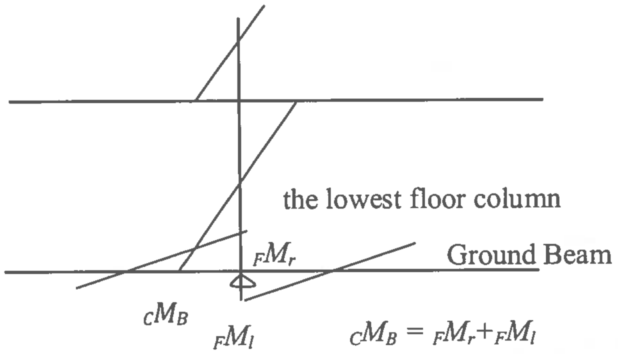


Figure A.3 Equilibrium of Bending Moment of Column and Ground Beams

Determine the size and reinforcing for a rectangular spread footing that supports a 16 in. square column founded on soil.

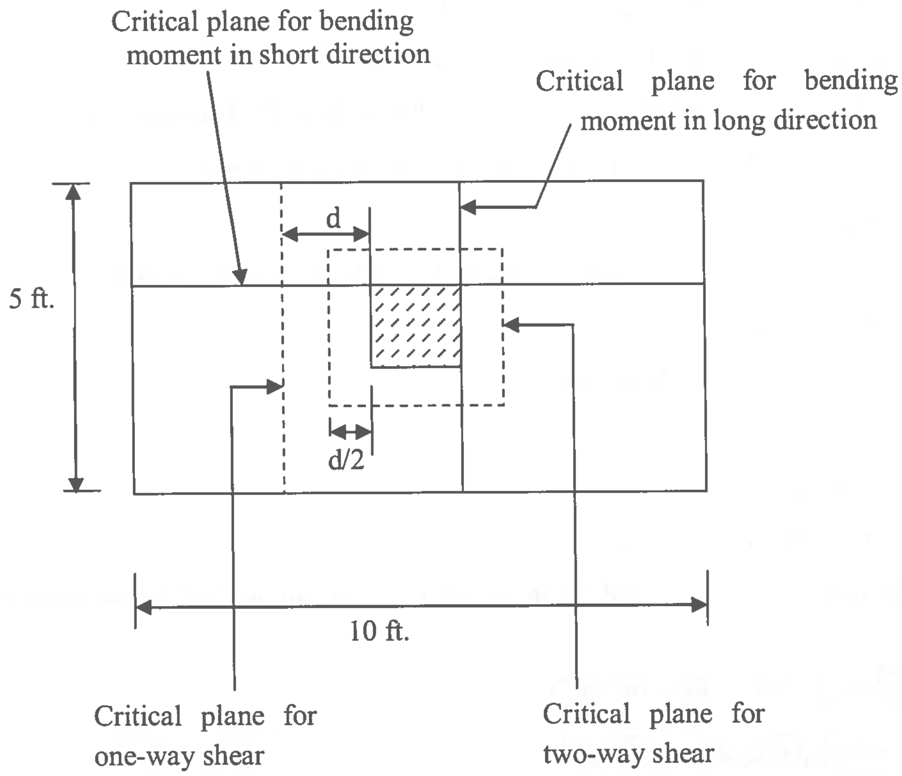


Figure A.4 Rectangular Spread Footing

Given:

$f_c' = 4$ ksi (28 N/mm^2)

$f_y = 60$ ksi (420 N/mm^2)

Dead Load = D = 180 K (820 kN)

Live Load = L=100 K (450 kN)

Earthquake O.T. = E = 120 K (540 kN)

(Axial load due to overturning under earthquake load)

Allowable soil bearing pressures

Due to D + L = 6 ksf (300 kN/m^2)

Due to D + L + E = 8 ksf (300×1.33=400 kN/m^2)

Procedure

① Sizing the footing

Ignoring the self-weight of the footing:

Areq.1 = (180 + 100) /6 = 46.7 sq. ft

Areq.2 = (180 + 100 + 120) /8 = 50 sq. ft

Use 5 ft ×10 ft

A = 50 sq. ft. is OK

② Required strength

U = 1.4 D = 1.4 (180) = 252 K or (252/50 = 5.1 ksf)

U = 1.2 D + 1.6 L = 1.2 (180) + 1.6 (100) = 376 K or (376/50 = 7.6 ksf)

U = 1.2 D + 1.0 E + 1.0 L = 1.2 (180) + 1.0 (120) + 1.0 (100) = 436 K or (436/50 = 8.72 ksf) (controls)

U = 0.9 D + 1.0 E = 0.9 (180) + 1.0 (120) = 282 K or (= 282/50 = 5.64 ksf)

③ Design for shear

φ shear = 0.75 Section 9.2 (From ACI 3.8-08, same hereinafter)

Assume Vs = 0 (no shear reinforcement)

$\varphi Vn = \varphi Vc$ Sec. 11.1.1

・Two-way action

Try d = 23 in = and h =27 in

b0 = 4 (16 + 23) = 156 in Sec. 11.11.1.2

According to Section 11.11.2.1, V_c shall be the smallest of (a), (b), and (c) for nonprestressed slabs and footings:

(a) $V_c = \left(2 + \frac{4}{\beta}\right) \lambda \sqrt{f_c'}\, b_0 d \quad (\beta = 16''/16'')$

$= \left(2 + \frac{4}{16/16}\right) \sqrt{f_c'}\, b_0 d = 6\sqrt{f_c'}\, b_0 d$

(b) $V_c = \left(\frac{\alpha_s d}{b_0} + 2\right) \lambda\sqrt{f_c'}\, b_0 d$ (αs = 40 for interior column)

$= \left(\frac{(40)(23)}{156} + 2\right)\sqrt{f_c'}\, b_0 d = 7.9\sqrt{f_c'}\, b_0 d$

(c) $V_c = 4\lambda\sqrt{f_c'}\, b_0 d = 4\sqrt{f_c'}\, b_0 d$ (Control)

$\therefore \emptyset V_c = 0.75(4\sqrt{4000} \times 156 \times 23)/1000 = 680.8k$

$V_u = \left\{(10' \times 5') - \left(\frac{16+23}{12}\right)^2\right\}(8.72) = 343.9k$

- One-way action (in short direction)

bw = 5×12″= 60 in and d = 23.5 in

$V_c = 2\sqrt{f_c'} b_w d$ (Sec. 11.2.1.1)

$\emptyset V_c = 0.75 \times (2\sqrt{4000}(60)(23.5))/1000 = 133.7k$

$V_u = 5\left\{\left(\frac{10}{2}\right) - \frac{8+23.5}{12}\right\}(8.72) = 103.6$

$\emptyset V_c = \emptyset V_c > V_u$ OK

- Bearing resistance of footing

Φ bearing = 0.65 (strength reduction factor φ, Section 9.3.2)

$\sqrt{(A_1/A_2)} = 2$ (Section 10.14.1 $\sqrt{(A_1/A_2)}$ is not more then 2)

$B_r = \emptyset\ (0.85\ f_c'\ A_1)\sqrt{(A_1/A_2)}$

=0.65(0.85) (4)(16)2(2)= 1131k >436k OK

④ Calculate moment at the column face (in long direction)

$M_u = \frac{1}{2} wl^2 = \frac{1}{2} \times (8.72) \times 4.33^2(5) = 408.7$ ft − k

$\Phi\ K_n = M_u$ (12,000)/(bd2)

φK_n =408.7(12,000)/{(5)(12)(23.5)2}=148 psi

For φK_n =148 psi, select ρ = 0.28%

$A_s = \rho_{bd} = 0.28 \times (5)(12)(23.5)/100 = 3.95\ in^2$

Check for As, min = 0.0018× (5) (12) (27) = 2.92 in^2< 3.95 in^2 OK

(Section 7.12.2, (b) Slab where grade 60 deformed bars or welded wire reinforcement are used 0.0018)

Use 8 # 7 bars distributed uniformly across the entire 5 ft width of footing

⑤ Calculate moment at the column face (in short direction)

—omitted—

⑥ Development length:

Critical sections for development length occur at the column face.

$$l_d = \left(\frac{3}{40}\right)(f_y/\sqrt{f_c'})[(\Psi_t\,\Psi_e\Psi_s\lambda)/(C_b + K_{tr})/d_b]d_b$$

$Ktr = 0$; and $(Cb + Ktr)/db = 2.5$ (maximum 2.5)

$$l_d = (3/40)(60{,}000/\sqrt{4{,}000})[(1.0)(1.0)(1.0)(1.0)/2.5]0.875$$

req.ld = 25 in for #7 bars

req.ld = 25 in < Pd (provided) = 4.33 (12) - 3 = 49 in

In the long direction: use straight #7 bars

req.ld = 25 in > Pd (provided) = 1.83 (12) – 3 = 19 in

in the short direction : use hooked #7 bars

$$l_d = (3/40)(60{,}000/\sqrt{4{,}000})[(1.0)(1.0)(0.8)(1.0)/2.5]0.625$$

req.ld = 14.2 in for #5 bars

req.ld = 15 in < Pd (provided) =22″- 3″ = 19 in

In the short direction: use straight #5 bars

Final Design

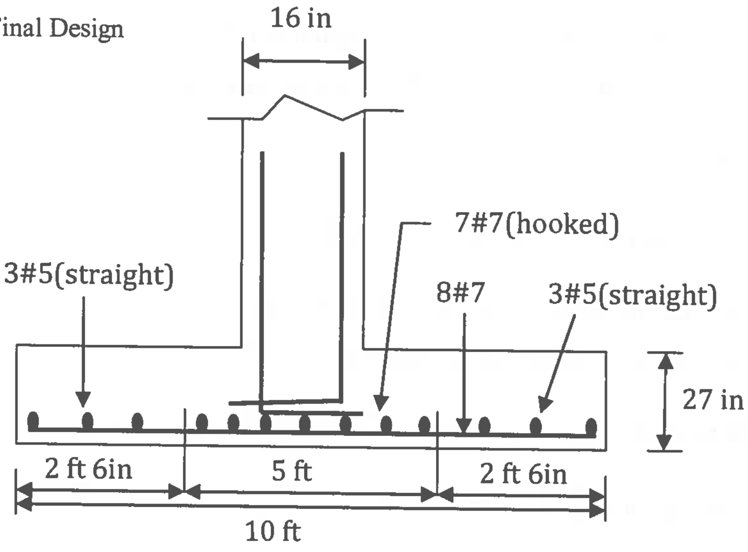


Figure A.5 Reinforcement Detail

A 4 LIQUEFACTION IN SEISMIC ZONE

A 4.1 LIQUEFACTION IN SAND SOIL (REF. 2)

If ground water level is high and ground surface stratum is composed of fine and loose sandy soil, liquefaction is likely to occur due to big earthquakes.

1) Occurrence condition, Mechanism and Damages

Soil Condition on occurrence of liquefaction

① Thick and loose sand stratum within 20m depth under the ground surface
② Less than N-value = 20
③ Sandy layer with less percentage of clay (Fine fraction content "F_c" < 35%)
④ High ground water level

↓

Big Earthquake Occurred (More than Seismic Intensity VIII of Modified Mercalli Intensity)

↓

Mechanism of Liquefaction

① Ground displacement
② Increasing of pore water pressure
③ Decreasing of affective vertical soil pressure
④ Deterioration of soil bearing capacity
⑤ Triggering liquefaction

↓

Damages of buildings and lifelines

① Ground Settlement
② Pile damages
③ Floating of sub-structures
④ Damages of Embankment or Fill
⑤ Land sliding (Lurching)

Figure A.6 Condition, Mechanism and Damages of Liquefaction

A 4.2 HIGH LIQUEFACTION POTENTIAL SOIL

A 4.2.1 Grain Size

- The percentage of silt and clay content is low
- Size of fine or medium sand in uniform
- Curve "B" has most liquefaction potential in three kinds of soil below

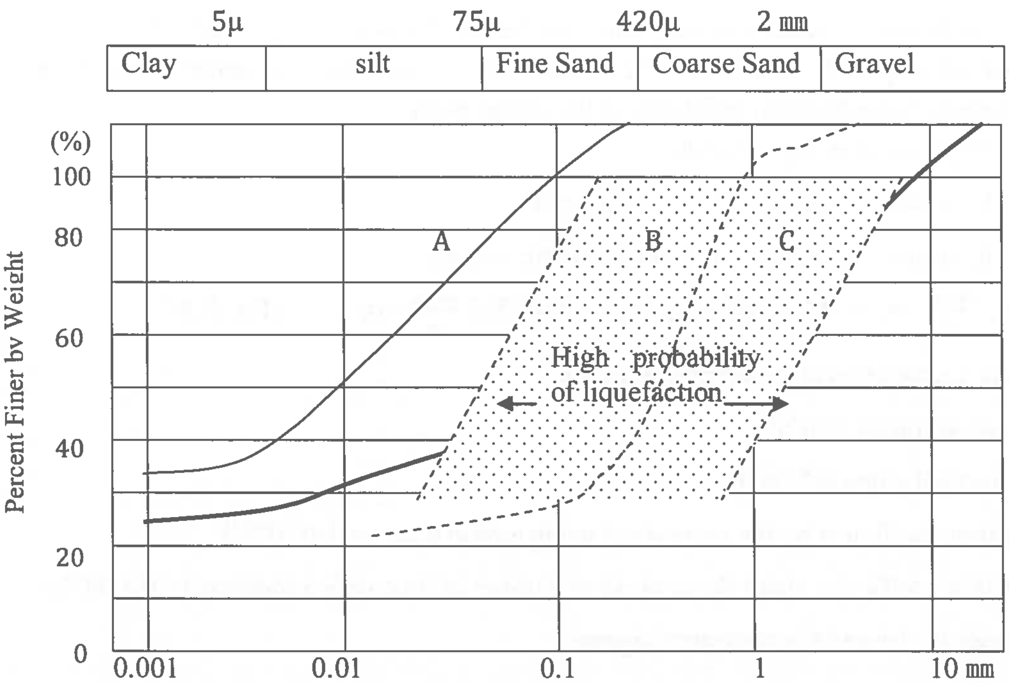


Figure A.7 High Liquefaction Potential Grain Size (Grain Size Distribution)

A 4.2.2 N-Value

An earthquake causes liquefaction to occur in loose sandy soil. Liquefaction potential is high in soils which N-value is less than 10. In case of severe earthquake, liquefaction may occur even if N-value is less than 20.

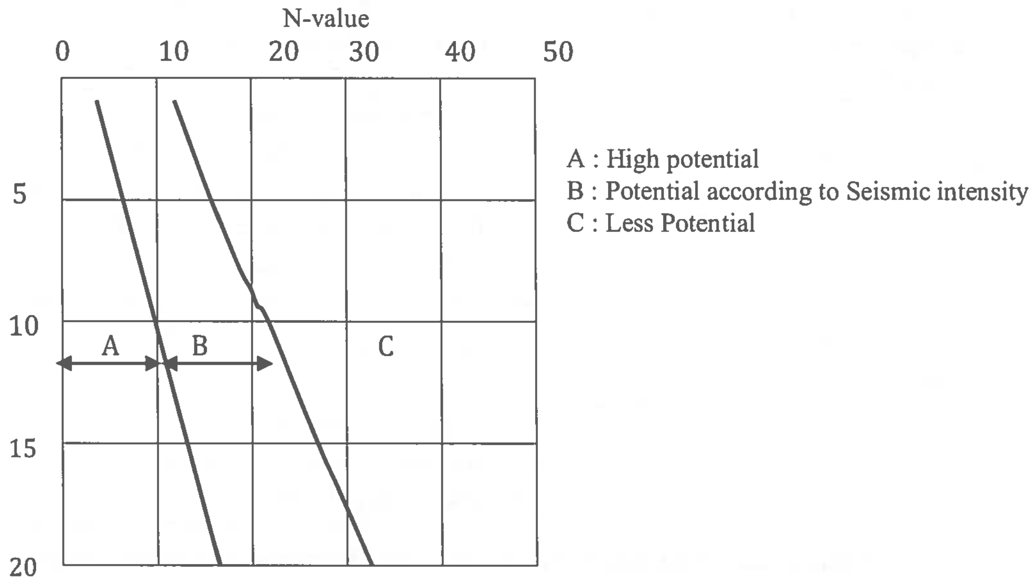


Figure A.8 N-Value and Liquefaction Potential

A 4.3 LIQUEFACTION POTENTIAL ANALYSIS

Liquefaction potential can be evaluated on the basis of FL value that is the ratio of shear strength "τ_d" occurred with earthquake and the liquefaction resistance "τ_L".

A 4.3.1 Shear Strength "τ_d" Occurred with Earthquake

Judgment of liquefaction potential is usually based on the comparison of "τ_d" and "τ_L".
"τ_d" is referred the amplitude of equivalent constant cyclic shear stress generated on the lateral plane, and "τ_L" is referred the liquefaction resistance at the lateral plane.
"τ_d" is obtained by the following methods.

1) The method by seismic response analysis of ground

2) The method by means of peak acceleration at ground surface

$$\tau_d = \gamma_n \cdot \gamma_d \frac{\alpha_{max}}{g} \cdot \sigma_v = 0.1 \times (M-1) \times (1-0.015Z) \frac{\alpha_{max}}{g} \cdot \sigma_v \qquad \text{(Eq. A.4-1)}$$

α_{max} : Peak acceleration at ground surface (cm/S^2)

g : Gravity acceleration (cm/S^2)

σ_v : Total vertical stress (kN/m^2)

γ_d : Reduction coefficient by the ground not being a rigid body (= 1-0.015Z)

γ_n : Correction coefficient about the equivalent number of times of a repetition (= 0.1(M-1))

M : Magnitude by Japan Meteorological Agency

Z : depth (m)

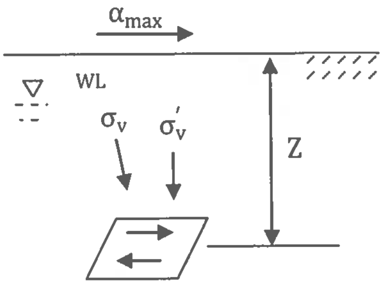


Figure A.9 Shear Stress in Soil

A 4.3.2 The Shear Strength in Liquefaction "τ_L"

"τ_L" is obtained directly by liquefaction test (cyclic undrained triaxial test on soils), or estimated by N-value, clay content ratio and effective stress σ_v' . Liquefaction resistance ratio "$R = \tau_L/\sigma_v'$" is obtained from Figure A.11.

The modified N-value (Na) in a certain depth is calculated

$$N_a = \sqrt{98/\sigma_v'} \times N + \Delta N_f \qquad \text{(Eq. A 4.2)}$$

Where

ΔN_f : Correction N-value -value increment according to fine fraction content ratio "F_C" obtained by Figure A.10.

N : observed N-value by Standard Penetration Test

σ_v' : Effective vertical stress below ground water

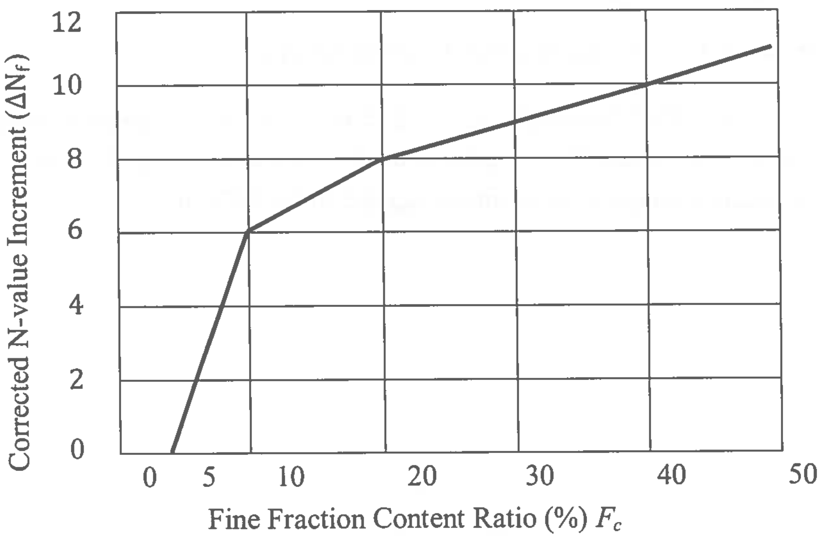


Figure A.10 Modified N-Value/Fine Faction Content Ratio

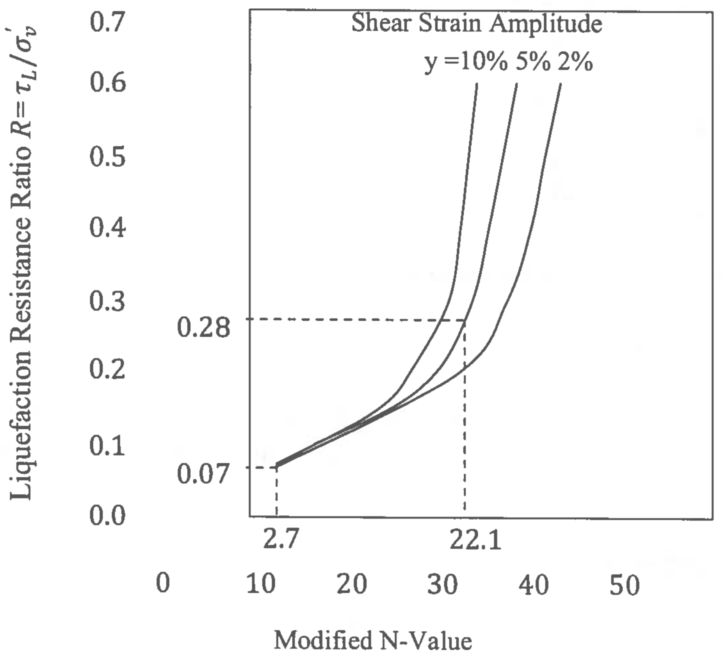


Figure A.11 Modified N-value/Liquefaction Resistance Ratio

A 4.3.3 Safe Factor F_L for Liquefaction Occurrence

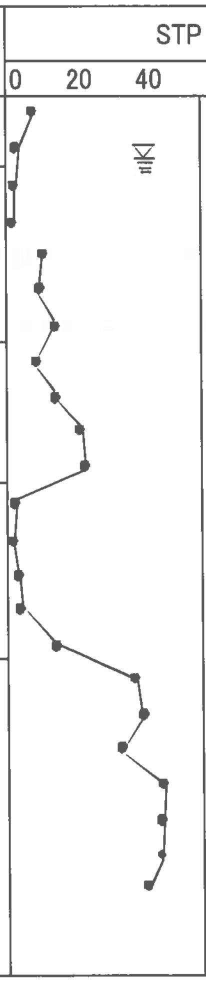
Liquefaction potential can be evaluated on the base of $F_L = {}^{\tau_L}/_{\tau_d}$ value as below.

F_L > 1.0 No probability of liquefaction

F_L < 1.0 Probability of liquefaction

A 4.3.4 Exercise for Prediction of Liquefaction Occurrence (Ref. 3)

To obtain the safety factor "F_L" for liquefaction potential in the alluvial soil shown at Figure A.11 below. The ground water level is 2.0m deep below ground level. Horizontal ground surface acceleration is assumed to be 150 (cm/sec2) and magnitude of an earthquake to be "M"= 6.5.



Depth (m)	Soil property name	STP N-Value	Unit Weight (kN/m³)	Effective Unit Weight (kN/m³)	Fine Fraction Contents F_c (%)
		8			25
		3	17.6	17.6	25
	Fine Sand	2			5
		2			5
5		12			5
		10			5
		15	18.6	8.6	5
	Fine Sand	10			5
		15			15
10		22			15
		23			15
	Clay	2			100
		2			100
		3	14.7	14.9	100
15		3			100
		16			100
	Sand with grovel	42			0
		45			0
		38			0
20		50			0
		50	18.6	8.6	0
		50			0
		48			0
25					

Figure A .12 Soil Condition

[Solution]

(1) Review at depth 4.0m below ground surface

From Eq. (A.4-1)

M = 6.5

γ_n = 0.1 (M-1) = 0.1 (6.5-1) = 0.55

α_{max} = 150 (cm/sec2)

σ_v = 17.6×2.0+18.6 (4.0-2.0) = 72.4 (kN/m²)

σ_v' = 17.6×2.0+8.8 (4.0-2.0) = 52.8 (kN/m²)

γ_d = 1- 0.015Z = 1-0.015×4 = 0.94

$$\frac{\tau_d}{\sigma_v'} = \gamma_d \cdot \gamma_n \cdot \frac{\alpha_{max}}{g} \cdot \frac{\sigma_v}{\sigma_v'} = 0.94 \times 0.55 \times \frac{150}{980} \times \frac{72.4}{52.8} = 0.109$$

The liquefaction resistance ratio τ_L/σ_v' is given by the following equation in accordance with the 5% shear strain amplitude curve.

$$\frac{\tau_L}{\sigma_v'} = 0.0410\left\{\sqrt{N_a} + 0.00903\left(\frac{N_a}{10}\right)^7\right\} \quad \text{(Eq. A.4-3) (Ref.4)}$$

The equation above is shown in Figure A.11.

$\sigma_v' = 52.8\ (\mathrm{kN/m^2})$

N = 2

$\therefore \Delta N_f = 0$ (According to Figure A.10)

$\therefore N_a = \sqrt{\frac{98}{52.8}} \times 2 + 0 = 2.7$

In accordance with Figure A.11 and Equation A. 4-3. $\frac{\tau_L}{\sigma_v'} = 0.0410\left\{\sqrt{2.7} + 0.00903\left(\frac{2.7}{10}\right)^7\right\} \fallingdotseq 0.07$

The safety factor "FL" for liquefaction potential is $F_L = \frac{\tau_L}{\tau_d} \cdot \frac{\sigma_v'}{\sigma_v'} = \frac{0.07}{0.109} = 0.642 < 1.0$

As the result, the liquefaction potential is height.

(2) Review at depth 9.0m below ground surface

$\sigma_v = 17.6 \times 2.0 + 18.6\ (9.0 - 2.0) = 165.4\ (\mathrm{kN/m^2})$

$\sigma_v' = 17.6 \times 2.0 + 8.8\ (9.0 - 2.0) = 96.8\ (\mathrm{kN/m^2})$

$\frac{\tau_L}{\sigma_v'} = 0.55 \times \frac{150}{980} \times \frac{165.4}{96.8} \times (1 - 0.015 \times 9.0) = 0.124$

The fine fraction content ratio "F_c" is 15%.

According to Figure A.10, $\Delta N_f = 7$

$\therefore N_a = \sqrt{\frac{98}{96.8}} \times 15 + 7 = 22.1$

From Figure A.11 and Eq. A. 4-3,

$\frac{\tau_L}{\sigma_v'} = 0.0410\left\{\sqrt{22.1} + 0.00903\left(\frac{22.1}{10}\right)^7\right\} = 0.29$

The safety factor "F_L" for liquefaction potential is $F_L = \frac{\tau_L}{\tau_d} \cdot \frac{\sigma_v'}{\sigma_v'} = \frac{0.29}{0.124} = 2.3 > 1$

As the result, the liquefaction potential is nothing.

A 5 SHALLOW FOUNDATION IN LIQUEFIABLE SOIL

If liquefiable layer is not deep, perhaps 2 to 3 ms total thickness of the layer, and for a not too tall buildings, the soil cement columns are available for soil improvement.

One of soil improvement method uses a special soil drill bit. The drill bit advances into the soil, cutting and grinding the soil and simultaneously injecting the cement slurry into the cuttings. During the drilling process, the soil is churned up by the rotation of a special bladed tool and mixed with low pressure injected cement power as the string is removed.

By mixing the soil and cement together with water, a stable and reinforced mass is quickly produced by the hydrating reaction. The only method to build the stable structure on liquefiable layer is to improve the liquefiable soil and to construct the stable foundation.

A 6. DESIGN OF PILE FOUNDATION

A 6.1 GENERAL DESCRIPTION OF PILE FOUNDATION

A 6.1.1 Analyzing Model

For ordinary buildings except super high rise buildings and structures designated as very essential facilities, the separated model for super structure and pile foundation is usually popular in designing piles. (See Figure A.13)

Though the interaction analysis involving super structure and pile foundation is sometime used for high rise building and very essential buildings, it is not usually applied for ordinary buildings. (See Figure A.14)

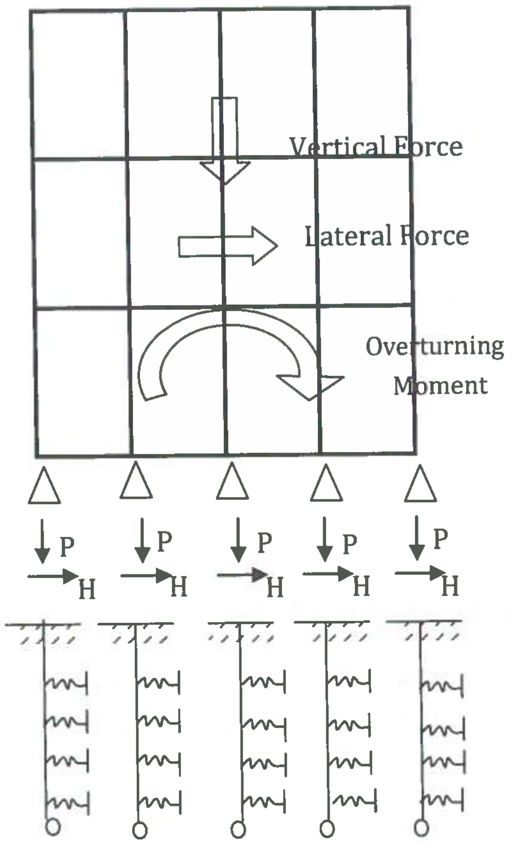


Figure A.13 Super Structure and Pile Foundation Separated Model

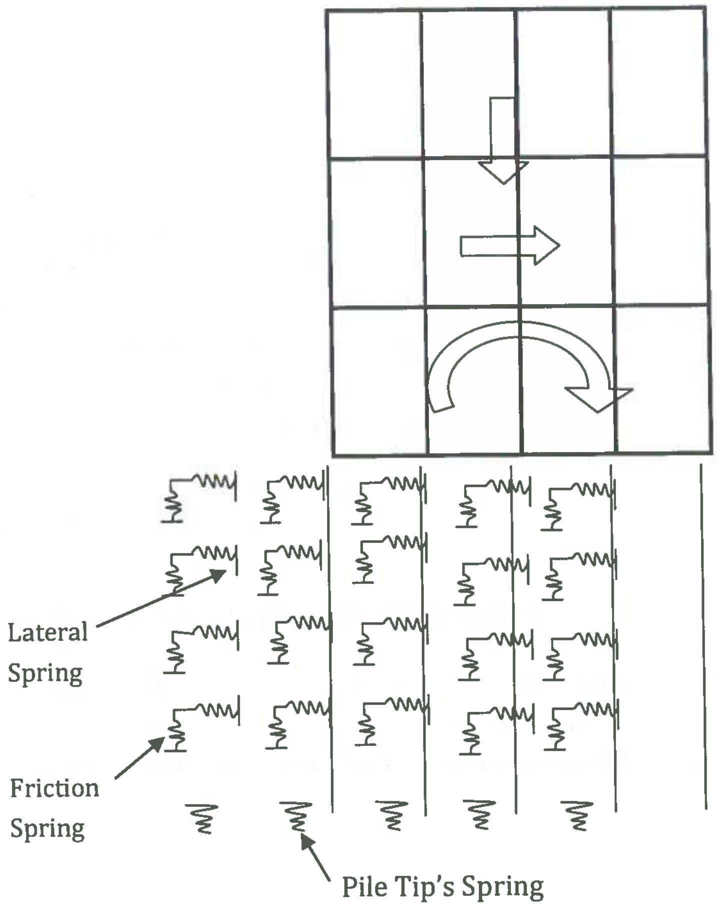


Figure A.14 Soil-Pile-Structure Interaction Model

A 6.1.2 Applied Forces to a Pile

When seismic force applies to the superstructure, design force for piles is the reaction forces that are occurred at column bottom supports when earthquake load applies to the superstructure.

- Vertical load = Gravity load + variable load in earthquake (compression or tension)
- Horizontal Load = Seismic lateral load acting on super structure shall be distributed to all piles in accordance with the lateral stiffness at each support

A 6.1.3 Design for Vertical Load

Vertical load, uplift resistance and load vs. settlement relation are described in BNBC-2015 as mentioned previously.

A 6.1.4 Design for Lateral Load

The relation of lateral force and displacement is required to obtain the lateral stiffness spring of piles. Total earthquake load acting on the building is distributed to each pile according to the lateral stiffness spring of piles. The pile is proportioned based on the displacement and bending moment created by the distributed force of piles.

Actual movement of pile foundations is not linear. The relation of the lateral force vs. lateral displacement at the top of pile becomes non-linear because of non-linearizing and plasticizing of soil or crack of concrete and yield of reinforcing bar of piles. (See Fig. A.15)

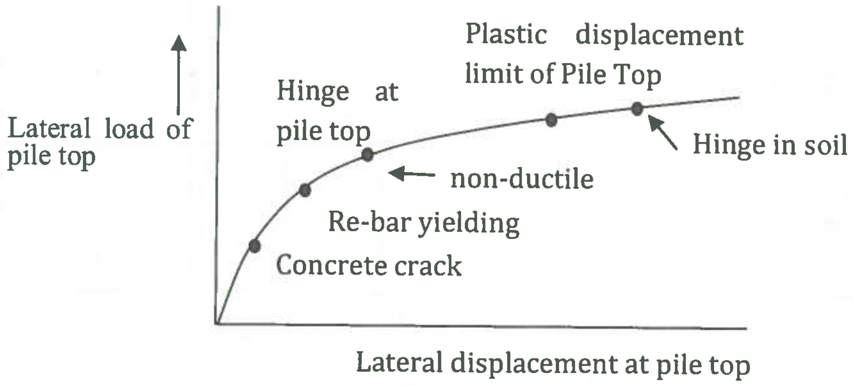


Figure A.15 Relation of Lateral Load vs. Displacement at Pile Top

The relation curves of M-φ or lateral force vs. displacement make difference between compressive pile and up-lift pile with increasing the lateral displacement. It means that the lateral load at top pile is variable depending on the vertical load value. (See Figure A.16)

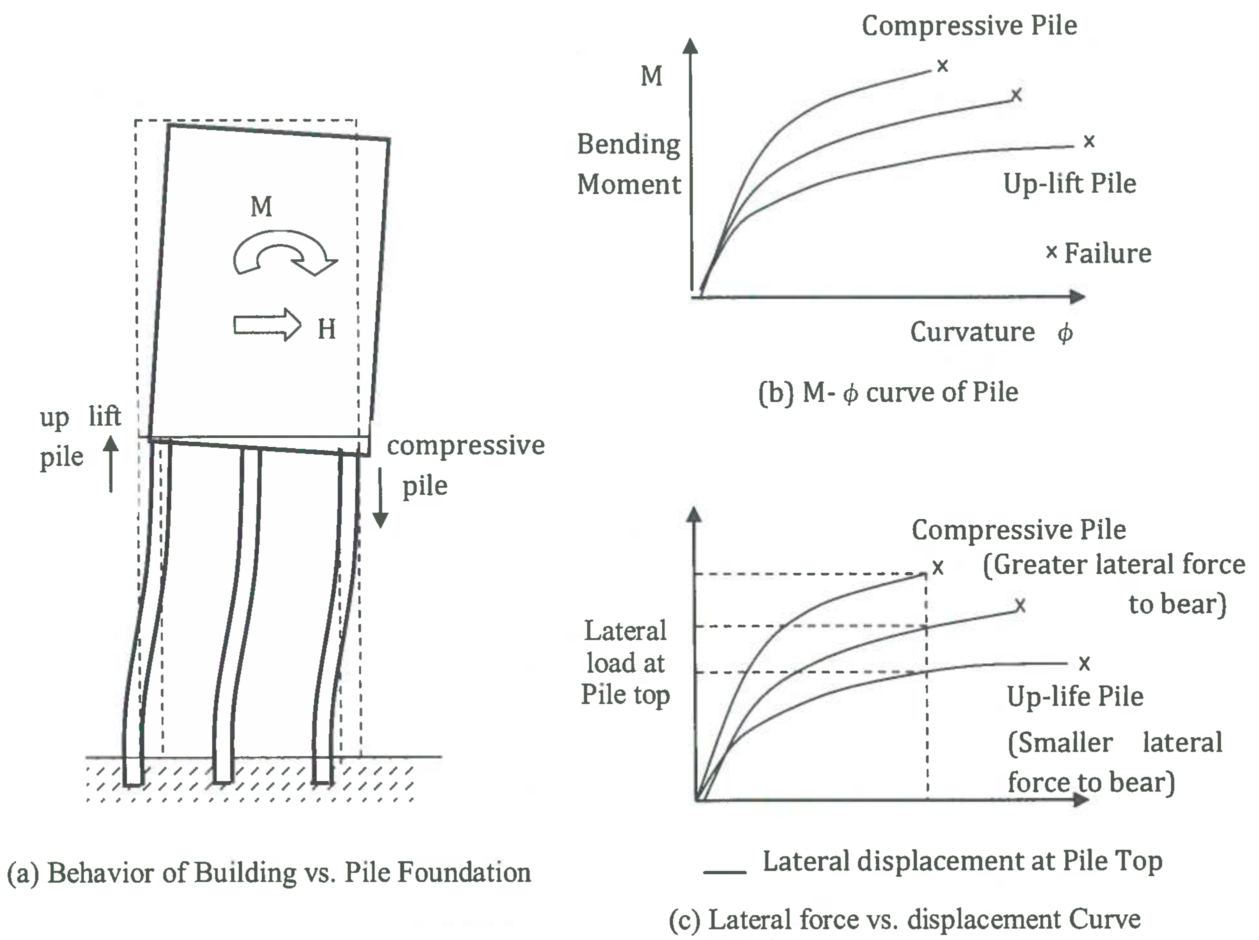


Figure A.16 Behavior of Pile Foundation in Earthquake

The boundary condition of pile top is different with joint detail of pile and pile cap, and is variable from fixed condition to free condition in accordance with axial force and deformation angle of pile top.

A 6.2 Calculation Formulas

The design methods of piles resisting lateral forces include an accurate design method strictly analyzing the embedded piles, and the simplified method under some conditions.

A 6.2.1 Formula for Lateral Displacement and Stresses of a Pile (Accurate Solution)

Basic differential equation is shown as below as assumed that the pile would be a beam possessing flexural stiffness and the soil would be lateral spring.

$$\frac{d^2}{dz^2}\left(K\frac{d^2y}{dz^2}\right)+B\,p=0 \qquad \text{(Eq. A.6-1)}$$

Where K : flexural rigidity considering non-linear of a pile

(K is equal to EI in case of elastic region) $(\mathrm{kN\cdot m^2})$

y : horizontal displacement of a pile (m)

z: depth from the ground surface (m)

p: horizontal soil reaction, $p = k_h \cdot y$ $(\mathrm{kN/m^2})$

k_h: coefficient of horizontal subgrade reaction (kN/m^3)

B: Pile diameter (m)

In Eq. A.6-1 lateral soil reaction "p" is related with lateral deflection "y" at a point with coordinate Z. The relation of lateral soil reaction "p" and deflection "y" is linear, it is shown as Eq. A.6-2.

$$p = k_h \cdot y \qquad \text{(Eq. A.6-2)}$$

Where

k_h is usually obtained from a coefficient of horizontal subgrade reaction and related with pile diameter.

Typical formula for lateral stiffness spring in non-linear soil is shown as follow.

$$p = k_h' \cdot y^{\frac{1}{2}} \qquad \text{(Eq. A.6-3)}$$

Eq. A.6-3 describes a parabola as shown in Figure A.17.

Apparent lateral spring (p/y) decreases in proportion to $y^{-\frac{1}{2}}$. The line of Eq. A.6-2 and the curve of Eq. A.6-3 are generally assumed to have a intersection at y = 1cm.

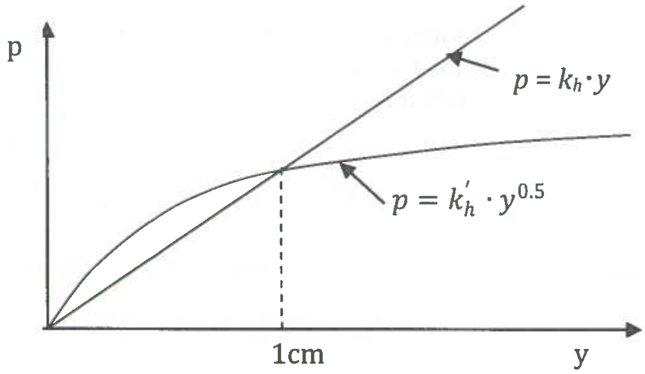


Figure A.17 *p-y* Curve

The actual behavior of piles to which lateral force applies in soil is obtained rather precisely under the condition of pile flexural stiffness decreasing and non-linear soil lateral stiffness in Eq. A.6-1. An analysis considering the decrease of pile flexural stiffness and non-linear soil lateral stiffness needs a repeated iterative calculation method.

A 6.2.2 Formula for Lateral Displacement and Stress of a Pile (Simplified Solution)

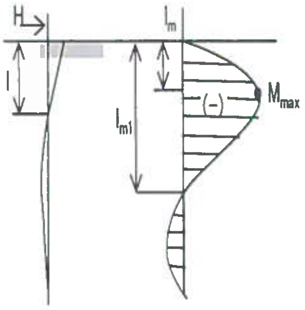
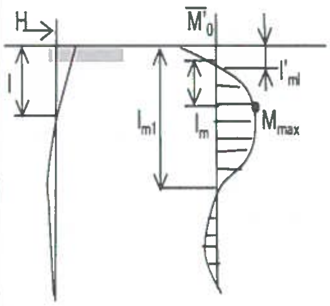
Most convenient and practical calculation method is given by the Eq. A.6-4 instead of Eq. A.6-1 based on the conditions below

- Pile section is constant without decline of flexural stiffness on account of pile crack or re-bar yielding.
- The p-y relation is linear as shown in Eq. A.6-2.

$$EI\frac{d^4y}{dz^4} + Bk_h \cdot y = 0 \qquad \text{(Eq. A.6-4)}$$

The solutions for the differential equation above are shown in Figure A.18.

(When lateral force and bending moment apply at pile top)

	Pile does not project from ground surface	
	Pile Top-Free End	Pile Top-Fixed End
Figure		
Pile Top Displacement	$y_t = \frac{H}{2EI\beta^3}$	$\overline{y_t} = \frac{H}{4EI\beta^3}$
Ground Surface Displacement	$y_0 = y_t$	$\overline{y_0} = \overline{y_t}$
Pile Top Slope Angle	$\theta_t = \frac{H}{2EI\beta^2}$	$\overline{\theta_t} = 0$
Pile Top Bending Moment	$M_0 = 0$	$\overline{M_0} = \frac{H}{2\beta}$
Maximum Bending Moment Under Ground	$M_{max} = -\frac{\sqrt{2}}{2\beta} e^{-\frac{x}{4}} H$ $=-0.3224H/\beta$	$M_{max} = -\frac{H}{2\beta} e^{-x/2}$ $=-0.2079\overline{M_0}$
A Point where Maximum Bending Moment appears Under Ground	$l_m = \frac{\pi}{4\beta}$	$l_m = \frac{\pi}{2\beta}$
The 1st Fixed Point Under Ground	$l = \frac{\pi}{2\beta}$	$l = \frac{3\pi}{4\beta}$
The 1st Zero Point of Bending Moment Under Ground	$l_{m1} = \frac{\pi}{\beta}$	$l'_{m1} = \frac{\pi}{4\beta}$ $l_{m1} = \frac{5\pi}{4\beta}$

$*\beta = [k_h \cdot B/(4EI)]^{1/4}$

Figure A.18 Solution for Elastic Supported Beam in a Uniform Soil

There are a few noteworthy items below

- The pile may be considered as semi-infinite length through satisfying Eq. A.6-5
- k_h is applicable in the range of $1/\beta$ in depth

$\beta L > 2.25$ Eq. A.6-5

Where

$\beta = \{K_h \cdot B/(4K)\}^{1/4}$ (1/m)

L = Embedded pile depth (m)

A 6.3 DESIGN EXAMPLE OF PILE FOUNDATION

A 6.3.1 Conditions for Calculation

1) Pile Condition (same conditions in case-1 (linear) and case-2(no-linear))

[Pile Dimension]

Embedded Length $L = 20$ m

Pile Diameter $B = 0.6$ m

Flexural Stiffness $EI = 1.46\times105$ kN$\cdot$m^2
(Refer to BNBC-1993 Part 6 Sec. 5.13.2 and assumed to be $f_c' = 24$ N/mm^2)

[Boundary Condition]

Pile Top : Fixed End, Pile Tip : Free End

2) Soil Condition (p～y relation shown in Figure A.17)

Soil condition and p～y relation in case-1 and case-2 assumed to be same constantly through underground.

[case-1 (linear)]

$p = k_h \cdot y,\quad k_h$=10000kN/m^3

[case-2 (non-linear)]
$p = k_h' \cdot y^{0.5}$ (provided that k_h' is linear when $y \leqq 1$mm), k_h'=1000kN/m$^{2.5}$

A 6.3.2 Calculation Results

1) Case-1

Earthquake lateral load of H=200kN applied at pile top the deformation and stress distribution are shown as below.

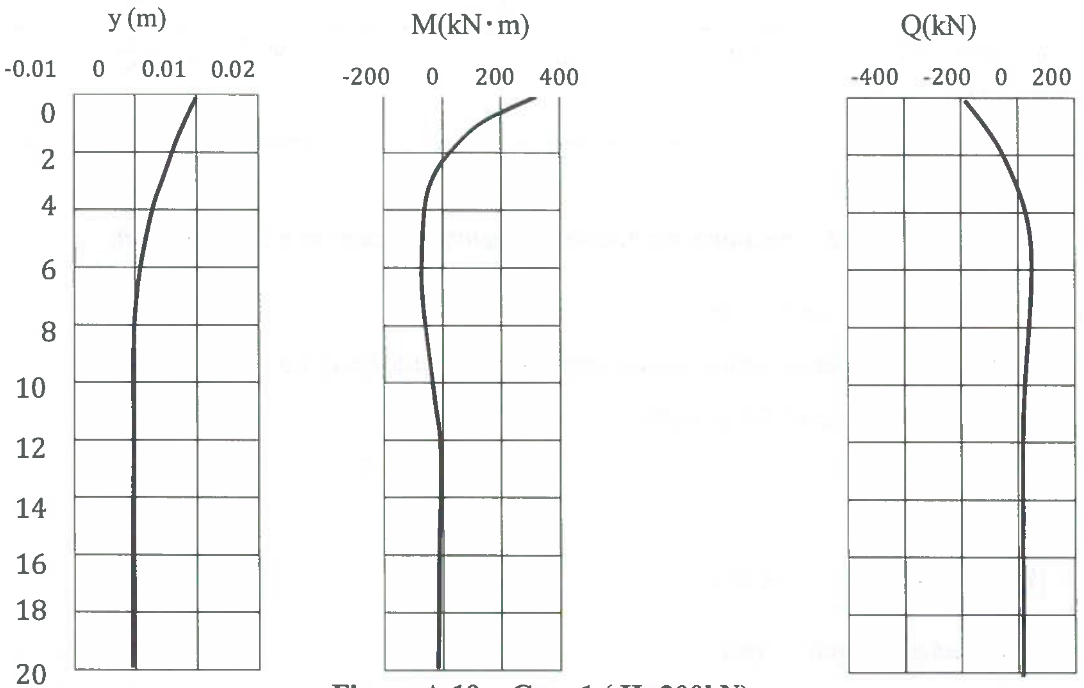


Figure A.19 Case-1 (H=200kN)

2) Case-2 Earthquake lateral load of H=200～600kN applied at pile top the deformation and stress distribution are shown as below.

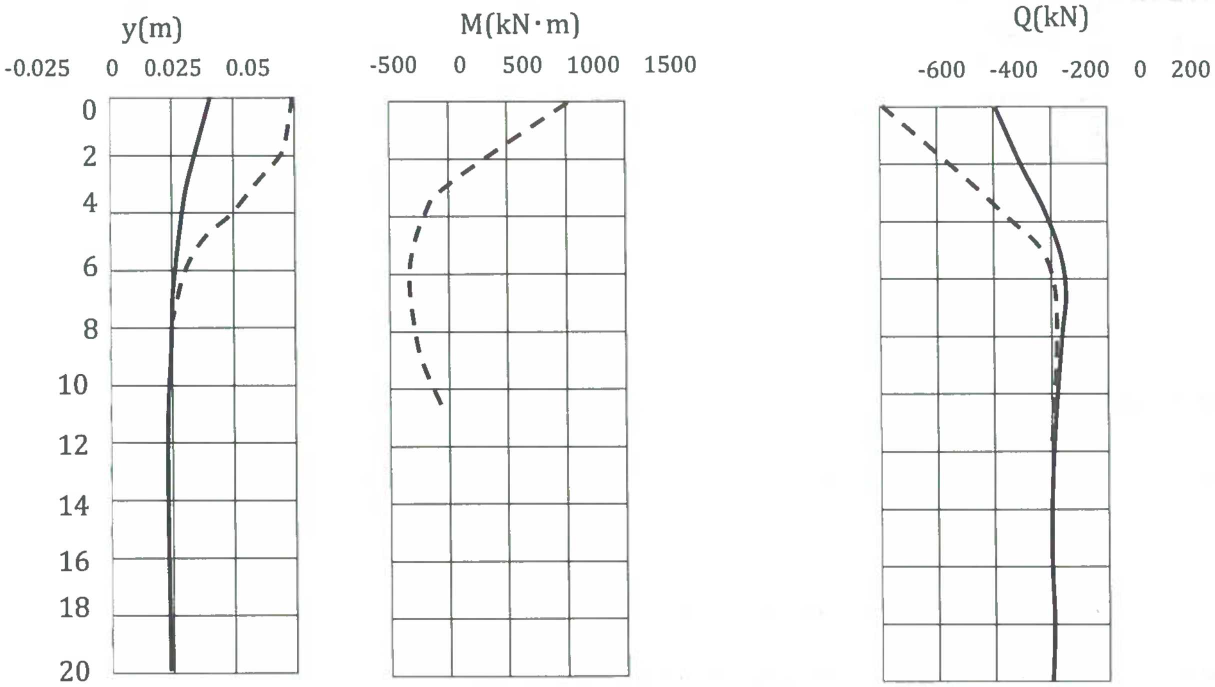


Figure A.20 Case-2 (H=200kN & 600kN) H=200kN(solid Curve) H= 600kN(dotted curve)

Calculation result in Case-1 by hand calculation is shown as follows. (Compare with Figure A.19)

- Boundary condition: Pile top (Fixed end), Pile tip (Free end)
- Pile length underground: L=20.0m
- Pile diameter: D=0.6m
- $\mathrm{E} = 4700\sqrt{f_c'} = 4700\sqrt{24} = 23 \times 10^3 \mathrm{N/mm^2}$, $\mathrm{I} = \pi \mathrm{D}^4/64 = 6.36 \times 10^{-3} (\mathrm{m}^4)$
- $\mathrm{EI} = 23 \times 10^6 \times 6.36 \times 10^{-3} = 146 \times 10^3$
- k_h =10000kN/m3 (For soil condition of N-value is 5～10)
- H =200kN

According to Figure A.18

Pile top displacement $\overline{y_t}$

$$\overline{y_t} = \frac{H}{4EI\beta^3} = \frac{200}{4 \times 146 \times 10^3 \times 0.32^3} = 0.01\mathrm{m}$$

Where

$$\beta = [K_h \cdot B/4EI]^{1/4} = [10000 \times 0.6/(4 \times 146 \times 10^3)]^{1/4} = 0.32$$

Pile top bending moment $\overline{M_o}$

$$\overline{M_o} = \frac{H}{2\beta} = \frac{200}{2\times0.32} = 312\ \mathrm{kN\cdot m}$$

The zero points of bending moment under ground

$$l'_{m1} = \frac{\pi}{4\beta} = \frac{3.14}{4\times0.32} = 2.4\ \mathrm{m}$$

$$l_{m1} = \frac{5\pi}{4\beta} = \frac{5\times3.14}{4\times0.32} = 12.3\mathrm{m}$$

Reference:

Ref. 1 「CHAPTER-5 FOOTING-SP17」 by S. Ali. Mirza, Published by Than Htet

Ref. 2 「Soil and Foundation for Architects」

Published by Kenchiku-Gijutu in 2001.2.1

Ref. 3 「Recommendation for Design of Building Foundations」

Revised in 2001; Published by Architectural Institute of Japan (AIJ)

Ref. 4 「Building Foundations」 by Dr. Yorihiko Osaki in 1991, Published by Giho-Do

APPENDIX B. UNREINFORCED (URM) MASONRY INFILL STRUCTURE

B 1. GENERAL

Since early eighties, the building construction practice in Bangladesh has been changing from pure masonry walls to RC frames infilled by unreinforced masonry (URM) walls. Many old buildings of PWD were constructed by pure masonry walls and they are still in service. Since early eighties, PWD also changed its building construction practice to RC frames infilled by URM walls similar to the private sectors in Bangladesh. Obviously the latest URM buildings are better than pure masonry buildings but URM buildings have also proven to be quite vulnerable in the past earthquakes. To enhance the seismic performance of URM, different methods for reinforcing masonry walls have been attempted over the years. Of these methods, two have proven to be particularly effective: reinforced masonry (RM), which involves the placement of reinforcing steel and concrete in the hollow cells of special masonry bricks/blocks, and confined masonry (CM).

The URM is also popular in countries like Indonesia, The Philippines, Turkey and in Central South America. The structural analysis and design of URM are complicated, especially to evaluate the stiffness and strength of URM in RC frames. Therefore, the stiffness and strength of URM are not considered in structural design, which makes the design method conservative. This may cause the effect of soft-storey more critical in the event of earthquakes. Also masonry walls with partial heights, provide in balcony and up to the level lower part of windows, make the columns very brittle against shearing force because of the decreasing ratio of the column's clear height to depth.

The effect of URM infill in RC frame cannot be neglected in seismic design. The stiffness and strength of URM must be addressed to avoid poor performance during earthquakes. The distribution of URM infill walls in the plan of a building should be outlined by the mutual discussions between structural engineer and architect, considering the horizontal eccentricities and vertical stiffness, and keeping in mind that URM infill walls are considerably effective to the seismic design of moderate earthquakes.

The HBRI (Housing and Building Research Institute), Engineering Universities in Bangladesh, PWD and other Research Institutions in Bangladesh should carry out more exclusive research works on URM infill walls keeping both safety against seismicity and economy in accounts.

However, the RM and CM buildings may be suitable for Bangladesh, if both seismic as well as economy are concerned. Reinforcing steel both in horizontal and vertical direction through special hollow bricks/blocks in infill walls are provided in RM buildings. These ensure confinement of masonry works with the RC frames as well as increase of shear strength and ductility of frames. Construction of masonry works may be gradually advanced with introduction of reinforcing bars in vertical as well as in horizontal direction through the hollows of bricks/blocks, though the progress of works is to be slow. To get rapid progress in construction works, the rebar may be fabricated first and hollow blocks/bricks later provided that the bricks/blocks should have openings in the facial sides.

CM is sometimes confused with another type of building system: reinforced concrete frames with masonry infill walls. These two systems often look the same (concrete beams and columns enclosing masonry walls) but behave very differently. In CM systems, the masonry wall carries the structural forces while the RC members just confine the masonry wall. As well, the RC members are usually the same thickness as the masonry wall in CM construction, and the sequence of construction is such that the masonry walls are built first and the tie columns and bond beams cast around the masonry wall. CM is a structural system consisting of unreinforced masonry wall panels surrounded by horizontal and vertical "confining" members called bond beams and tie columns [Ref. B1].

B 2. TECHNICAL REPORTS

This section contains summary of twelve technical reports concerning URM infill walls

B 2.1 EVALUATING STRENGTH AND STIFFNESS OF UNREINFORCED MASONRY INFILL STRUCTURES

Guidelines on evaluating the lateral load capacity of in-filled panels for in-plane and out-of-plane bending. (by US Army Corps of Engineers, Engineer Research and Development Center, January 2002).

B 2.2 ROLES OF MASONRY IN-FILL WALLS IN AN EARTHQUAKE

Masonry in-fill walls may have significant effect on the collapse of buildings and loss of life depending on the nature of the earthquake level, geology, building size, shape and irregularities. This paper highlights the importance of the selection of the building wall materials, and the shortcomings of the most commonly used frames structures with masonry in-fills irrespective of residential or office buildings. (by DINCEL CONSTRUCTION SYSTEM in Australia).

B 2.3 HYBRID MASONRY CONSTRUCTION

Hybrid masonry is a structural system that utilizes reinforced masonry walls with a framed structure. The discussion here will include steel frames in combination with reinforced concrete masonry walls. Hybrid masonry offers many benefits and complements framed construction. (by Technology Brief of the International Masonry Institute).

B 2.4 SEISMIC DESIGN AND ANALYSIS OF MASONRY IN-FILLED FRAMES

A simple analytical procedure for the seismic design of masonry-in-filled frame is presented. The analysis procedure, based on the experimental and analytical studies reported in the literature, accounts for the effect of infills in all three stages, namely, in computing seismic loading, in predicting response of the in-fills frame, and in determining the strength of the in-filled frame. (NRCC-38839 by V. K. R. Kodur, M. A. Erki, and J. H. P. Quenneville, National Research Council Canada, June 1995).

B 2.5 STUDY THE REINFORCED CONCRETE FRAME WITH BRICK MASONRY INFILL DUE TO LATERAL LOADS

In this paper the behavior of reinforced concrete frames with brick masonry infill for various parametric changes have studied to observe their influences in deformation patterns of the frame. (by Kashif Mahmud, Md. Rashadul Islam and Md. Al-Amin)

B 2.6 LATERAL STIFFNESS OF MASONRY INFILLED REINFORCED CONCRETE FRAMES WITH CENTRAL OPENING

The present study proposes a reduction factor for effective width of diagonal strut over that of the solid reinforced concrete in-filled frame to calculate its initial lateral stiffness when a central window opening is present. (by Goutam Mondal and Sudhir K. Jain, M. EERI)

B 2.7 STATIC RESPONSE OF INFILLED FRAMES USING QUASI-STATIC EXPERIMENT

This paper treats an experimental investigation of gravity-load steel frame in-filled with unreinforced masonry (concrete block) walls and slowly applied cyclic lateral loads. Quasi-static experimentation can be defined as a testing procedure where cyclic loading is slowly applied to the tested structure to simulate lateral loads produced by seismic forces. (by Khalid M. Mosalam, Richard N. White, and Peter Gergly JOURNAL OF STRUCTURAL ENGINEERING/NOVEMBER 1997)

B 2.8 EFFECT OF INFILL MASONRY WALLS ON THE SEISMIC RESPONSE OF REINFORCED CONCRETE BUILDINGS SUBJECTED TO THE 2003 BAM EARTHQUAKE STRONG MOTION: A CASE STUDY OF BAM TELEPHONE CENTER

A 3-dimension nonlinear analysis of the Bam telephone center-reinforced concrete building, subjected to the horizontal components of the recorded strong motion, was carried out to obtain an analytical explanation of the almost linear performance of the building during the earthquake. (by Hossein Mostafaei and Toshimi Kabeyasawa, Bull. Earthq. Res. Inst. Univ. Tokyo)

B 2.9 SEISMIC ASSESSMENT OF RC FRAME BUILDINGS WITH BRICK MASONRY INFILL

Five reinforced RC framed building with brick masonry infill were designed for same seismic hazard in accordance with IS code taking into consideration of effect of Masonry.The investigation has been made to study the behavior of RC frames with various arrangement of infill when subjected to dynamic earthquake loading. (by Mulqund G. V., Dr. Kulkarni A. B, IJAEST-International Journal of Advanced Engineering Sciences and Technologies)

B 2.10 MASONRY INFILL WALLS: AN EFFECTIVE FOR SEISMIC STRENGTHENING OF LOW-RISE REINFORCED CONCRETE BUILDING STRUCTURES

Field evidence has shown that continuous infill masonry walls can help reduce the vulnerability of a reinforced concrete structure. In order to test this hypothesis, a full-scale three-storey flat-plate structure was strengthened with infill brick walls and tested under displacement reversals. The results of this test were compared with the same building without infill walls. (by Santiago Pujol, Amadeo Benavento-Climenty, Mario E Rodriguez, and J. Paul Smith-Pardo, the 14th world Conference on Earthquake Engineering).

B 2.11 SEISMIC PERFORMANCE OF REINFORCED CONCRETE FRAME STRUCUTRUE WITH AND WITHOUT MASONRY INFILL WALLS

This research quantifies the effect of the presence and configuration of masonry infill walls on seismic collapse risk. Seismic performance assessments indicate that, of the configurations considered (bare, partially infilled and full-infilled frames), the full infilled frame has the lowest collapse risk ad the bare frame is found to be the most vulnerable to earthquake-induced collapse. (by Siamak Sattar, Abbie B Liel University of Colorado)

B 2.12 MODELING OF MASONRY INFILL PANELS FOR STRUCTURAL ANALYSIS

A macro model for masonry infill is proposed in this paper. The proposed analytical development assumes that the contribution of the masonry infill to the response of the infilled frame can be modeled by "replacing the panel" by a system of two diagonal masonry compression struts. A simulation study was performed in order to verify the proposed analytical approach. The analytical evaluation was done using IDARC2D Version 4.0 (Valles et al. 1996). A computer program for the inelastic damage analysis of buildings. (by A. Madan, A. M. Reinhorn, Fellow, ASCE, J. B MANDER, Member, ASCE, and R. E. Valles)

In the technical reports above, Sec. B2.8 is mostly understandable. It may be applied for the design of the buildings with URM infill walls.

B 3. MASONRY INFILL

The Technical Report Sec. B2.8 titled "Effect of Infill Masonry Walls on the Seismic Response of Reinforced Concrete Buildings Subjected to the 2003 Bam Earthquake Strong Motion: A Case Study of Bam Telephone Center" is introduce here in detail as it is to understand the masonry infill.

B 3.1 COMPRESSIVE STRENGTH OF A MASONRY PRISM

According to Hilsdrof H. K. (1969) report, a schematic of a masonry prism subjected to compression, with the brick units and mortar joints being in bi-axial C-T and bi-axial C-C states is presented in Figure B-1 [Ref. B2].

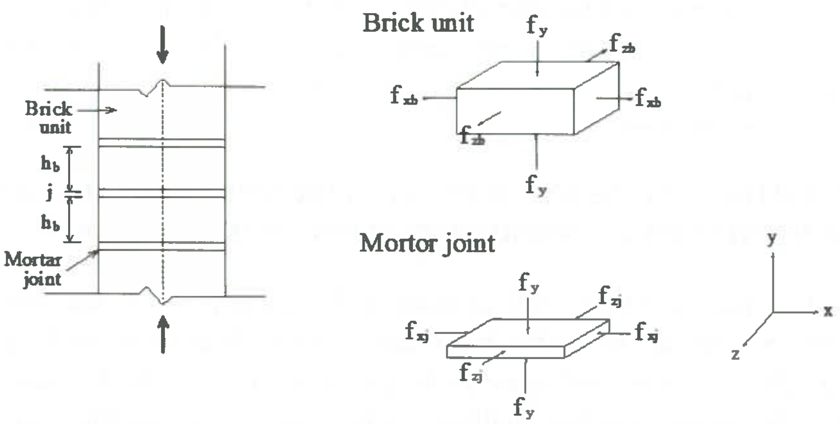


Figure B.1 Stress Developed in the Brick Units and the Mortar Joints of a Masonry Prism Subjected to Compression [*Ref.* B 2]

The relationship between the normal stress, *fy*, and transverse stresses, *fxb* and *fzb* developed in the brick units in a bi-axial tension-compression state, can be assumed to be linear and is given by

$$f_{xb} = f_{zb} = f'_{tb}(1 - f_y / f'_{cb}) \qquad \text{(B1)}$$

Eq. (B1) is converted to Eq. B1a.

$$\frac{f_{xb}}{f_{tb}} + \frac{f_y}{f_{cb}} = 1 \qquad \text{(B1a)}$$

The failure envelope proposed by Hilsdrof is shown in Figure B2 [Ref. B3].

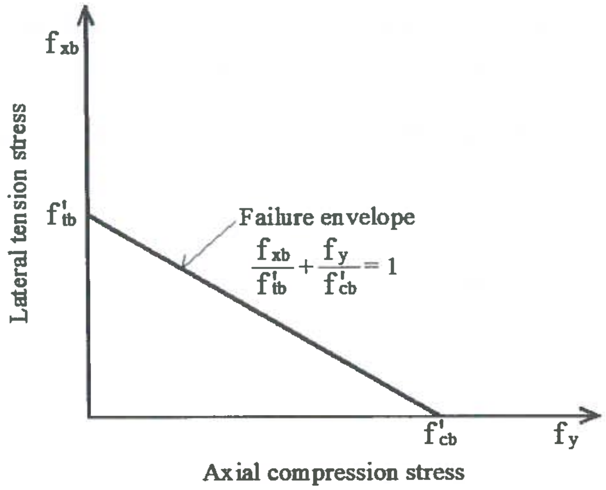


Figure B.2 Bi-Axial Compression-Tension (C-T) Relationship of Brick Units [*Ref.* B 3]

Where, f'_{cb} and f'_{tb} are uniaxial compression and biaxial tension strengths of the masonry unit, and *fy* is the axial compression stress occurring in conjunction with lateral tension stress *fxb*, at failure.

The relationship between the normal stresses, *fy*, and transverse stresses, *fxj* and *fzj*, developed in the mortar joints in a bi-axial compression-compression state, can also be assumed to be linear and is given by the equation below [Ref. B2]

$$f_{xj} = \left(\frac{f_y - f'_{cj}}{4.1}\right) \qquad \text{(B.2)}$$

Where f'_{cj} is the uniaxial compressive strength of the mortar joint. This relationship is shown in Figure B3.

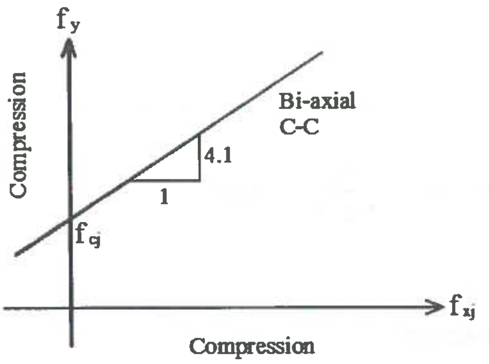


Figure B.3 Bi-Axial Compression-Compression (C-C) Relationship of Mortar Joints [*Ref.* B 2]

Assuming that the lateral stresses, *fx*, in the brick units and the mortar joints are uniformly distributed over the height of the bricks and the joints (width is the same for both) (see Figure B1), then from force equilibrium

$$fxb \cdot hb = fxj \cdot j \qquad \text{(B.3)}$$

Where *hb* and j are the brick unit and mortar joint heights, respectively.

Substituting Eq. B1 and Eq. B2 in Eq. B3, the following expression proposed by Hilsdorf (1969) for predicting the compressing strength of masonry prism, f_p' is obtained.

The compressive strength of the masonry prism *fp'* is determined from an equation recommended by Paulay and Priestly (1992) as follows.

$$f_p' = \frac{f_{cb}'(f_{tb}' + \alpha f_j')}{U_u(f_{tb}' + \alpha f_{cb}')} \qquad \text{(B4)}$$

Where,

$$\alpha = \frac{j}{4h_b}$$

f_p' = compression strength of the masonry prism
f_{tb}' = tension strength of the brick
f_{cb}' = compression strength of the brick
f_{cj}' = mortar compression strength
h_b = the height of the masonry unit
j = the mortar joint thickness and
U_u = the stress non – uniformity coefficient equal to 1.5

According to the Building and Housing Research Center of Iran, an average of f_{cb}'=75 kg/cm^2 may be assumed for the compression strength of the bricks in the selected building.
The height of the masonry unit "*hb*" assumed to be 60mm and the mortar joint thickness "j" to be 15mm .
The tension strength of the solid bricks may be determined by T. Paulay and M. J. N. Priestly (1992) as follows:

$f_{tb}' = 0.1\, f_{cb}'$ therefore, f_{tb}'=7.5 kg/cm^2

Hence, compression strength of a masonry prism [f_p'] is obtained as:

$$f_p' = \frac{75(7.5 + \alpha 50)}{1.5(7.5 + \alpha 75)}, \quad \text{where } \alpha = \frac{1.5}{4.1 \times 6} = 0.061$$

$$f'_p = \frac{75(7.5+0.061\times50)}{1.5(7.5+0.061\times75)} = 44 \text{ kg/cm}^2$$

B 3.2 SHEAR STRENGTH OF INFILL WALLS [REF. B4]

There are several potential failure modes for infill masonry walls, and the following two failure modes are critical.

- Sliding shear failure of masonry walls horizontally and
- Compression failure of diagonal strut.

Failure modes other than above are:

- Tension failure of the tension column resulting from applied overturning moments;
- Diagonal tensile cracking of the panel and
- Flexural or shear failure of the columns.

B 3.3 SLIDING SHEAR FAILURE

The Mohr-Coulomb failure criteria can be applied to assess the maximum shear strength for this kind of failure mechanism:

$$\tau_f = \tau_o + \mu\sigma_N \tag{B.5}$$

Where, τ_o = cohesive capacity of the mortar beds

μ = sliding friction coefficient along the bed joint

σ_N = vertical compression stress in the infill walls

Applying the panel dimension, maximum horizontal shear force Vf is assessed as follows:

$$V_f = \tau_o t l_m + \mu N \tag{B.6}$$

Where,

t = infill wall thickness

lm = length of infill panel

N = vertical load in infill walls

$$N = t l_m E_m \gamma^2 \tag{B.7}$$

Where,

Em = Young's modulus of the masonry

γ = the inter-storey drift angle

If deformations are small, then $V_f = 0$ because σ_N may only result from the self-weight of the panel. However, if these inter storey drifts become large, then the bounding columns impose a vertical load due to shortening of the height of the panel. The vertical shortening strain in the panel is given by (EFMA 306 Sec. 8.3.1.b, see Figure B4):

$$\varepsilon = \frac{\delta}{h} = \gamma.\frac{\Delta}{h} = \gamma^2 \tag{B8}$$

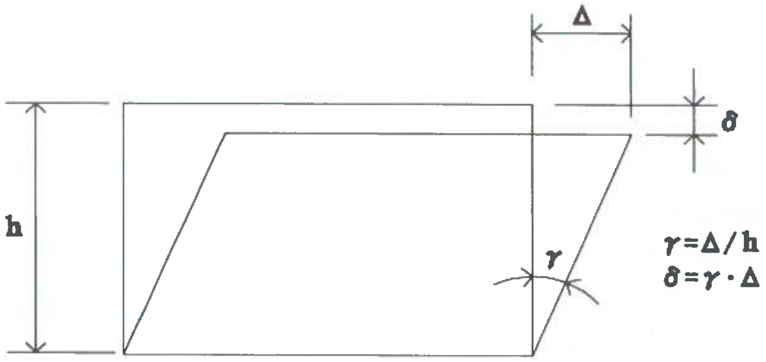


Figure B.4 Geometrical Configurations of Shear Wall Drift Where

$\delta =$ Downward movement of the upper as a result of the panel drift angle, γ

h = Inter-storey height (center – to – center of beams)

$\Delta =$ Inter-storey drift (displacement) and

$\gamma =$ Inter-storey drift angle

In this study, N is estimated directly as a summation of applied external vertical load on the panel and the vertical component of the diagonal compression force RC as shown in Figure B5.

The external vertical load is zero for the infill walls of the building and only the vertical component of the strut compression force is considered.

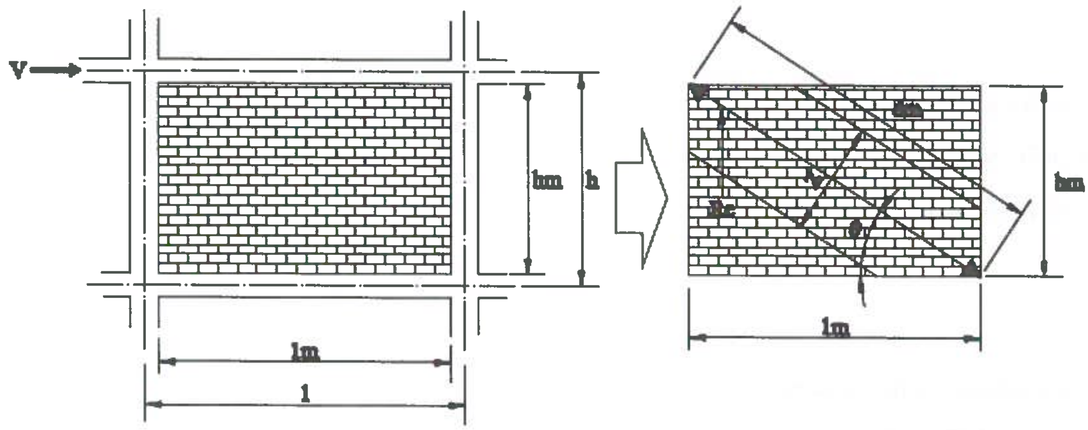


Figure B.5 Infill Masonry Walls and the Equivalent Diagonal Compression Action Parameters [*Ref.* 4]

Therefore, maximum shear force can be calculated as:

$$V_f = R_c \cos\theta = \tau_0 t l_m + \mu R_c \sin\theta \tag{B9}$$

$$V_f = \frac{\tau_0 t l_m}{(1-\mu\tan\theta)} \tag{B10}$$

Typical ranges of τ_o are $1 \le \tau_o \le 15$ kg/cm². For evaluation analysis purposes, it may be assumed typically as $\tau_o = 0.04\ f_m' = 0.04 \times 44 = 1.76$ kg/cm² by Paulay and Priestley (1992).

Although the different test methods of shear strength in masonry construction tend to give different shear strength, all indicate a strong dependence of shear stress τ_i on transverse compression stress (fm). The general form of the shear strength equation used is [Ref. B5]:

$$\tau_i = \tau_o + \mu f_m \tag{B11}$$

as shown in Figure B6:

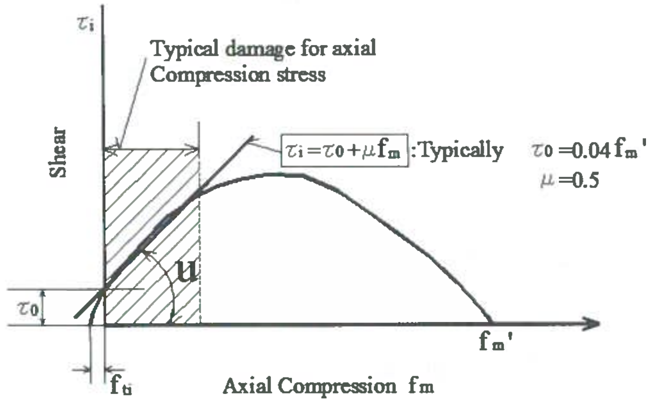


Figure B.6 Shear - Axial Compression Interaction [*Ref.* B5]

It is seen that the linear Eq. B11 is valid for the typical range of axial compression for uncracked masonry, and invalid after cracking. Values for the constants τ_0 and μ vary with test method and type of masonry.

As for μ, it is determined from the experimental results of Chen(2003) in the following equations;

$$\mu = 0.654 + 0.000515 f_j' \tag{B.12}$$

Where, f_j' is the mortar block compression strength (50 kg/cm^2).

$$\therefore \quad \mu = 0.654 + 0.000515 \times 50 = 0.68$$

Therefore, the maximum shear strength for each panel of the selected building, based on the sliding shear failure mechanism, can be obtained as;

$$V_f = \frac{1.76 t l_m}{(1 - 0.68 \tan\theta)} \tag{B.13}$$

B 3.4 COMPRESSION FAILURE

Compression failure of infill walls occurred due to the compression failure of the equivalent diagonal strut. The shearing force (horizontal component of the diagonal strut capacity) can be calculated from an equation suggested by Stafford-Smith and Carter (1969), however, the equivalent strut width Z, in Figure B5, is computed using a modification recommended by FEMA 306 (1998),

Stafford Smith (1962) suggested the shearing force (horizontal component of the diagonal strut capacity) from the following equation. The equivalent strut width z in Figure B5 is computed using a modification recommended by FEMA 306(1998).

$$V_c = ztf'_m \cos\theta \qquad \text{(B14)}$$

Where,

f'_m = Masonry compression strength, which for ungrouted clay brick masonry, (Paulay and Priestley, 1992):

f'_m = 44 kg/cm^2 = 4.4 MPa

z = equivalent strut width obtained by the following equation FEMA 306 (1988):

$$z = 0.175(\lambda h)^{-0.4} d_m \qquad \text{(B15)}$$

Where

$$\lambda = \left[\frac{E_m t \sin 2\theta}{4E_c I_g h_m}\right]^{1/4} \qquad \text{(B16)}$$

h = column height between centerlines of beams (cm)

h_m = height of infill panel (cm)

E_c = expected modulus of elasticity of frame material

= 240,000 kg/cm^2

E_m = expected modulus of elasticity of infill material

= 750 f'_m = 33,000 kg/cm^2 by Paulay and Priestley (1992)

I_g = moment of inertial of column (cm4)

d_m = diagonal length of infill panel (cm)

t = thickness of infill panel and equivalent strut (cm)

θ = angle whose tangent is the infill height - to - length aspect ratio, as:

$$\theta = \tan^{-1}\left(\frac{h_m}{l_m}\right) \qquad \text{(B17)}$$

Where,

l_m = Length of infill panel

B 4 EXAMPLE

The masonry wall shown in Figure B7 is to be designed for the equivalent strut width (Z). Clear span length 435-cm (171-in) and clear storey height 290-cm (114-in) are given as shown in the figure, and size of columns 45×45 (cm) (18×18(in)).

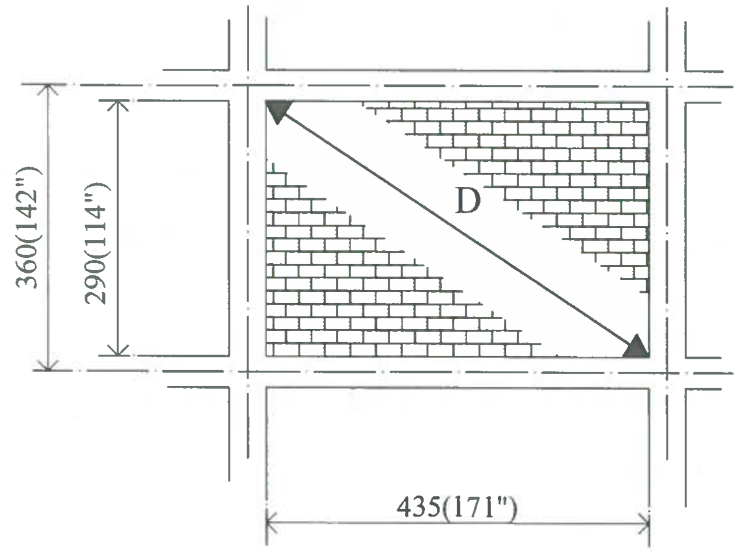


Figure B.7 Example Model of URM Infill

$$\lambda = \left[\frac{33,000 \times 20 \times 0.93}{4 \times 240,000 \times 340 \times 10} \right]^{1/4} = 0.009$$

$$D = \sqrt{290^2 + 435^2} = 520(cm) = 205"$$

$$\sin\theta = \frac{290}{520} = 0.556$$

$$\theta \approx 34^o$$

Z = 0.175 (0.009×360) - 0.4×520 = 57 cm

$$\sin 2\theta = \sin 68 \approx 0.93$$

$Vc = Z\,t\,f'm$ Cos θ = 57 × 20 × 44 × Cos 34°≈ 41 × 103 kg = 41 ton

B 4.1 MODELING OF MASONRY INFILL WALLS (WITH NO OPENING)

A masonry infill panel can be modeled by replacing the panel with a system of 2 diagonal masonry compression struts, Modan (1997). Figure B8 shows the analytical model and the strength envelope for masonry infill walls.

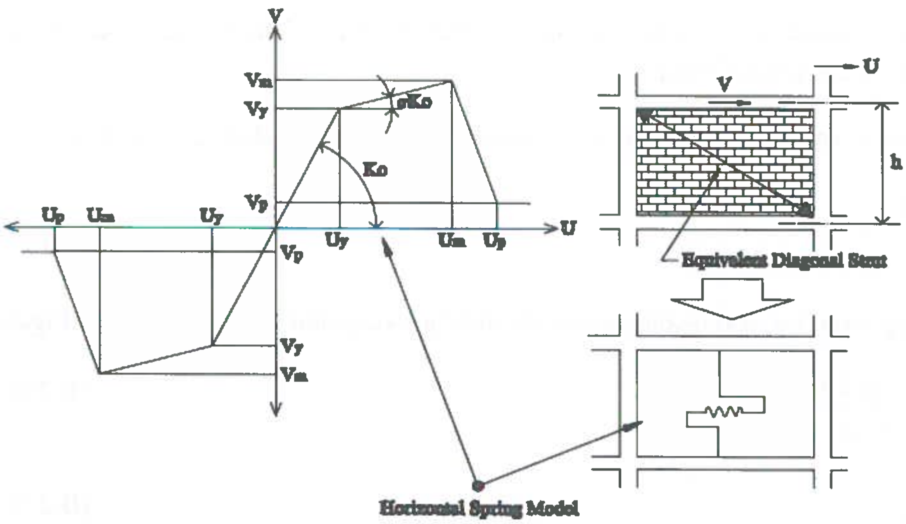
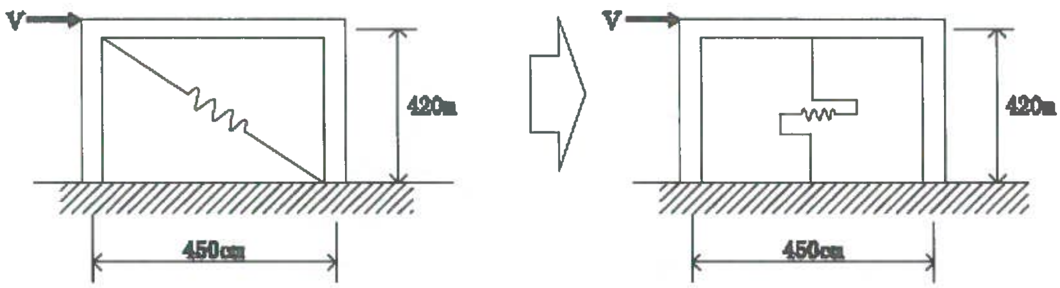


Figure B.8 Strength Envelope for Conventional Masonry Infill Walls and the Analytical Model [*Ref.* B4]

B 4.2 HORIZONTAL SPRING MODEL (WITH NO OPENINGS)



(a) Diagonal Model (b) Horizontal-Spring Model

Figure B.9 Diagonal-Spring and Horizontal-Spring as Two Masonry Infill Wall Models [Ref. B4]

Shear strength and the corresponding-displacements

The main factors of the envelop model, in Figure B8 are,

The assumed yield point: *Vy - Uy*

The maximum point: *Vm - Um*

The post-peak residual shear strength: *Vp - Up*

The maximum lateral strength, *Vm* should be obtained from the sliding shear and compression failures. The maximum displacement at the maximum lateral force is estimated by Eq. (B18), Mandan et al. (1997)

$$U_m = \frac{\varepsilon'_m d_m}{\cos\theta} = \frac{0.0018 \times 545}{0.73} = 1.34 \text{ cm} \tag{6.18}$$

Where

ε'_m = masonry compression strain at the maximum compression stress, = 0.0018

dm = diagonal strut length

The maximum drift limitation (Um/hm) is 0.8% which is implied from the experimental results of Mehrabi et al. (1996) and Chen (2003).

The initial stiffness K_o can be estimated by the following equation, Mandan et al. (1997).

$$K_0 = 2\left(\frac{V_m}{U_m}\right) \tag{B.19}$$

The lateral yielding force Vy, and displacement Uy may be calculated from geometry in Figure B8:

$$V_y = \frac{V_m - \alpha K_o U_m}{1-\alpha} \tag{B.20}$$

$$U_y = \frac{V_y}{K_0} \tag{B.21}$$

Here, the value of α is assumed to be equal to 0.2. Up and *Vp* can be defined from the previews of experimental results.

The average value of drift ratio at the 80% post-peak point, defined as a point on the envelope curve, in Figure B8, with a shear level 80% of the maximum shear strength, is about 1.5% for concrete block infill walls, Mehrabi et al., (1996). It is assumed as (3/4)1.5% = 1.0%, for solid bricks walls.

The Vp and Up should be determined considering that the line connecting the peak of the envelope and the point (Vp, Up) passes through the 80% post - peak point. Therefore;

Assuming: $V_p = 0.3\ V_m$ (B.22)
It may lead to $U_p = 3.5\ (0.01h_m) - 2.5\ U_m$ (B.23)
Eq. B23 is obtained in the following way:

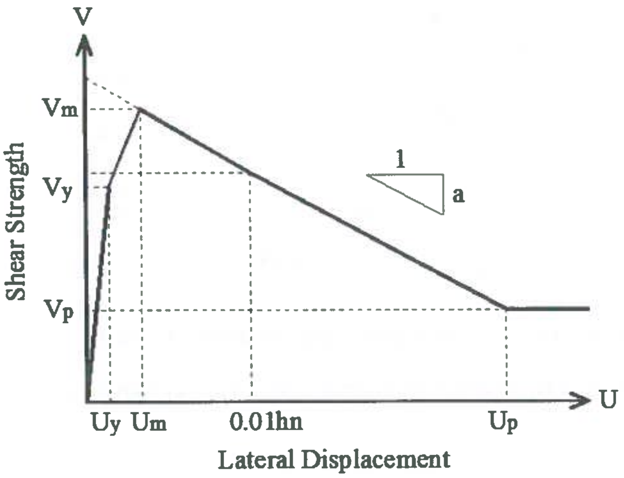


Figure B.10 Strength Envelope for Infill Wall

$$V = -aU + b \tag{B.24}$$

$$V_m = -aU_m + b \tag{B.25}$$

$$0.8V_m = -a(0.01h_n) + b \tag{B.26}$$

The numerical constants "a" and "b" are obtained from the following equations:

$$a = \frac{0.2V_m}{0.01h_n - U_m} \qquad b = V_m + \frac{0.2V_m}{0.01h_n - U_m} \times U_m$$

$$\therefore V = -\frac{0.2V_m}{0.01h_n - U_m} \times U + (Vm + \frac{0.2V_m}{0.01h_n - U_m} \times U_m)$$

When $Vp = 0.3V_m$ is substituted for V,

$$0.3V_m = -\frac{0.2V_m}{0.01h_n - U_m} \times U + (Vm + \frac{0.2V_m}{0.01h_n - U_m} \times U_m)$$

$$\therefore U = U_p = 3.5(0.01h_n - U_m) + U_m = 3.5(0.01h_n) - 2.5U_m$$

To show the applicability of the horizontal spring model instead of a diagonal spring model, an example of one - bay frame with an infill masonry wall is presented here. The infilled frame is analyzed with both horizontal and diagonal models (See Figure B11).

Pushover analysis was employed for two infilled frame models in Figure B9. As a result, the diagonal strut model and the horizontal spring have almost the same performance of infilled frames.

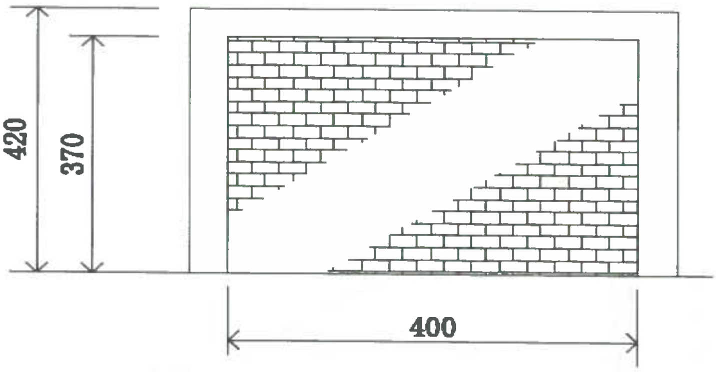


Figure B.11 Configuration of Infilled Frame

The lateral force capacity, for this wall in the compression failure mechanism, can be calculated from Eq. (B27) as:

$$V_c = ztf'_m \cos\theta \tag{B27}$$

Where

$$\lambda = \left[\frac{E_m t \sin 2\theta}{4E_c I_g h_m}\right]^{1/4} \qquad \theta = \tan^{-1}(h_m / l_m) = \tan^{-1}(370/400) = 42.76°$$

$$= \left[\frac{33,000 \times 35 sin2\theta}{4 \times 240,000 \times 833,333 \times 370}\right]^{1/4} = 0.0079 \qquad E_m = 750 f'_m$$

z = 0.175 (λh) - 0.4 dm = 0.175 (0.0079×420) - 0.4×545 = 59.03cm

$Vc/tlm = zt\, f'_m \cos\theta\ /tlm$ = 59.03×35×44×0.73/ (35×400) = 4.77 kg/cm^2

Lateral force assuming the sliding shear failure is determined by Eq. (B13):

$$V_f / tl_m = \frac{\tau_o}{1 - \mu \tan\theta} = \frac{1.76}{1 - 0.68 \times 0.925} = 4.74 kg / cm^2$$

Selecting the minimum shear strength from the two mechanisms:

$\therefore V_m = 4.74 \times 400 \times 35 = 66{,}360$ kg

From Eq. (B18):

$$U_m = \frac{\varepsilon'_m d_m}{\cos\theta} = \frac{0.0018 \times 545}{0.73} = 1.34 \text{ cm}$$

$$K_o = 2\frac{V_m}{U_m} = 2 \times \frac{66{,}360}{1.34} = 98{,}750 kg / cm$$

$$V_y = \frac{V_m - \alpha K_0 U_m}{1-\alpha} = \frac{66{,}360 - 0.2 \times 98750x1.34}{1-0.2} = 49{,}770 \text{ kg}$$

$$U_y = \frac{V_y}{K_0} = \frac{49770}{98750} = 0.504 \text{ cm}$$

By applying Eq. (B22) and (B23);

$$V_p = 0.3V_m = 19.908kg$$

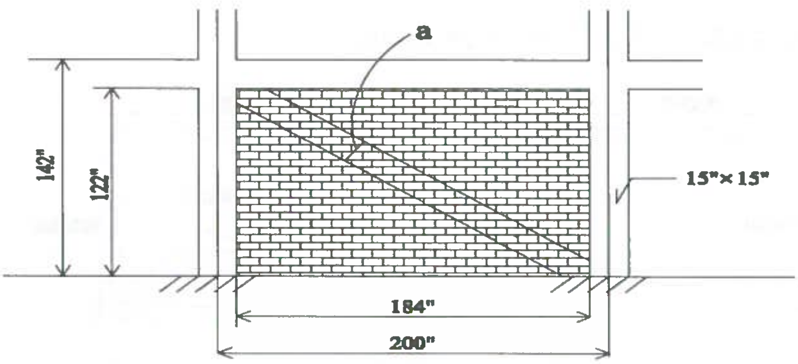
$$U_p = 3.5(0.01h_m) - 2.5U_m = 3.5 \times 0.01 \times 370 = 2.5 \times 1.34 = 9.6cm$$

The calculation example is shown below in order to compare the stiffness of the bare frame and the frame with URM infill wall, according to FEMA 273, Sec. 7.5.2.1 (see Figure B12 and B13).

The elastic in-plane stiffness of a solid unreinforced masonry infill panel prior to cracking shall be represented with an equivalent diagonal compression strut of width, a, given by equation below.

The equivalent strut shall have the same thickness and modulus of elasticity as the infill panel it represents.

$$a = 0.175(\lambda_1 h_{col})^{0.4} \gamma_{\inf}$$



Figuure B.12 Equivalent Diagonal Compression Action Parameters

Where

$$\lambda_1 = \left[\frac{E_{me} t_{\inf} \sin 2\theta}{4E_{fe} I_{col} h_{\inf}}\right]^{1/4}$$

h_{col} = Column height between centerline of beam, in.

h_{inf}= Height of infill panel, in.

E_{fe}= Expected modulus of elasticity of frame material, psi.

E_{me}= Expected modulus of elasticity of infill material, psi.

I_{col} = Moment of inertia of column, in4.

L_{inf} = Length of infill panel, in.

γ_{inf} = Diagonal length of infill panel, in.

t_{inf} = Thickness of infill panel and equivalent strut, in.

θ = Angle whose tangent is the infill height-to-length aspect ratio, radians.

λ_1= Coefficient used to determine equivalent width of infill strut.

h_{col} = 142 in.

h_{inf} = 122 in.

E_{fe} = 3000×103 psi

E_{me} = 300×550 = 165×103 psi

(According to FEMA 273, Sec. 7.3.2.1, the masonry compressive strength in poor condition is 3000 psi. The modulus of elasticity is 550 times the expected masonry compressive strength)

$$I_{col} = \frac{15\times15}{12} = 4219 \text{ in}^4 \qquad t_{\inf} = 10L_{\inf} = 184''$$

$$\theta = \sin^{-1}\left(\frac{122}{220}\right) = 33.5^o \qquad \gamma_{\inf} = \sqrt{122^2 + 184^2} = 220''$$

$$\lambda_1 = \left[\frac{E_{me} t_{\inf} \sin 2\theta}{4E_{fe} I_{col} h_{\inf}}\right]^{1/4} = \left[\frac{165\times10\times0.92}{4\times3000\times4219\times122}\right]^{1/4} = 0.022$$

$$\therefore a = 0.175 \times 220(0.022 \times 132)^{-0.4} = 25''(\fallingdotseq 64\text{cm})$$

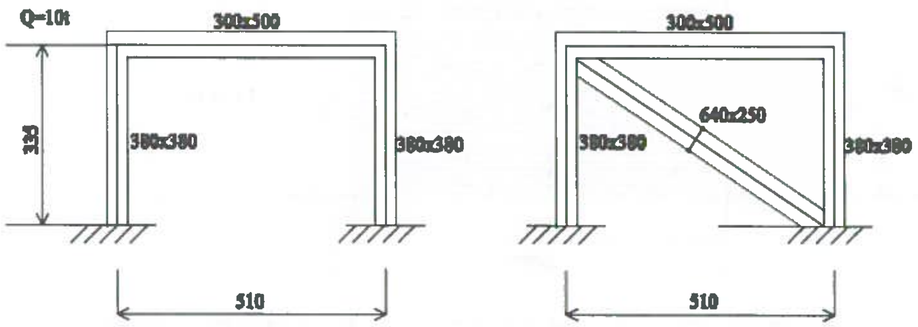


Figure B.13 Modeling of Bare Frame and Strut Frame

Frame Stiffness KB = Q/δ=10/0.59 =17t/cm; Frame Stiffness KURM =Q/δ=10/0.13= 77t/cm

The stiffness of frame with URM in fill is approximately 4.5 times the bare frame.

An example of the multi-spring infilled frame is illustrated in Figure B14. The result of the pushover analysis, for the infilled frame in Figure B14, are illustrated in Figure B15. The results show multi-spring (S1, S2, S3) model and the single equivalent (Se) model responses are almost the same.

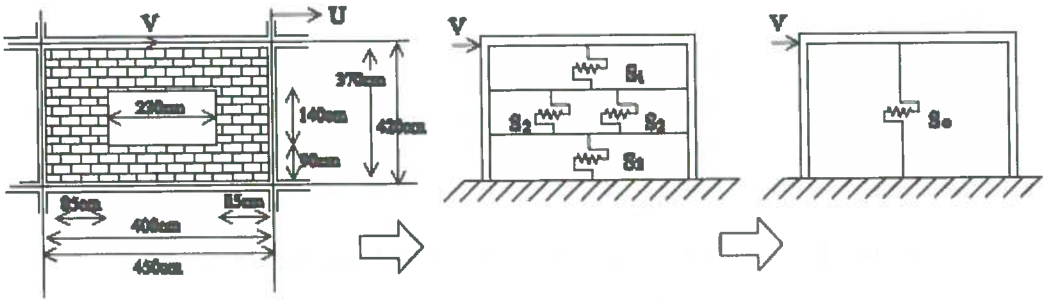


Figure B.14 An Unfilled Frame with Window Opening and its Equivalent with a Single-Spring [*Ref.* B4]

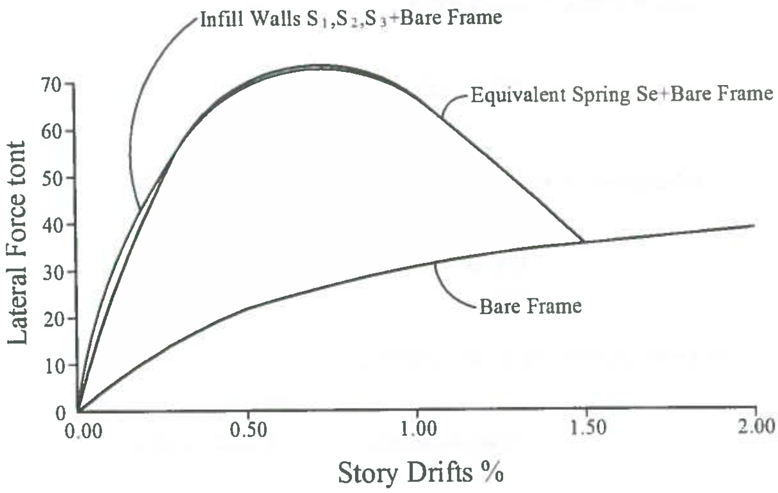


Figure B.15 Correlation of Multi-Spring Model and Equivalent Single-Spring Model Responses [*Ref.* B4]

B 5 MASONRY INFILL IN OUT-OF-PLANE

As mentioned before, there are many buildings with unreinforced masonry infill wall in Bangladesh and they will be constructed in future also.

Masonry infill walls in-plane have the problems such as the soft-storey effect, the horizontal eccentricities and vertical stiffness irregularity.

On the other hand, masonry infill walls out-of-plane also have the serious problem, and the failure of the face loaded wall will occur "out from the building", not "into the building" by seismic force.

The response acceleration of exterior walls loaded in out-of-plane direction may be much greater than the floor accelerations, due to the possibility of resonant response as a result of near coincidence of natural period of out-of-plane response of the wall, and transverse in-plane response of the structure as a whole. Seismic load and energy path are shown in Figure B16 and Figure B17 respectively [Ref. B6].

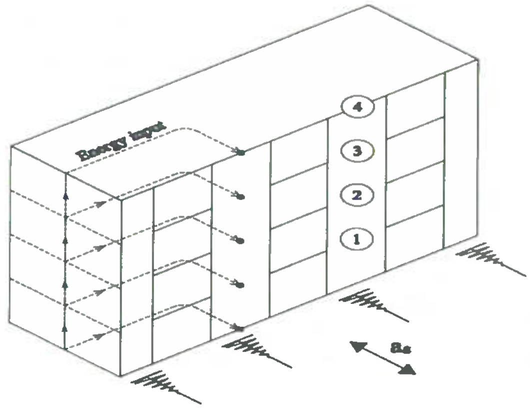


Figure B.16 Seismic Load Path for URM Buildings [*Ref.* B6]

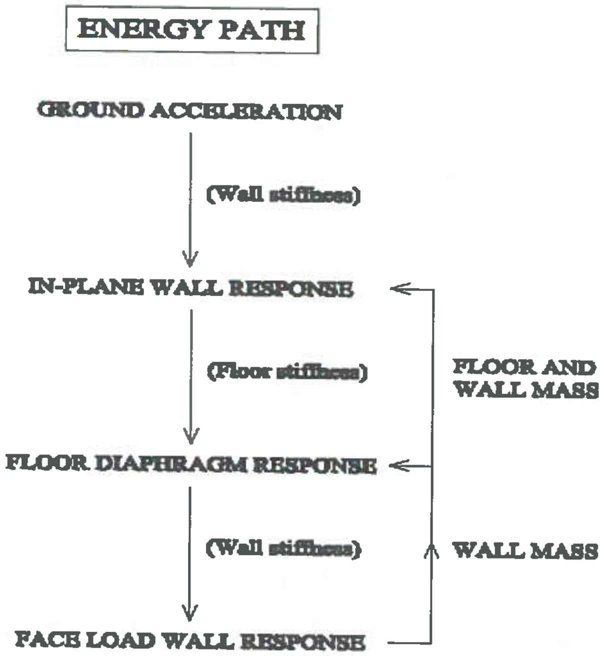


Figure B.17 Seismic Energy Path [*Ref.* B6]

Unreinforced masonry walls subjected out-of-plane excitation must resist their own inertia forces, determined from the wall self-weight multiplied by the acceleration at the mid height of the wall.

The maximum out-of-plane acceleration will produce maximum distributed inertia forces acting transversely to the plane of the wall and forcing the wall to bend like a floor slab

When the URM infill wall out-of- plane is designed, two models are considered. One is a two-way action by constraining the infill to the columns as well as the top beam and bottom support, the other is a one–way action as a single support beam that spans from the top beam to bottom beam. The crack pattern proves that the masonry wall loaded out-of-plane behaves very much like a solid 2-way reinforced concrete slab

As an Example, a design method of the reinforced masonry wall modeled by a one-way action is presented here. The seismic lateral force is based on the regulation by Japanese Code.

According to the Japanese code, in a case where a building with four or more stories excluding the basement level or building higher than 20 m, and in which an elevator tower or other part similar thereto protrudes above the roof of the building or outside stair or other part similar thereto protrudes from the outside wall of the buildings installed, the forces produced in the said part or in elements essential for structural resistance connected to the said part under the acting loads and external forces (seismic force shall be $P = kW = 1.0 \times$ W) are calculated, and the structural calculation are performed to confirm safety. An example is shown below [Ref. 6.7].

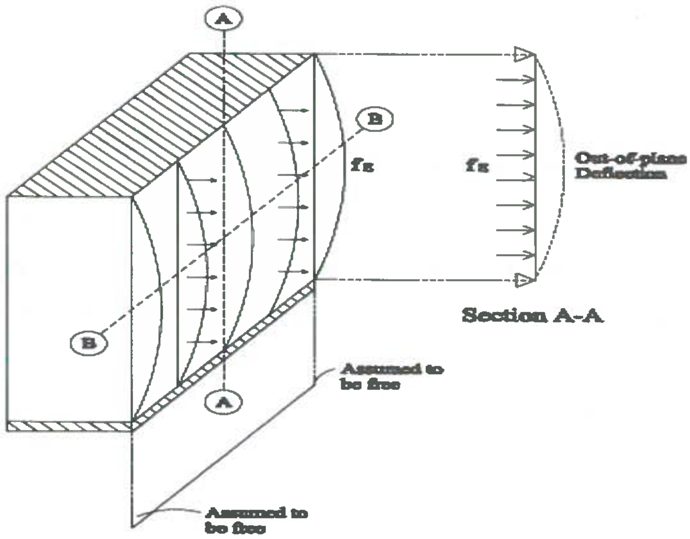


Figure B.18 Out-Of-Plane Deflection of Wall Modeled with Side Free under Inertia Force [Ref. B7]

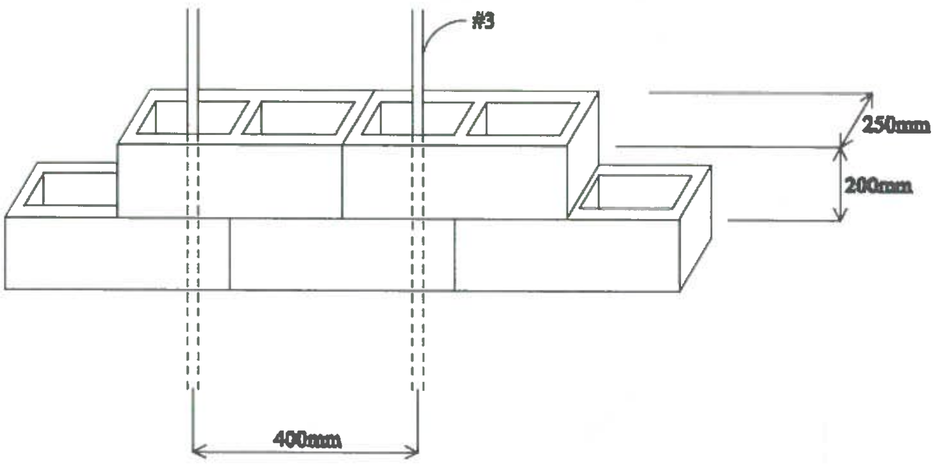


Figure B.19 Reinforced Masonry

$fE = m(a)$

fE = distributed seismic forces that act out-of-plane of the wall

m = mass per unit area of the wall
a = out-of-plane acceleration (a = 1.0(g) according to the Japanese Code)
fE =(1.0(g) × w/g = 1.0 × w
w = 0.25(1.0) (1.0)20 kN/m3 + 0.05(1.0) (1.0) 20 kN/m^3 = 6 kN/m^2
fE = 1.0 × 6.0 kN/ m^2 = 6.0 kN/m
Clear storey height "H" = 3.50 m
Seismic load intensity in Dhaka is assumed to be one-third of the Japanese one.

$M_o = (fE \times 1/3) \times H2 / 8$
$= (6.0 \times 1/3) \times 3.52 / 8 = 3.6$ kNm/m
D = 250 mm, d = 125 mm, j = 0.875 × d = 10.9 cm, f_y = 414 MPa = 60 ksi
req $A_t = M/(f_y \times j) = 360/(414 \times 10.9) = 0.8$ cm^2 /m
One deformed bar of #3 is placed at every 400 mm of the longitudinal size of the block. (A provided = 0.72 × 2.5 = 1.8 cm^2 /m)

B 6. MASONRY INFILL WALL WITH OPENINGS

The effect of opening in the lateral stiffness of masonry infill walls is evaluated by considering to the reduction factor (R1)i. The equivalent strut width, z, shall be multiplied by a reduction factor (R1)i to account for the loss in strength due to the opening. The reduction factor (R1)i is calculated using Eq. B27 [Ref. B8]:

(R1) = 0.6(Aopen/Apanel) 2-1.6(Aopen/Apanel) + 1 (B.27)

z red = (R1) i× z (B.28)

Where
Aopen = area of the opening
Apanel= area of the infill panel
Note : If the area of the openings (Aopen) is greater than or equal to 60 percent of the area of the infill panel (Apanel), then the effect of the infill should be neglected, i.e., (R1)i = 0.
Eq. B28 is represented by Figure B20 below:

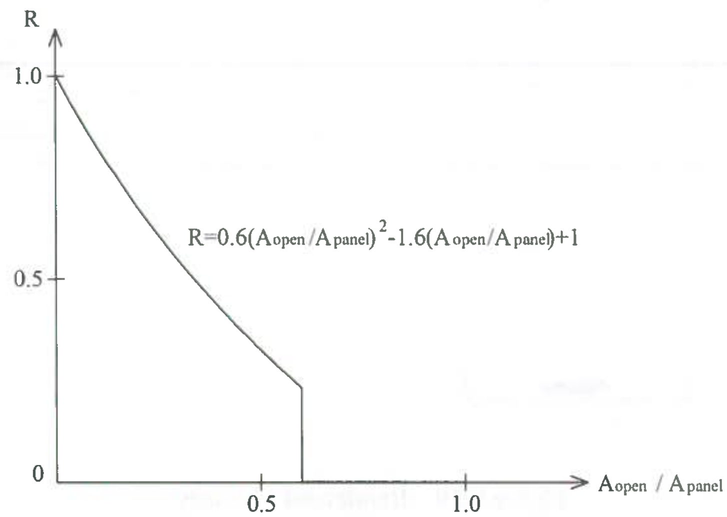


Figure B.20 Infill Panel Stiffness Reduction Factor in Relation to the Opening Percentage [*Ref.* B8]

According to the technical report by Panagiotis G. Asteris et. al, using a finite element technique, analytical results are presented on the influence of the opening size on the seismic response of masonry infilled frames. Figure B21 shows the variation of the λ factor as a function of the opening percentage (opening area/infill wall area), for the case of an opening on the compressed diagonal of the infill wall (with aspect ratio of the opening the same as that of the infill). As expected, the increase in the opening percentage leads to a decrease in the opening percentage leads to a decrease in the frame's stiffness. Specifically, for an opening percentage greater than 50% the stiffness reduction factor tends to zero.

Although considerable difference of the stiffness reduction factor between Figure B20 and B21 exists, it is almost same that the stiffness reduction factor will be zero for the opening percentage greater than around 50% or 60%.

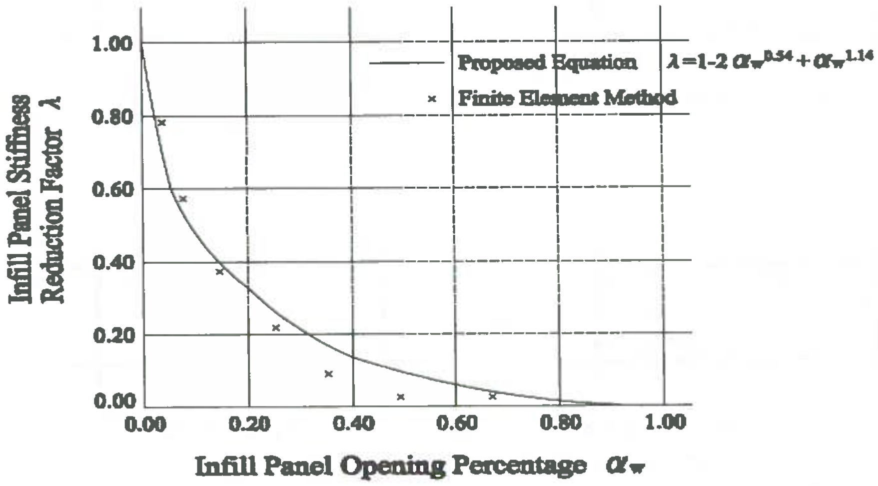


Figure B.21 Infill Panel Stiffness Reduction Factor in Relation to the Opening Percentage [*Ref.* B9]

$$\lambda = 1\text{-}2\alpha w0.54 + \alpha w1.14 \qquad \text{(B.29)}$$

in which αw is the infill wall opening percentage (area of opening to the area of infill wall).

When the influence of openings on the behavior of URM infills is considered, the followings figures are introduced as a practical engineering judgments.

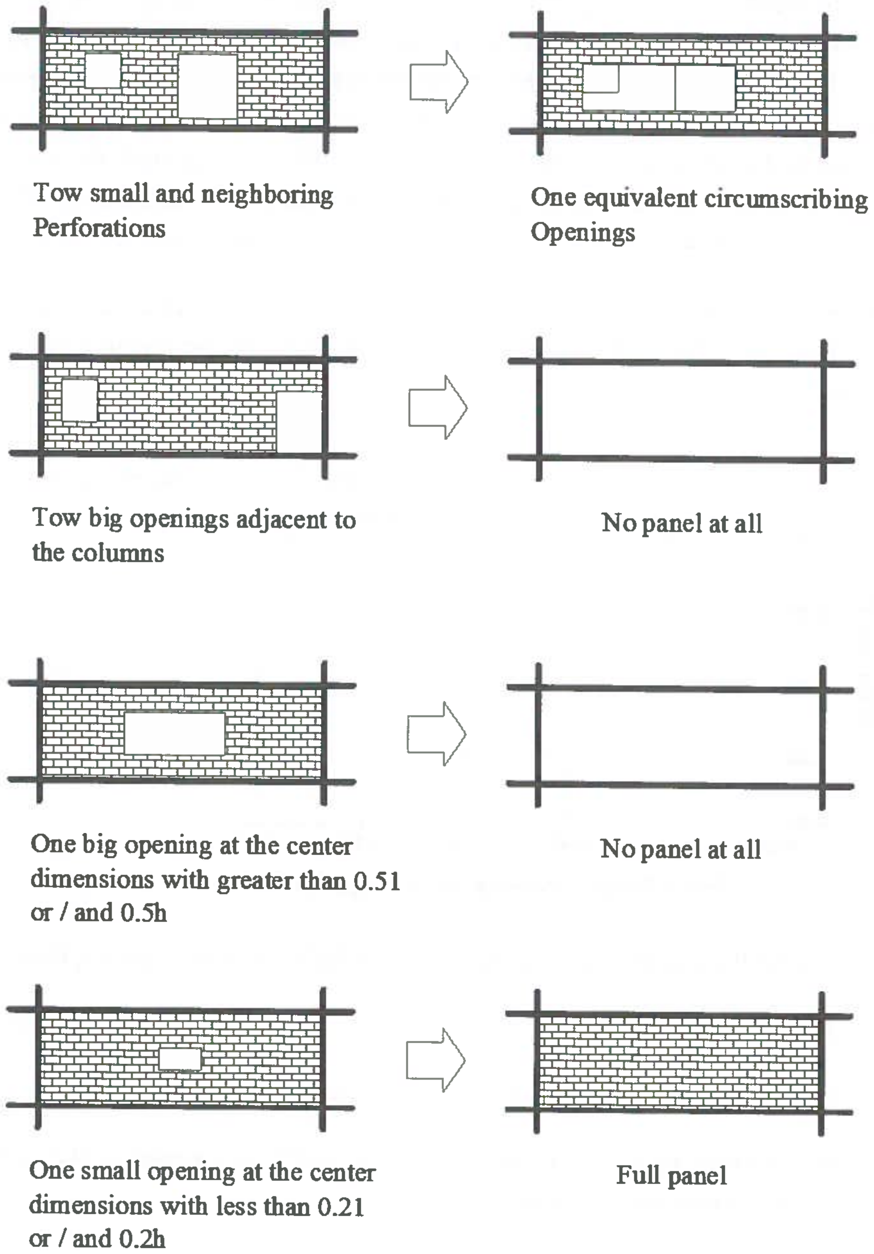


Figure B.22 "Black" and "White" Decisions Regarding Openings of URM Infills [*Ref.* B10]

B 7 MASONRY INFILL IN OUT-OF-PLANE WITH SHORT COLUMN

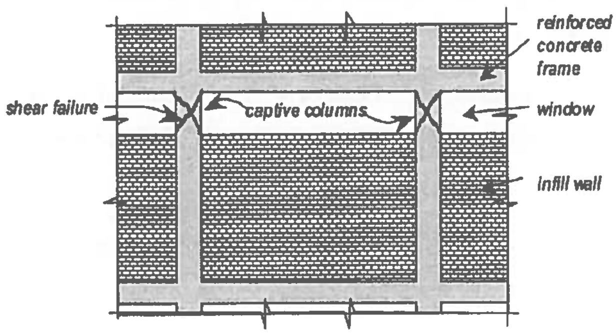


Figure B.23 Captive Short Column Effect [*Ref.* B 11.]

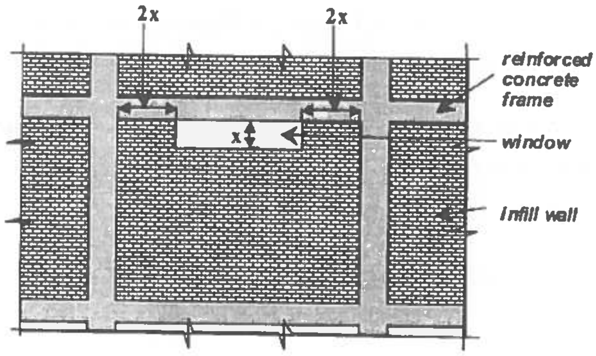


Figure B24 Alternative to account for the short or captive [*Ref.* B 11.]

The influence of masonry infills on the structure's lateral force behavior such as captive or short columns (Figure B23) should be addressed by the designer, either by considering such influence in the design process or by isolating the masonry from the structure. In the second alternative, appropriate measures should be taken to maintain the out-of-plane stability of the masonry when subjected to seismic or wind lateral loads.

Two alternative corrective measures should be considered. One is to separate the infill wall from the column by providing a gap between wall and column. It should be approximately 1.5% of the storey height. The masonry wall should be anchored to prevent its overturning when subjected to out-of-plane lateral loads. The other is to locate a much shorter window in the central part of the span. The distance between the column face and the window should be at least twice the vertical dimension of the gap left by the window. (Figure B.24)

If both corrective measures are not satisfied, hoops should be provided over the full column length as indicated by Sec.2.11.2.5 of the Manual.

References:

B1 "Evaluation of Confined Masonry Guideline for Earthquake-Resistant Housing" prepared by UBC EERI Standard Chapter Committee on Confined Masonry Construction.

B2 "SEISMIC RETROFIT OF UNREINFORCED MASONRY INFILLS IN NON-DUCTILE REINFORCED CONCRETE FRAMES USNG ENGINEERED CEMENTITIOUS COMPOSITES" by MARIOS A. Kyriakides March 2011, p.80.

B3 "SEISMIC DESIGN OF REINFORCED CONCRETE and MASONRY BUILDINGS" by T.Paulay, M.J.N. Priestley p.110.

B4 Hossein Mostafaei and Toshimi Kabeyasawa; Effect of Infill Walls on the Seismic Response of Reinforced Concrete Buildings Subjected to the 2003 Ban Earthquake Motion: A Case Study of Ban Telephone Center, Bull. Earthq. Res. Inst. Univ. Tokyo Vol. 79(2004) pp. 133-156.

B5 "SEISMIC DESIGN OF REINFORCED CONCRETE and MASONRY BUILDINGS" by T.Paulay, M.J.N. Preistley p.565.

B6 M.J.N. Priestley; SEISMIC BEHAVIOR OF UNREINFORCED MASONRY WALLS, BULLETIN OF THE NEW ZEALAND NATIONAL SOCIETY OF EARTHQUAKE ENGINEERING, Vol.18, No.2 June 1985 p.194.

B7 Recep KANIT, Ergin ATIMTAY; Experimental Assessment of the Seismic Behavior of Load-Bearing Masonry Walls Loaded Out of Plane, Turkish J. Eng. Sci 30(2006), p.111.

B8 US Army Corps of Engineers, Engineer Research and Development Center, ERDC/CERL TR-02-01 p.20 & p.62.

B9 Panagiotis G. Asteris, Ioannis P. Giannopoulos and Christis Z. Chrysostomou; Modeling of Infilled Frames With Openings, The Open Construction and Building Technology Journal 2012.6.(Suppl1-M6) p.84.

B10 M. Chronopoulos and P. Chronopoulos; Recent Greek Provisions For Re Structures with Urm Infills, The Open Construction and Building Technology Journal 2012.6.(Suppl1-M7) p.97.

APPENDIX C. CASE STUDY ON SEISMIC CAPACITY OF RC BUILDING FRAME WITH THE EFFECT OF INFILL BRICK MASONRY WALL

C 1 INTRODUCTION

In Bangladesh, development of design and construction of RC buildings are not satisfactory in many cases considering the current seismic demand specified in the National Building Code. The Public Works Department (PWD) is attempting to upgrade those public and private buildings with the technical cooperation of JICA. The CNCRP Project has been in operation since March, 2011 to help achieve this, and several buildings in Bangladesh are being assessed and retrofitted as a result. The manual devised for the analysis of buildings in Bangladesh is mainly based on JBDPA (The Japan Building Disaster Prevention Association) Standard procedure. However, as this procedure does not take into consideration the effect of infill brick masonry walls within RC frame buildings, the aim of this study is mainly to address this issue.

A Pushover Analysis Method following guidelines in ATC-40/FEMA 273/FEMA 356/FEMA 440 EL was considered. Infill brick masonry was modeled as a diagonal strut, and the ETABS v13.2.2 used as the structural analysis tool. A double bay typical frame of the model building has been considered to understand the seismic behavior.

The model building was a multi-purpose building used by the Bangladesh government secretariat as garage cum office building. The Public Works Department (PWD) is involved in the construction and routine maintenance of the building. It is a five-storied RC framed building with infill brick masonry, constructed in 1985, i.e., before the first Bangladesh National Building Code (BNBC 1993) was implemented in 1993. At the same time, a building was designed considering the same architectural design and the BNBC 1993 guidelines to understand the seismic behavior. The outcome of the assessment considering the building designed by BNBC has been summarized in this appendix C.

To calibrate the analysis procedure using ETABS v13.2.2 software, several test results on building frames performed as part of CNCRP.

C 2 CALIBRATION OF STRUCTURAL TEST DATA AND SOFTWAREANALYSIS DATA

To calibrate the software analysis and the laboratory test data, several models which confirmed to the test frame criteria were prepared and analyzed. Brick infill walls were modeled as "diagonalstrut" of structural equivalence.

C 2.1 MATERIAL PROPERTIES OF RC FRAME AND BRICK MASONRY WALL

For the software analysis, the compressive strength of concrete was considered f'_c = 10.6 MPa and the rebar yield strength has been considered f_y = 275 MPa as per the laboratory test results. The structural tests frames were designed following the actual building configuration in Bangladesh. High axial load ration about 0.67, low compressive strength concrete were considered.

There was no laboratory test data for brick units or the masonry wall unit. Several considerations have been made to establish Compressive Strength (f'_m) and Modulus of Elasticity (E_m) of infill masonry brick wall. As shown in BNBC 1993, the compressive strength of masonry wall (f'_m) considered 3 MPa, where, the ratio of cement and sand is 1:6 common practice in the construction sector of Bangladesh. Modulus of Elasticity (E_m) was set as per BNBC 1993 guidelines as $E_m = 750\text{x}f'_m = 750\text{x}3 = 2250$ MPa.

Several evaluations were made to verify the values of f'_m and E_m. ACI 530.1-02 gives the minimum compressive strength of masonry wall (f'_m) as 6.9 MPa which includes the compressive strength of a masonry unit is 14.48 MPa and N type (5.2 MPa as per ASTM C270.) mortar, which is much higher in comparison to BNBC 1993.

The Modulus of Elasticity (E_m) was verified with the state-of-the-art methods of computation as shown in the Figure C.1. Here, $E_m = k\,x\,f'_m$, k is variable ($k = 200$ to 2000) depending on methodology.

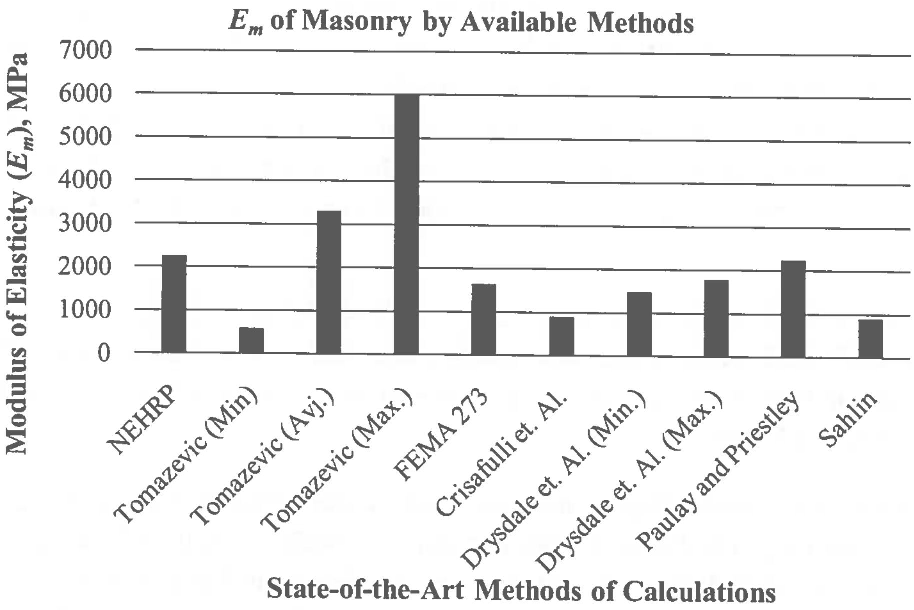


Figure C.1 Graph Showing Modulus of Elasticity of Infill Brick Masonry Walls by using State-of-the-Art Methods of Computation

C 2.2 COMPUTATION AND FIXATION OF EQUIVALENT DIAGONAL STRUT WIDTH

A comparative study was performed to obtain the width of an equivalent diagonal strut from the test specimens. There is no indication for about such computation in BNBC 1993; however, several non-Bangladeshi publications and guidelines provide methods to computing an equivalent diagonal strut width. Generally, the formulae either give a ratio with the diagonal length of the strut or the stiffness of the frame materials and the infill materials. The strut width computed with the state-of-the-art methods are varies from 156 mm to 770 mm for specimen no. 4 as shown in the Figure C.1. The equivalent strut width for the test specimen was 386 mm, following "Paulay and Priestley", which provided analysis result closest to the test results.

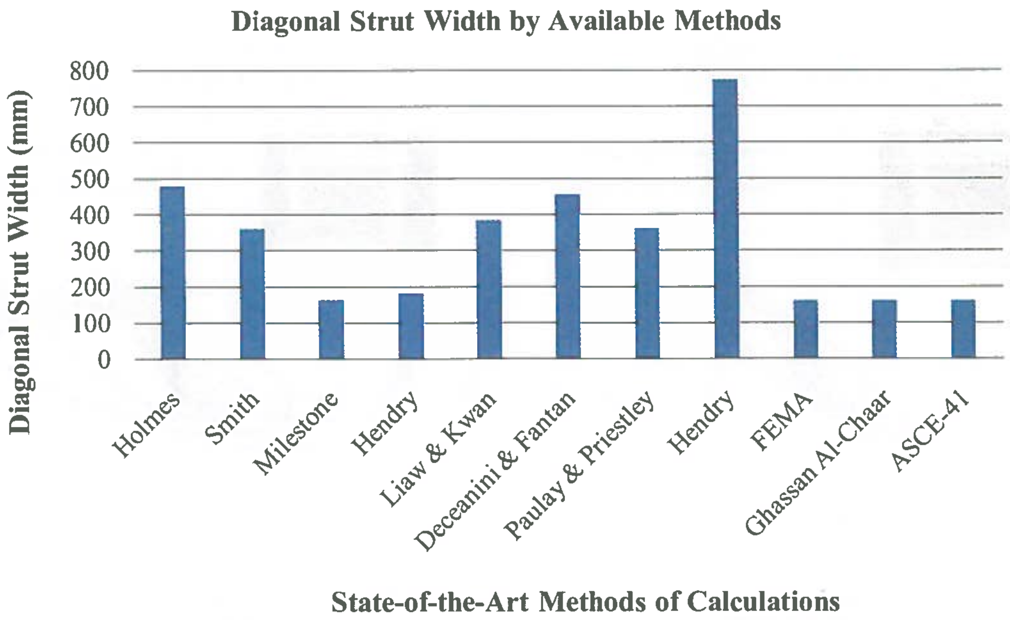


Figure C.2 Graph Showing Width of Infill Brick Masonry Wall by Using State – of – the - Arts Methods of Computation

C 2.3 CONSIDERATIONS FOR SOFTWARE MODELING AND ANALYSIS

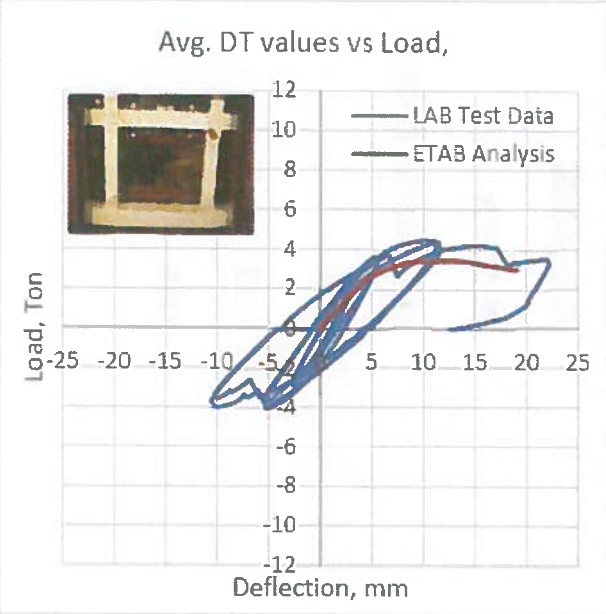
In the ETABS analysis, the equivalent strut has been modeled as a brittle axial hinge in the center and pin connected to the RC frame members. Axial, flexural fiber hinges and shear hinges were considered for the columns. For the beam, flexural and shear hinge mechanisms were considered. The hinge properties in ETABS follow the ASCE 41 and relevant guidelines. The pushover analysis was displacement-based.

C 2.4 COMPARISON OF STRUCTURAL TEST DATA AND SOFTWARE ANALYSIS

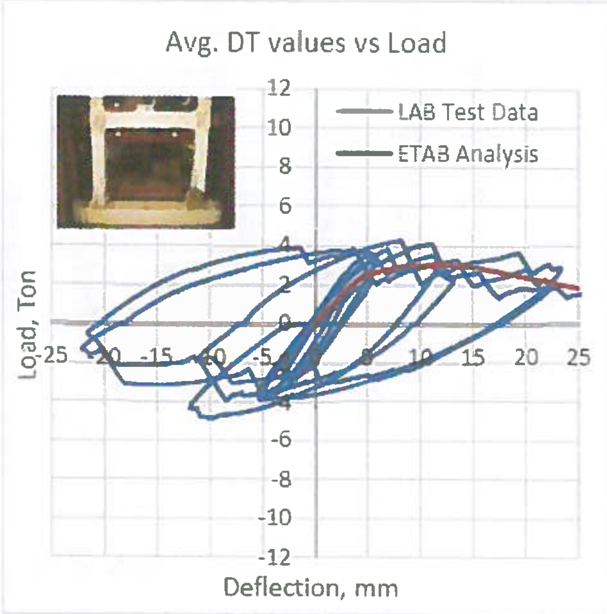
From ETABS analyses using the above criteria, pushover curves were plotted on the hysteresis curves obtain from laboratory test results as shown in Figure C.3. Specimen no. 1 and 2 provided very good simulation for test results and software analysis as shown in the Figure C.3 and the Table C.1.

The ratio between the test result and the analysis result is 77% and 72%. But in the case of specimens no. 3 the shear capacity ratio reached 84% and in case of specimen no. 4, the test result and analysis result ratio is 64%. In case of specimen no. 4, the shear capacity was much higher than the calculated shear capacity of columns and infill brick masonry.

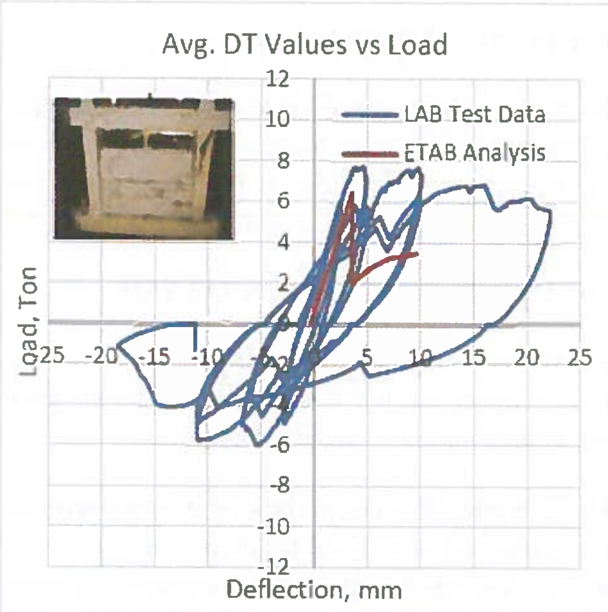
There may have scope of shear increment of the infill masonry wall due to high axial load of columns. The confinement of infill brick masonry by beam and column and high axial load on columns seems to increase the overall shear capacity of the frame which is not a factor in the equivalent diagonal strut width calculation or the shear modulus of the infill masonry wall. The considerations on compressive strength and elastic modulus of masonry infill need more tests and analysis.



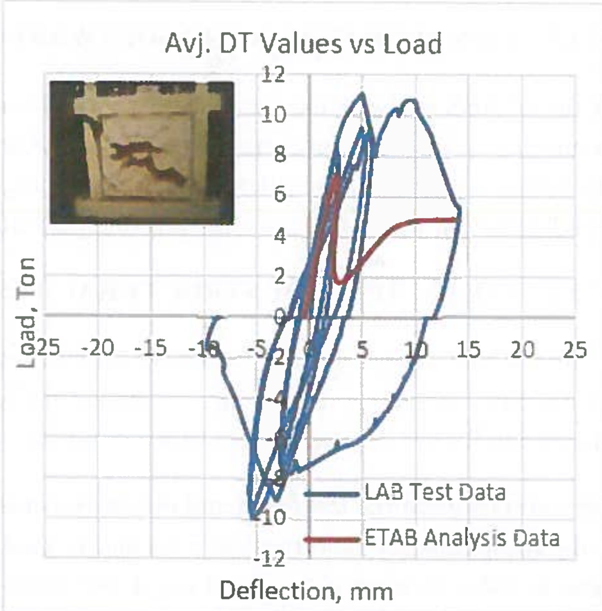
(a) **Specimen no. 1**



(b) **Specimen no. 2**



(c) **Specimen no. 3**



(d) **Specimen no. 4**

Figure C.3 Comparison of Structural Test and the Software Analysis Graphs

Table C.1 Comparison of Maximum Force and Displacement in Structural Tests and Software Analysis

Specimen No.	Frame Description	Maximum capacity		
		Test (Tons)	Analysis (Tons)	Ratio (ETAB/Tests)
1	Bare frame continuous beam	4.54	3.48	0.77
2	Bare frame with discontinuous beam	4.28	3.05	0.72
3	Partial infill masonry wall	7.75	6.46	0.84
4	Fully infill masonry wall	11.11	7.15	0.65

C 3.1 PERFORMANCE BASED ANALYSIS OF MODEL BUILDING FRAME (DESIGNED BY BNBC 1993)

The model design was based on an existing building and the structural considerations of BNBC 1993. The seismic zone coefficient was Z = 0.15, response modification factor R = 8, structural importance factor I = 1, soil type S3. Live load was 3.0 kN/Sq. meter, floor finish and other loads were considered as per code. Material strength parameters for the design were: compressive strength of concrete f'_c = 24 MPa and yield tensile strength of reinforcing steel was f_y = 400 MPa. The compressive strength of infill brick masonry wall was considered f'_m = 3.0 MPa.

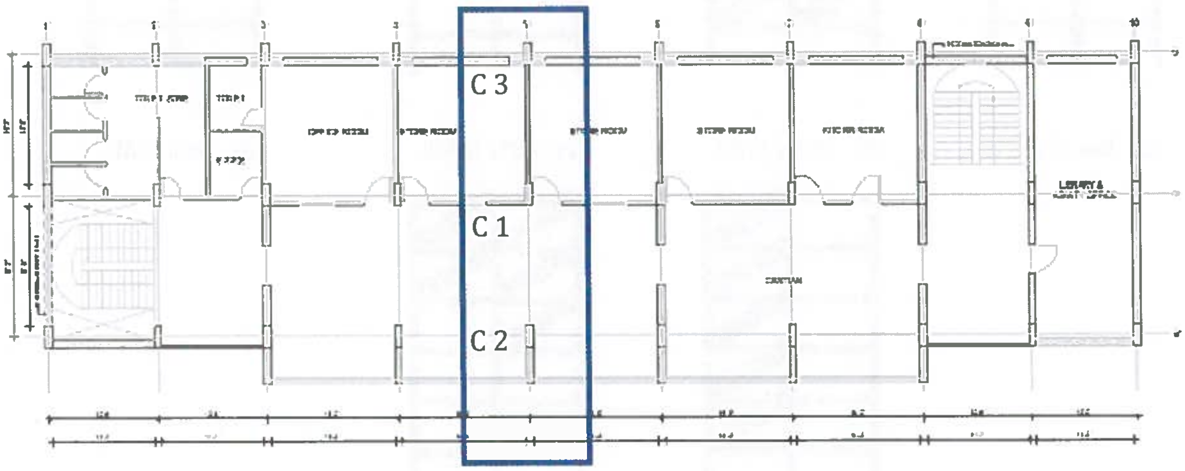


Figure C.4 Typical Architectural Floor Plan of the Model Building

As shown in the Figure C.4, the building is a multi-purpose office cum dormitory building having infill brick masonry wall as partitions. The panel (05-05) was considered for the analysis process. The building is a five storied building and the ground floor is open for car parking.

Table C.2 and C.3 shows the member dimension and reinforcement plan of the typical columns, beams and grade beams which are considered during analysis.

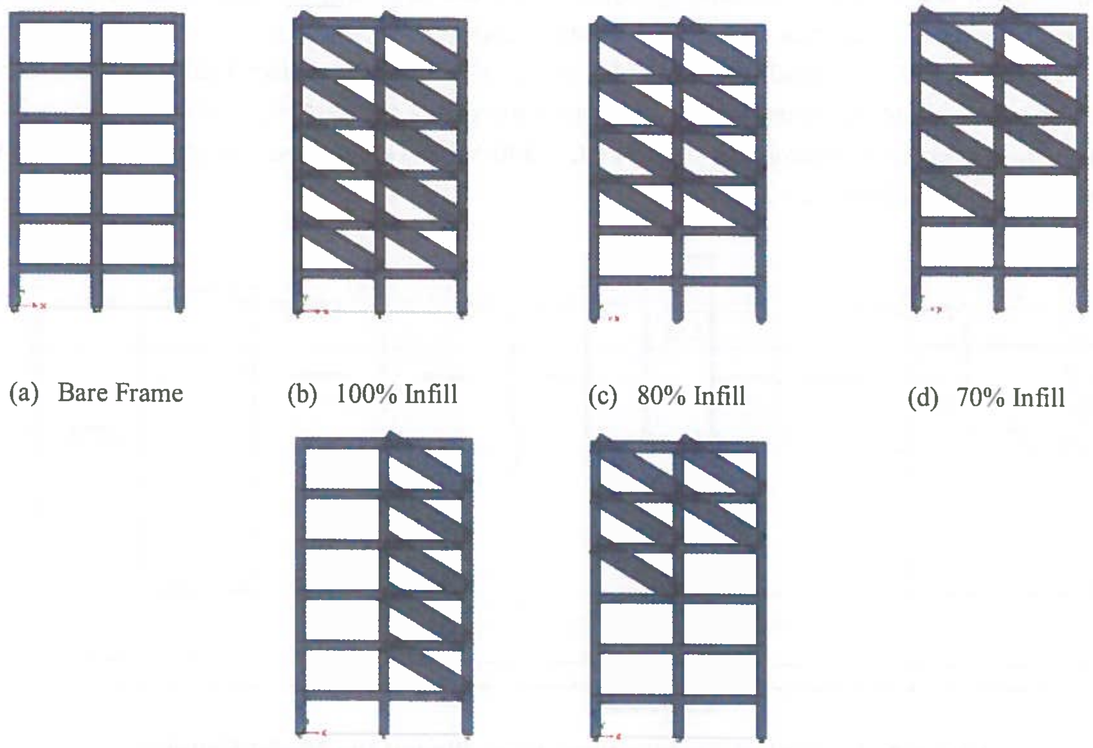
Table C.2 Study Frame Column Rebar Details

Column Mark	Size (B × D), mm	Main Rebar up to 3rd Floor	Main Rebar 4th and 5th Floor	Shear reinforcement/Ties
C-1	300 × 600	12-D 20	12-D16	D10 @100 mm c/c
C-2	300 × 500	12-d 20 mm	12-d16mm	d10 @100 mm c/c
C-3	300 × 500	12-d 20 mm	12-d16mm	d10 @100 mm c/c

Table C.3 Study Frame Beam Rebar Details

Beam Mark	Size (B × W), mm	Main Rebar at Top	Main Rebar at Bottom	Shear Rebar/Stirrups (mm)
B2	300 × 500	6-d 20mm	4-d 20mm	d10 @100/200/100 c/c
GB	300 × 450	3-d 20mm	3-d 20mm	d10 @100/200/100 c/c

As shown in the Figure C.5, the frames for analyses were taken as bare frame and frame with infill walls setups. The placements of walls have been selected considering the masonry infill use pattern in different buildings of Bangladesh. There are many buildings with 100% infill masonry wall above grade beam pattern as shown in the figure. The soft storey effect is an important issue in Bangladesh buildings having lower one or two storey as car parking.



(a) Bare Frame (b) 100% Infill (c) 80% Infill (d) 70% Infill

(e) 50% Infill- 1 Bay (f) 50% Infill- 2 Bays

Figure C.5 Study Frames of the Model Building with Different Infill Quantities and Arrangements as Modeled in ETAB

80% infill masonry wall pattern frame represents the issue. There are many buildings in the country having one bay open for corridor, especially in office, educational institute etc. and some building are open in the lower stories for market, and factory etc. and upper part is residential. So, all these frames are considering the present situation and to understand the behavior.

Figure C.6 shows the maximum displacement profile for bare frame and different infill setups. The maximum displacement behavior represents the vertical irregularity of the building, particularly the soft storey behavior on the ground floor which is not included in the design considerations.

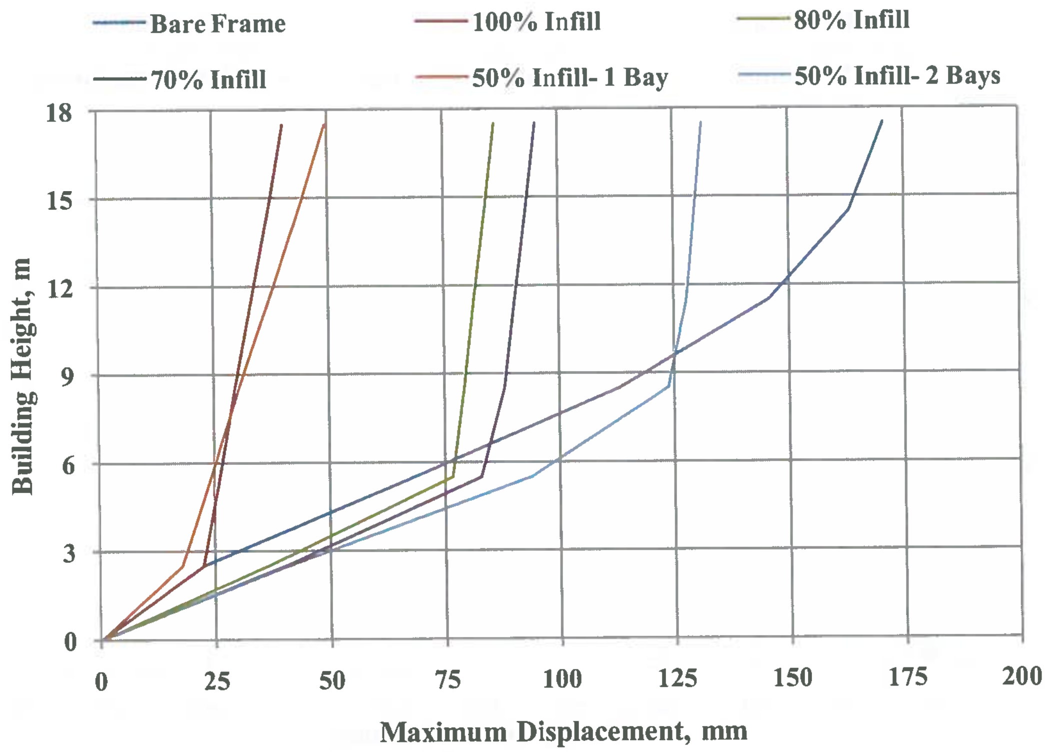


Figure C.6 Maximum Displacement Profiles Comparison for Bare Frame and Infilled Frames with Various Infill Quantities

Figure C.7 shows the pushover curves of the frame in bare and infilled frames. The pushover curves were influenced by the quantity and placement style of the infill brick masonry. Compared to bare frames shear capacity increased and displacement ability decreased with infilled frames.

Figure C.8 shows the variation in base shear ratios (lateral shear capacity of infilled frame divided by that of bare frame) between bare frame and the infilled frames. Base shear ratio increased with infilled frames with different percentages of the infill masonry wall area in respect to the total area of the frames. The shear ratio also depended upon the placement pattern of the walls.

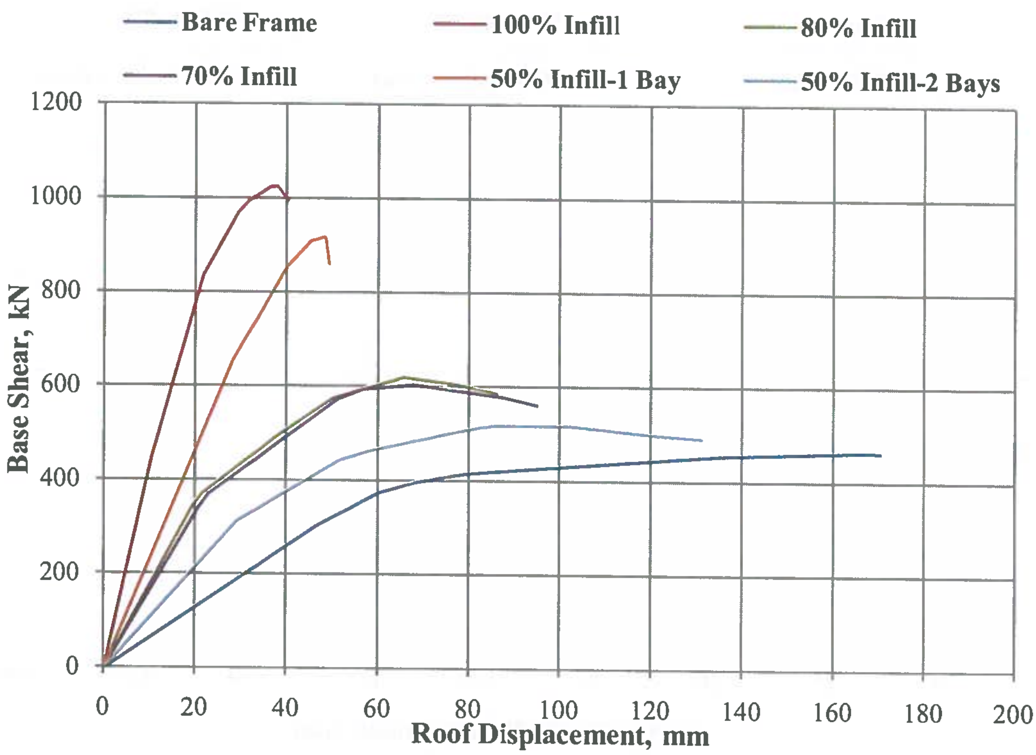


Figure C.7 Comparison of Pushover Curves of Bare Frame and Infilled Frames with Various Infill Quantities

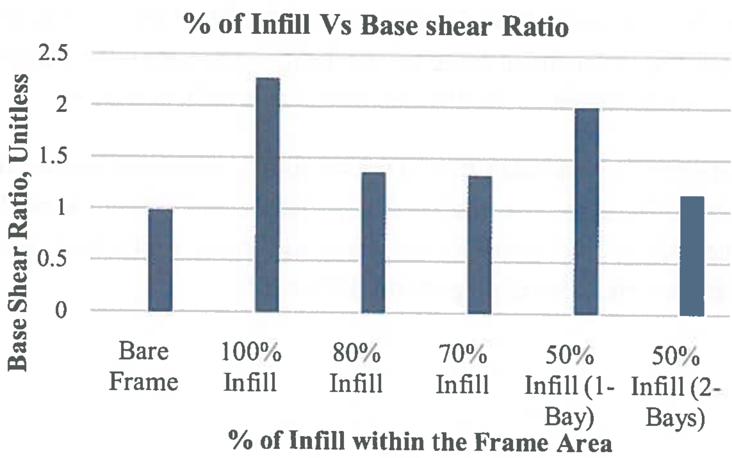


Figure C.8 Comparison of Base Shear Ratio of Bare Frame and Infilled Frames with Various Infill Quantities

Figure C.9 shows the variation of stiffness between bare frame and the infilled frames. Different percentages and placement style of the infill masonry wall increased the stiffness of the frame and negatively affected ductile characteristics.

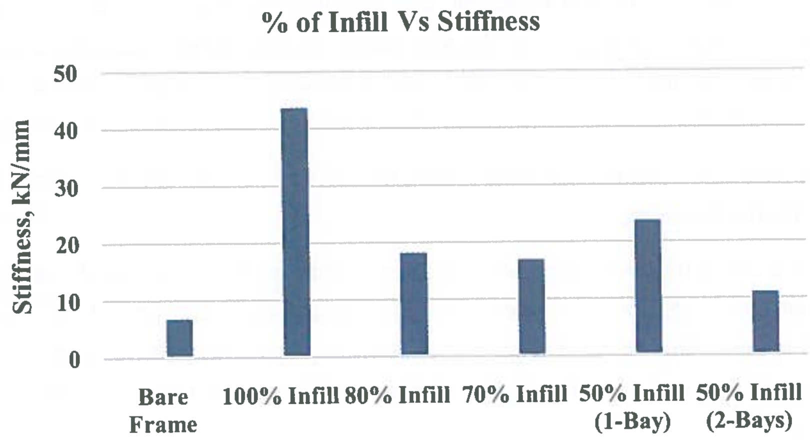


Figure C.9 Comparison of Stiffness of Bare Frame and Infilled Frames with Various Infill Quantities

Knowing the parameters of storey drift is an important part of understanding and quantifying the changing displacement characteristics of a building. Non-linear pushover analysis can determine this behavior as performed in this study. Storey drift is defined as the ratio of the total lateral displacement that occurs in a single storey divided by the height of that storey; e.g. storey drift ratio (of storey 2) = (displacement storey 2- displacement storey 1)/ storey height. Gradual changes in displacement ensure structural stability, uniform stiffness and decrease the probability of non-uniform plastic hinges forming.

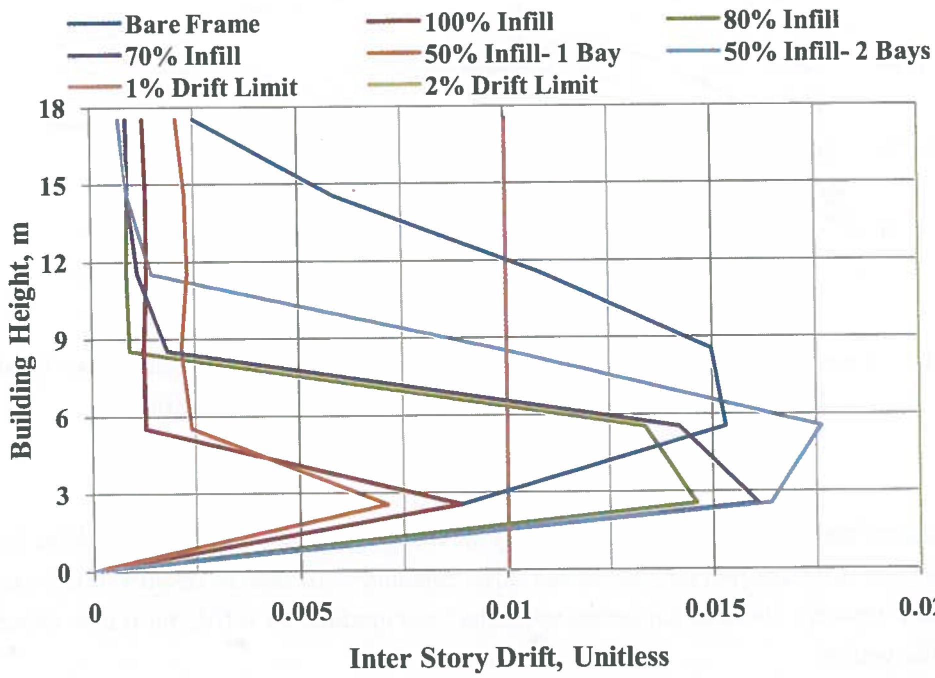


Figure C.10 Comparison of Maximum Inter Storey Drift Ratio Profiles Comparison for Bare Frame and Infilled Frames with Various Infill Quantities

In the case of infilled masonry frames behavior was found to be worse; there were sudden displacement changes in the soft storey under specific lateral loads. A soft storey usually causes greater drift in successive stories above it, and this is similar to other structural responses. For a 100% infilled frame, and a frame with one single bay 50% infilled, the drift ratio was lower, as shown in Figure C.10.

BNBC 2015 proposes the control of drift ratio up to 1% in occupancy category IV, which was represented in the model building.

Figure C.11 shows a comparison of capacity curves plotted against the same demand response spectrum in the ADRS format which shows the seismic acceleration and displacement of the building frame in bare and different infill setups. The demand response spectra is plotted according to the BNBC 1993 with 5% damping factor and also from 11% to 17.5% damping factor associated with the demand of different performance points.

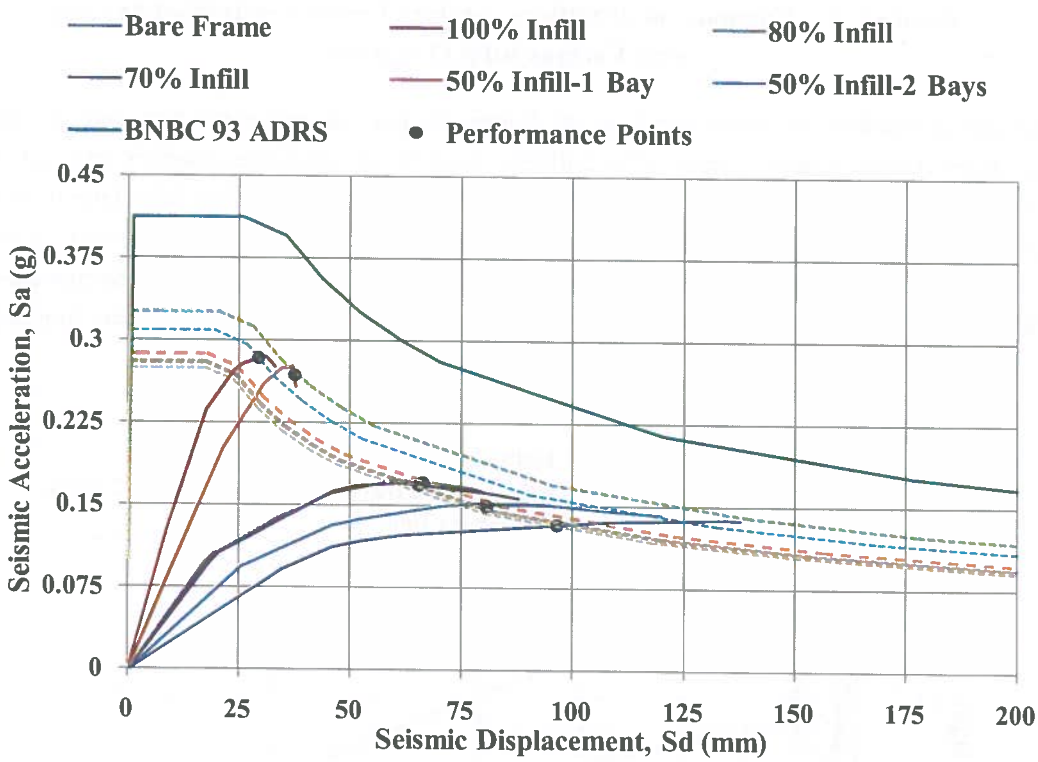


Figure C.11 Comparison of Capacity Curves Showing the Performance Points (Dot) of Bare Frame and Infilled Frames with Various Infill Quantities

Figure C.12 shows the variation of shear capacity at the performance point of the bare frame and the infilled frames with different percentages of the infill masonry wall area in respect to the total area of the frame. The shear capacity not only depended upon the total quantity of infill, but it also depends upon the placement of the walls.

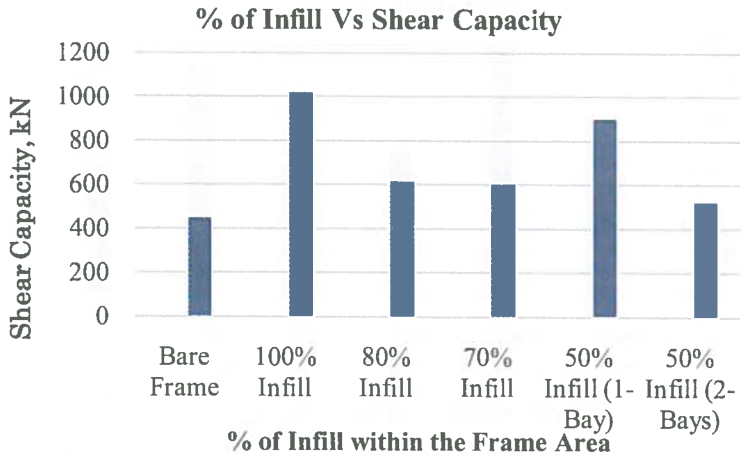


Figure C.12 Comparison of Shear capacity at Performance Point of Bare Frame and Infilled Frames with Various Infill Quantities

Figure C.13 shows the variation of displacement at the performance point of the bare and the infilled frames with different percentages of the infill masonry wall area in respect to the total area of the frame. The shear capacity not only depends upon the total quantity but it also depended upon the placement of the walls.

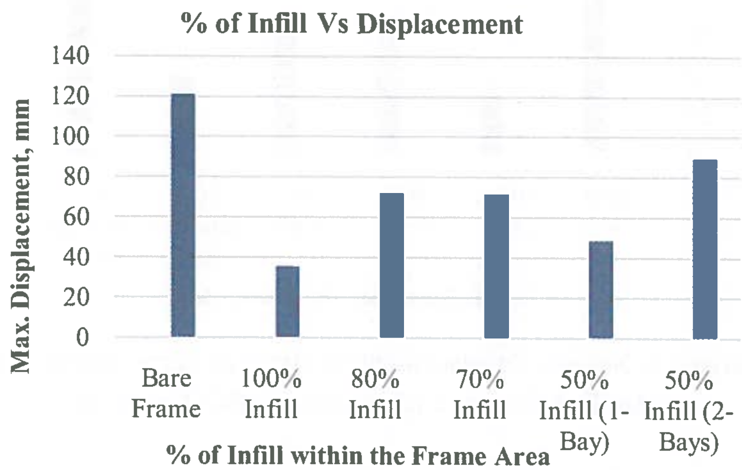


Figure C.13 Comparison of Displacement at Performance Point of Bare Frame and Infilled Frames with Various Infill Quantities

Figure C.14 and Figure C.15 show the variation of spectral accelerations and displacements at the performance point of the bare and the infilled frames with different percentages of the infill masonry wall area in respect to the total area of the frame. According to different quantities and placements style the seismic acceleration of the frame increased but the displacement capacity decreased and converted the frame type from ductile to strength dominant.

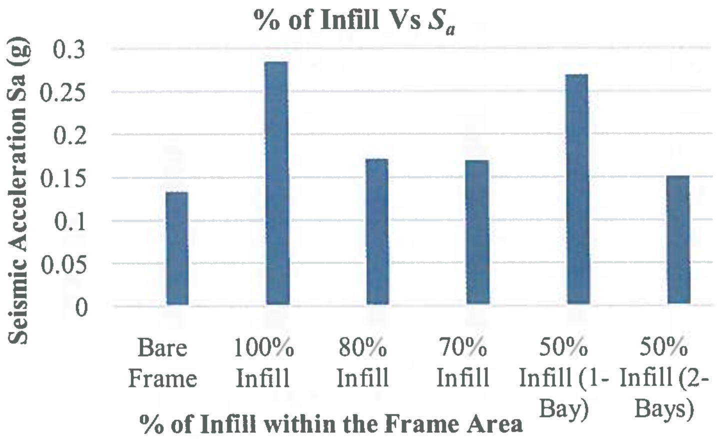


Figure C.14 Comparison of Seismic Acceleration Sa (g) at Performance Point of Bare Frame and Infilled Frames with Various Infill Quantities

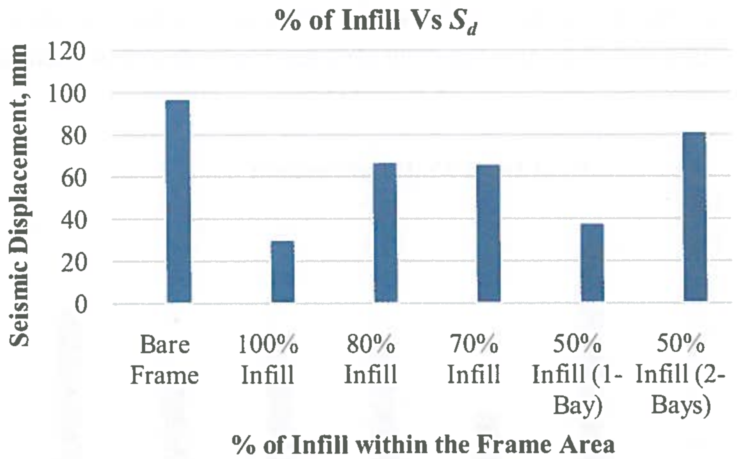


Figure C.15 Comparison of Seismic Displacement Sd (mm) at Performance Point of Bare Frame and Infilled Frames with Various Infill Quantities

Usually the RC framed buildings with infill brick masonry wall are designed considering the time period of a bare frame structure. Figure C.16 shows the variation in effective time period of the bare frame and the infilled frames with different percentages of the infill masonry wall area in respect of the total area of the frames exhibited. Bare frame shows the maximum effective time period at the performance point compared with the infilled frames.

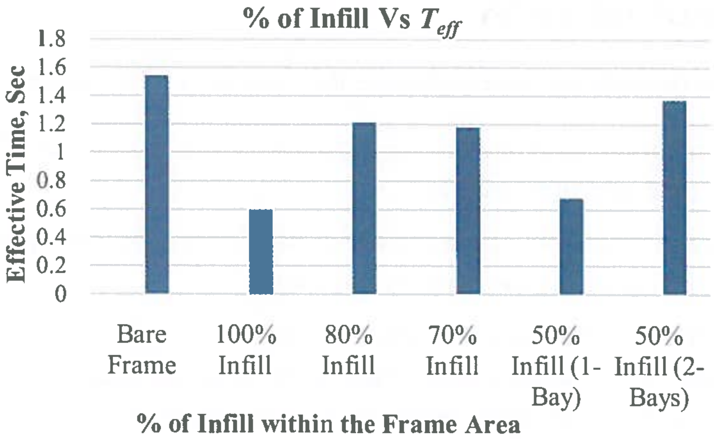


Figure C.16 Comparison of Effective Time Period (T_{eff}) at Performance Point of Bare Frame and Infilled Frames with Various Infill Quantities

C 3.2 REMARKS ON RESULTS

Followings are the observation results summarized after numerical and graphical evaluation data of the model building frame;

From the model of masonry units and calibration of analytical model;

a) The masonry unit property and the masonry infill wall properties are dependent on various materials properties and construction procedure.
b) Methods to calculate the compressive strength and modulus of elasticity of infill masonry wall are in a wide range of variables. So, it needs proper testing for model analysis for specific structures representing the similar materials and method of construction.
c) Fixation of strut width needs to be calibrated with the test result and state-of-the-art methods to get the equivalent strut width applicable for buildings in Bangladesh.

From the parametric study for model frames;

d) The base shear capacity of bare frame was lower than those of the infilled frames. In case of infilled frames, quantity of base shear was very much depended on the amount and style of placement of infill masonry walls within the frame. Roof displacement of study frame was larger in case of bare frame.

e) Inter storey displacements in soft stories were very large. Which indicates that the ductility demand in soft stories become very high. Stiffness of the above stories was much higher than soft storey or stories.

f) Inter storey drift ratio is depending upon the placement and amount of infill masonry unit. Lateral storey drift ratio is not uniform in case of frame having soft storey criteria.

g) Performance point of the building frame designed by the BNBC showed satisfactory parameters of seismic demand criteria in case of bare frame but it is adversely influenced when infill masonry wall considered.

C 3.3 LIMITATIONS OF THE STUDY

The following are further research options considering the limitations of this study.

a) The test results in this study reflected only frames tested in 2013 under CNCRP Project. Other frames tested in 2012 need to be analyzed.

b) Further testing plans should be prepared considering different material properties, strength and stiffness of structural elements and masonry infill walls.

c) This study mainly considers lateral load and the displacement behavior of the structural frame. A detailed study shall be made on internal stress-strain behavior and collapse mechanisms of the structural frame elements. To better understand the structural elements, hinge length and hinge formation mechanisms need more study.

d) This individual study considers only one typical frame of one model building. Several frames and whole buildings need to consider for better judgment.

e) The behavior of a building or building frames can be studied by modeling the infill masonry as finite elements. The results obtained using finite elements models can be compared with results obtained from strut analysis models and laboratory test results.

f) Out-of-plane behavior of the building frame was not studied, and this is necessary.

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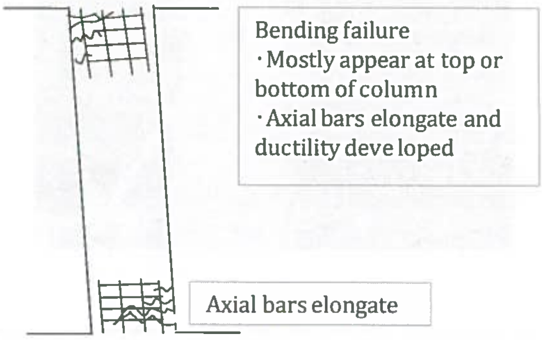
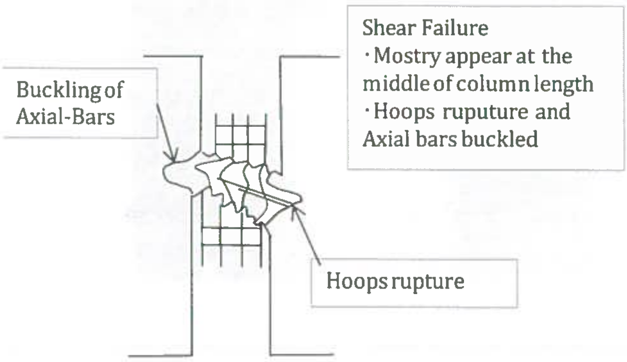
APPENDIX-D FAILURE MODE WITH PICTURES

Figure D1. Sides way by Flexural Failure of Column (a)

Figure D2. Shear Failure in Column (a)





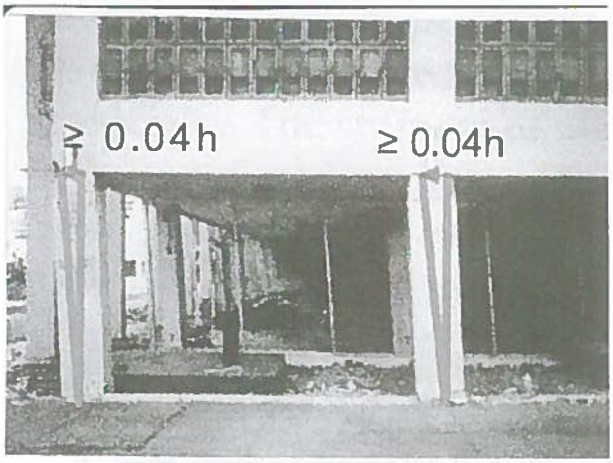


Figure D3. Sides way by Flexural Failure of Column (b)

Figure D4. Shear & Flexural Failure at Column Top

Figure D5. Shear Failure in Column (b)

Figure D6. Shear Failure of Column due to Short Column

Figure D7. Shear Failure due to Short Column

Figure D8. Shear Failure at Beam-Corner Column Joint (a)

Figure D 9. Shear Failure at Beam-Corner Column Joint (b)

Figure D10. Shear Failure at Beam-Corner Column Joint (c)

Figure D11. Ground Soft Storey Failure

Figure D12. 1st Storey Failure

Figure D13. Exterior Brick Walls Collapse

Figure D14. Exterior Brick Walls Collapse

References:

D-1 Report the Damage Investigation of the 1992 Turkey Earthquake by Architectural Institute of Japan (AIJ)

D-2 Ditto

D-3 Seismic Behavior and Retrofit of Infilled Frames Mohammad Reza Tabeshpour, Amir Azad and Ali Akbar Golafshani

D-4 Report the Damage Investigation of the 1992 Turkey Earthquake by Architectural Institute of Japan (AIJ)

D-5 The Great East Japan Earthquake

D-6 Reconnaissance Report on The 2010 Chile Off Maule Earthquake by Architectural Institution of Japan (AIJ)

D-7 The Great East Japan earthquake

The Kenchiku Gijutsu 2011 September No.740

D-8 1985 Mexico Earthquake

D-9 Report on the Damage Investigation of the 1990 Luzon earthquake by Architectural Institution of Japan (AIJ)

D-10 Ditto

D-11 Report the Damage Investigation of the 1992 Turkey Earthquake by Architectural Institute of Japan (AIJ)

D-12 Strength and drift demand of columns of RC framed buildings with soft ground storey by Sharany Haque and Khan Mahmud Amanat

D-13 Report the Damage Investigation of the 1992 Turkey Earthquake by Architectural Institute of Japan (AIJ)

D-14 Report the Damage Investigation of the 1992 Turkey Earthquake by Architectural Institute of Japan (AIJ)

* D-8, D-9 , and D-10 are provided by the courtesy of Shunsuke Otani (Professor

Emeritus, Tokyo University)

PART-II

CHAPTER 1. OVERVIEW OF JAPANESE PROVISION FOR EARTHQUAKE ANALYSIS & DESIGN

1.1 GENERAL

Japan, a country believed to be seismically very active. There are about two thousand active faults in Japan, more than in most other countries. Most part of Japan is susceptible to strong earthquakes of magnitude 7 or more on the Richter scale. The earthquake resistance design, methods of earthquake resistance calculation, structural requirements, and construction methods had been developed and/or changed in this country, especially after the great Kanto Earthquake of 1923, and the experience gained and lessons learned from subsequent major earthquakes occurred in the other parts of Japan.

1.2 BIRTH OF SEISMIC DESIGN IN JAPAN

An official announcement of Urban Building Law of Japan was made in 1920, but seismic design provision was introduced in 1924 through the revision of Urban Building Law as a consequence of the great Kanto Earthquake (Table1.1). This introduction first incorporated that lateral seismic coefficient would be not less than 0.1, and it was the first seismic provision not only in Japan but also in the world. In fact, this 0.1 coefficient was 3 times effective, means equivalent to 0.3, because of using 1/3rd ultimate strengths of materials in design. The seismic coefficient was raised to 0.2, long- and short-term load specified separately in design procedure and building height limited to 31m in Japan Building Standard 3001 published in 1947 by the Ministry of Construction as a further development. In 1950 after the Second World War, the Urban Building Law was replaced by Building Standard Law of Japan, in which the seismic coefficient 0.2 was limited up to 16*m* height and gradual increase of the coefficient above that height was instructed. Also, instruction was made that some specified parts of buildings, for instance, penthouse that should be designed by using the seismic coefficient 0.3.

The restriction on the limitation of building height 31m, had been abolished in 1963 by the revision of Building Standard Law, and technical guidance on high-rise buildings was published by Architectural Institute of Japan (AIJ) in 1964.

In 1968, the Tokachi Oki Earthquake occurred, which claimed not so large number of lives but caused severe damage to 15 percent of code-designed reinforced concrete buildings and some other buildings. In consequence, partial revision of the Building Standard Law and introduction of Ultimate Strength Design (USD) in shear of reinforced concrete by AIJ were in effect in 1971. Furthermore, a five-year-project from 1972 to 1977 was conducted by the Ministry of Construction, in the aim of establishment of a new seismic design method. The ductility of members was especially addressed in this project after learning lessons from 1968 earthquake. The Ministry of Construction released the proposal in 1977 and in the same year, Japan Association for Building Disaster Prevention published a review procedure of existing building for seismic safety.

In the following year (1978), the Miyagiken Oki Earthquake occurred. Many buildings suffered severe damage due to torsional effect demonstrating more complicated characteristics of urban disaster. The similar type of damage was observed in the 1975 Oitaken Chubu Earthquake. The lessons learned from these earthquakes enabled to revise and establish the so called New Seismic Design Method with the introduction of Two-Phase Design Method through the Enforcement Order of Building Standard Law in

1981. Guidelines to Structural Calculation Based on the Revised Building Standard Law had been published by the Building Center of Japan by the assistance of Housing Bureau and Building Research Institute, Ministry of Construction in 1981.

Table 1.1 Major Earthquakes and Development Building Design Standard in Japan

Major Earthquakes					Development		Remarks
Year	Name	M	Persons Killed	Building Damaged	Year	Building Design Standard	
					1920	Announcement of Urban Building Law (UBL)	
1923	Great Kanto Earthquake	7.9	142,807	128,266	1924	Revised the UBL, introduction of Seismic Coefficient 0.1	
1945	Nankaido Earthquake	8.0	1,330	9,060	1947	Seismic Coefficient increased to 0.2 and Long- and Short-term loads distinguished	
1948	Fukui Earthquake	6.3	3,895	35,420	1950	Building Standard Law (BSL) replaced UBL, Seismic Coefficient above 16m increased with height, and in soft sub-soil area 0.3	
					1963	BSL revised and height limitation abolished	
1964	Niigata Earthquake	7.5	26	1,960	1964	Guideline on high-rise building by AIJ	
1968	Tokachi Oki Earthquake	7.9	52	673	1968	Formed Special Committee for Highrise Buildings under Building Center of Japan	The first 30-storey highrise Kasumiga-seki building built
					1971	BSL revised, introduction of Ultimate Strength Design (USD) in shear of RC buildings	Column tie pitch decreased from 300mm to 150mm
1975	Oitaken Chubu Earthquake	6.7	0	58	1977	Proposal for New Seismic Design Method	
1978	Miyagiken Oki Earthquake	7.4	28	1,183	1981	Published New Seismic Design Method (Two-Phase Design Method)	
1983	Nihonkai Chubu Earthquake	7.7	104	934	1988	Design guideline for earthquake resistant RC buildings based on Ultimate Design Concept (Draft) by AIJ	
1995	Hyogoken Nanbu Earthquake	7.2	5,472	81,972	1995	Introduction of Building Rehabilitation Law	
					2000	Introduction of Performance Based Design (response and limit capacity calculation)	

The Hyogoken Nanbu Earthquake (Great Hanshin Earthquake or Kobe Earthquake) occurred on 17th January 1995. The buildings built after the introduction of new earthquake code in 1981, had survived without any or major structural damage. However many older structures (before 1981) suffered significant damage. Therefore, Ministry of Construction advised the owners of pre-1981 buildings for retrofitting, though not by issuing a legal notice but giving partial financial assistance.

The Building Standard Law was substantially revised and the *"Response and Limit Capacity Calculation"* was introduced to innovate the Building Law from the Specification-Based Design Method to the Performance-Based Design Method in 2000. The Performance-Based Design Method is introduced in Chapter 3 of the Sub-Manual.

The development of building design method with the earthquakes is given in Table 1.1 (*Ref. 8.1*).

1.3 STRUCTURAL SEISMIC DESIGN

A flow for Seismic Design Method is given in Figure 1.1. It is composed of two stages of design, namely First-Phase Design and Second-Phase Design. All buildings are divided into five groups. Group 1 includes the houses of single-story and the building area less than $200m^2$. Group 2 includes houses constructed by timber, and the other buildings, which do not require second-phase design, specified by the Ministry of Land, Infrastructure, Transport and Tourism (MLIT). Group 3 through Group 5 are divided distinctly based on the height of the buildings. The buildings belongs to Group 2 through Group 4 are usually designed by following the specification and guidance recommended by the MLIT. The buildings belonging to Group 5 are designed by the time history response analysis. These buildings are subjected to the review of the Structural Review Committee for Highrise Buildings at the Building Center of Japan. Then an approval from the MLIT is issued for structural design and construction.

The buildings in Group 2 through Group 4 are first subjected to the First-Phase Design or in other words the conventional design through the checking for allowable stress in all members due to long-and short-term loading. The short - term loading obviously include the seismic loading so on. Group 3 and Group 4 require going through the Second-Phase Design, which is not required for Group 2. Both Group 3 and Group 4 enter Second-Phase Design demanding check for story deformation angle. Therefore, Group 3 may follow either Route-2 or Route-3, a judgment is made by experienced engineer. If Route-2 is followed, the checking for rigidity factor, eccentricity factor, and specifications of the MLIT is required. The checking may be avoided if Route -3 is followed by checking for horizontal load carrying capacity. Group 4 directly approaches for checking horizontal load carrying capacity. The horizontal load carrying capacity means each story shear resisting capacity at the formation of a collapse mechanism. The buildings in Group 5 are subjected to special approval.

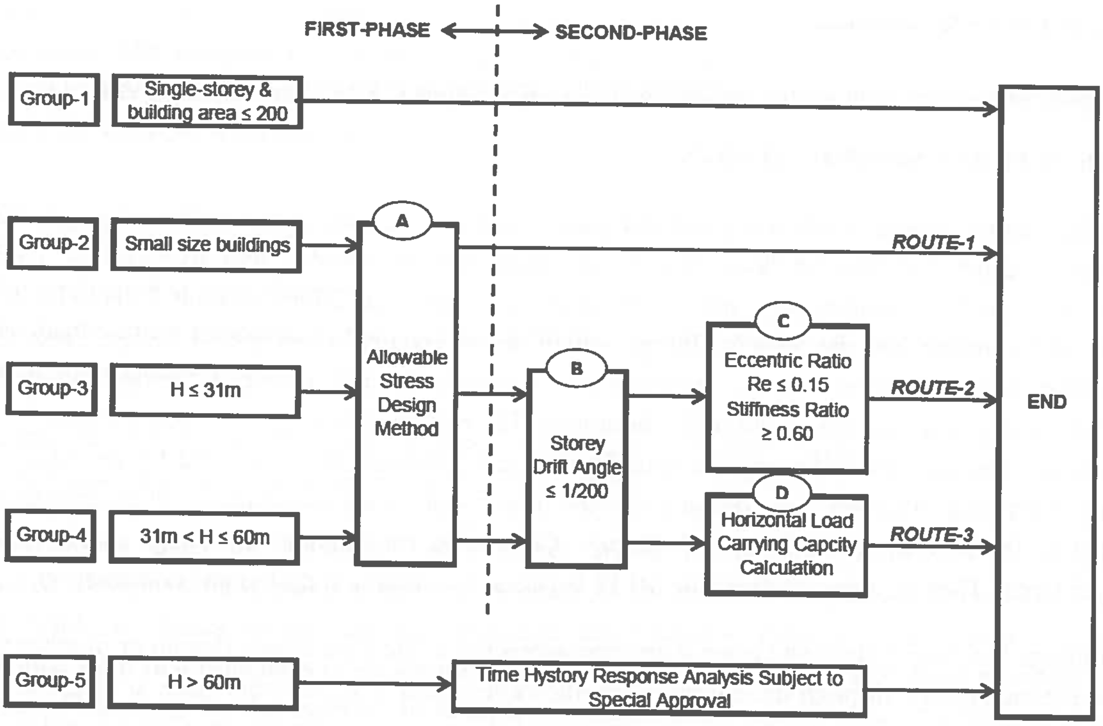


Figure 1.1 Flowchart for Seismic Design (*Ref.* 8.1)

1.3.1 First-Phase Design

1.3.1.1 Design Process

The first-phase design, basically the conventional design, requires checking for allowable stress in all elements. Therefore, the design is based on working stress design method. The purpose of this design is to protect buildings with almost no damage in the case of moderate earthquakes which may occur several times during the life time of the buildings.

For structural calculations, the procedure adopted by the designer in practice is to (a) assume sectional properties of all members and joints, (b) estimate member stiffness for modeling frame structure, (c) calculate permanent and temporary loads, and the resulting stress in every members (d) calculate stress in every section resulting from stress in member, and (e) check for the sectional stress whether exceed the allowable stress of the materials that would be used for construction. Generally speaking the second phase design governs the sectional properties of the building members rather than the first phase design.

1.3.1.2 Lateral Static Earthquake Force above Ground Level

The Q_i which is the lateral seismic shear of i^{th} storey above the ground level, shall be determined in accordance with the following equations.

$$Q_i = C_i \sum_{i=1}^{n} W_i \qquad (1.1)$$

$$C_i = Z R_t A_i C_0 \qquad (1.2)$$

Where,

W_i = Weight of building above 1[th] storey (this includes dead load, reduced live load, and snow load in snowy area)

n = Number of stories

C_i = Seismic storey shear coefficient of the aboveground part of a building at a given height

Z = A value specified by the Minister of Land, Infrastructure, Transport and Tourism within a range between 1.0 and 0.7 reflecting the extent of earthquake damage, seismic activity and other seismic characteristics based on the record of earthquake in the region concerned

R_t = A value representing vibration characteristics of buildings obtained by the calculation method specified by the Minister of Land, Infrastructure, Transport and Tourism according to the natural period in the elastic range of building components and ground category

A_i = lateral shear distribution factor in vertical plane, which shall be determined by the fundamental natural period and the weight distribution of the building

$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i \right) \frac{2T}{1+3T} \tag{1.3}$$

Where,

$$\alpha_i = \frac{\sum_{j=1}^{n} W_j}{\sum_{j=1}^{n} W_j} \tag{1.4}$$

α_i = Value of the ratio of the dead load and the live load of parts supported by parts with the height for which A_i of the building is to be calculated to the total of the dead load and the live load of above ground parts of the said building.

T = Natural period of the building calculated by the equation A_i is originated that the earthquake load is given by the lateral shear force coefficient, and weight the standard weight α_i as a parameter that shows the weight distribution but not in accordance with the height.

w_j = The weight of the 1-th storey

C_o = Standard shear coefficient, which shall be not less than 0.2 (0.3 for wooden building in soft subsoil area) for the first-phase design and 1.0 for the second-phase design. The vibration characteristics factor R_t is extrapolated from the following empirical expressions for three types of soil.

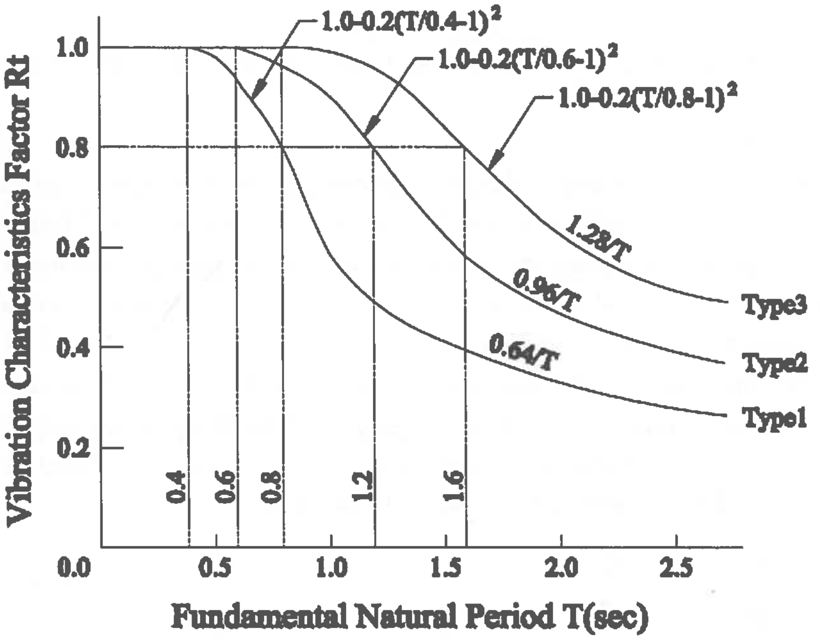


Figure 1.2 Fundamental Natural period vs Vibration Characteristics Factor

The vibration characteristics factor R_t is a function of natural period T and the type of subsoil evaluated from the following equation:

$$R_t = 1 \qquad \text{for } T < T_c \tag{1.5}$$

$$R_t = 1 - 0.2\left(\frac{T}{T_c} - 1\right)^2 \qquad \text{for } T_c \le T \le 2T_c \tag{1.6}$$

$$R_t = 1.6\left(\frac{T_c}{T}\right) \qquad \text{for } 2T_c \le T \tag{1.7}$$

Where,

T = Fundamental natural period of structure;

T_c = Critical period of subsoil (0.4 sec for Type 1 soil composed of rock, still sand and gravel, 0.6 sec for Type 2 soil composed of others, and 0.8 sec for Type 3 soil composed of alluvium). The fundamental natural period T is to be calculated from the following empirical expression.

$$T = h(0.02 + 0.01\alpha) \tag{1.8}$$

Where,

h = Height of the building in meter,

α = The ratio of the height of stories, which consist of steel columns and beams, to the total height h. This is a very handy calculation method of T, but in many cases T is determined from the stiffness calculation. Therefore, for reinforced concrete structures $T = 0.02h$, and for steel structures $T = 0.03h$. The resulting value of R_t is shown in Figure 1.2.

1.3.1.3 Lateral Static Earthquake Force in the Basement Stories

The lateral seismic shear in basement Q_B shall be determined in accordance with the following equation.

$$Q_B = Q_p + kW_B \tag{1.9}$$

Where,

Q_p = Portion of the seismic storey shear force in the adjacent upper storey that is carried by columns and shear walls above the basement being considered;

k = Seismic design coefficient of the basement as determined in accordance with Equation. (8.10);

W_B = Weight of the basement due to dead and live loads in the storey being considered. The seismic design coefficient of basement k shall be determined in accordance with the following expression.

$$k \geq 0.1\left(1-H/_{40}\right)Z \tag{1.10}$$

Where,

H = Depth of the basement from ground level in meter (if the depth exceeds 20m, it should be taken 20m). Figure 1.3 shows a building with basement storey and the distribution of seismic design coefficient k.

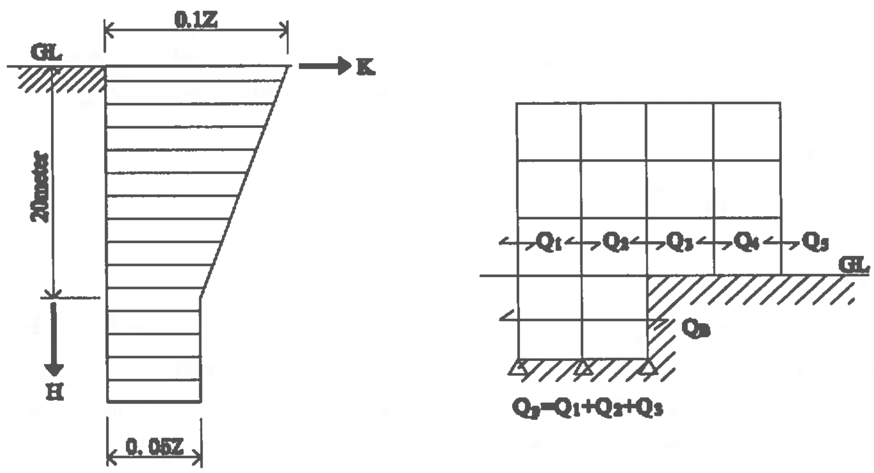


Figure 1.3 Lateral Seismic Shear Q_B and Design Seismic Coefficient k of the Basement

1.3.1.4 Lateral Static Earthquake Force on Appendages

The lateral seismic shear of penthouse, chimney, tower, cistern, parapet and other appendages on buildings Q shall be determined in accordance with the following equation.

$$Q = k_a W \tag{1.11}$$

Where,

k_a = Seismic design coefficient of appendages (it shall be greater than 1.0, but the value can be minimized down to 0.5 in case no harm will occur);

W = the weight of the appendage.

1.3.2 Second-Phase Design

1.3.2.1 Design Procedure

The second-phase design could be achieved through the lessons learned especially from the Tokachi-Oki Earthquake in 1968, the Miyagiken-Oki Earthquake of 1978, and the Oita-ken Earthquake of 1975. They revealed that the regular shape buildings responded very nicely but the performance of the irregular buildings was either not satisfactory or resulted into severe damage and collapse of the structures. The vulnerability of the irregular buildings gave a great momentum to the concerned authorities, and therefore the necessity of second-phase design came into action. In case of buildings of *Group 3* in Figure 1.1, experienced designers make judgment whether to adopt Route 1 or 2. Usually, Route 1 is adopted for regular shape buildings and Route 2 is adopted for irregular shape buildings because the latter cannot fulfill the requirements of Route 1.

The purpose of the second-phase design is to protect the buildings with minor damage but no collapse nor harm of human lives by the severe earthquake which may occur less than one time in the life time of the buildings. The second phase design, basically several checking for some structural demands, and the specifications prescribed by the MLIT as shown in [B], [C] and [D] of Figure 1.1. What structural fabrication are made in the first-phase design, they are demanded for checking in the second-phase design.

1.3.2.2 Check for Drift Angle ([B] of Figure 1.1)

The storey drift at every floor level δ_i under the action of lateral seismic shear force Q_i prescribed in Eq. (1.1) is calculated by the elastic analysis. Then the storey deformation angle R_i is to be checked by the following expression.

$$R_i = {}^{\delta_i}\!/_{h_i} \leq {}^{1}\!/_{200} \text{ rad} \tag{1.12}$$

Where,

h_i : height of i^{th} storey.

The value of R_i can be increased up to 1/120 rad in the case the non-structural members shall have no severe damage at the increased storey drift limitation. The storey drift angle calculated under the action of seismic shear force prescribed by Eq. (1.1), *i.e.* for the first-phase design. Under the action of severe earthquake prescribed in the second-phase design, the storey drift angle shall be much larger than that prescribed by Eq. (1.12), that may be between 1/50 and 1/75.

1.3.2.3 Check for Rigidity Factor ([C] of Figure 1.1)

In the design of a building, uniform rigidity in the vertical direction should be maintained if possible, or should be designed very carefully when that cannot be maintained under some special circumstances. Because of the non-uniformity in the vertical direction, the deformation may concentrate in the weak-storey as shown in Figure 1.4. In the event of an earthquake, the collapse of the buildings due to the lack of uniformity in the vertical direction, or in other words, due to the concentration of the dissipation of energy in the weak-storey, had been observed very often. To prevent this type of collapse, check for rigidity factor R_{si} is necessary, and which is to be done by the following expressions.

$$R_{si} = {}^{r_{si}}\!/_{r_s} \geq 0.6 \tag{1.13}$$

$$r_{si} = {}^{1}\!/_{R_i} \tag{1.14}$$

$$r_s = \sum_{i=1}^{n} {}^{r_{si}}\!/_{n} \tag{1.15}$$

Where,

R_{si} = Stiffness ratio of each storey

r_{si} = Reciprocal of the storey drift angle of each storey

r_s = Arithmetic mean of r_{si} concerning the building

In one of more stories do not satisfy this requirement, the buildings must be checked for the horizontal load carrying capacity, i.e. to go back to follow the Route 2 of Figure 1.1.

1.3.2.4 Check for Eccentricity Factor ([C] of Figure 1.1)

As of the rigidity in the vertical direction of a building, the regularity in the planning of the building should be maintained. A building with irregular plan is subjected too much greater vulnerability of earthquake than a building with regular plan. The eccentricity factor should be checked for a building to avoid poor performance in the event of a major earthquake. As shown in Figure 1.5, (g_x , g_y) is the center of gravity of the total mass above the storey being considered. The center of gravity can be obtained from the following equations.

$$g_x = \frac{\sum Nx}{W} \tag{1.16a}$$

$$g_y = \frac{\sum Ny}{W} \tag{1.16b}$$

Where,

N = the axial force in column and shear wall

$W = \sum N$ in the storey being considered

The center of rigidity i.e. the center of rotation (l_x , l_y) under the action of torsional moment can be obtained from the following equations.

$$I_x = \frac{\sum K_y x}{\sum K_y} \tag{1.17a}$$

$$I_y = \frac{\sum K_x y}{\sum K_x} \tag{1.17b}$$

Where, K_x and K_y are the transitional stiffness in x and y axes. The eccentric distances in the axes of x and y are e_x and e_y, as shown in the Figure 1.5 and can be obtained from the following equations.

$$e_x = |I_x - g_x| \tag{1.18a}$$

$$e_y = |I_y - g_y| \tag{1.18b}$$

The rotationally stiffness at each storey exists at each storey. Each seismic resisting element is in the (X_i , Y_i) coordinate shown as below,

$$X_i = X - I_x \tag{1.19a}$$

$$Y_i = Y - I_y \tag{1.19b}$$

The point in the new coordinate where the center of stiffness is the origin of coordinate. The rotational stiffness around the center of stiffness is

$$K_R = \sum\left(K_x Y_i^2\right) + \sum\left(K_y X_i^2\right) \tag{1.20}$$

The eccentricity factors R_{ex} and R_{ey} are to be checked by the following expressions.

$$R_{ex} = {e_y}/{r_{ex}} \leq 0.15 \tag{1.21a}$$

$$R_{ey} = {e_x}/{r_{ey}} \leq 0.15 \tag{1.21b}$$

Where, r_{ex} and r_{ey} are the elastic radii defined by the following equations.

$$r_{ex} = \sqrt{\frac{\sum\left(K_x Y_i^2 + \sum K_y X_i^2\right)}{\sum K_x}} \tag{1.22a}$$

$$r_{ey} = \sqrt{\frac{\sum\left(K_x Y_i^2 + \sum K_y X_i^2\right)}{\sum K_y}} \tag{1.22b}$$

If one or more stories do not satisfy the requirement of Eq. (1.21), the building must be checked for the horizontal load carrying capacity, *i. e.* to go back to follow the rigidity and eccentricity factors.

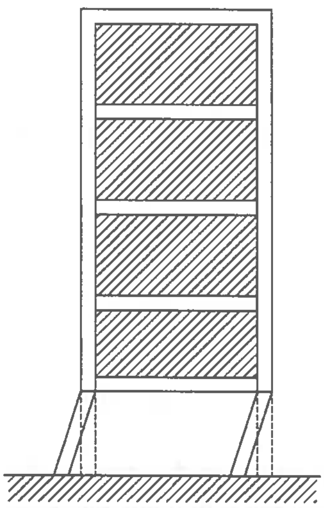


Figure 1.4 Concentration of Energy Dissipation in Weak-Storey (Soft Storey)

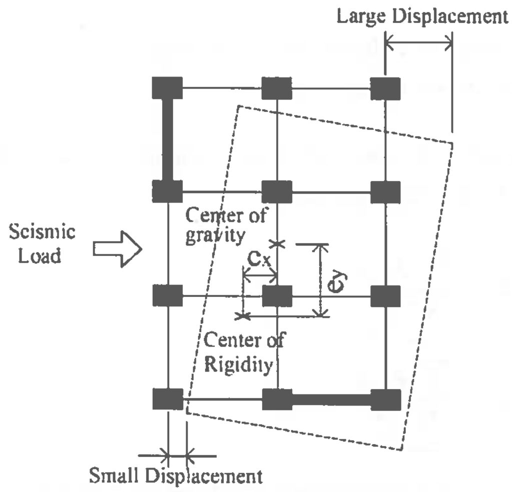


Figure 1.5 Building with Eccentricity

1.4 HORIZONTAL LOAD CARRYING CAPACITY ([D] OF FIGURE 1.1)

1.4.1 Required Horizontal Capacity "Q_{un}"

The ultimate lateral shear strength (Q_u) of each storey shall not be less than the lateral shear Q_{un} determined in accordance with the following equation.

$$Q_u \geq Q_{un} = D_s F_{es} Q_{ud} \tag{1.23}$$

Where,

Q_{un} = Required value of horizontal load carrying capacity of each storey

Q_u = Computed ultimate lateral capacity (lateral load bearing capacity) of each storey

D_s = Structural characteristics factor (Table 1.2);

Q_{ud} = Seismic shear in a storey for severe earthquake (calculated by Equation (1.1), (1.2) taking C_o not less than 1.0);

F_{es} = The shape factor which shall be the product of F_s and F_e as given bellow.

$$F_{es} = F_e F_s \tag{1.24}$$

Where,

F_e = Basic shape factor determined as a function of the eccentricity factor R_e as of Eq. (1.21a, 1.21b).

$$F_e = 1.0 \qquad \text{for } R_e \le 0.15 \tag{1.25a}$$

$$F_e = 1.0 + \frac{0.5}{0.15(R_e - 0.15)} \qquad \text{for } 0.15 < R_e < 0.3 \tag{1.25b}$$

$$F_e = 1.5 \qquad \text{for } R_e \ge 0.3 \tag{1.25c}$$

F_S = Basic shape factor determined as a function of the rigidity factor R_S as of Equation (1.13).

$$F_s = 1.0 \qquad \text{for } R_s \ge 0.6 \tag{1.26a}$$

$$F_s = 1.0 + \frac{0.5}{0.15(R_e - 0.15)} \qquad \text{for } 0.3 < R_s < 0.6 \tag{1.26b}$$

$$F_s = 1.5 \qquad \text{for } R_s \le 0.3 \tag{1.26c}$$

The figures presenting Eq. (1.25) and (1.26) are shown in Figure 2.6 in Chapter 2.

Table 1.2 Structural Characteristics Factor D_s

Behavior of Members		Type of Reinforced Concrete structures		
		(1) Ductile moment frame $\beta_u \le 0.3$	Frame other than listed in (1) & (3) $0.3\beta_u \le 0.7$	(3) * * $\beta_u > 0.7$
A	Members of excellent ductility	0.3	0.35	0.4
B	Members of good ductility	0.35	0.4	0.45
C	Members of fair ductility	0.4	0.45	0.5
D	Members of poor ductility	0.45	0.5	0.55

* * Frames with shear walls for reinforced concrete buildings.

β_u = Ratio of horizontal force carried by shear walls to total storey shear of the storey being considered.

1.4.2 Horizontal Load Carrying Capacity "Q_u"

In order to obtain the horizontal load carrying capacity (Q_u) in each storey, the pushover analysis method is widely and actually used in Japan.

When the lateral seismic loads applies to the structure to get to the collapse mechanism the summation of ultimate shearing strength of columns and seismic walls is the horizontal load carrying capacity (Q_u).

The seismic loads shall be applied to the structure till the storey drift of the structure gets to 1/75～1/100, in order to obtain the exact value of D_s.

The joint distribution method is shown in Figure 1.6 to understand the horizontal load carrying capacity.

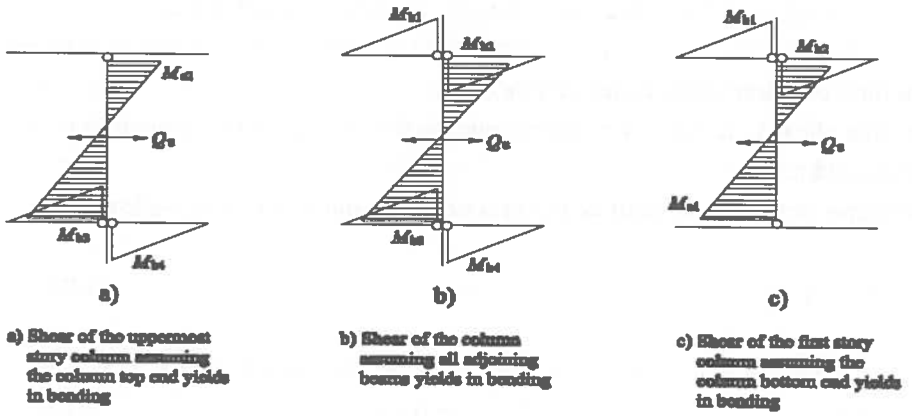


Figure 1.6 Shear of Columns, Q_u (Joint Distribution Method)

1.4.3 D_s Value of RC Structure

1.4.3.1 Method of calculating D_s

Method of calculating D_s in Japanese Seismic Design Code is presented here. This value is decided by the evaluation of damping and ductility of frames, and equivalent to the reverse value of R (Response modification coefficient for structural systems) given in upcoming BNBC.

D_s value is calculated by following sequences,

i. Collapse mechanism of a frame is calculated by a push over analysis etc.
ii. Category of beam/column and wall is estimated based on Table 1.3 and Table1.4.
iii. Category as a member group is estimated based on Table1.5.
iv. Ds value is estimated based on Table1.6, Table1.7 and Table1.8.

Category of beam and column is decided based on Table 1.3, and in case that member with different type is connected, category of column is decided as follows.

i. In case that category FC and type FD does not exist, category FB is applied.
ii. In case that category FD does not exist and category FC exists, category FC is applied.
iii. In case that category FD exists, category FD is applied.

Table 1.3 Category of RC Column and Beams

Classification of Beam and Column							Category of Beam and Column
Member	**Beam and Column**	**Column**				**Beam**	
Conditions	Failure mode	h_o/D	σ_o/F	P_t	τ_u/F_c	τ_u/F_c	
	No brittle failure such as shear, bond crack, and compressive failure	≧2.5	≦0.35	≦0.8	≦0.10	≦0.15	FA
		≧2.0	≦0.45	≦1.0	≦0.125	≦0.20	FB
		---	0.55	---	≦ 0.15	---	FC
	Other than FA, FB and FC						FD

h_o = Internal height of column (cm)
D = Width of column (cm)
σ_o = Axial stress at collapse mechanism (N/mm^2)
F_c = Material strength of concrete (N/mm^2)
P_t : Tensile reinforcement ratio (%)
τ_u : Average shear stress at collapse mechanism (N/mm^2)

Table 1.4 Category of RC Walls

Classification of Wall				Category of bearing wall
Member	**Bearing Wall**	**Bearing wall other than wall type structure**	**Bearing wall of wall type structure**	
Conditions	Failure mode	τ_u / F_c	τ_u / F_c	
	No brittle failure such as shear failure	$\leqq 0.2$	$\leqq 0.1$	WA
		$\leqq 0.25$	$\leqq 0.125$	WB
		---	$\leqq 0.15$	WC
	Other than WA, WB, and WC			WD

Note, τ_u, and F_c, refer to Note of Table1 1

Table 1.5 Category as a Member Group

	Ratio of Strength of Members	Category as a Member Group
(1)	$\gamma_A \geqq 0.5$ and $\gamma_C \leqq 0.2$	A
(2)	$\gamma_C < 0.5$ (excludes type A member group)	B
(3)	$\gamma_C \geqq 0.5$	C

Notes: γ_A denotes the value of total strength of columns with type FA divided by total strength of columns excluding columns with type FD for column and beam group members, and denotes the value of total strength of walls with type WA divided by total strength of walls excluding walls with type WD for wall group members;

γ_C denotes the value of total strength of columns with type FC divided by total of columns excluding columns with type FD for column and beam group members, and denotes the value of total strength of walls with type WC divided by total strength of walls excluding walls with type WD for wall group members.

Table 1.6 D_s Value for RC Frame Structure

Category of Member Group for Beam and Column Structure	D_s Value
A	0.3
B	0.35
C	0.4
D	0.45

Table 1.7 D_s Value for RC Wall Structure

Category of Member Group for Bearing Wall Structure	D_s Value
A	0.45
B	0.5
C	0.55
D	0.55

Table 1.8 D_s Value for RC Frame and Wall Structure

			Category of Member Group for Beam and Column Structure			
			A	B	C	D
Category of Member Group for Wall Structure	A	$0 < \beta_u \leqq 0.3$	0.3	0.35	0.4	0.45
		$0.3 < \beta_u \leqq 0.7$	0.35	0.4	0.45	0.5
		$\beta_u > 0.7$	0.4	0.45	0.45	0.55
	B	$0 < \beta_u \leqq 0.3$	0.35	0.35	0.4	0.45
		$0.3 < \beta_u \leqq 0.7$	0.4	0.4	0.45	0.5
		$\beta_u > 0.7$	0.45	0.45	0.5	0.55
	C	$0 < \beta_u \leqq 0.3$	0.35	0.35	0.4	0.45
		$0.3 < \beta_u \leqq 0.7$	0.4	0.45	0.45	0.5
		$\beta_u > 0.7$	0.5	0.5	0.5	0.55
	D	$0 < \beta_u \leqq 0.3$	0.4	0.4	0.45	0.45
		$0.3 < \beta_u \leqq 0.7$	0.45	0.5	0.5	0.5
		$\beta_u > 0.7$	0.55	0.55	0.55	0.55

Note: β_u = Means the ratio of strength of RC bearing walls against the total horizontal strength of the storey

Q_{un} = Required value of horizontal load carrying capacity of each storey. (kN)

D_s = A value specified by the Ministry of Land, Infrastructure Transport and Tourism, representing structural characteristics considering damping characteristics and ductility of each storey corresponding to the structural methods of elements necessary for structural resistance of buildings.

F_{es} = A value calculated by the method specified by the Minister representing form characteristics of each storey corresponding to its stiffness ration and eccentricity ratio.

Q_{ud} = Horizontal force acting upon each story due to seismic force. (kN)

D_s value of the reinforcement structures is between 0.3 and 0.5.

1.4.3.2 D_s value Based on the Newmark Theory

Required value of horizontal load-carrying capacity is based on the Newmark Theory.

D_s value of RC is 0.3～0.55 as shown in Figure 1.7 and 1.8.

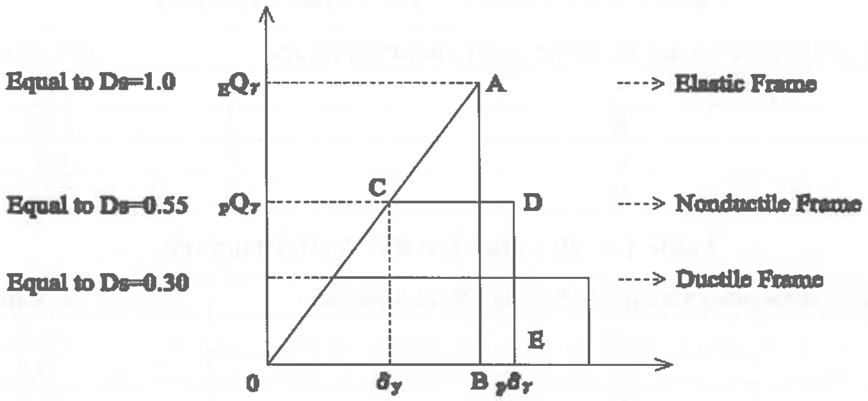


Figure 1.7 Elastic and Post-Elastic Response

Under the condition of Area $OA\delta_L = OBC\delta_N$, the following equations are obtained.

$$Q_L \frac{\delta_L}{2} = Q_Y \frac{\delta_Y}{2} + Q_Y(\delta_N - \delta_Y)$$

$$\delta_L = \left(\frac{Q_L}{Q_Y}\right) - \delta_Y$$

$$\delta_N = \left[\left(\frac{Q_L}{Q_Y}\right)^2 + 1\right]\frac{\delta_Y}{2}$$

$$\therefore D_s = \frac{Q_Y}{Q_L} = \frac{1}{\sqrt{2\mu - 1}}$$

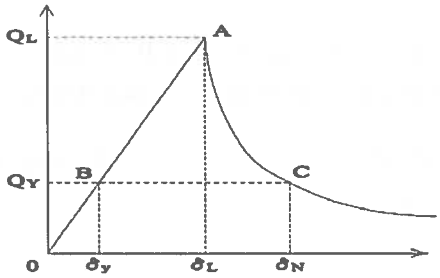


Figure 1.8 Illustration for Newman Theory

Where,

μ= ductility factor

1.4.3.3 D_S Value

The structural characteristic factor D_S is a reduction factor to the earthquake force which would come into the building in the elastic region.

When the seismic ground motion acts to the SDOF system with elasto-plastic restoring force characteristic, the seismic energy induced to the elastic response system and the elasto-plastic response system is same. It means that the area $OA\delta_L$ and $OBC\delta_N$ in Figure is same. This concept is based on the N. M. Newmark's theory.

The same factor as D_s are defined in other courtiers' code like "k_o" in ISO 3010, "q" factor in Euro code and "1/R" in IBC/ASCE 7-05.

The structural factor, "k_o" in ISO 3010, is used to reduce design seismic force or shear forces, taking into account the ductility, acceptable deformation, restoring force characteristics and overstrength (or overcapacity) of the structure. The behavior factor "q" in Euro code 8 is the factor that reduces the elastic spectrum to use as the design spectrum instead of inelastic structural analysis in design.

The basic value "q_o" composing "q" varies from the maximum 5 to minimum 2.

The response modification coefficient "R" in IBC/ASCE 7-05 varies from the maximum value 8 for special moment frames to the minimum value 3.5 for ordinary steel moment frames.

References

1.1 M.A.A Mollick, "Seismic Design Provisions of Buildings in Japan, Journal of the Civil Engineering Division, The Institution of Engineers, Bangladesh, Vol. CE 23 No.1, 1995, pp 37-58.

1.2 Yuji ISHIHARA, "Introduction to Earthquake Engineering and Seismic Codes in the World", February 2011, pp.33.

CHAPTER 2. EXAMPLE OF HORIZONTAL LOAD CARRYING CAPACITY BASED ON JAPANESE EARTHQUAKE PROVISION

2.1 CASE STUDY- SIX STORIED BUILDING AT DHAKA

2.1.1 Building Data

(a) Structural System: Reinforced Concrete Frame

(b) Number of stories: 6

(c) Storey heights: 3.66 m (approx), total height: 24.69 m

(d) Floor area: 735.41 m^2, total area: 4326.20 m^2

(e) Number of spans in x-direction: 11 (clear spans: 5.49m)

(f) Number of spans in y-direction: 3 (clear spans: 6.40m, 5.49m and 3.96 m)

(g) Foundation type: Reinforced Concrete Bore Pile

(h) Special feature: One of intermediate stories is on bare frame

(See **Figure 2.1, 2.2, 2.3** for plan and elevations)

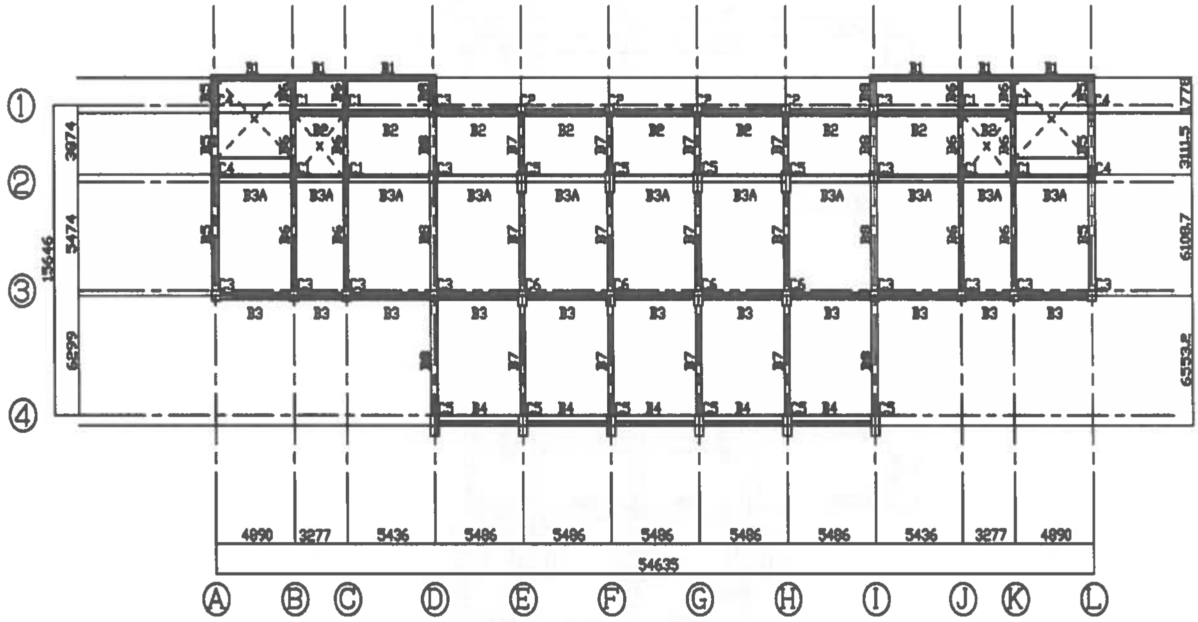


Figure 2.1 1st Floor Plan

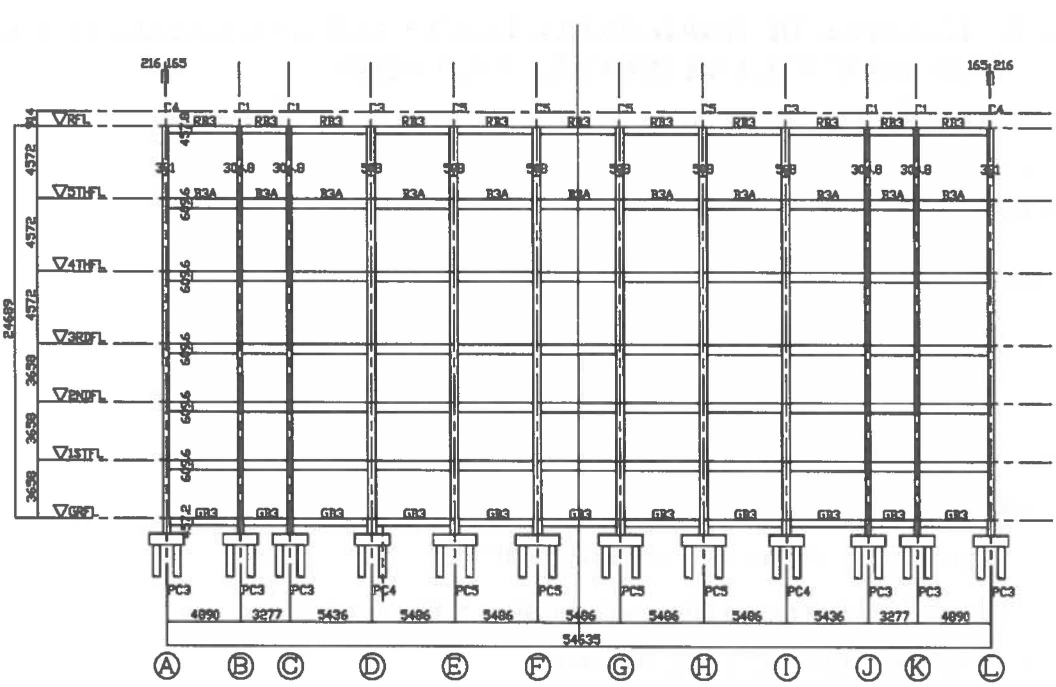


Figure 2.2 2-Line Elevation

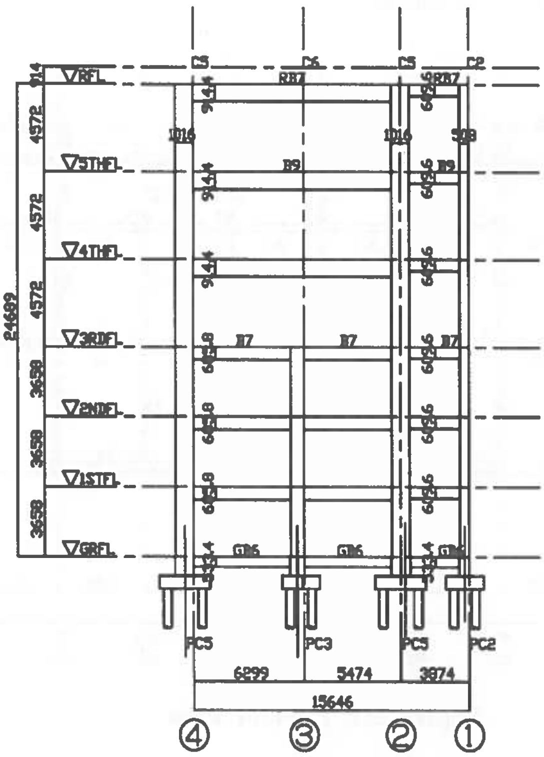


Figure 2.3 F-Line Elevation

2.1.2 Structural Design Description

a) Column & Beam List

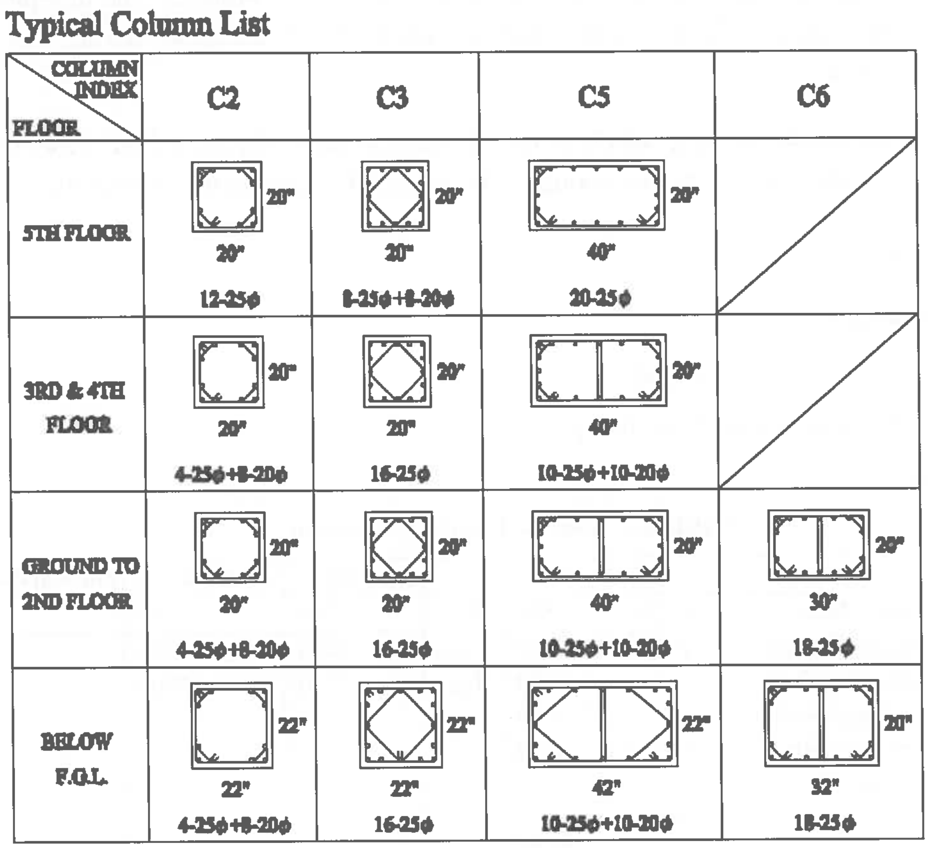


Figure 2.4 List of Typical Columns

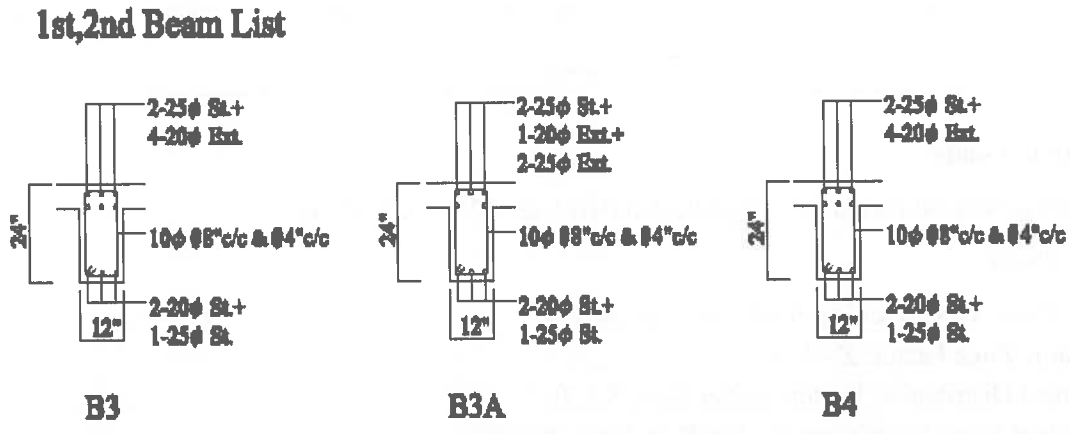


Figure 2.5 Typical Beam Sections

2.1.2.1 Structural Design Criteria

A typical plan and elevation of a 6 storey building are shown in Figure 2.1 to 2.3. The structural system of the building is a moment-resisting frame system with an essentially complete frame that provides support for the gravity loads. The computation of the seismic design forces according to the Japanese Code of "Horizontal Load-Carrying Capacity Calculation" is illustrated below.

As illustrated in the Chapter 1, the Japanese seismic design code has two-phases. The first-phase design is the allowable stress design for the safety and serviceability of buildings during medium-level earthquake ground motion.

The second-phase earthquake design, which is the ultimate strength design called "Horizontal Road Carrying Capacity Calculation" targets preventing the building collapse to ensure life safety.

2.1.2.2 Design Data

(i) Material Properties

Concrete: $f'_c = 24$ N/mm^2 (= 3,500 psi)

Reinforcement: $f_y = 390$ N/mm^2 (= 60,000 psi)

(ii) Service Loads

Table 2.1 Service Load Calculation

					For Slab	For Frame	For Earthquake
Roof	Finish Mortar	t=25	500	D.L.	6700	6700	6700
	Cinder Concrete	t=100	1600	L.L.	1000	600	400
	Water Proofing		200	T.L.	7700	7300	7100
	Leveling Mortar		500				
	Concrete Slab	t=150	3700				
	Ceiling		200				
			6700				
Study Room	Finish Mortar		1000	D.L.	4900	4900	4900
	Concrete Slab		3700	L.L.	2300	2100	1100
	Ceiling		200	T.L.	7200	7000	6000
			4900				

(iii) Seismic Loads

- Building Natural Period: $T = h(0.02 + 0.01\alpha), \alpha = 0, h = 24.69$ m

$T = 0.49$ sec

- Soil Type: Type II soil (soft) $T_c = 0.6\ sec,\ R_t = 1.0$
- Seismic Zone Factor: $Z = 1.0$
- Vertical Distribution Factor: A_i (Sec Sect. 8.3.3)
- Standard Shear Coefficient C_o: For First-Phase $C_i = 0.2$

  For Second-Phase $C_i = 1.0$
- Storey Shear Coefficient: $C_i = ZR_tA_iC_0$

The seismic loads for first-phase are obtained according to the above data of Table 2.2.

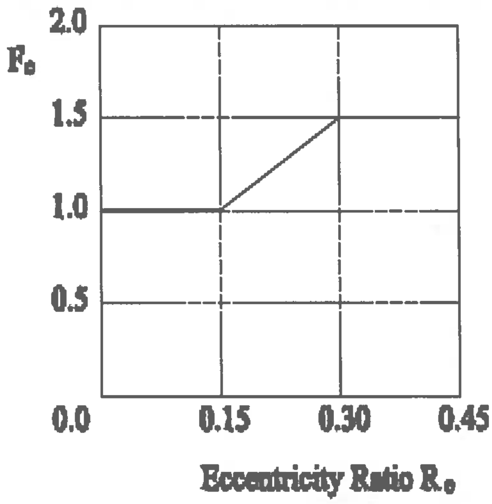
Table 2.2 Seismic Load for First Phase

Direction	Storey	W_i (kN)	$\sum W_i$ (kN)	α_i	A_i	C_i	Q_i (kN)
X,Y	6	7068.7	7068.7	0.148	1.972	0.394	2788.3
	5	8148.5	15217.2	0.320	1.575	0.315	4795.0
	4	8135.7	23352.9	0.491	1.371	0.274	6407.1
	3	8186.9	31539.8	0.664	1.223	0.244	7720.9
	2	8147.1	39686.8	0.835	1.102	0.220	8752.6
	1	7798.0	47484.8	1.000	1.000	0.200	9497.0

The drift angle, the stiffness ratio, eccentricity ratio are obtained as listed in Table 2.3

Table 2.3 Storey Drift, Drift Angle, Stiffness Ratio, Eccentricity Ratio (For X-direction)

Storey	Floor Height (mm)	Storey Drift (mm)	Drift Angle	Stiffness Ratio (R_s)	Elastic Radii (m)	Eccentricity (m)	Eccentricity Ratio (R_e)
6	4572	10.53	1/434	1.38	19.80	0.21	0.011
5	4572	14.75	1/310	0.99	20.92	0.27	0.013
4	4572	18.06	1/253	0.81	21.35	0.36	0.017
3	3658	11.73	1/312	1.00	19.88	0.62	0.031
2	3658	12.51	1/292	0.93	20.2	0.58	0.029
1	3658	12.97	1/282	0.90	21.98	0.37	0.017



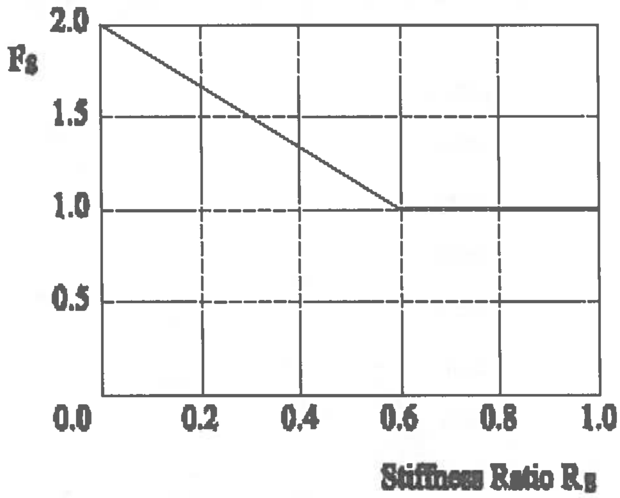


Figure 2.6 Shape Factor $F_{es} = F_e F_s$

2.1.3 Lateral Load Carrying Capacity

Generally speaking, the lateral load carrying capacity is obtained through the pushover analysis. The ultimate lateral load carrying capacity is calculated by means of incremental vertical distribution of horizontal force loading when the storey drift reached the designated value such as 1/75～1/50, or the brittle failures occur in the columns or beams. In this case, the brittle failures occur in the 2nd floor for incremental steps of 34th.

Table 2.4 Ultimate Lateral Load Carrying Capacity Q_{un} in X(L) Direction

Direction	Floor	D_s	F_e	F_s	F_{es}	Q_{ud}	Q_{un}	Q_u	Q_u/Q_{un}	
X(L)	6	0.30	1.00	1.00	1.00	13941.6	4182.4	1821.0	0.43	NG
X(L)	5	0.45	1.00	1.00	1.00	23974.9	10788.7	3131.6	0.29	NG
X(L)	4	0.45	1.00	1.00	1.00	32035.4	14415.9	4184.5	0.29	NG
X(L)	3	0.45	1.00	1.00	1.00	38604.5	17372.0	5042.6	0.29	Ng
X(L)	2	0.45	1.00	1.00	1.00	43763.0	19693.3	5716.4	0.29	NG
X(L)	1	0.45	1.00	1.00	1.00	47484.8	21368.1	6202.6	0.29	NG

The required lateral load carrying capacity "Q_{un}" is

$$Q_{un} = D_s F_{es} Q_{ud} \tag{2.1}$$

$$Q_{ud} = Z R_t A_i C_0 W_i \tag{2.2}$$

and the ultimate lateral load carrying capacity "Q_u" for each storey must exceed "Q_{un}". For example, Q_{un} is obtained for the 2nd floor of X (L) direction as below.

Where, $Z = 1.0$, $R_t = 1.0$, $A_i = 1.102$, $C_o = 1.0$, $W_i = 43763.0$ kN

$$Q_{ud} = Z R_t A_i C_0 W_i = 1.0 \times 1.0 \times 1.102 \times 1.0 \times 47484.8 = 43763.0 = 43763.0$$

$$Q_{un} = D_s F_{es} Q_{ud} = 0.45 \times 1.0 \times 43763 = 19693.3\ kN$$

The relation of the lateral force (Q) and the lateral displacement (δ) by the pushover analysis is shown in Figure 2.7. The relation of the required (Q_{un}) and ultimate (Q_u) lateral load capacity is shown in Figure 2.8.

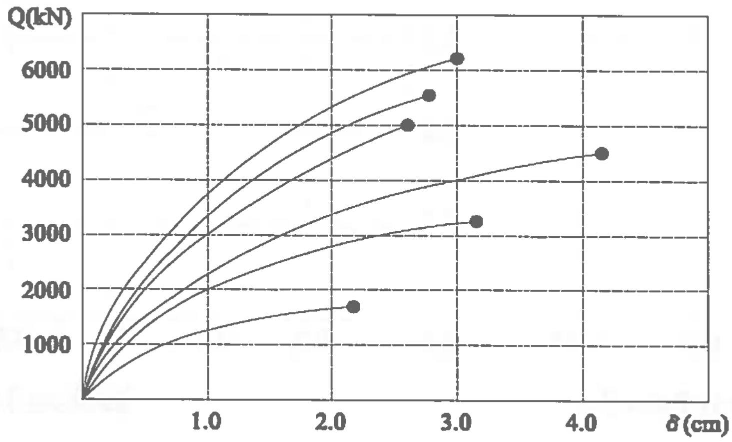


Figure 2.7 Q - ∂ Curve

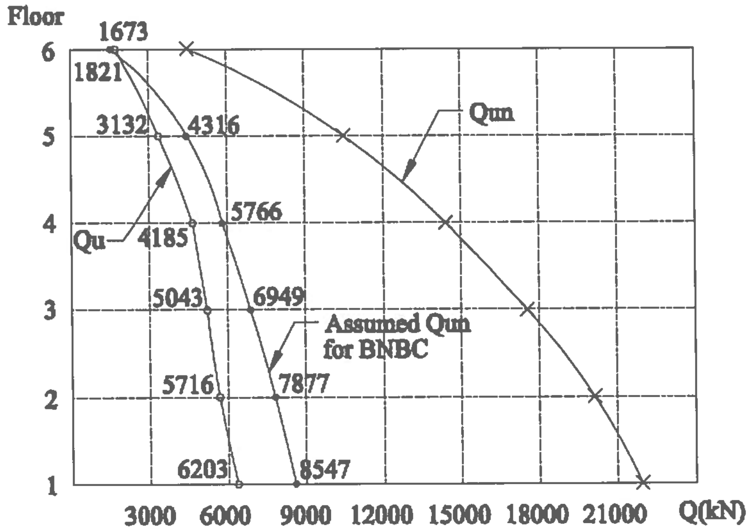


Figure 2.8 Ultimate Storey Shear Strength and Demand Curve

Q_{un} is very large comparing to Q_u (Q_u / Q_{un} < 1.0). This is because the base shear coefficient "C_0" for the major earthquake in the Japanese Code is 1.0 which is applied to the Q_{un} of Eq. (2.1). When the earthquake intensity scale of Bangladesh is compared to the Japanese scale, the base shear coefficient "C_0" for the major earthquake in Bangladesh is assumed to be between 1/2.5 and 1/3. The base of the value 1/2.5 or 1/3 is from Figure 2.9 (presented by Osamu Miyoshi on the Technical Discussion dated 1st March 2012).

The beams of the Academic Building occur brittle shear failure at the early step of the increment loads at the pushover analysis, and the ultimate shear strength of the storey is comparatively small. Provided the shear strength of the beams is increased by the closer stirrups, the ultimate shear strength of the storey increased and the Q_u/Q_{un} is very close to "1.0".

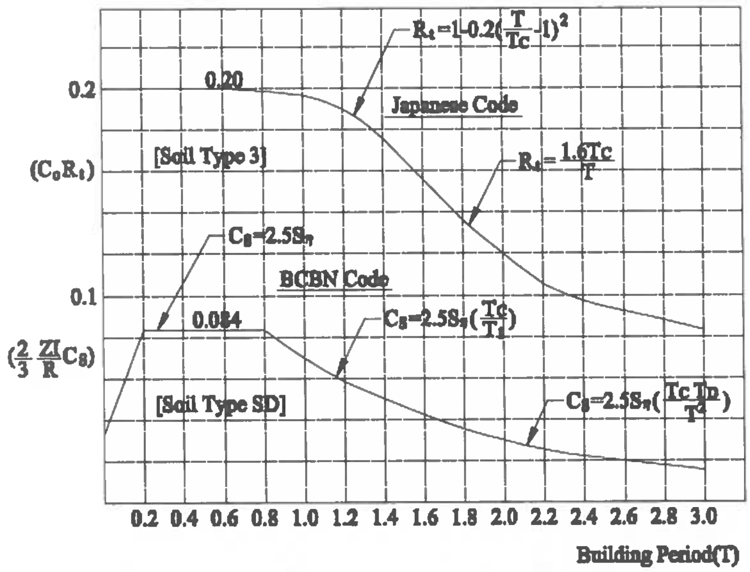


Figure 2.9 Base Shear Coefficient According to BNBC Code & Japanese Code

(Soil Type-SD & -3)

CHAPTER 3. RESPONSE & LIMIT CAPACITY CALCULATION AS PER JAPANESE EARTHQUAKE PROVISION

3.1 PERFORMANCE-BASED DESIGN

This chapter and section briefly introduces the concept of performance-based design in Japan. The reason why it is introduced is prescribed in the preface of "the Main-Manual." The seismic design code in Japan was revised in 2000 to implement performance-based design. The performance objectives are (i) life safety and (ii) damage control of a building at two corresponding levels of earthquake ground motions.

3.1.1 Concept of Performance-Based Design

The design earthquake motions are defined in terms of the acceleration response spectra specified at the engineering bedrock. On the other hand, seismic performance shall be verified by comparing the predicted response values with the building's estimated limit values. The design earthquake load is provided as accurately as possible, and the building response spectra are estimated according to the seismic load. Seismic performance shall be verified by confirming that the response spectra are less than the building's estimated limit values.

3.1.2 Earthquake Force

The earthquake force is determined from the acceleration response spectrum at the engineering bedrock which has adequate stratum thickness and stiffness extending deep underground and which confirms with the standards stipulated in (i) to (iii) below (*Ref.* 3.1).

i. The shear wave velocity of the soil is approximately 400m/second or higher.
ii. The thickness of the ground is 5m or more.
iii. Within a range equal to 5 times the thickness of the surface ground layer on a point directly under the building, the ground is inclined less than 5-degrees assuming the ground depth is uniform.

The acceleration response spectrum at the level of the structure foundation is amplified with the relation between the soil and superstructure.

3.1.3 Response

The earthquake response value is derived from the equivalent linearization of a single degree of freedom. The equivalent linearization is used as part of a nonlinear static procedure. It is a SDOF with a equivalent effective mass (M_u) using an effective period, T_{eff} and effective damping, β_{eff}.

The flow and procedure of dynamic analysis is shown by Figure 3.1-3.3 (*Ref. 3.2*).

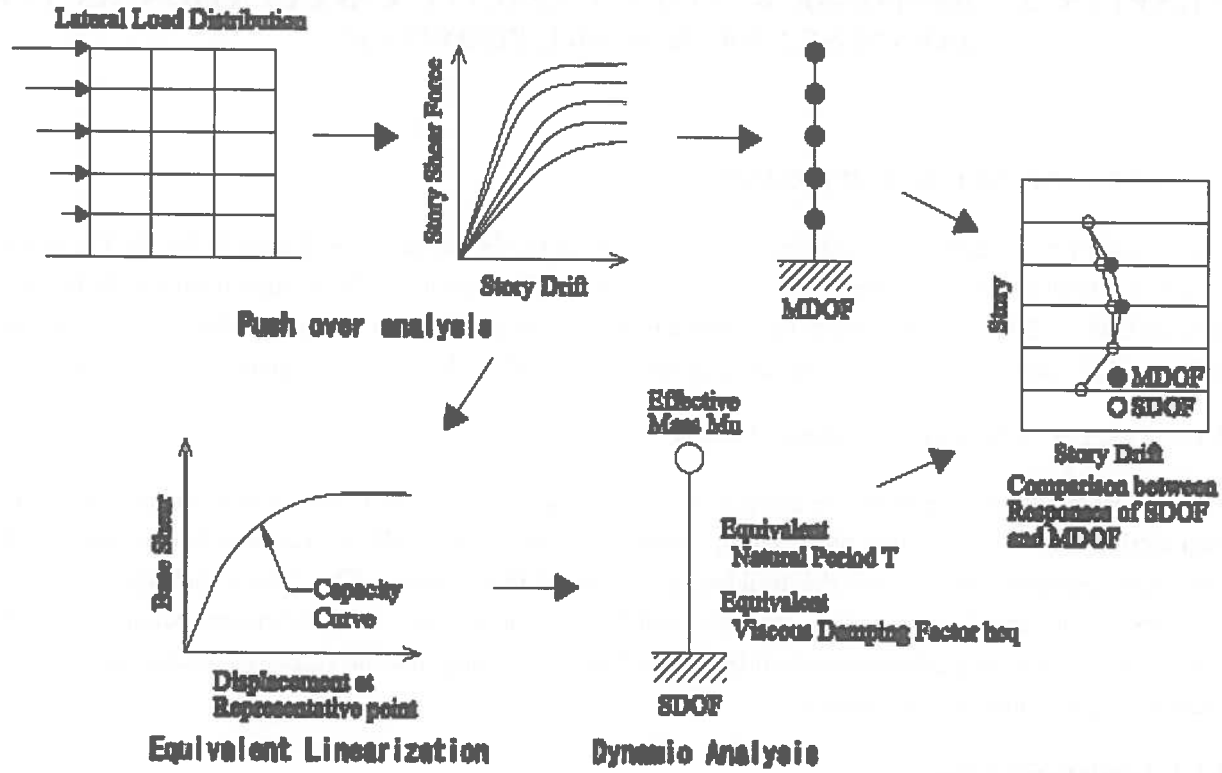


Figure 3.1 Flow of Dynamic Analysis

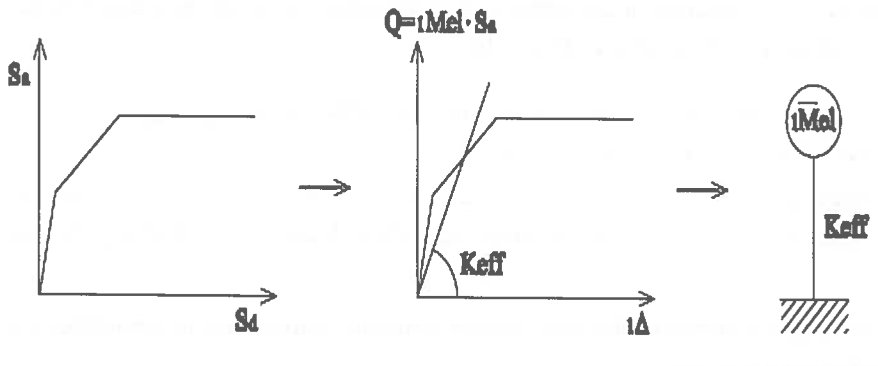


Figure 3.2 Skeleton Curve for SDOF System

In order to execute earthquake response analysis of SDOF system, modeling of the restoring force characteristics is required. As shown in Figure 3.2, multiplying the S_a components of the skeleton curve by the equivalent mass corresponding to the first mode for elastic, ${}_1M_{el}$, a skeleton curve for the representative shear vs representative displacement relation is obtained.

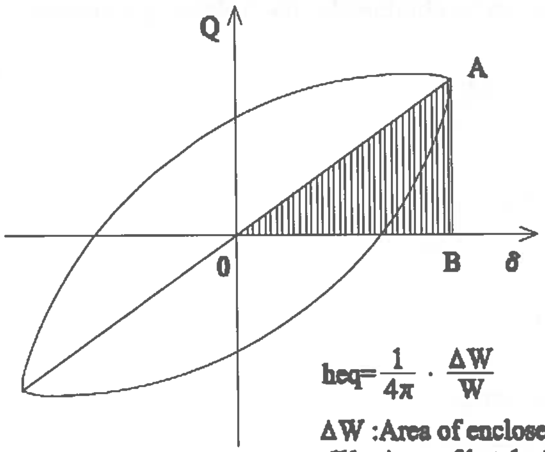


Figure 3.3 *Q-S* Curve and Energy Dissipation

The derivation of damping for spectral reduction is shown in Figure 3.3,

Where,

W=Maximum strain energy = Area of ΔOAB

ΔW=Energy dissipated by damping = Area of hysteresis loop

The procedure of analysis is as follows (*Ref. 3.2*):

(a) Obtain the shear versus displacement relation of each storey from a non-linear pushover analysis with the external force distribution corresponding to the first mode of vibration;

(b) Make the capacity spectrum (Sa-Sd curve using Eq. 3.1 and 3.2) and the analytical results obtained from step (a), namely information on the external force and displacement of each storey and base shear in each loading step;

(c) Make the skeleton curve for the base shear versus representative displacement relation by converting the Sa-Sd curve into a tri-linear curve and multiplying Sa by the equivalent mass for the elastic first mode, ${}_1\overline{M}_{el}$ (see Figure 3.2);

(d) Execute earthquake response analys for SDOF system using the equivalent mass for the elastic first mode, ${}_1\overline{M}_{el}$ and a restoring force charecteristic with the skeleton curve made in step (c) and the assumed hysteresis rule for the structural type of analyzed buildings mentioned above;

(e) Seek a loading step on the Sa-Sd curve made in step (b), which is corresponding or nearest to the maximum response of SDOF system obtained in step (d);

(f) Seek the displacement at each storey on the shear versus displacement obtained from the push-over analysis in step (a), which is corresponding to the loading step obtained in step (e);

(g) Execute earthquake response analyses for MDOF system and

(h) Compare the results of SDOF system (step (f)) with those of MDOF system.

The example of a building response and a performance limit is shown in Figure 3.5. In this figure the maximum response under the design earthquake motion is defined as the intersection (called performance point) of the demand spectrum and the capacity spectrum. The capacity spectrum presents the relation of the base shear (Sa) and the equivalent displacement (Sd) in SDOF system. (See Figure 3.4)

The capacity Spectrum (Sa-Sd curve) can be obtained by the following equation:

$$S_a = (\sum_{i=1}^{N} m_i \delta_i^2 / (\sum_{i=1}^{N} m_i \delta_i)^2) Q_B \qquad (3.1)$$

$$S_d = (\sum_{i=1}^{N} m_i \delta_i^2 / (\sum_{i=1}^{N} P_i \delta_i)^2) S_a \qquad (3.2)$$

Where

m_i : lumped mass in the i-th storey

δ_i : displacement of the i-th storey

Q_B: base shear at the maximum response

P_i: external force applied at the i-th storey

Using Eqs. (3.1) and (3.2) and the information on the external forces and displacements of each story and the base shear in each loading step obtained from non-linear push-over analysis with external force distribution proportioned to the first mode, a Sa-Sd curve can be described as shown in Figure 3.4.

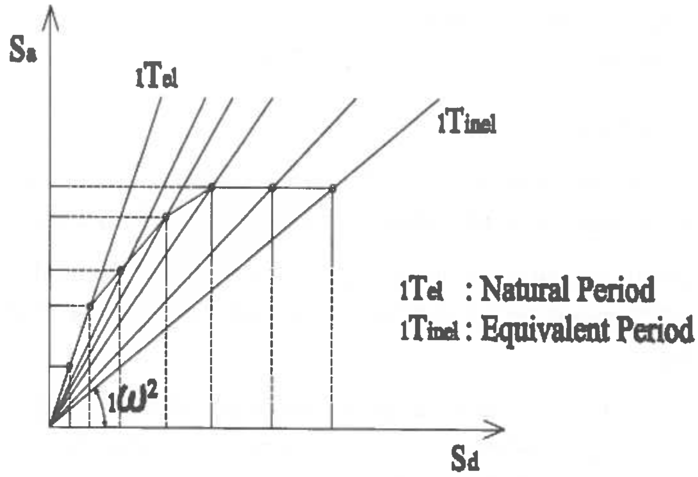


Figure 3.4 Capacity Spectrum

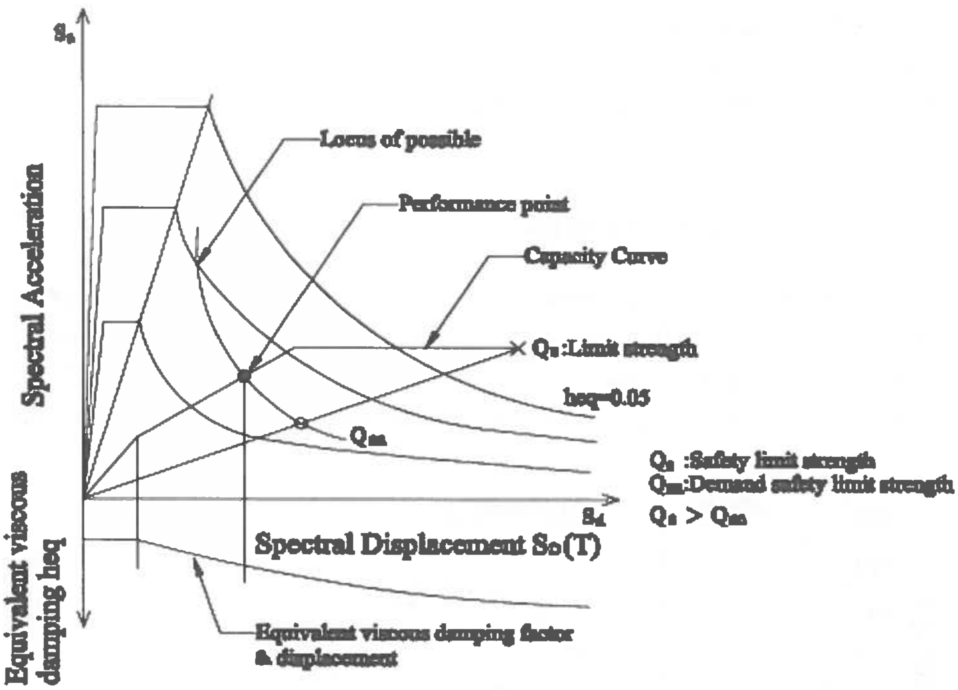


Figure 3.5 Performance Criteria Using Demand Spectrum of Design Earthquake Motions and Capacity Curve of an Equivalent SDF System (*Ref.*3.3)

3.1.4 Performance Verification

The performance verification is proved by confirming that the response point (mark o) is below the limit point in Figure 3.5 (mark ×).

3.1.5 Required Performance

(i) The required strength at the damage limit of the buildings is not greater than the strength at damage limit. The damage limit of the building is defined that the building shall not be damaged at the middle class earthquake.

(ii) The required strength at the safety limit of the buildings is not greater than the strength at safety limit. The safety limit of the building is defined that the building shall not be collapsed at the great earthquake.

3.2 CALCULATION OF RESPONSE AND LIMIT CAPACITY

The numerical example for calculation of response and limit capacity is shown as below.

3.2.1 Design Acceleration Response Spectrum

The design acceleration response spectrum for the earthquake ground motion can be derived as

$$S_A = G_s(T) Z S_0 \qquad (3.3)$$

Where

S_A : Design acceleration response spectrum

$G_S(T)$: Amplification factor by surface geology
Z : Seismic hazard zoning coefficient
S_O : Standard acceleration response spectrum
T : Natural period (sec)

(i) Standard Acceleration Response Spectrum

The standard acceleration response spectrum is set on the engineering bedrock exposed in the deep underground as the design acceleration response spectrum. The definition of the engineering bedrock is prescribed at the previous section (Sec. 3.1.2). The standard response spectrum S_o (T) at exposed engineering bedrock for the safety limit is represented by the following equations (See Figure 3.6).

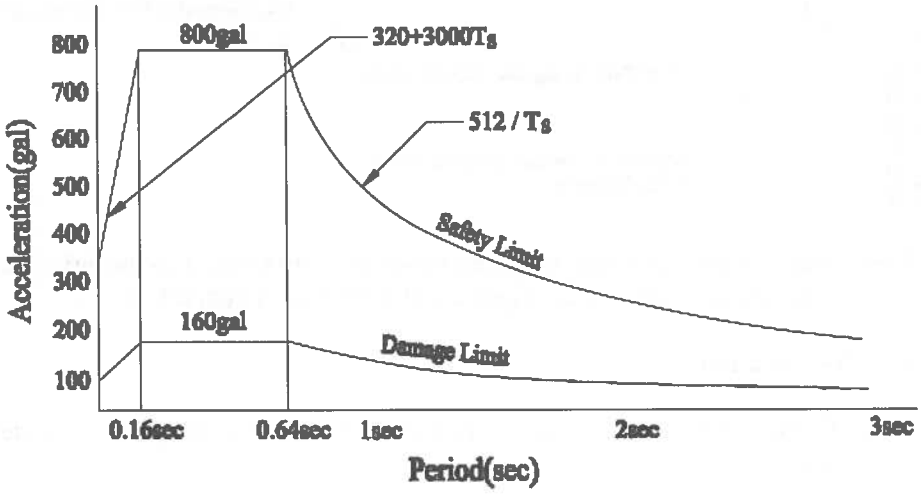


Figure 3.6 Standard Acceleration Response Spectrum

This is an extremely rare earthquake, which is expected to occur once in approximately 500 years.

$$S_0(T) = \begin{cases} 320 + 3000T_s & \text{for } T_s < 0.16 \quad (3.4a) \\ 800 & \text{for } 0.16 < T_s < 0.64 \quad (3.4b) \\ 512/T_s & \text{for } 0.64 < T_s \quad (3.4\ c) \end{cases}$$

Where,
S_o (T) is the spectrum ordinate (m/Sec^2)

The standard response spectrum for the damage limit is scaled as 1/5 of for the safety limit shown below.

$$S_o(T) = \begin{cases} 64 + 600T_d & \text{for } T_d < 0.16 \quad (3.5a) \\ 160 & \text{for } 0.16 < T_d < 0.64 \quad (3.5b) \\ 120.4/T_d & \text{for } 0.64 < T_d \quad (3.5c) \end{cases}$$

Where, T_S and T_d are the natural period of the building for the safety limit and the damage limit, respectively.

(ii) Amplification factor by the surface ground

The value G_s representing the amplification factor of acceleration according to the geological data at the site is determined in accordance with the soil types of type 1, type 2 and type 3.

The value G_S in soil type 1 is:

$$G_s = \begin{cases} 1.5 & \text{for } T < 0.576 \quad (3.6a) \\ 0.864/T & \text{for } 0.576 \le T < 0.64 \quad (3.6b) \\ 1.35 & \text{for } 0.64 < T \quad (3.6c) \end{cases}$$

The value G_S in soil type 2 or 3 is:

$$G_s = \begin{cases} 1.5 & \text{for } T < 0.64 \quad (3.7a) \\ 1.5\left(T/0.64\right) & \text{for } 0.64 \le T < T_u \quad (3.7b) \\ g_v & \text{for } T_u < T \quad (3.7c) \end{cases}$$

Where

T : Natural Period of the building (sec)

$$T_u = 0.64\left(g_v/1.5\right)$$

$$g_v = \begin{cases} 2.025 & \text{for soil type-2} \\ 2.7 & \text{for soil type-3} \end{cases}$$

Soil type 1: Soil layer consisting of rock, stiff sand gravel, and pre-tertiary deposits.

Soil type 2: Soil layer other than type 1 and type 3.

Soil type 3: Alluvium layer mainly consisting of humus and mud whose depth is more than 30m, or filled land of more than 3m deep and worked within 30 years.

3.2.2 Maximum Response under the Earthquake Motion

In "the Response and Limit Capacity Calculation", it is confirmed that the spectral acceleration of a structure at a limit state should be higher than the corresponding acceleration of the demand spectrum.

The principle of the verification procedures is that the predicted response values caused by earthquake ground motions should not exceed the estimated limit value. In the case of a major earthquake, the maximum strength and displacement response value should be smaller than the ultimate capacity for strength and displacement.

It shall be verified that the response values predicted on the basis of the response spectra satisfy the condition of being smaller than the limit values. The limit values mean the damage-initiation limit state and the life-safety limit state.

The acceleration of the demand strength point should be less than the acceleration of the limit strength point in Figure 3.7. Conversion into SDOF system is obtained according to the following steps. (From (i) to (vi))

For the life-safety limit state, the mark or symbol with suffix "d" for the damaged initiation limit state is replaced to suffix "s", unless other wire specified.

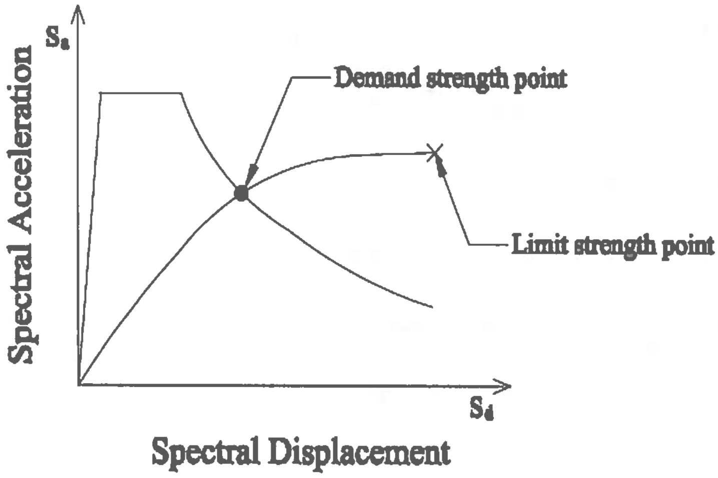


Figure 3.7 S_a-S_d **Curve of SDF System**

(i) Vertical Distribution of Lateral Force. The distribution factor of the lateral load on each storey along the building height is as below.

$$b_{di}=\begin{cases}\left\{1+\left(\sqrt{\alpha_i}-\alpha_i^2\right)\dfrac{2h(0.02+0.01\lambda)}{1+3h(0.02+0.01\lambda)}\dfrac{\sum_{j=1}^{N}m_j}{m_i}\right. & \text{for top floor} \quad (3.8a)\\[2ex] 1+\left(\sqrt{\alpha_i}-\sqrt{\alpha_i+1}-\alpha_i^2+\alpha_{i+1}^2\right)\dfrac{2h(0.02+0.01\lambda)}{1+3h(0.02+0.01\lambda)}\dfrac{\sum_{j=1}^{N}m_j}{m_i} & \text{for floors except the top floor} \quad (3.8b)\end{cases}$$

Where

α_i = The value obtained by dividing the sum of the permanent load and the imposed load of the parts supported by the parts of the height which is used to calculate for b_{di} of the building by the sum of the permanent load and imposed load of the building above ground.

h = The building height (unit: meters)

λ = The ratio to h of buildings of which the majority of the columns and beams are wooden or steel construction

m_j = The mass in the j[th] storey

N = The storey number of the building.

The nonlinear static seismic analysis (push-over analysis) is carried out based on the above mentioned vertical distribution of the lateral force in order to gain the shear force (Q_{di}, Q_{si}) and the floor displacement (δ_{di}, δ_{si}) of each storey from the basement, for the damage-initiation limit state and the life-safety limit state respectively.

(ii) Conversion to equivalent single-degree of freedom system. The equivalent single degree of freedom system is captured through the load and displacement in each storey calculated by the push-over analysis.

The effective mass of the building "M_{ud}", the natural period "T_d" and the equivalent displacement "Δ_d" is defined as,

$$M_u = \frac{\sum (m_i \delta_{di})^2}{\sum m_i \delta_{di}^2} \tag{3.9}$$

$$\Delta_d = \frac{\sum m_i \delta_{di}^2}{\sum m_i \delta_{di}} \tag{3.10}$$

$$T_d = 2\pi \sqrt{M_{ud} \frac{\Delta_d}{Q_d}} \tag{3.11}$$

Where,

M_{ud}	: the affective mass of a building (ton)
m_i	: the mass in the i-th storey (ton)
Δ_d	: the equivalent displacement of buildings (mm)
T_d	: natural period at damage limit (seconds)
δ_{di}	: the i-th storey displacement from the basement (mm)
Q_d	: the strength at damage limit of buildings (Base shear force, (N))

(Iii) The Damping Characteristics

The value (h) of the damping characteristics of buildings shall be obtained by the following equation considering damping effect of the building or its members against earthquake response,.

"h" at damage limit is 0.05.

$$h = \gamma_1 \left(1 - \frac{1}{\sqrt{D_f}}\right) + 0.05 \tag{3.12}$$

Where,

γ_1 = The coefficient representing the damping characteristics corresponding the structural system of each member as shown in Table3.1

D_f = the value representing the ductility of each member, shall be obtained by the following equation (it shall be 1 when it is less than 1)

$$D_f = \frac{\Delta_s Q_d}{\Delta_y Q_s} \tag{3.13}$$

Table 3.1 Value of γ_1

Structural system	γ_1
Member of which the composing material and the connecting of adjacent members are connected firmly	0.25
Other members or bracing members sharing the compressive force of which strength reduced by the buckling when earthquake motion affects on	0.20

(vi) Reduction Factor of Acceleration

The reduction factor for acceleration caused by the damping in the system during the earthquake motion shall be calculated by the following equation:

$$F_h = \frac{1.5}{1+10h} \tag{3.14}$$

(v) The distribution factor of the Acceleration

The distribution factor of the acceleration on each storey is obtained by the multiplying of the effective mass, the vertical distribution factor of the lateral force and the correction factor *p, q* shown by the Table 3.2 and Table 3.3.

Table 3.2 "*p*" Value

Storey \ Response period at damage limit	0.16 sec. and less	More than 0.16 sec.
1	$1.00 = \frac{0.20}{0.16} T_d$	0.8
2	$1.00 = \frac{0.15}{0.16} T_d$	0.85
3	$1.00 = \frac{0.10}{0.16} T_d$	0.90
4	$1.00 = \frac{0.05}{0.16} T_d$	0.95
More than 5	$1.00 = \frac{0.20}{0.16} T_d$	1.00
In this table, T_d shall represent the natural response period at damage limit of the building.		

p: The value calculated by the equation in this table based on the number of stories and the response period at damage limit.

Table 3.3 "q" Value

Effective mass ratio	Less than 0.75	$0.75\dfrac{\sum m_i}{M_{ud}}$
	0.75 and more	1.0
Effective mass ratio is the ration of the effective mass to the whole mass of the building		

q: The value calculated by the equation in this table according to the effective mass ratio.

The vertical distribution factor of the acceleration "B_{di}" and "B_{si}" is obtained by the following equation.

$$B_{di} = pq\left(\frac{M_{ud}}{m_j}\right)B_{si} \tag{3.15}$$

(vi) The Seismic Force

The seismic force acting on a storey of the building during the earthquake shall be calculated as the summation of the forces generated horizontally in the storey and above stories calculated by the formula shown by the equations below according to the response period at damage limit and safety limit, respectively.

$$P_{di} = \begin{cases} \left(0.64+6T_d\right)m_i B_{di} Z G_s & \text{for } T_d < 0.16 & (3.16a) \\ 1.6 m_i B_{di} Z G_s & \text{for } 0.16 \le T_d < 0.64 & (3.16b) \\ \dfrac{1.024 m_i B_{di} Z G_s}{T_d} & \text{for } 0.64 \le T_d & (3.16c) \end{cases}$$

$$P_{si} = \begin{cases} \left(3.2+30T_s\right)m_i B_{si} F_h Z G_s & \text{for } T_s < 0.16 & (3.17a) \\ 8 m_i B_{si} F_h Z G_s & \text{for } 0.16 \le T_s < 0.64 & (3.17b) \\ \dfrac{5.12 m_i B_{si} F_h Z G_s}{T_s} & \text{for } 0.64 \le T_s & (3.17c) \end{cases}$$

Where

P_{di}, P_{si}: Forces acting horizontally on each storey (kN)
T_d, T_s : Response period of building (at damage limit & safety limit)
B_{di}, B_{si}: A value that represents the distribution of the acceleration generated in each storey of the building (at damage limit & safety limit)
m_i: Mass of each storey(ton)
Z: Zoning Factor (1.0~0.7)
G_s: A value that represents the amplification factor of the acceleration caused by the surface ground
F_h: Response spectrum reduction factor in natural period at safety limit

References

3.1 Notification No 1457 of the Ministry of Construction/May 31, 2000 in Japanese Building Standard Law.

3.2 Hiroshi KURAMOTO, Masaomi TESHIGAWARA, Toshifumi OKUZONO, Norihide KOSHIKA, Masaharu TAKAYAMA and TomihiroHORI; PREDICTING THE EARTHQUAKE RESPONSE OF BUILDING USING EQUIVALENT SINGLE DEGREE OF FREEDOM SYSTEM; 12 WCEE 2000.

3.3 S.Otani, H. Hiraishi, M.Midorikawa and T. Teshugawara; New Seismic Provision in Japan

CHAPTER 4. EXAMPLE OF RESPONSE AND LIMIT CAPACITY CALCULATION BASED ON JAPANESE EARTHQUAKE PROVISION

4.1 CASE STUDY-DHAKA MEDICAL COLLEGE EXTENSION BUILDING

4.1.1 Building Data

a) Number of Storey: 6
b) Floor Area: 735.41 m^2 Total Area: 4326.20 m^2
c) Structural System: Reinforced Concrete Frame
d) Height: 24.69 m
e) Foundation: reinforced concrete Bore-Pile

4.1.2 Structural Design Description

Structural design description is refereed to Sec. 12.1.1 & 12.1.2.

4.1.3 Structural Design Principle

As mentioned in Sec. 12, the Japanese seismic design code was revised in June 2000 to implement a performance-based structural engineering framework. The code provides two performance objectives: life safety and damage limitation of a building at two corresponding levels of earthquake motions.

The buildings shall be designed so that not only the building as a whole but also no storey of the building should collapse against the maximum earthquake ground motion based on the historical earthquake data. The building damages shall be prevented against the earthquake ground motion that may occur more than once in the lifetime of the building. A return period for such seismic ground motion may be 30 to 50 years at once or twice in the building lifetime.

But, actually speaking of the structural engineering field in Japan now, the design method of "horizontal Load-Carrying Capacity Calculation" is used extensively by the structural engineers.

4.1.3.1 Design Data

(i) Material Properties

Concrete: $f'_c = 24$ N/mm^2 ($\fallingdotseq$ 3,500 psi)
Reinforcement: $f_y = 390$ N/mm^2 ($\fallingdotseq$ 60,000 psi)

(ii) Service Loads

See Table 4.1 below.

Table 4.1 Service Load Calculation (N/m^2)

					For Slab	For Frame	For Earthquake
Roof	Finish Mortar	t = 25	500	D.L.	6700	6700	6700
	Cinder Concrete	t = 100	1600	L.L.	1000	600	400
	Water Proofing		200	T.L.	7700	7300	7100
	Leveling Mortar		500				
	Concrete Slab	t =150	3700				
	Ceiling		200				
			6700				
Study Room	Finish Mortar		1000	D.L.	4900	4900	4900
	Concrete Slab		3700	L.L.	2300	2100	1100
	Ceiling		200	T.L.	7200	7000	6000
			4900				

4.1. 3. 2 The Flow-chart for Calculation

Figure 4.1 shows the flow-chart for response and limit calculation as per Japanese earthquake provision

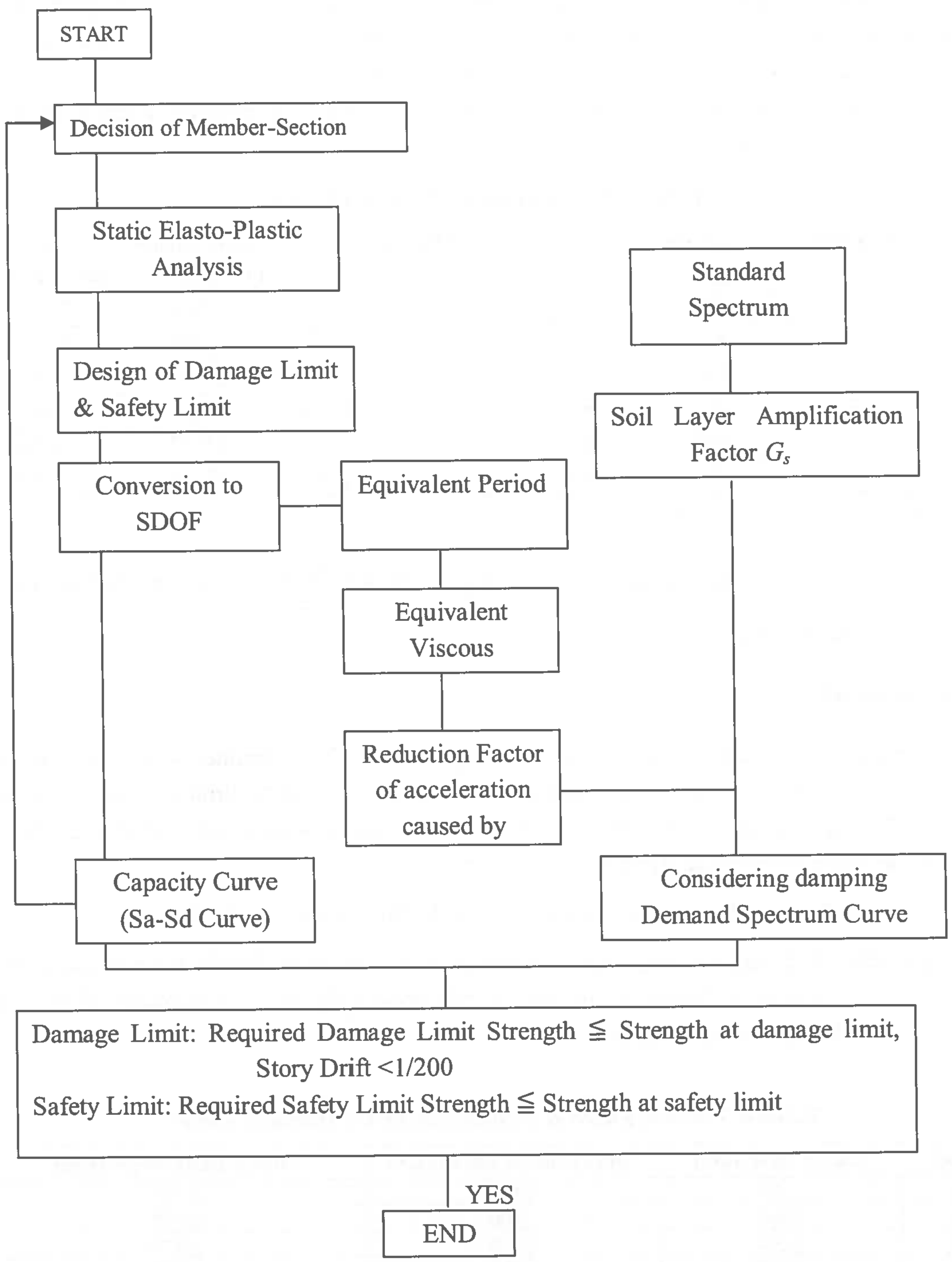


Figure 4.1 Flow of Response & Limit Capacity Calculation

4.2 ANALYSIS

4.2.1 Static Elasto-Plastic Analysis

The computer soft program is "the static and elasto-plastic incremental analytic program, which is consistent and performs calculation of component rigidity, load, member stress, a calculation route, a proportioning of members, horizontal load-carrying capacity, etc.
The lateral force is controlled by the lateral shear distribution factor A_i. Regarding A_i and α_i refer to Eqs. (1.3) and (1.4) in Section 1.3.1.

Table 4.2 Calculation of Lateral Force

Floor	Mass/floor m_i (t)	Total Mass $\sum m_i$ (t)	α_i	Natural Period T (sec)	A_i	Story Shear Force Q_i (kN)	Lateral Force P_i (kN)
6	721	721	0.148	0.49	1.972	13934	13934
5	831	1552	0.320		1.575	23955	10021
4	830	2382	0.491		1.371	32004	8049
3	835	3217	0.664		1.223	38557	6553
2	831	4048	0.835		1.102	43717	5160
1	796	4844	1.000		1.000	47471	3754

Building natural period $T= 0.02h = 0.02 \times 24.69$ (m) = 0.49 sec.

The lateral force is obtained being assumed, C_o =1.0 and $Q_i = ZR_t A_i C_0 \sum_{i=1}^{n} W_i$ where, Z=1.0 (Zone Factor), R_t (Vibration Characteristics Factor) = 1.0.

4.2.2 Analysis Results

The interaction curve of the lateral shear force (Q) and the story drift (δ) obtained by the static and elasto-plastic incremental analysis is shown in Figure 4.2. Damage and safety limit are defined as below, respectively. This diagram shows the relation of the storey shear force and storey drift when the lateral force applies the frame from left side and in X-direction.

(i) Damage Limit: Either column or Beam is yield. The 4th floor beam is yield first.

(ii) Safety Limit: Drift angle of any one of storey exceed 1/50 or the brittle failures occur at either columns or beams. In this case, brittle shear failures occur in middle floor beams in the life safety limitation state.

Table 4.3 Share Force & Displacement for Damage Limit

Floor	Storey Drift (cm)	Storey Shear Force (kN)	Storey Drift Angle (rad)
6	1.31	1450	1/351
5	2.04	2500	1/225
4	2.59	3320	1/178
3	1.67	4000	1/222
2	1.73	4520	1/214
1	1.80	4930	1/205

Table 4.4 Share Force & Displacement for Safety Limit

Floor	Storey Drift (cm)	Storey Shear Force (kN)	Storey Drift Angle (rad)
6	2.02	1822	1/228
5	3.18	3133	1/145
4	4.06	4186	1/113
3	2.69	5044	1/138
2	2.86	5718	1/129
1	3.00	6205	1/123

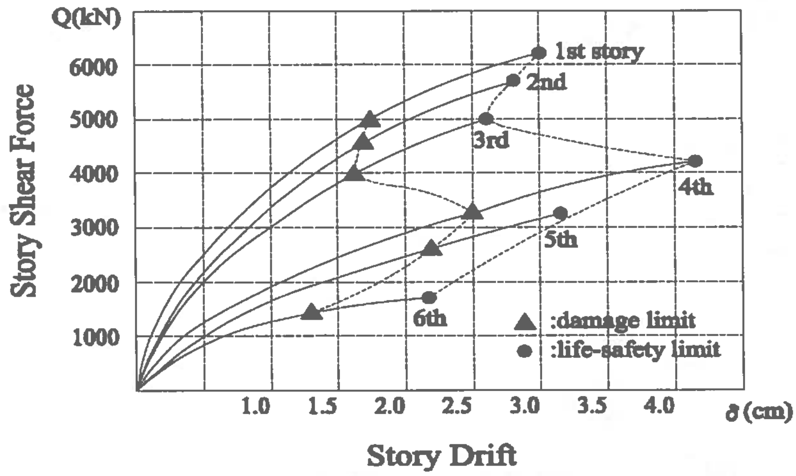


Figure 4.2 Diagram for Storey Shear Force-Storey Drift

The ultimate strength of the Hospital is controlled by the beam shear reinforcement. The damage limit strength and the safety limit strength are shown in Table 4.3 and Table 4.4 respectively. The base-shear coefficient (C_B) of the safety limit strength is 0.13. (C_B= 6205/ (4844 × 9.8) = 0.13)

4.3 CONVERSION TO SINGLE DEGREE OF FREEDOM SYSTEM

The capacity curve of a converted single degree of freedom system is obtained by the Eq.(4.1) to Eq. (4.3),where the base shear force obtained by a static and elasto-plastic analysis at every step, the lateral and relative displacement (δ_i) of ith storey from the foundation and every floor's mass are combined.

The effective mass ratio and the equivalent natural period are shown in Eq. (4.4) and Eq. (4.5) respectively.

The relation of the shear force and the relative displacement obtained by the static and elasto-plastic analysis is shown in Figure 4.3. The capacity curve of SDOF is shown in Figure 4.4.

$$S_a = \frac{\sum_{i=1}^{N} m_i \delta_i^2}{\left(\sum_{i=1}^{N} m_i \delta_i\right)^2} Q_B \tag{4.1}$$

$$S_d = \Delta = \sum_{i=1}^{N} \frac{m_i \delta_i}{M_u} = \frac{\sum_{i=1}^{N} m_i \delta_i^2}{\sum_{i=1}^{N} m_i \delta_i} \tag{4.2}$$

$$M_u = \frac{\left(\sum_{i=1}^{N} m_i \delta_i\right)^2}{\sum_{i=1}^{N} m_i \delta_i^2} \tag{4.3}$$

$$\textit{Effective mass ratio} = \frac{M_u}{\sum_{i=1}^{N} m_i} \tag{4.4}$$

$$T = \frac{2\pi}{\omega} = 2\pi\sqrt{M_u/K} = 2\pi\sqrt{M_u\Delta/Q_B} = 2\pi\sqrt{S_d/S_a} \tag{4.5}$$

Where
S_a = Response acceleration (Gal)
S_d = Response displacement (cm)
m_i = ith storey mass (t)
δ_i = Relative displacement from Foundation
Q_B = Shear Force at each step (kN)
Δ = Equivalent displacement at each step (cm)
M_u = Equivalent mass at each step (t)
K = Equivalent stiffness at each step

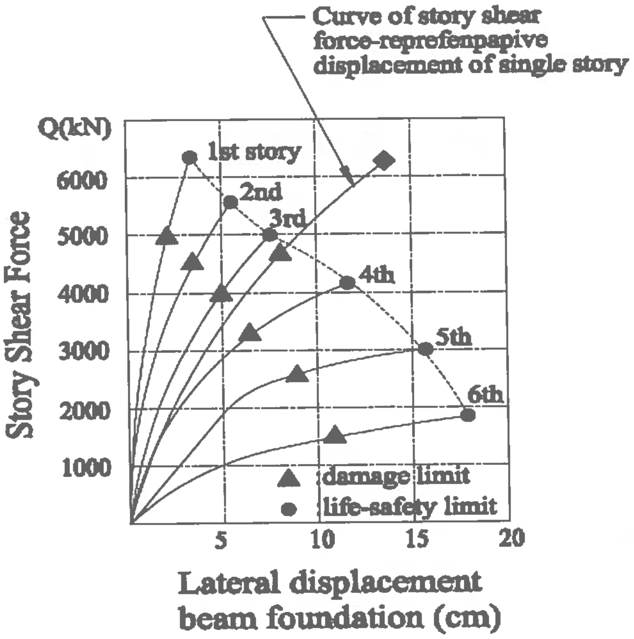


Figure 4.3 Storey Shear Force vs Relative Displacement

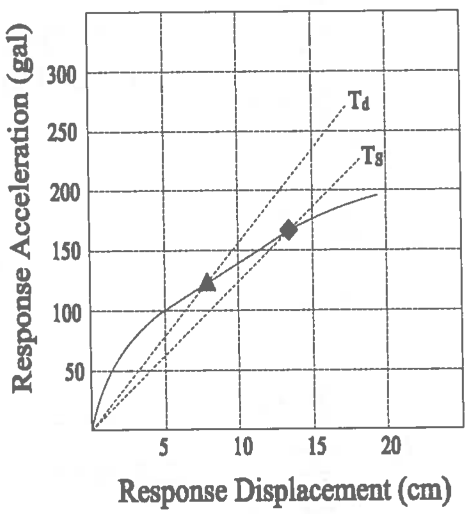


Figure 4.4 S_a - S_d Curve

Table 4.5 m_i, $\sum m_i\delta_i$, $\sum m_i\delta_i^2$ for Damage Limit

Floor	m_i (t)	δ_i^2 (cm)	δ_i^2	$m_i\delta_i^2$	$m_i\delta_i$	$\Sigma m_i\delta_i$
6	721	11.14	124.1	89476.1	8031.9	
5	831	9.88	96.6	80274.6	9168.7	
4	830	7.79	60.7	5038.1	6465.7	
3	835	5.20	27.0	2254.5	4342.0	
2	831	3.53	12.5	10387.5	2933.4	
1	796	1.80	3.2	2547.2	1432.8	
				255611.4	31374.5	

Table 4.6 m_i, $\sum m_i\delta_i$, $\sum m_i\delta_i^2$ for Safety Limit

Floor	m_i (t)	δ_i^2 (cm)	δ_i^2	$m_i\delta_i^2$ (10^3)	$m_i\delta_i$	
6	721	17.81	317.20	228.7	12841	
5	831	15.79	249.32	207.2	13121	
4	830	12.61	189.01	132.0	10466	
3	835	8.55	93.10	61.0	7139	
2	831	5.86	34.34	28.5	4870	
1	796	3.00	9.00	7.2	2388	
				664.6×10^3	50825	

The capacity curve for the damage limit

$$S_a = \frac{\sum_{i=1}^{N} m_i \delta_i^2}{\left(\sum_{i=1}^{N} m_i \delta_i\right)^2} Q_B = \frac{255{,}611.4}{(31{,}374.5)^2} Q_B = 2.6\times10^{-4} Q_B = 1.27 \square 130(Gal)$$

$$S_d = \frac{\sum_{i=1}^{N} m_i \delta_i^2}{\sum_{i=1}^{N} m_i \delta_i} = \frac{255{,}611.4}{31{,}374.5} = 8.1\ cm$$

$$M_u = \frac{\left(\sum_{i=1}^{N} m_i \delta_i\right)^2}{\sum_{i=1}^{N} m_i \delta_i^2} = \frac{31{,}374.5^2}{255{,}611.4} = 3850\ ton$$

$$Effective\ mass\ ratio = \frac{M_u}{\sum_{i=1}^{N} m_i} = \frac{3850}{4844} = 0.795$$

$$T = 2\pi\sqrt{S_d / S_a} = 2\times3.14\sqrt{8.1/130} = 1.57\sec$$

The capacity curve for the safety limit

$$S_a = \frac{\sum_{i=1}^{N} m_i \delta_i^2}{\left(\sum_{i=1}^{N} m_i \delta_i\right)^2} Q_B = \frac{664.6\times10^3}{(50825)^2} Q_B = 2.57\times10^{-4}\times6205 = 1.59 \square 160(Gal)$$

$$S_d = \frac{\sum_{i=1}^{N} m_i \delta_i^2}{\sum_{i=1}^{N} m_i \delta_i} = \frac{664.6\times10^3}{50825} = 13.0\ cm$$

$$M_u = \frac{\left(\sum_{i=1}^{N} m_i \delta_i\right)^2}{\sum_{i=1}^{N} m_i \delta_i^2} = \frac{(50825)^2}{664.6\times10^3} = 3887\ ton$$

$$EffectiveMassRato = \frac{M_u}{\sum_{i=1}^{M} m_i} = \frac{3887}{4844} = 0.802$$

$$T = 2\pi\sqrt{S_d / S_a} = 2\times3.14\sqrt{13/160} = 1.79\sec$$

4.4 ACCELERATION REDUCTION FACTOR F_h

4.4.1 Based on the ${}_m h_{ei}$ of Each Member

The reduction factor of acceleration caused by the damping in the system during the earthquake motion, F_h shall be calculated by the following equation.

$$F_h = \frac{1.5}{(1+10h)} \tag{4.6}$$

Where, h is the damping ratio of the building.

Equivalent damping ratio *"h"* is obtained by the following methods.

$${}_m h_{ei} = \gamma_1 \left(1 - \frac{1}{\sqrt{{}_m D_{fi}}} \right) \tag{4.7}$$

Where

${}_m h_{ei}$ = equivalent vicious damping ratio of the members

γ_1 = 0325 (For members rigidly connected to adjacent members).

${}_m D_{fi}$ = ductility parameter of the member " i "

$$h = \frac{\sum_{i=1}^{N} {}_m h_{ei}\, {}_m W_i}{\sum_{i=1}^{N} {}_m W_i} + 0.05 \tag{4.8}$$

Where

${}_m W_i$ is the product of the deformation and the capacity, that is strain energy.

4.4.2 Based on the Conversion to SDF

The equivalent viscous damping ratio *"h"* is obtained by the following equation in accordance with the Figure 13.4 showing the capacity curve of the converted SDOF.

$$h = \gamma_1 \left(1 - \frac{1}{\sqrt{D_f}} \right) + 0.05 \tag{4.9}$$

$$D_f = \frac{K_d}{K} = \Delta \frac{Q_d}{(\Delta_d Q)} \tag{4.10}$$

$$K = \left(\frac{2\pi}{T} \right)^2 M_u = \omega^2 M_u \tag{4.11}$$

Where

D_f = Ductility parameter of the building.

K_d = Stiffness for damage limit.

K = Equivalent stiffness at each step.

$\Delta \cdot Q$ = Spectral displacement and base shear at each step.
Δ_d, Q_d = Displacement at damage limit and the ultimate strength at damage limit.
T = Equivalent period at each step.

$$D_f = \frac{\Delta Q_d}{(\Delta_d Q)} = \frac{130\times 13}{8.1\times 160} = 1.3$$

$$h = \gamma_1\left(1-\frac{1}{\sqrt{D_f}}\right)+0.05 = 0.25\left(1-\frac{1}{\sqrt{1.3}}\right)+0.05 = 0.081$$

$$F_h = \frac{1.5}{(1+10h)} = \frac{1.5}{(1+10\times 0.081)} = 0.83$$

4.5 DEMAND SPECTRUM

4.5.1 Standard Spectrum

The standard spectrum is defined as the acceleration response spectrum with the damping ratio 5% at the exposed engineering bedrock (shear wave velocity is or more than about 400m/s).

The standard spectrum at the major earthquake is given by the Eq. (4.13a-c), according to the Japanese Code. The assumed earthquake ground motion in Bangladesh may be one third or two and half times of the earthquake ground motion in Japan, approximately.

In Japan

$$S_{a0}(T, h=0.05) = \begin{cases} 320+3000T_s & \text{for } T_s < 0.16 & (4.13a) \\ 800 & \text{for } 0.16 \le T_s < 0.64 & (4.13b) \\ 512/T_s & \text{for } 0.64 \le T_s & (4.13c) \end{cases}$$

In Bangladesh (assumed to be one third of Japanese Code)

$$S_{a0}(T, h=0.05) = \begin{cases} 107+1000T_s & \text{for } T_s < 0.16 & (4.14a) \\ 267 & \text{for } 0.16 \le T_s < 0.64 & (4.14b) \\ 170/T_s & \text{for } 0.64 \le T_s & (4.14c) \end{cases}$$

The standard spectrum at the moderate earthquake is one-fifth of the major earthquake (see Figure 4.5).

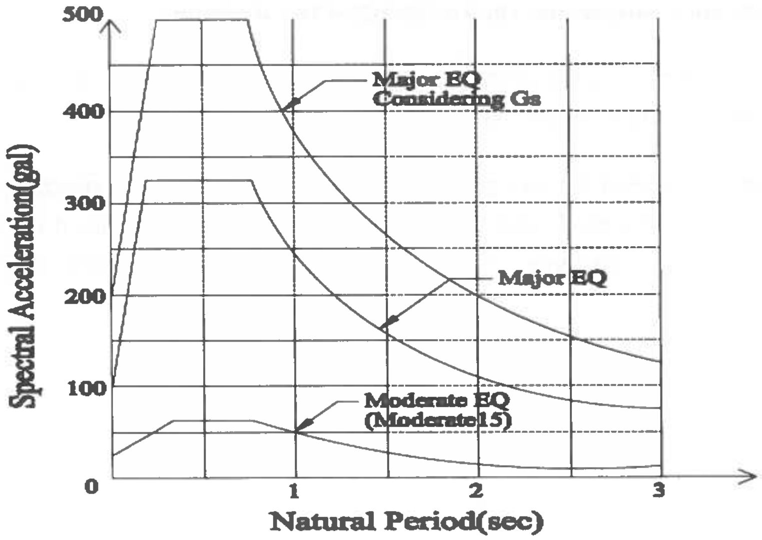


Figure 4.5 Spectral Acceleration vs Natural Period

4.5.2 Amplification Factor by the Surface Ground

The value G_s in soil type 2 is assumed in this design example as follows.

$$G_s = \begin{cases} 1.5 & \text{for } T < 0.64 \quad (4.15a) \\ 1.5\left(T/0.64\right) & \text{for } 0.64 \le T < T_u \quad (4.15b) \\ g_v & \text{for } T_u < T \quad (4.15c) \end{cases}$$

The demand spectrum considering the amplification factor by the surface ground is shown in Figure 4.6 (The curves in Figure 4.6 are assumed to be half than Japanese Code).

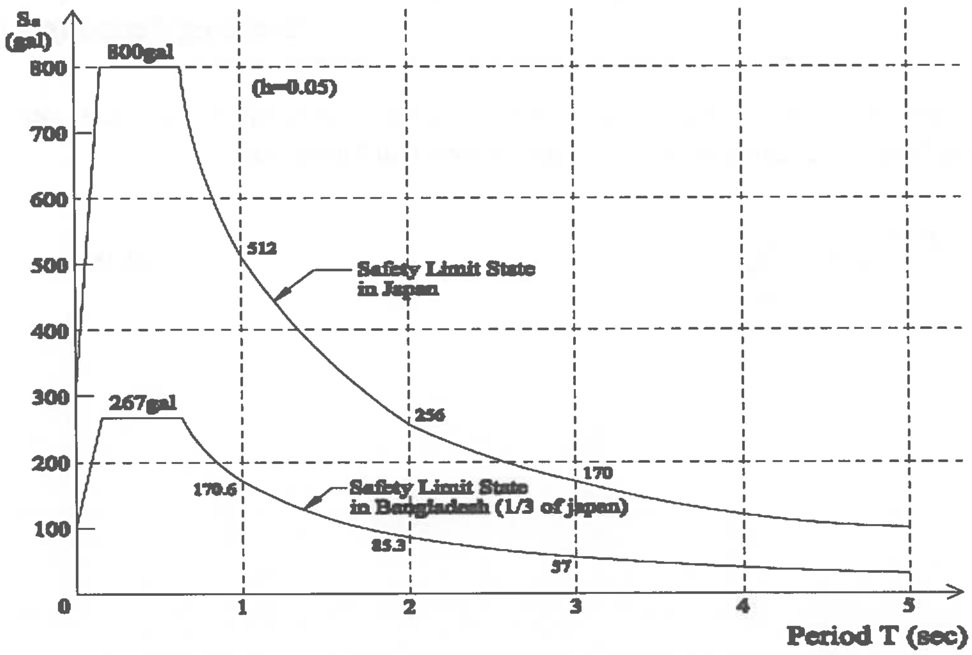


Figure 4.6 An Assumed Response Spectrum of JP & BAN

4.5.3 Demand Spectrum Considering the Damping of the Building

The demand spectrum considering the damping is obtained by multiplying "F_h" to the demand spectrum considering the soil amplification factor "G_s"

The demand spectrum considered the damping is shown in Figure 4.7. The spectrum graph shown in this figure is based on the natural period, and the building capacity curve is based on the response spectral displacement. The spectrum graph shown in this figure shall be transferred to S_a-S_d Curve.

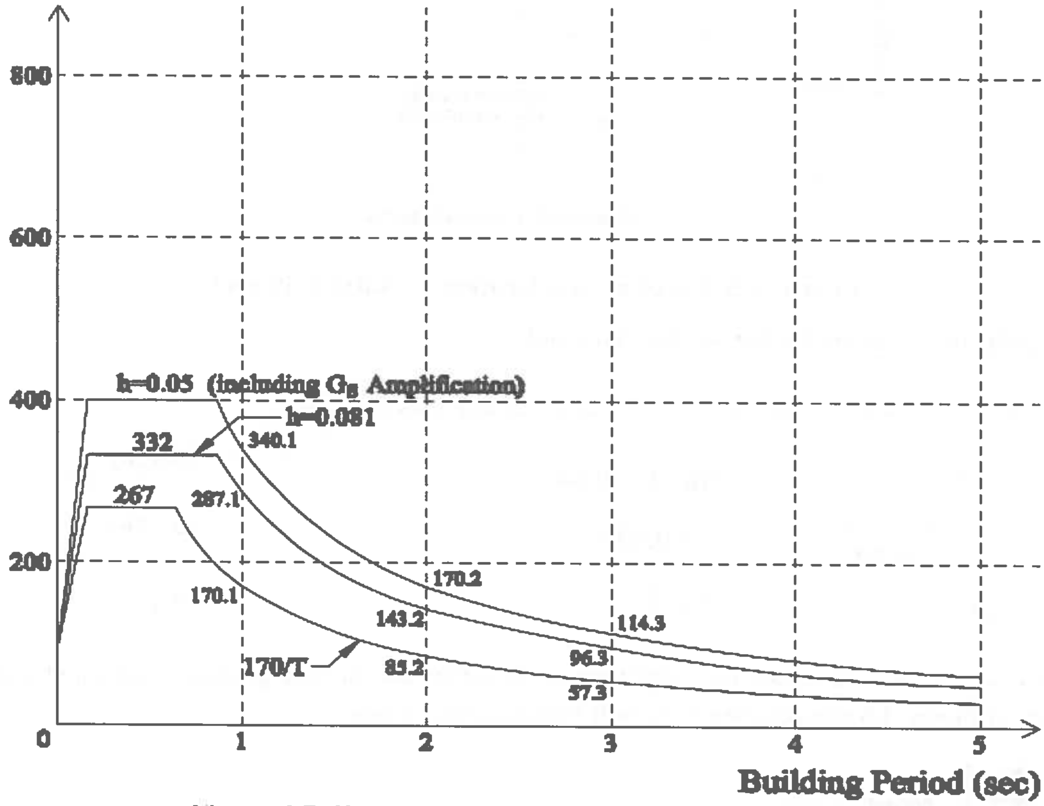


Figure 4.7 Response Spectrum Considering G_s and h_{eq}

The spectral displacement (S_d) is obtained by the following equation using the equivalent period at each step. The demand spectrum displayed by S_a-S_d curve is shown in Figure 4.8.

$$S_d = \left(\frac{T}{2\pi}\right)^2 S_a = \frac{S_a}{\omega^2} \tag{4.16}$$

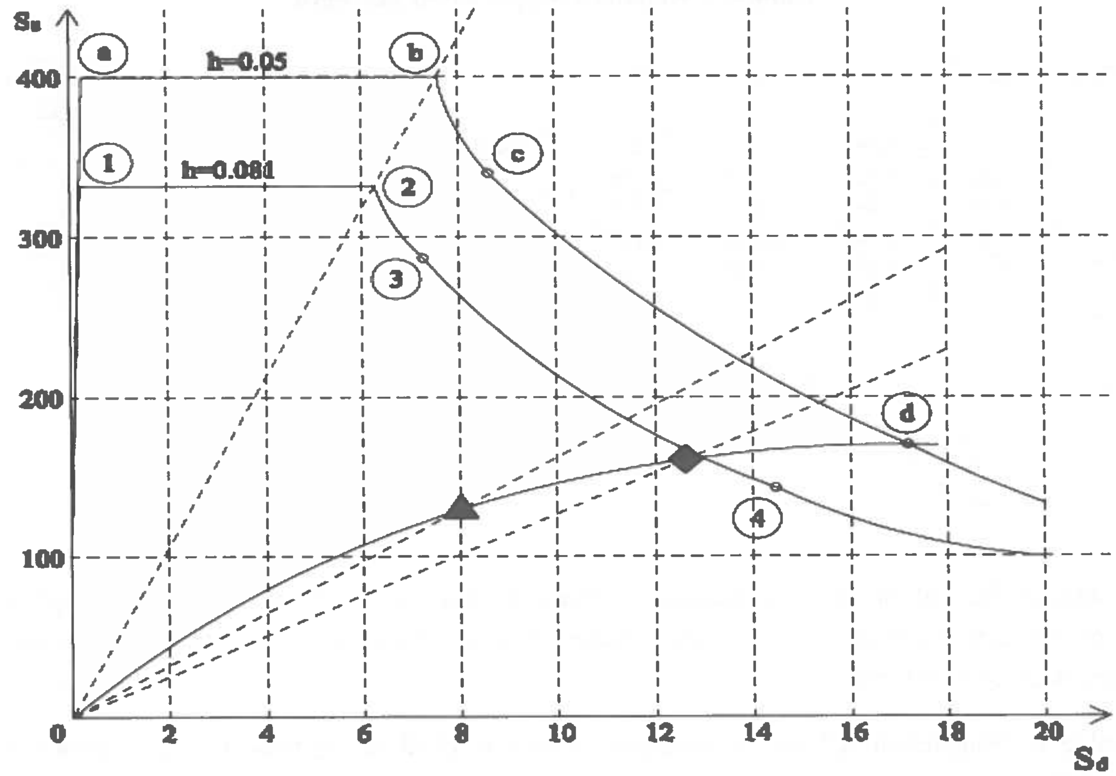


Figure 4.8 S_a-S_d Curve

4.6 VERIFICATION OF THE SAFETY LIMIT

4.6.1 Performance Point

The response spectrum of the building is defined as an intersection of the demand spectrum (based on S_a-S_d curve) of the design earthquake motion considering the damping and the capacity curve (S_a-S_d Curve) of an equivalent SDF system (Figure 4.8).

4.6.2 Required Ultimate Strength Q_{sn} at Safety Limit

The required ultimate strength "Q_{sn}" at safety limit is obtained as an intersection of the equivalent stiffness line (dotted line in Figure 4.8) at the safety limit strength and the demand spectrum considering the damping ratio.

The required safety limit strength is given in Table4.7.

The required shear strength considering "F_h" is compared with the safety limit strength (ultimate shear strength), and it is confirmed that the latter values are greater than the former values. In this building example, the required shear strength is over the ultimate shear strength in every storey.

Table 4.7 Required Safety Limit Strength

Floor	m_i (t)	α_i	b_{si}	B_{si*}	P_{si}^{**}(kN)	Required Shear Strength	Ultimate Shear Strength
6	721	0.148	1.971	1.582	1828	1828	1822
5	831	0.320	1.233	0.986	1313	3141	3133
4	830	0.491	0.992	0.794	1056	4197	4186
3	835	0.664	0.834	0.667	893	5090	5044
2	831	0.835	0.602	0.482	642	5732	5718
1	796	1.000	0.475	0.380	485	6216	6205

h=24.69m, M_{us}=3887, $\sum m_i$=4844, T_s=*1.79*, G_s=2.025, F_h=0.83

$$B_{si*} = pq\frac{M_{su}}{\sum_{j=1}^{N} m_j} b_{si}$$

P_{is} is obtained by multiplication of standard acceleration spectrum "S_o", zone factor "Z", amplification factor by the surface ground "G_s", reduction factor of the acceleration "F_h", vertical distribution factor "B_{si}" and mass of i-th storey.

Sao value in Bangladesh (Dhaka) is assumed to be one third of Japanese Code as shown in Eq. (4.14a~14c).

$$P_{si} = \frac{1}{3} \times \frac{5.12}{T_s} \cdot m_i \cdot B_{si} \cdot F_h \cdot Z \cdot G_s$$

In case of 6th floor,

$$P_{si} = \frac{1}{3} \times \frac{5.12}{1.79} \times 721 \times 1.582 \times 0.83 \times 1.0 \times 2.025 = 1828$$

When "1/3" is applied, the ultimate shear strength just satisfy the required shear strength, and "1/2.5" is applied, the ultimate shear strength is a little shorter than required one.

From Eq. (13.16):

(1) $[0.159 \times 0.16]^2 \times 332 = 0.21$ cm; (a) $[0.159 \times 0.16]^2 \times 400 = 0.26$ cm

(2) $[0.159 \times 0.864]^2 \times 332 = 6.26$ cm; (b) $[0.159 \times 0.864]^2 \times 400 = 7.55$ cm

(3) $[0.159 \times 1.0]^2 \times 287 = 7.25$ cm; (c) $[0.159 \times 1.0]^2 \times 340 = 8.60$ cm

(4) $[0.159 \times 2.0]^2 \times 143 = 14.46$ cm; (c) $[0.159 \times 2.0]2 \times 170 = 17.19$ cm

APPENDIX: PART-II

APPENDIX I. PILE FOUNDATION DESIGN IN LIQUEFACTION SOIL

Section 4.5 Liquefaction of ground, and Section 6.6 Lateral resistance and lateral displacement in "Recommendations for Design of Building Foundations (AIJ) 2001"is originally translated for materials of Pile Foundation Design in Liquefaction Soil as below,

1. Section 4.5 Liquefaction of ground (page 61 – 71 of original copy)

1. Evaluate the possibility of liquefaction occurrence in an earthquake by appropriate means for saturated sand soil.
2. Evaluate the degree of liquefaction, deformation of ground after liquefaction, degree of deformation and ground rigidity, fall in subgrade reaction, etc. for ground where the possibility of liquefaction has been judged to be high.
3. In foundation design for ground where the possibility of liquefaction has been judged to be high, foundation design methods considering the effects of liquefaction must be selected as well as providing measures as required.

Grounds that have liquefied will cause settling and leaning of the spread foundation by its complete loss of bearing capacity or the apparent loss of rigidity. Furthermore, dynamic and residual lateral displacement and settling by liquefaction and lateral flow ground may lead to damage to the pile foundation. For retaining walls and sub-structures, the soil pressure will increase by liquefaction and could result in possible damages. In addition, liquefied soil will behave like a liquid with unit weight of almost double that of water, and by this, underground installations with small unit weight will float with the increase in buoyancy and reduced friction. To prevent these kinds of damages, in the basic design for liquefied ground, in addition to predicting the possibility of liquefaction, the decrease in ground rigidity and subgrade reduction, increase in ground deformation, changes in soil pressure, buoyancy and friction must be determined, and its effects appropriately considered and measures be taken appropriately as required.

The degree of liquefaction and damage will vary widely with the density of the soil. In loose sand, deformation will progress while strength and rigidity will remain small, leading to severe damage, while with compacted sand, the ground strength will recover when deformation reaches a certain extent, and the damage will be relatively smaller. This phenomenon, to distinguish it from liquefaction, shall be named "cyclic mobility." The extent of damage will also differ with the depth of the liquefied stratum. In this guideline, as an index to evaluate the degree of liquefaction, dynamic lateral displacement of the ground surface caused by the shear deformation of the liquefied stratum shall be applied.

The design flow for the foundation structure for liquefied ground shall be divided into 1) Liquefaction judgment and ground deformation forecast and 2) Design of foundation considering liquefaction. Liquefaction judgment, ground deformation forecast, evaluation of subgrade reaction and rigidity and the concept of foundation design are described hereunder.

1. Liquefaction judgment

(1) Soil stratum to be considered

Saturated soil stratum requiring liquefaction judgment is of an alluvial formation, generally located in depth of around 20 m or less below the ground surface, and the type of soil to be considered is soil with fine fraction content of 35% or less. However, there have been reports where liquefaction has occurred in artificially created ground such as reclaimed ground, ultra plastic silt with fine fraction content of 35% or higher and silt with soil water content close to its liquid limit, therefore liquefaction study must be made for soil with clay content (soil fraction of soil diameter of 0.005 mm or less) of 10% or less, or reclaimed or filled ground with a plasticity index of 15% or less. Possibility of liquefaction cannot be denied for gravel with fine fractions or surrounded by soil stratum of low permeability, and study for liquefaction must also be made in these cases.

(2) Liquefaction risk forecast

Liquefaction judgment may be in accordance with the following procedure, using Figures 4.5.1 through 4.5.4.

(a) Equivalent cyclic shear stress ratio generated at each depth of the ground concerned shall be obtained from the following equation:

$$\frac{\tau_d}{\sigma_z'} = \gamma_n \frac{\alpha_{max}}{g} \frac{\sigma_z}{\sigma_z'} \gamma_d \qquad (4.5.1)$$

Where, τ_d is the amplitude of equivalent constant cyclic shear stress (kN/m^2) generated on the lateral plane; σ'_z is the effective overburden pressure (effective vertical stress) (kN/m^2); γ_n is the correction coefficient concerning the equivalent number of cycles and is 0.1 (M-1); M is the seismic magnitude; α_{max} is the design lateral acceleration at ground surface (cm/s^2); g is the gravity acceleration (980 cm/s^2); σ_z is the total overburden pressure at the depth concerned (total vertical stress) (kN/m^2); and γ_d is the reduction coefficient of the ground not being a rigid body, and expressed by the following:

$$\gamma_d = 1 - 0.015z \qquad (4.5.2)$$

Where, z is the depth from the ground surface expressed in meters.

(b) Corrected N-value (N_a) corresponding to the depth is obtained from the following equation:

$$N_1 = C_N \qquad (4.5.3)$$

$$C_N = \sqrt{98/\sigma_z'} \qquad (4.5.4)$$

$$N_a = N_1 + \Delta N_f \qquad (4.5.5)$$

Where, N_1 is the reduced N-value, C_N is the conversion coefficient concerning confining pressure, ΔN_f is the corrected N-value increment corresponding to fine fraction content F_C as per Figure 4.5.2. N shall be actually measured N-value by the pigtail-hook drop method or the automatic drop method.

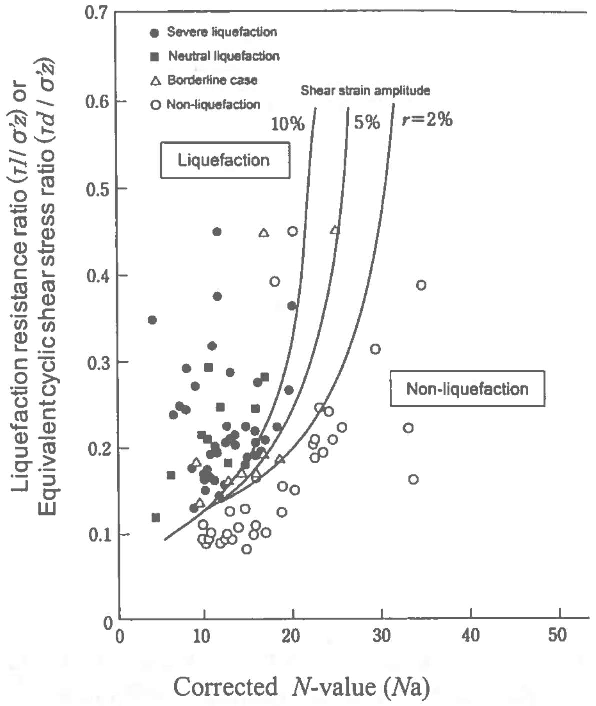


Figure 4.5.1 Relationship Between Corrected N-Value, Liquefaction Resistance and Dynamic Shear Strain[4.5.3)]

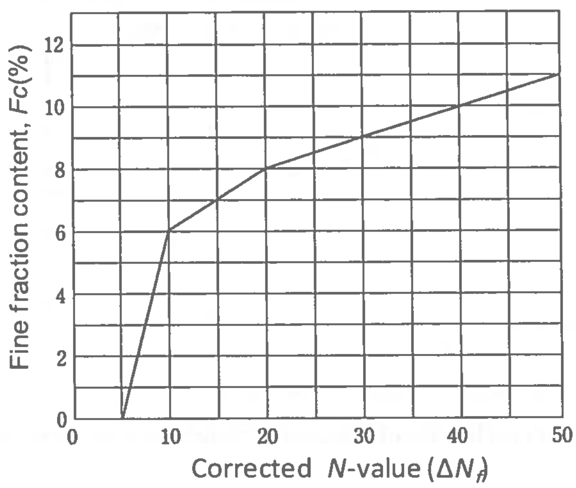


Figure 4.5.2 Fine Fraction Content and Correction Coefficient for N-Value[4.5.3)]

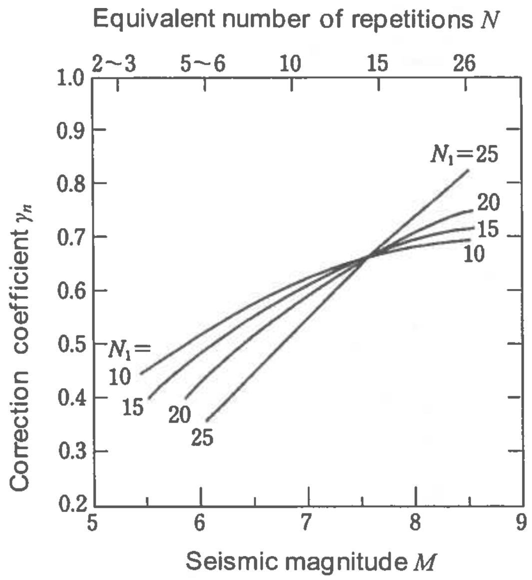


Figure 4.5.3 Relationships Among Corrected *N*-Value, Magnitude, Number of Repetitions and Correction Coefficient[4.5.3)]

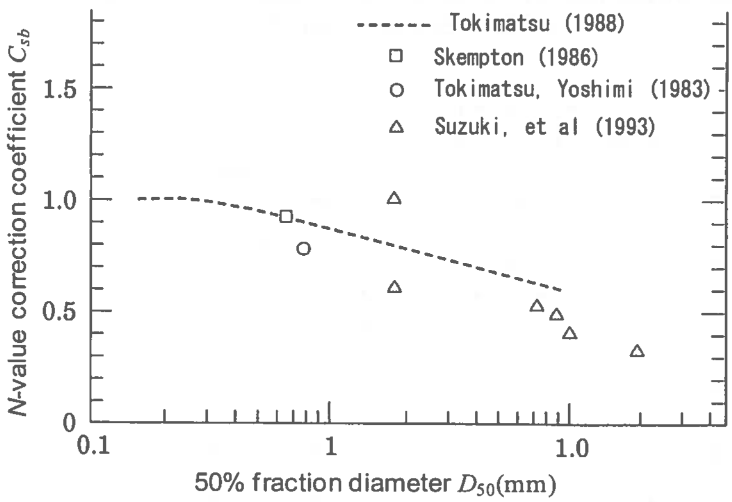


Figure 4.5.4 *N*-Value Correction Coefficient for Sand/Gravel Ground[4.5.3)]

(c) Using the limit shear strain curve at 5% in Figure 4.5.1, liquefaction resistance ratio $R = \tau_l \,/\, \sigma'_z$ for saturated soil layer corresponding to corrected N-value (N_a) is obtained. Here, τ_l is the liquefaction resistance at the lateral plane.

(d) Safety factor F_l against the generation of liquefaction at each depth shall be calculated by the following equation:

$$F_l = \frac{\tau_l / \sigma_z'}{\tau_d / \sigma_z'} \qquad (4.5.6)$$

It is judged that there shall be no liquefaction for soil stratum where F_l is greater than 1, but on the other hand, if it is 1 or less, the possibility exists and the smaller the number, the larger the risk of liquefaction, and the greater the depth of the soil stratum where F_l is 1 or less, the greater the risk.

In the above procedure, the lateral acceleration at ground surface in the calculation of cyclic shear stress ratio (τ_d / σ'_z) is the result of ground response, and is greatly influenced by the ground properties. However, we recommend the use of 150 to 200 cm/s^2 hereunder, for check of the damage limits, and 350 cm/s^2 for the check of ultimate limit study. 350cm/s^2 corresponds almost to the maximum value observed on liquefied ground in the 1995 South Hyogo Prefecture Earthquake. If shear stress needs to be obtained more appropriately, it can be obtained by defining the input seismic vibrations against the engineering foundation by maximum velocity and spectrum and (1) obtain the depth distribution of shear stress through response analysis or (2) obtain shear stress by the method indicated in (a) above after assuming ground surface acceleration. The accuracy of γ_d in Eq. (4.5.1) will worsen with the increase in depth. When this situation is to be assumed, the use of response analysis is recommended. Equivalent linearization analysis may be permitted for this analysis and in such a case, it can be done by factoring γ_n in Eq. (4.5.1) to the maximum shear stress ratio to obtain (τ_d / σ'_z) and following the procedures in the guideline, as follows. In addition, it is also possible to determine γ_n from Figure 4.5.3 considering the calculated effective number of repetitions of the seismic wave and the ground density.

For gravel type soil where the *N*-value tends to be large, *N*-value correction coefficient C_{sb} in Figure 4.5.4 may be used temporarily for its 50% fraction diameter D_{50}. However, in view of its reliability, it is desirable that it be studied in a comprehensive manner together with estimation method using large-scale penetration tests and estimation method using *S*-wave velocity. Furthermore, for soil with relatively high fine-fraction with low reliability *N*-value, the use of estimation method to use cone penetration test or indoor-testing method for undisturbed samples to obtain liquefaction resistance, without relying on estimation method using *N*-value, is recommended.

Figure 4.5.5 indicates the relationship between the cone penetration resistance and liquefaction strength. From Eq. (4.5.7), by defining the compensated cone penetration resistance value q_{c1}, compensated for the effects of confining pressure and gradation, liquefaction strength can be assumed from Figure 4.5.5 and the safety factor against liquefaction can be estimated from Eq. (4.5.6).

$$q_c = F(I_c)q_c C_N \qquad (4.5.7)$$

Where, q_c is the cone penetration resistance measured at the original location; C_N is the correction coefficient for confining pressure, the same value as Eq. (4.5.4); $F(I_C)$ is the correction coefficient for gradation (behavioral property of the soil) obtained from Figure 4.5.6; and I_C is the soil behavioral property index, which is given by the following equation:

$$I_c = \{(3.47 - \log Q_t)^2 + (\log F_R + 1.22)^2\}^{0.5} \qquad (4.5.8)$$

Where,

$$Q_t = (q_c - \sigma_z)/\sigma_z' \qquad (4.5.9)$$

$$F_R = f_s/(q_c - \sigma_z) \times 100 \qquad (4.5.10)$$

Where, f_s is the peripheral surface friction resistance.

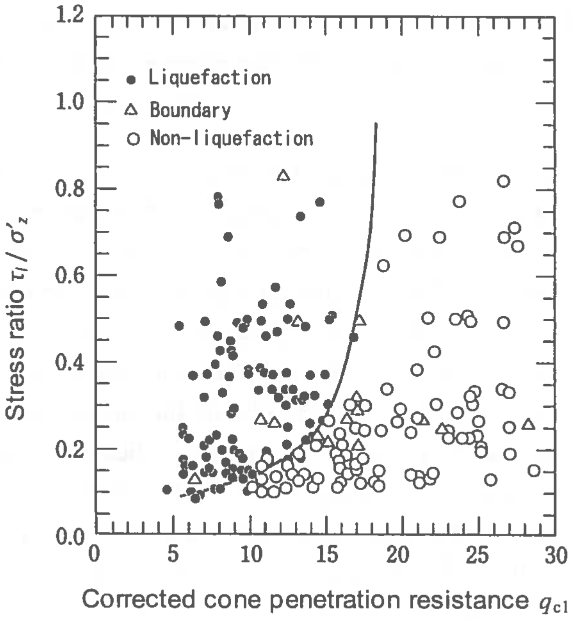


Figure 4.5.5 Relationship Between Cone Penetration Resistance and Liquefaction Strength[4.5.2)]

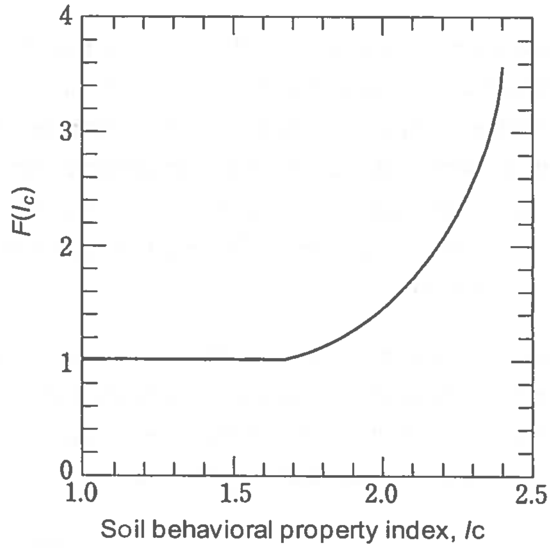


Figure 4.5.6 Fine Fraction Content and Cone Penetration Resistance Correction Coefficient[4.5.2)]

2. Ground property accompanying liquefaction and forecasting of ground displacement

For grounds where the possibility of its liquefaction has been judged to be high, the information required for the foundation design of the building shall be evaluated as follows:

(1) The degree of liquefaction and the forecasting of ground displacement accompanying liquefaction and lateral flow of soil.

(a) The degree of dynamic lateral displacement, residual lateral displacement, amount of settlement, degree of liquefaction and the forecasting of dynamic lateral displacement of lateral ground may be conducted by the following procedure after liquefaction judgment, in addition to an appropriate response analysis.

(i) Estimate the cyclic shear strain for each stratum corresponding to N_a and τ_d / σ'_z from Figure 4.5.7.

(ii) Assuming shear strain of each stratum γ_{cy} to generate in one-single direction, the maximum lateral displacement distribution during vibration is taken by integration method of shear strain in the vertical direction.

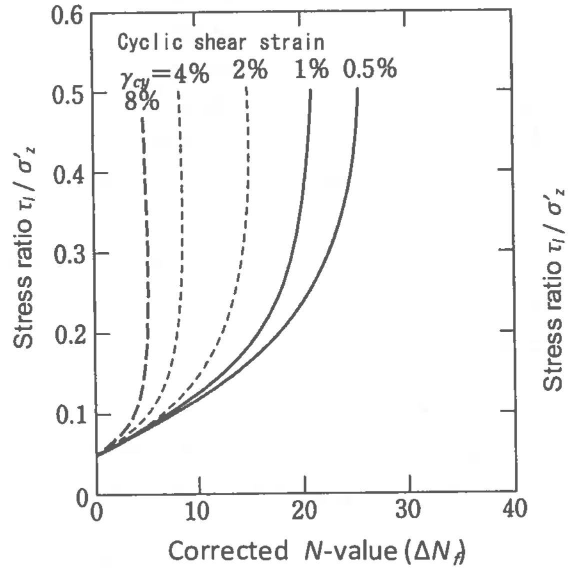


Figure 4.5.7 Relationship Between Corrected *N*-value and Cyclic Shear Strain[4.5.2)]

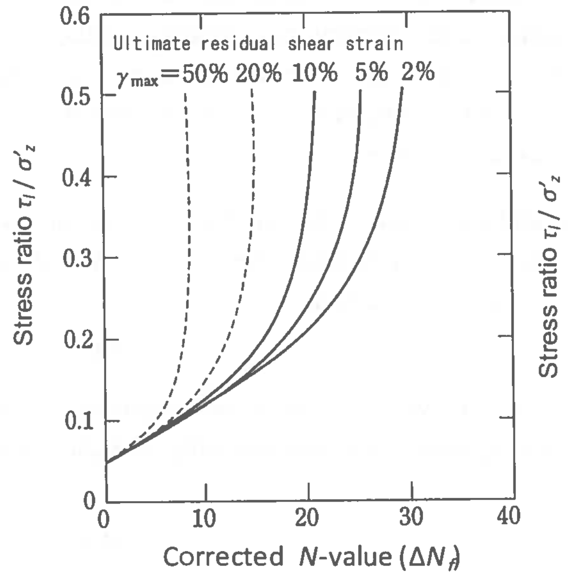


Figure 4.5.8 Relationship Between Corrected *N*-Value and Ultimate Residual Shear Strain[4.5.3)]

(iii) Ground surface displacement D_{cy} is used as an index for the degree of liquefaction. The degree of liquefaction shall be evaluated as per the value of D_{cy} in Table 4.5.1.

Table 4.5.1 Relationship Between D_{cy} and the Degree of Liquefaction

D_{cy} (cm)	Degree of liquefaction
0	none
- 05	slight
05 -10	small
10 - 20	medium
20 - 40	large
40 -	extremely large

Likewise, when amount of settlement S is to be obtained, use Figure 4.5.7 as is but replacing γ_{cy} to read as volumetric strain ε_v[4.5.4].

Where, when a ground with liquefying stratum depth H = 8 m and N_a= 10 should liquefy, then γ_{cy}= 4% from Figure 4.5.7, and therefore D_{cy} = S = 32 cm. When a ground of H = 5 m and N_a= 20 should liquefy, then γ_{cy}= 1%, and therefore D_{cy} = S = 5 cm.

(b) The possibility and amount of lateral flow, the horizontal distribution of the amount of lateral flow, and the ground displacement near embankments and gentle slopes when lateral flow accompanying liquefaction occurs. The lateral displacement of ground near embankments is also estimated from Eq. (4.5.11):

$$D_0 = \min(D_W, D_{\max}) \qquad (4.5.11)$$

Where, D_w is the embankment displacement, D_{max} is the ultimate residual lateral displacement of the lateral flow ground after the quake, and obtained by replacing Figure 4.5.7 and γ_{cy} of the process indicated in (a) above by Figure 4.5.8 and γ_{max}, respectively. D_w shall be estimated separately by other appropriate means, however, when the seismic resistance of the embankment is extremely low and a large seismic motion is assumed, D_0 can be taken to be D_{max}.

The relationship between the ground displacement D_0near the embankment and the zone of influence of lateral flow L is indicated in Figure 4.5.9 for ground where the liquefaction stratum depth H is almost uniform and is represented by the following equation:

$$D_0\, L \fallingdotseq 50 D_0 \qquad (4.5.12)$$

Furthermore, the relationship between lateral displacement amount D and distance x from the embankment can be indicated and expressed non-dimensionally in Figure 4.5.10 and by the following equation.

$$D(x)/D_0 = (1/2)^{5x/L} \qquad (4.5.13)$$

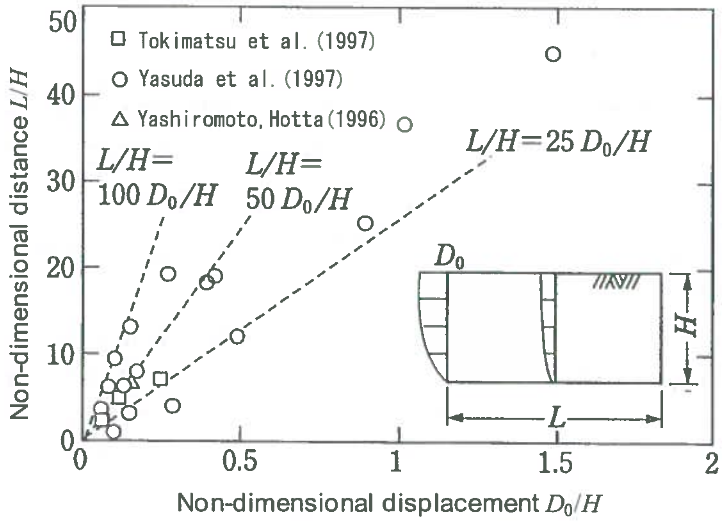


Figure 4.5.9 *D* Embankment Displacement and Zone of Influence of Lateral Flow[4.5.3)]

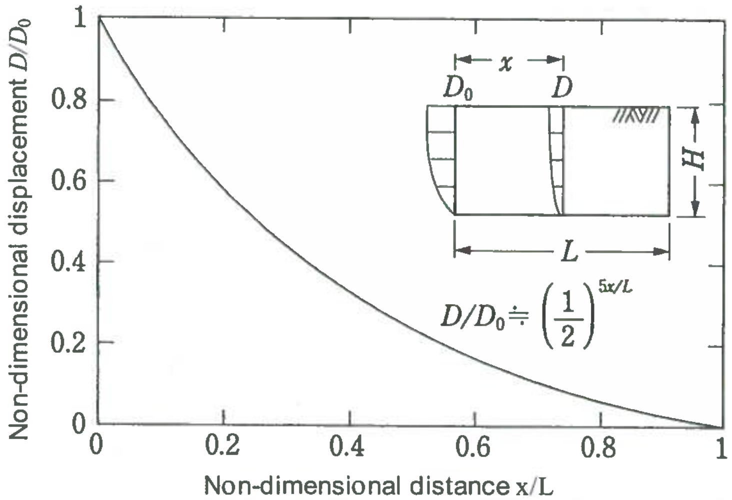


Figure 4.5.10 Lateral Flow Amount and Distance from Revetment[4.5.3)]

Considering a liquefied ground of H = 10m, γ_{max} = 20%, the amount of ground displacement near the embankment and its zone of influence L are expected to be approximately D = 2 m and L = 100 m for a d embankment displacement of 4 m, and approximately D = 1 m and L = 50 m in case of a embankment displacement of 1 m. The distribution of lateral displacement in the depth direction can be approximated by the following equation:

When $z < z_w$,

$$f(z,\ x) = \mathrm{D}(x) \tag{4.5.14}$$

When $z \geq z_w$,

$$f(z,\ x) = \mathrm{D}(x)\cos(\pi(z - z_w)/2H) \tag{4.5.15}$$

or

$$f(z,\ x) = \mathrm{D}(x)(1 - (z - z_w)/H) \tag{4.5.16}$$

Where, z_w is the depth of the topmost edge of the liquefied stratum.

(2) Changes in ground properties

(a) Fall in ground rigidity

The evaluation of ground rigidity to forecast the amount of settlement of spread footings on liquefied ground, and of ground deformation by equivalent linear response analysis may be obtained by the following methods:

1) Estimate equivalent rigidity compatible to the strain of each layer from the relationship between the rigidity fall rate and strain in Figure 4.5.11.

2) When the influence of the fall in effective stress is to be evaluated, in the case where the safety factor against liquefaction F_l is 1 or higher, water pressure increase rate γ_u is obtained from either Figure 4.5.12 or the following formula, and the equivalent rigidity is estimated by assuming the rigidity to be in proportion to the square root of the effective stress. The above has been taken into consideration in Figure 4.5.11.

$$\gamma_u = F_l^{-7} \tag{4.5.17}$$

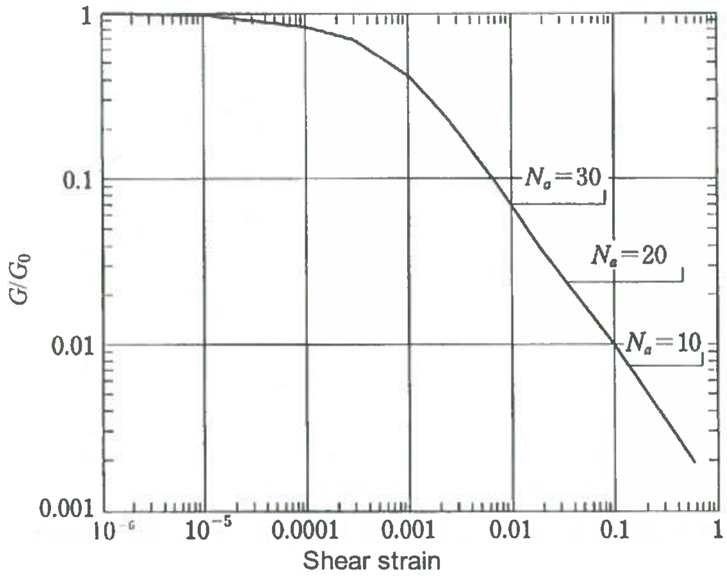


Figure 4.5.11 Relationship Between Corrected *N*-value and Rigidity Fall Rate

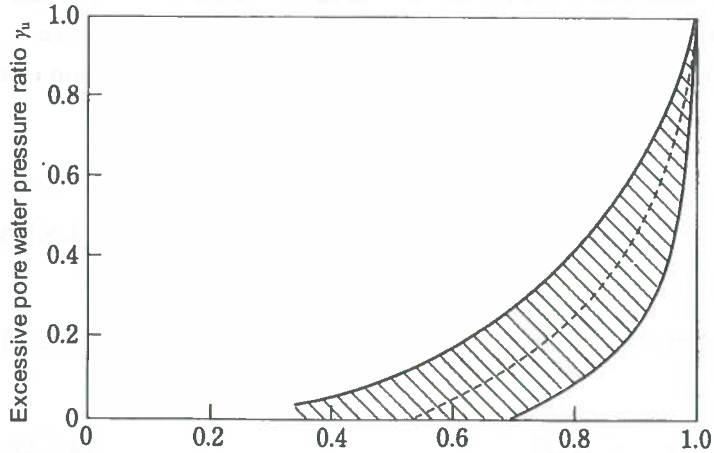


Figure 4.5.12 Relationship Between Safety Factor and Water Pressure Increase

(b) Decrease in lateral modulus of subgrade reaction

In the study concerning lateral resistance of piles in a liquefied ground, the lateral modulus of subgrade reaction k_h and the plastic lateral subgrade reaction p_y are reduced by the following equation (Figure 4.5.13).

$$k_{hl} = \beta k_{h0} \cdot y_r^{-1/2} \tag{4.5.18}$$

$$p_{yl} = \alpha p_{y0} \tag{4.5.19}$$

Where, $\beta$is a correction coefficient related to the N_a-value in Figure 4.5.14, and k_{h0} is the lateral modulus of subgrade reaction given in Eq. (6.6.4). y_r is the relative displacement of the pile and ground taking liquefaction into consideration and p_{y0} is the plastic lateral modulus of subgrade reaction given by Eq. (6.6.5).

The reduction value$\alpha$of plastic lateral modulus of subgrade reaction is estimated to be around 0.05 to 0.2, calculated backwards from the measured deformation mode of the pile foundation. Therefore, tentatively, $\alpha$is taken to be equivalent toβ ($\alpha = \beta$). This responds to the maximum subgrade reaction of the liquefied ground being 0.2 to 1 times the total overburden pressure.

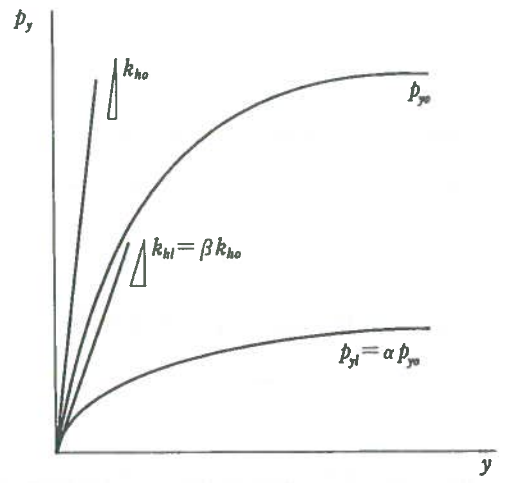


Figure 4.5.13 Pile Lateral Subgrade Reaction and Displacement Relationship Model

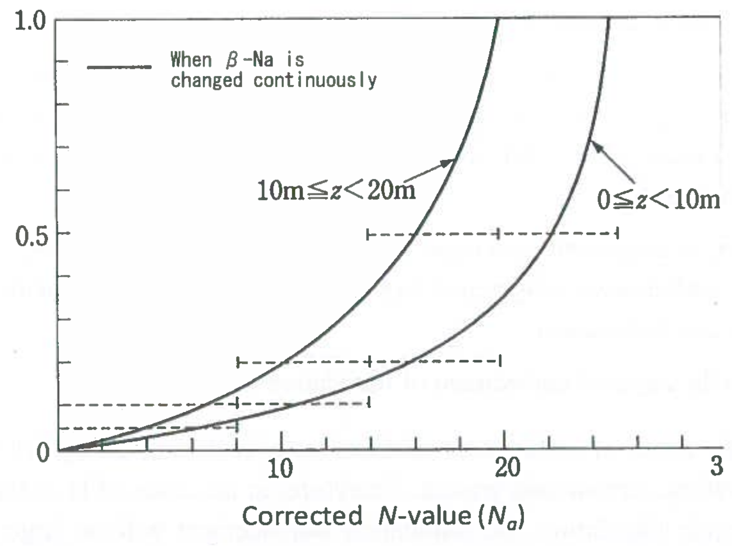


Figure 4.5.14 Reduction Ratio of Lateral Modulus of Subgrade Reaction[4.5.3)]

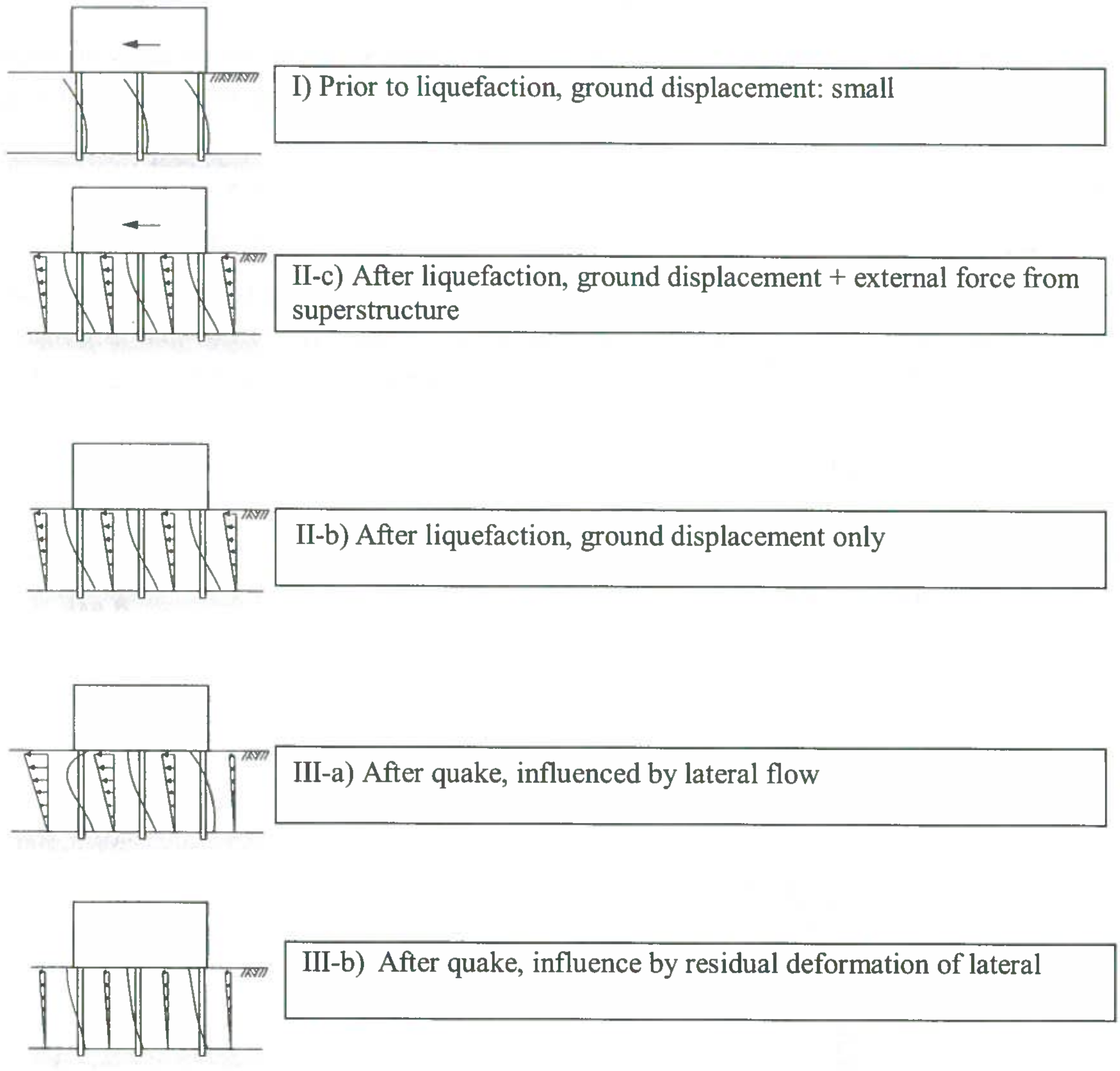


Figure 4.5.15 Schematic Diagram of Ground Deformation and Interaction Between Piles and Building in Liquefied Ground[4.5.1)]

(c) Consideration for decrease in the force of friction and buoyancy

In general, force of friction acting on building sides and bottom and the circumference surface of piles are ignored for parts where liquefaction has occurred. Furthermore, when rise in excessive pore water pressure is expected from Figure 4.5.12, force of friction shall be reduced depending on its degree, even if liquefaction has not occurred.

When the ground liquefies, underground structures with apparent specific gravity of 1.9 or less will not be able to resist buoyancy with its own weight and may float. There is a need to provide measures against these situations when they can be foreseen.

(d) Soil pressure acting on the depth of embedment of foundation

Soil pressure acting on the depth of embedment of foundation is dominated by relative displacement between the foundation and the surrounding ground. Therefore, in the cases of I) or II) in Figure 4.5.15, where there is a flexible pile foundation, the foundation displacement will be larger than the ground displacement influenced by the force of inertia. As indicated in Figure 4.5.16, the soil pressure shall be passive on the left side of the foundation and active on the right side. When there is a rigid pile foundation in the cases of II) and III), the ground will push the foundation and conversely, the soil pressure will be passive on the right side of the foundation and active on the left side of the foundation. In the case of III) with a flexible pile foundation, soil pressure acting on the depth of embedment will be influenced by the soil pressure acting on the pile, and both cases could be considered. Especially in the case where the ground pushes the foundation, lateral forces that have not been considered before will act on the piles. In the design of the foundation, these effects must be appropriately taken into consideration.

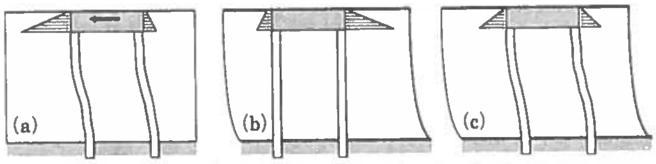


Figure 4.5.16 Soil Pressure Acting on the Depth of Embedment[4.5.4)]

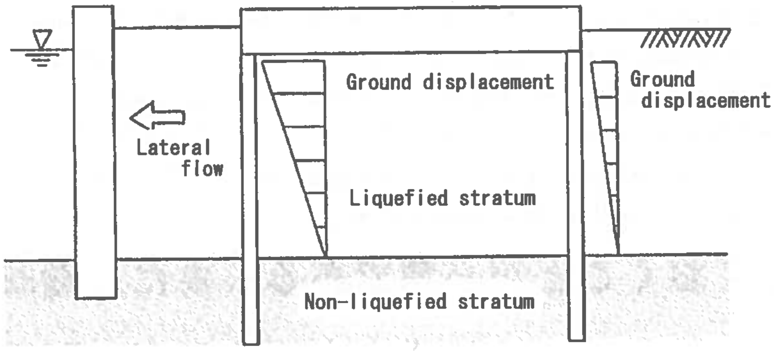


Figure 4.5.17 Example of Lateral Flow Analysis Model for an Unified Pile Foundation and Building System[4.5.1)]

3. Structural planning of foundation in liquefied ground

When risk of liquefaction is predicted, prevention measures such as: a) ground improvement + spread foundation, (b) ground improvement + pile foundation, c) no ground improvement + structural measures, or other measures shall be selected. In general, in addition to the concept of not letting the ground liquefy, there is the concept to provide structural countermeasures for not letting the ground liquefy at the damage limit, while allowing a certain amount of liquefaction to take place against the ultimate limit. Structural countermeasures can be separated into ground deformation trailing type (SC piles, steel pipe piles, etc.) and ground deformation resistance type (wall piles, retaining walls). In the case where there is a possibility of lateral flow of ground near embankment where consideration of its effects is believed to be appropriate, it is advisable to provide ground deformation resistance type measures. However, it should be noted that ground improvement for the purpose of preventing liquefaction or the adoption of a highly rigid foundation may possibly result in seismic load being increased in the superstructure and the foundation structures.

Since the seismic response characteristics of liquefied ground indicates a strong non-linearity, it is recommended to conduct various response analysis for the verification of foundation displacement and stress. The response analysis may be substituted by seismic deformation method. In this case, changes in mutual action of the ground-building system accompanying liquefaction indicated below must be taken into account (refer to Figure 4.5.15). Furthermore, results obtained from 2. (1) may be used for the depth distribution of lateral displacement of the ground.

I) When external force from the superstructure acts on the pile-head when ground displacement prior to liquefaction is small.

II) When ground displacement and external force from the superstructure acts on the piles after liquefaction.

III) When piles receive soil pressure from residual deformation of ground in the latter half or after the main quake.

In pile foundation where pile-head rotation restraining conditions are met, in the case of I), the shear force and flexural moment will be maximized at the pile-head, while in the cases of II) and III), in addition to the areas around the pile-head, shear force and flexural moment will increase at the top and bottom edges of the liquefaction stratum. In the case of III), the residual displacement of lateral ground will be smaller than the dynamic displacement under a normal earthquake, however, when lateral flow should occur, its shear deformation will exceed the vibration component. Therefore, in lateral ground where there is no possibility of lateral flow, the safety of piles for cases I) and II) should be studied, and in the case where lateral flow occurs, case III) should be studied additionally. However, the amount of lateral flow will differ with the distance from the embankment and it will be necessary to use models, as in Figure 4.5.17. In this case $f(z)$ shall be those obtained in 2.(2).

In the case of a spread foundation, the first to be considered is in conjunction with ground improvement. When ground improvement proves to be difficult for being a wooden structure, etc., RC strip footing or mat foundation should be adopted. Providing sufficient rigidity and strength to the foundation will lead to reducing the damage to the superstructure after liquefaction. However, if the building is eccentric or the ground not uniform, there is a possibility that the superstructure will incline. Even in these circumstances, having foundation rigidity should be advantageous in conducting repairs, such as jack-up.

When estimating the amount of foundation settlement, estimation can be made by using the settlement equation for spread foundation, taking into consideration the fall in ground rigidity in 2.(2). In such a foundation design, it is important to provide consideration to recovery measures in case of differential settlement, by minimizing the eccentric load of the building, decreasing the building aspect ratio and by providing a highly rigid foundation to withstand differential settlement. In the case where there is a very loose ground immediately beneath the foundation or when ground rigidity is uneven, considerations, such as providing ground improvement will be necessary. Liquefaction judgment, deformation forecast after compacting of ground may be conducted as per 1.(2) and 2.(2).

2. Section 6.6 Lateral resistance and lateral displacement (page 262 – 280 original copy)

1. Discussion items and limit values for design

(1) Discussion items for piles receiving lateral force are as follows:

a. Lateral resistance determined by the pile - ground system.

b. Lateral displacement determined by the pile - ground system.

(2) Limit values for design of pile foundation receiving lateral force shall be as per Table 6.6, in general.

Table 6.6 Limit Values for Design of Pile Foundation Receiving Lateral Force

Performance level (Limit state)	Limiting values for design		
	Pile-body	Ground	Lateral displacement (Amount)
Ultimate limit state	Reliability strength or plastic deformation limit	Maximum lateral resistance	Value to be set for each required performance level of the superstructure
Damage limit state	Elastic strength limit	Maximum lateral resistance x 1/2	
Use limit state	Cracking strength limit and strength with sufficient allowance against creep deformation		

2. Evaluation method

Lateral resistance and lateral displacement of a single pile shall be evaluated by either of the following methods. For group piles, evaluation shall be conducted by factoring their influence.

(1) Lateral load test

(2) Lateral resistance calculation formula

3. Items to note

(1) For piles installed in ground where the influence to the pile caused by the deformation of the ground by an earthquake on the pile-body itself cannot be ignored, influence on lateral resistance and lateral displacement shall be considered in the evaluation.

(2) For piles installed in ground possible to be liquefied in an earthquake, in the evaluation of lateral resistance and lateral displacement, their influence must be considered.

(3) When sloped ground exists in the vicinity of a pile foundation, its influence must be considered.

(4) When the unification of the pile foundation and the superstructure becomes an issue, their influence must be considered.

For lateral loads acting on a pile, there are relatively short period cyclic loads caused by earthquakes, slightly long period cyclic load from storms, single dimensional static load caused by actions of distorted soil pressure and others, and the acting form of loads are diverse. Of the above, storms and distorted soil pressure can be generally handled as problems of lateral force acting on a pile-head. However, in an earthquake, the ground will also vibrate and depending on the ground stratum structure, it may be necessary to consider the effects of ground displacement on the pile, in addition to the lateral force on the pile-head by the force of inertia of the superstructure. Furthermore, the effects on the piles from the lateral flow of the ground caused by liquefaction, etc. could also become an issue.

In addition to the above, note should be taken for cases where piles of different diameters are used in a single building or when short and long piles are mixed and used where the supporting stratum is sloped or when the piles have been connected with a highly-rigid beams. In these situations, there will be variances in lateral rigidity of each pile against lateral load, resulting in variances in lateral load borne by each pile. Especially, the variances in load borne by the piles will influence the rigidity of the superstructure including its foundation beam, therefore, the adoption of a unified analysis for both the superstructure and the pile foundation should be made, as required. Furthermore, in the case where the distance between the piles are small, piles will mutually affect each other through the ground, which result in phenomena particular to group piles, where, based on an equal pile-head lateral displacement, the lateral load per single pile will be smaller for group piles than that for a single pile or the lateral load borne by each pile is different, etc. These effects must be considered in design, in the case where the effects of the group piles cannot be ignored.

For pile foundation of structures with embedment, such as a structure with a basement floor, it is important to appropriately evaluate and design the lateral load input to the piles under an earthquake. Unfortunately, the lateral load may decrease or increase, by the embedment caused by the reciprocal conditions of superstructure, pile and the ground, and cannot be determined outright. In addition, for pile foundation with embedment, it is difficult to measure the required data such as the load bearing of the embedment and pile foundation by testing, and with the analysis modeling of the total system of embedment, pile foundation and ground also being difficult, it results in many unsolved items to be cleared, even from the research perspective. Please refer to Section 3.5 for such evaluation of lateral load under an earthquake for pile foundation of a structure with embedment.

1. Discussion items and limit values for design

(1) Discussion items

The required performances below of a pile foundation receiving lateral force can be confirmed by discussing the two items indicated in the text 1. (1).

(a) Ultimate limit

i) That the pile-body shall not be under brittle failure.

ii) That the pile-body shall not reach the limit value of its plastic deformation capacity.

iii) That the pile shall not reach its limit value of lateral resistance by the destruction of surrounding ground.

iv) That the connections and pile-head joints shall not be damaged.

v) That the superstructure shall not be destructed by the lateral displacement of the pile.

In many cases, piles that receive lateral force will reach its ultimate limit by the fracture of the pile-body; however, for short piles, the surrounding ground will destruct for the entire length of the pile prior to the fracture of the pile-body, and reach its ultimate limit. iii) Above assumes this kind of a limit situation.

(b) Damage limit

i) That the pile-body, connections and the pile-head joints do not reach damage strength limit.

ii) That no excessive residual lateral displacement develops in the pile foundation.

iii) That there will be no damage that requires structural repairs or strengthening to the superstructure by the lateral displacement of piles.

(c) Use limit

i) That the pile-body, connections and pile-head joints do not reach cracking strength limit or the cracking width limit.

ii) That there shall be no disturbance in its usability, functionality and durability of the superstructure by the lateral displacement of the piles.

Of the above, the required performance for the effects on the superstructure by the lateral displacement of piles shall be set under load conditions to be envisaged covering the entire pile foundation. For others, the required performances shall be, for each pile that comprises the pile foundation, not to reach these conditions. However, concerning the ultimate limit, this will not apply in case it is guaranteed that there will not be any devastating influence to the superstructure from differential settlement, etc. for the entire structure. But in the case where this kind of design is to be the objective, then it will be necessary to conduct studies on the safety of structures after load has been applied, such as after an earthquake, etc. In addition, the required specifications should also be established for underground installation such as lifelines connected to the structures.

Fundamentally, it is desirable to adopt a combined dynamic interaction analysis as a comprehensive building-pile-ground system for the verification of seismic load. However, adopting such dynamic interaction analysis in the design of general buildings involves complicated analysis and, as it stands now, such analysis is not adopted in design of buildings other than important buildings and special buildings. Compared to many analysis data available from static load tests, there are only few dynamic tests conducted on actual sized piles up to regions where the non-linearity of the ground or the pile-body would cause major influence, and it is difficult to obtain analysis data that can respond to design reaching the ultimate condition of the pile foundation. In consideration of the current situation, for the basic study items concerning lateral resistance and lateral displacement of piles, the flow of pile seismic resistance design focused mainly on static analysis is as indicated in Figure 6.6.1.

The study method is based on static lateral load equivalent to the seismic force acting on the pile-head which has been adopted from the past, and in addition to this, in the case of a ground where liquefaction is presumed to occur or for soft ground where its natural period is relatively long or for ground with intermediate layers of extremely different rigidity, the influence from ground displacement to the piles requires to be studied.

The study of pile-body at ultimate limit should be conducted, using strength as an index for piles where ductility cannot be expected, and in case ductility can be expected from the piles and design has been made taking this into consideration, then, the plastic deformation amount shall be used as the index. This being the case, the analysis method to be adopted for piles against lateral load must also respond to such methods. Generally, by the lateral load acting under the assumed axial force, the shear force generated on each part of the piles, the stress such as flexural moment, etc. or the deflection of the angle of ration or the curvature, etc. of the pile-body shall be obtained to confirm that they do not reach the design limit strength or the plastic deflection limit.

In addition to the above, studies should be conducted for the effects of sloped ground existing in the vicinity of the pile foundation and the unification with the superstructure, etc., as necessary.

(2) Limit values for design

When lateral load is acted to the head of a sufficiently embedded single pile, the relationship between the pile-head lateral load and the pile-head lateral displacement is as indicated in Figure 6.6.2. Starting out with an almost linear state in the low-load range, it develops into a non-linear state, and after reaching a certain load, it will start to display a sudden increase in lateral displacement without the increase in load or a fall in the lateral load and then result in reaching its ultimate state.

This non-linear behavior of the pile is the manifestation of synergistic phenomena of the non-linear behavior of the ground in the vicinity of the pile together with the progressive failure of the ground from the ground surface to the depth, accompanying the increase in lateral displacement and the plasticizing of the pile-body from an elastic state. The ultimate state will occur when the destruction mechanism of the pile-ground system is established, and the destruction mechanism will either be (i) where the ground is destructed over the entire length of the pile or (ii) in the case where the pile-head is fixed, the two cross sections of the pile-head and the embedded mid-section will constitute a hinged state (in the case of the pile-head free, the embedded part will constitute a hinged state), and the ground between the two cross sections are destructed.

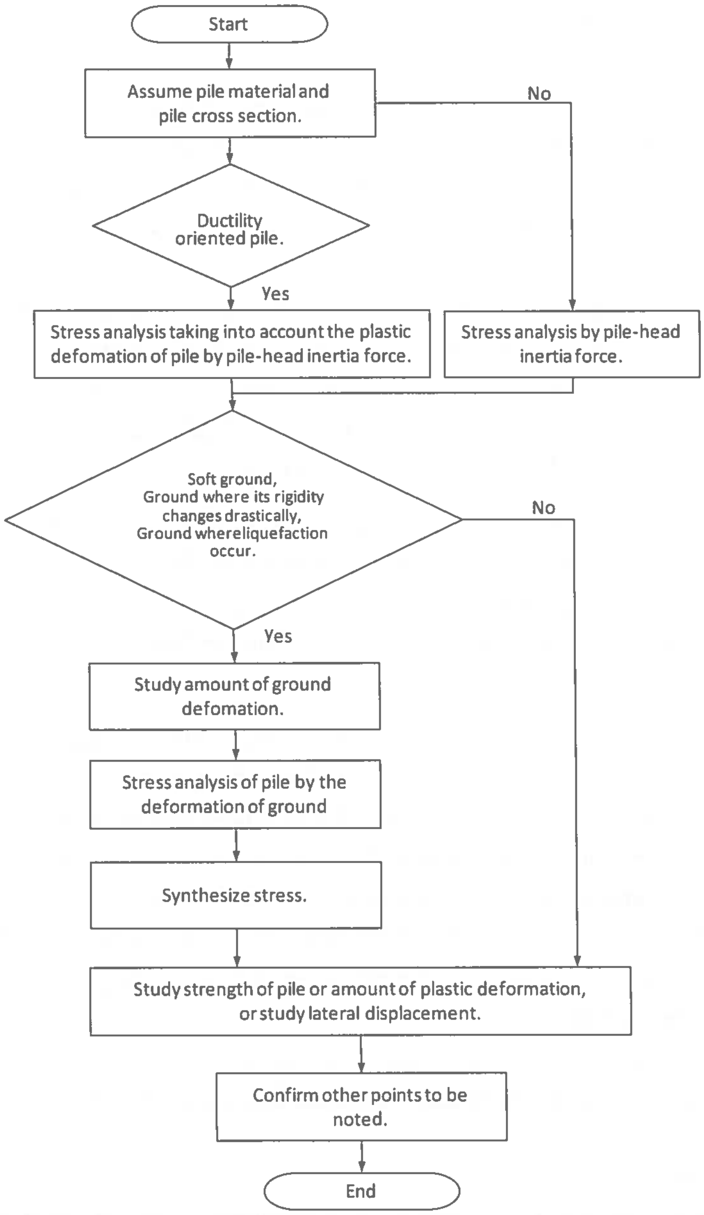


Figure 6.6.1 Flow Chart for Pile Seismic Resistance Design

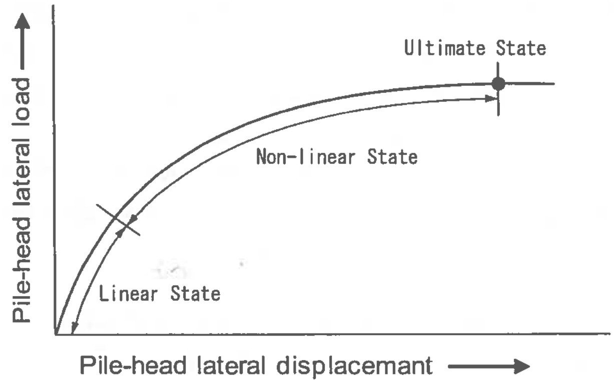


Figure 6.6.2 Relationship Between Lateral Load and Lateral Displacement of Pile-Head

In this guideline, the pile that forms the destruction mechanism of (i) above shall be defined as a "Short Pile," while the case of (ii) defined as the "Long Pile." In other words, it is no longer classified by the product of the relative rigidity β-value between the pile-body and the ground adopted in the former guideline(1988)[6.6.1)] (= $[k_h \cdot B/(4EI)]^{1/4}$; k_h= lateral modulus of subgrade reaction, B = pile diameter, EI = pile-body flexural rigidity) and η-value ($=[n_h/EI])^{1/5}$; n_h= lateral modulus of subgrade reaction) by the pile length, but classified by the relationship between the pile-body strength and the ground strength. This is because, in this guideline, in the design against lateral load, the basics of the design is to incorporate the non-linear characteristics between the pile-body and ground strength, and since the rigidity of the pile-body and the ground are not uniform in the depth direction, the relative rigidity of the two cannot be uniformly set.

For the general lateral behavior of a single pile mentioned above, the relationship with the state of a pile-body for a Long Pile that has its pile-head secured can be explained, in most cases, as indicated in Figure 6.6.3. In the concrete piles, cracks will form on the pile-head, leading to the yielding of outermost edge reinforcing-bar yielding, then onto plastic hinging. However, for a steel pile, the outermost edge will first yield at the pile-head and then enter into a plastic hinge state. For any further displacements after this, the pile-head lateral load will fall drastically for piles that do not have ductility, but for piles with ductility, the pile-head lateral load will increase and a plastic hinge will occur in the embedded part, forcing the pile to reach its ultimate state. However, should the pile-head reach its plastic deformation limit before the plastic hinge is generated in the embedded part, then this point shall be considered as the ultimate state of this pile.

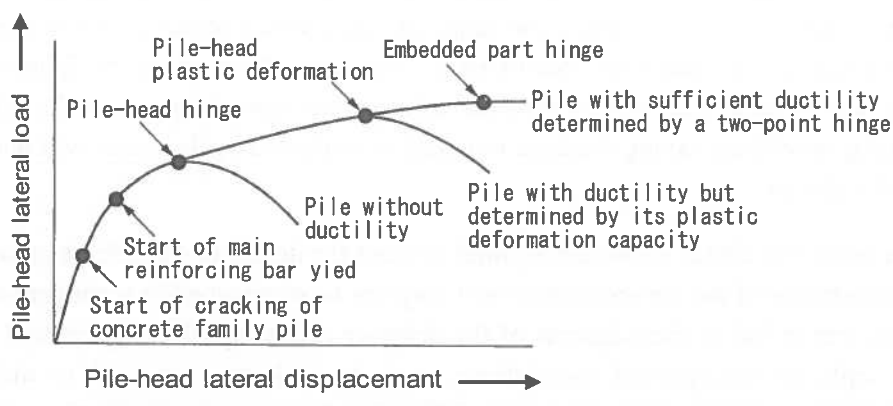


Figure 6.6.3 Each State in the Relationship Between Lateral Load and Lateral Displacement at the Pile-Head (In the Case of a Concrete Long Pile)

For piles satisfying the conditions for a free pile-head, Short Piles with insufficient embedded length, or piles where the effects on the pile-body by ground deformation in an earthquake cannot be ignored, the state of the pile-body will be different from the above explanation, and in these cases, it must be studied under their individual conditions. Especially for piles where the effects of ground deformation are large, care must be taken to note that there is a possibility where the pile body that is close to the stratum boundary where the ground rigidity changes drastically or close to the boundary between liquefied and non-liquefied strata will reach its ultimate state prior to the other parts.

The limit values for design corresponding to each performance level are indicated in Table 6.6, separated into pile-body, ground and lateral displacement. For pile-body, shear failure and flexural compression failure under high axial force in concrete piles or local buckling of steel piles may lead to a brittle failure mode generally accompanied by a rapid fall in bearing capacity and, even for pile-body with ductility, it may result in the decrease in bearing capacity if it exceeds the limit of deformation performance. Therefore, from the viewpoint of securing vertical supporting capacity, the ultimate limit value for design shall be either the reliable strength or the plastic deformation limit that does not lead to the abovementioned failure. This will be followed by the damage limit for design of the pile-body, which shall be the elastic limit strength, where the pile-body can be re-used without repairs of reinforcement. For Long Piles, if the pile-body does not exceed the elasticity limit strength, it is considered to satisfy the required performance of "no excessive residual lateral displacement develops. "The use limit for design shall be the "strength with sufficient allowance against creep deformation" so that even if creep deformation of the pile-body should progress, it will not affect the usability or the function of the superstructure. In addition, only applicable to concrete piles, from the viewpoint of securing durability, it shall be the crack limit strength.

For Short Piles, even after reaching the maximum pile-head lateral load indicated in Figure 6.6.2 by the failure of ground for the entire length of the pile, lateral resistance may not fall, depending on the type of ground, the possibility of expressing a large deformation performance is high. However, considering that a quantitative evaluation is currently difficult, the maximum lateral resistance shall be the ultimate limit for design instead of adopting the deformation capacity as the index. For the damage limit value for design, from the requirement performance of generating no excessive residual displacement on the pile, while the use of the conventional method where about 2/3 of the maximum lateral resistance is taken to be the short-term equivalent lateral resistance of the ground could be considered, there are no materials available to back this up, so 1/2 of the maximum lateral resistance shall be adopted, to be on the safe side. This shall not apply in the case where the required performance under each limit condition has been confirmed through lateral load tests. Additionally, with Short Piles, from its definition, they are piles where a destruction mechanism is formed by the failure of the surrounding ground for the entire length of the pile, but it goes without saying the there is a need to confirm the pile bodies will not reach each of the limit values for design.

On the other hand, the lateral displacement limit amount for design of the pile must be set to respond to the various conditions of the superstructure, and may not be determined in a uniform way. Therefore, in this guideline, this is left to the judgment of the designer and no specific figures will be indicated. The same shall apply to underground installations, such as lifelines connected to the building. In the establishment of limit values, it should be noted that, for structures in general, the relative displacement of the ground at the bottom of the foundation and, for underground installations, the relative displacement of the structure and the surrounding ground shall be the subject of the study.

2. Evaluation methods

When a general pile foundation where an almost rigid floor assumption is established receives a lateral load on its pile-head, the lateral resistance limit for design corresponding to each performance level shall be evaluated under the following procedures:

(a) Obtain the load - lateral displacement curve at the head of each pile that comprises the pile foundation.

(b) Of the pile-head lateral displacement that reaches the limit value of design concerning the pile-body or the ground of each pile, obtain the minimum value y_{0min}.When lateral load acts on the all the group piles, the pile that gives this y_{0min} will be the first to reach the limit state. In the case where the lateral displacement limit for design governed by the superstructure is smaller than this value, then such value shall be y_{0min}.

(c) Obtain the lateral load when each pile that comprises the pile foundation reaches the pile-head lateral displacement y_{0min} and the sum of the values shall be the lateral resistance limit for the design of the whole pile foundation.

When the rigid floor assumption does not stand, then the foundation beam + pile foundation model or a unified analysis model shall be used to appropriately evaluate the lateral load acting on each pile, and the lateral resistance limit for design of the total pile foundation is obtained.

The pile-head lateral displacement of a single pile receiving lateral load on its pile head, the stress on pile-body such as shear force, flexural moment, etc., the deformation of the pile-body such as deflection angle and curvature, the lateral resistance of the ground, etc. are obtained by using either method (1) or (2) indicated below: The lateral resistance limit of a single pile shall be given as the lateral load when the various quantities above have reached their limit values for design. The limit strength and plastic deformation limit are greatly influenced not only by lateral load but also by vertical load, and their evaluation methods are mentioned comprehensively in Section 6.7 "Cross section design of pile-body."

(1) Evaluation by lateral load test

Lateral load test shall be conducted according to the "Method for lateral load test of piles and commentaries" of the Japanese Geotechnical Society. If lateral load test can be conducted under the same conditions as those assumed in the design, lateral resistance limit can be directly evaluated from the test results, as indicated below. For concrete piles, judgment shall be based on lateral resistance upon cracking, lateral resistance upon the outermost edge reinforcing bar yield point and the ultimate lateral resistance, and, for steel piles, the lateral resistance upon the outermost edge steel yield point and the ultimate lateral resistance, etc. However, in order to reproduce a state where vertical load is acting on a fixed joined pile-head as assumed in the design will require a special loading device and the lateral load test is generally conducted under a free pile-head without any vertical load. Pile axial direction displacement and stress distribution when identical load is applied for a pile with a free pile-head and a fixed jointed pile-head will show differences as indicated in Figure 6.6.4. In addition, as mentioned before, the lateral resistance limit shall vary greatly depending on the strength of axial force. Therefore, in the case of a test that does not match the design conditions, the test shall be positioned as a test to obtain various constants of the soil to be applied in the lateral resistance calculation equation. Using the equation used for design, various constants for the ground are obtained so that the behavior of the pile obtained from the test can be approximated, and, with these constants, obtain the lateral resistance limit or lateral displacement of the pile by re-calculation under the conditions assumed in the design.

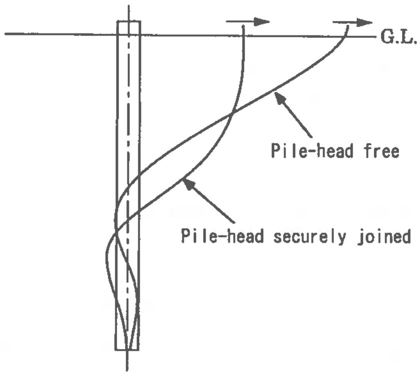


Figure 6.6.4 Difference in Pile Displacement Distribution By the Difference in Pile-Head Conditions

The soil constant can be obtained from soil investigations, such as *N*-value from a standard penetration test or lateral load test inside boreholes as mentioned later, etc., but their accuracy is generally low. In contrast, soil constant obtained from a lateral load test on piles that are actually installed in the current location reflects almost all the influences of the complex factors, including the change in properties of the ground surrounding the piles accompanying the installation of the piles, allowing for a highly-accurate evaluation.

(2) Evaluation by lateral resistance calculation equation

There is a method available as a representative method to obtain lateral resistance and lateral displacement of pile against lateral force acting on the pile-head, and this is indicated as follows:

a. Calculation method using an analysis model where the pile is assumed to be a rod with flexural stiffness and the soil as a spring:

(a) A method to assume both the pile and the soil to be elastic, for piles set in a uniform Soil condition.

(b) A method to assume both the pile and the soil to be elastic, for piles in a multi- layered stratum.

(c) A method to consider pile and soil non-linear, for piles in a multi- layered stratum.

b. Broms method using the ultimate balance method
All methods in a. use the solution for the following basic differential equation concerning the assumed analytical model, to obtain lateral displacement, flexural moment, shear force, etc. of a pile at any given depth, including the pile-head.

$$\frac{d^2}{dz^2}\left[K\frac{d^2y}{dz^2}\right] + pB = 0 \qquad (6.6.1)$$

Where,

- K: Flexural stiffness taking into account the pile to be non-linear (kN·m^2)
  (Equals EI in elastic range)
- y: Lateral displacement of pile (m)
- z: Depth (m)
- p: Lateral soil reaction at depth z (kN/m^2) and expressed by the following equation: $p = k_h \cdot y$
- k_h: Lateral soil modulus at depth z (kN/m^3)
- B: Pile diameter (m)

The method in (a) can be used to evaluate the lateral resistance of a Short Pile by solving the differential equation by giving the boundary conditions at the tip of the pile. In the case where the pile length is long enough to satisfy Eq. (6.6.2)[6.6.1)], a simplified equation to obtain the major values for design, such as the displacement of pile-head, pile-head flexural moment, maximum embedded flexural moment and its location, etc., has been obtained allowing it to be calculated by hand (refer to table 6.6.1).

$$\beta L > 2.25 \qquad (6.6.2)$$

Where,

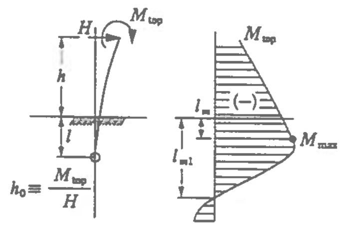
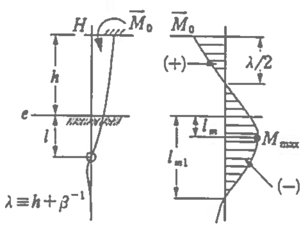
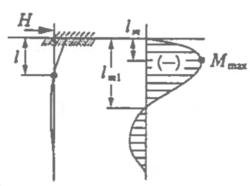
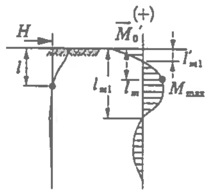
$\beta = [k_h \cdot B \,/\, (4K)]^{1/4}$(1/m)

L: Embedded pile depth (m)

In this method, the soil is assumed to be elastic, but by decreasing the lateral soil modulus along with the increase in pile-head displacement, non-linearity of the soil can also be considered. Therefore, if the soil is uniform, this method can be adopted up to the evaluation of lateral resistance damage limit, where the pile is retained almost within the elastic range. The range of soil, where influence of the lateral resistance of the pile receiving lateral force on its pile-head is dominant, is approximately ($1/\beta$) in depth from the surface, and this method can be adopted if the soil within this range is almost uniform.

The method indicated in (b) is the method in (a) extended to cover multi-layered stratum. The pile and the soil will be divided into multiple layers and for each pile in each layer, Eq. (6.6.1) is applied and the general solution for displacement, deflection angle, flexural moment, shear force, etc. are obtained. The behavior of the pile can be obtained by solving the conditions of continuity, and the boundary conditions of the pile-head and the pile tip at the boundary of each layer, and solving the multi-dimensional simultaneous linear equation. Although there is the limitation of assuming the pile-body to be elastic, this method can also be adopted when the flexural stiffness of the pile and the soil structure changes in the pile axial direction. Eq. (6.6.1) can also be applied to obtain lateral resistance and lateral displacement of a pile for soil with intermediate stratum with drastic change in the rigidity of the soil, when the effects of soil displacement against lateral resistance of the pile under an earthquake is to be considered. Likewise as in the method for (a), this method can be applied up to the evaluation of lateral resistance damage limit.

Table 6.6.1 Solution for Elastic Supported Beam in a Uniform Soil (Lateral Resistance of Pile Receiving Lateral Force and Flexural Moment to the Head)

	Case where pile is protruding on the ground		**Case where pile is not protruding on the ground**	
Model	Pile-head free 	Pile-head fix 	Pile-head free 	Pile-head fix 
Pile-head Displacement.	$y_t = \frac{(1+\beta h)^3+1/2}{3EI\beta^3}H + \frac{(1+\beta h)^2}{2EI\beta^2}M_{top}$	$\bar{y}_t = \frac{(1+\beta h)^3+2}{12EI\beta^3}H$	$y_t = \frac{H}{2EI\beta^3}$	$\bar{y}_t = \frac{H}{4EI\beta^3}$
Surface Displacement.	$y_0 = \frac{1+\beta(h+h_0)}{2EI\beta^3}H$	$\bar{y}_t = \frac{1+\beta h}{4EI\beta^3}H$	$y_0 = y_t$	$\overline{y_0} = \overline{y_t}$
Angle of Deflection of Pile-head.	$\theta_t = \frac{(1+\beta h)^2}{2EI\beta^2}H + \frac{1+\beta h}{EI\beta}M_{top}$	$\bar{\theta}_t = 0$	$\theta_t = \frac{H}{2EI\beta^2}$	$\bar{\theta}_t = 0$
Bending Moment of Pile-head.	$M_0 = -M_{top}$	$\bar{M}_0 = \frac{1+\beta h}{2\beta}H = \frac{\lambda}{2}H$	$M_0 = 0$	$\overline{M_0} = \frac{H}{2\beta}$
Bending Moment of Underground part maximum.	$M_{max} = -\frac{H}{2\beta}\sqrt{(1+2\beta(h+h_0))^2+1}$ $\exp\left[-\tan^{-1}\frac{1}{1+2\beta(h+h_0)}\right]$	$M_{max} = -\frac{H}{2\beta}\sqrt{1+(\beta h)^2}$ $\exp[-\tan^{-1}(1/\beta h)]$	$M_{max} = -\frac{\sqrt{2}}{2\beta}e^{-\pi/4}H$ $= -0.3224H/\beta$	$M_{max} = -\frac{H}{2\beta}e^{-\pi/2}$ $= -0.2079\overline{M_0}$
Bending Moment of Underground part maximum. Originating point	$l_m = \frac{1}{\beta}\tan^{-1}\frac{1}{1+2\beta(h+h_0)}$	$\overline{l_m} = \frac{1}{\beta}\tan^{-1}\frac{1}{\beta h}$	$l_m = \frac{\pi}{4\beta}$	$l_m = \frac{\pi}{2\beta}$
1st unmoving points of Underground	$l = \frac{1}{\beta}\tan^{-1}\frac{1+\beta(h+h_0)}{\beta(h+h_0)}$	$l = \frac{1}{\beta}\tan^{-1}\frac{\beta h+1}{\beta h-1}$	$l = \frac{\pi}{2\beta}$	$l = \frac{3\pi}{4\beta}$
Bending Moment of Underground 1st Zero point	$l_{m1} = \frac{1}{\beta}\tan^{-1}\left[\frac{\beta(h+h_0)}{1+\beta(h+h_0)}\right]$	$l_{m1} = \frac{1}{\beta}\tan^{-1}\frac{1-\beta h}{1+\beta h}$	$l_{m1} = \frac{\pi}{\beta}$	$l'_{m1} = \frac{\pi}{4\beta}$ $l_{m1} = \frac{5\pi}{4\beta}$

[Notes] $\beta = [\kappa \cdot B/4EI]^{1/4}$

The method indicated in (c) is where the effects of the non-linearity of the pile and the soil are introduced to the method indicated in (b), where the pile shifts from a linear state to a non-linear state with the increase in lateral load, allowing the lateral resistance and lateral displacement of each limit up to the ultimate state to be evaluated. An example of the calculation model is indicated in Figure 6.6.5. The pile and the soil shall be divided into elements along the pile axis direction, giving consideration of the ground stratum judged from soil survey results. Within each of the elements, the relationship between the flexural moment M of the pile and curvature ϕ, and the relationship curve between lateral soil reaction p and lateral displacement shall be established in consideration of the non-linear properties of each. The $M - \phi$ relationship and p - y relationship curves are expressed by curves of an appropriate function or a number of polygonal lines including bi-linear interpolations, and the calculation considering these non-linear properties shall be made either by the direct repetition method or by the incremental method.

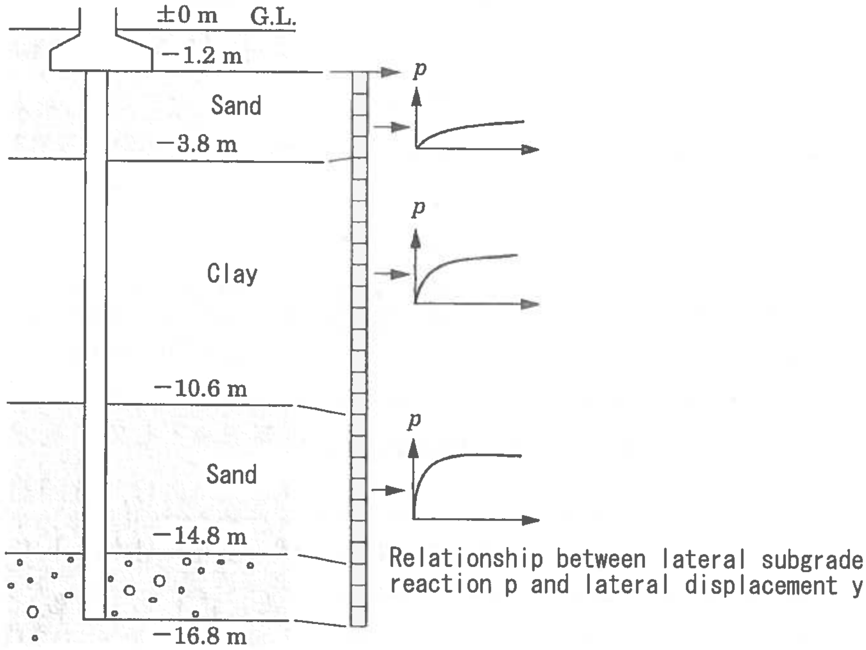
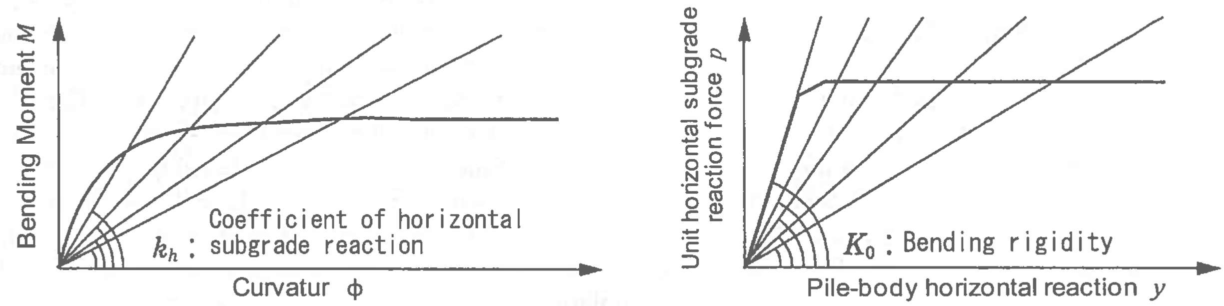


Figure 6.6.5 Actual Pile Design and Analysis Model

(* An ideal division would be for each element length to be smaller than the pile diameter)

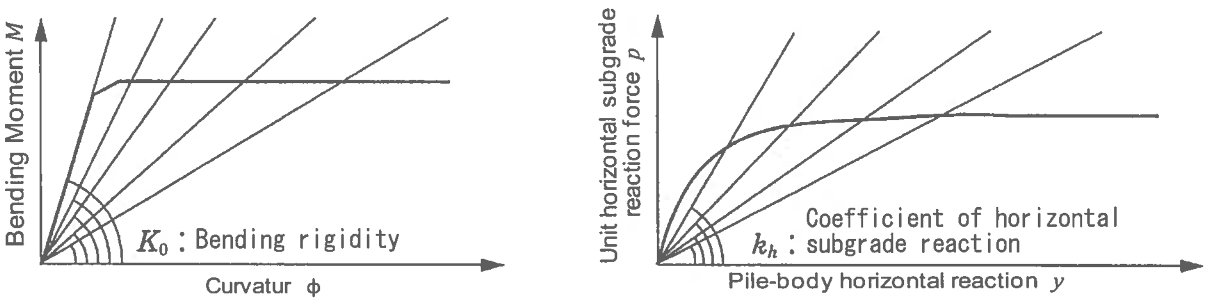
In the direct repetition method, as indicated in Figure 6.6.6, the flexural rigidity EI (kN·m^2) and the modulus of lateral soil reaction k_h (kN/m^3) are provided as secant moduli of both the M- ϕ and $p - y$ relationship curves and the flexural moment M and lateral displacement y for the acting load are obtained by solving Eq. (6.6.1).

Then, a new K and k_h corresponding to both M and y shall be obtained from Figure 6.6.6 and Eq. (6.6.1) is again solved using these values. This shall be repeated until the difference with the previous calculation result fall within the allowance range, and once convergence has been achieved, it will then move on to the next load stage. In contrast, with the increment method, the acting load is divided into small quantity and assumes the pile and soil to behave linearly within the range of each small load quantity, and as indicated in Figure 6.6.7, the solutions for M and y or others are calculated by assuming that flexural rigidity of the pile and the lateral modulus of small reaction corresponding to polygonal line gradient should exist, and sequentially increasing it on each of the polygonal lines. However, in terms of calculation accuracy, although both methods can assure given accuracy, the direct repetition method is favorable in practice, for its relationship with the calculation for group piles and piles embedded in liquefied ground.



(1) Soil model surrounding pile (2) Model for pile

Figure 6.6.6 Direct Repetition Method



(1) Pile-body model of subject element (2) Pile-body surrounding ground model for subject element

Figure 6.6.7 Incremental Method

Under the assumed load conditions, when it is evident that plasticization occurs only at the head of the pile and the other parts of the pile remain in the elastic range, a model with a non-linear rotary spring attached to the pile-head, as indicated in Figure 6.6.8, may also be established, as an intermediate calculation model of (b) and (c). With this model, in addition to the plasticization of the pile-head, calculations taking various conditions into consideration, such as the phenomenon where the rotary rigidity decreases by the yielding of the connection between the pile-head and the pile cap with the increase in load or when flexural rigidity of the foundation beam connecting the pile caps are small can be conducted by the establishment of a rotary spring.

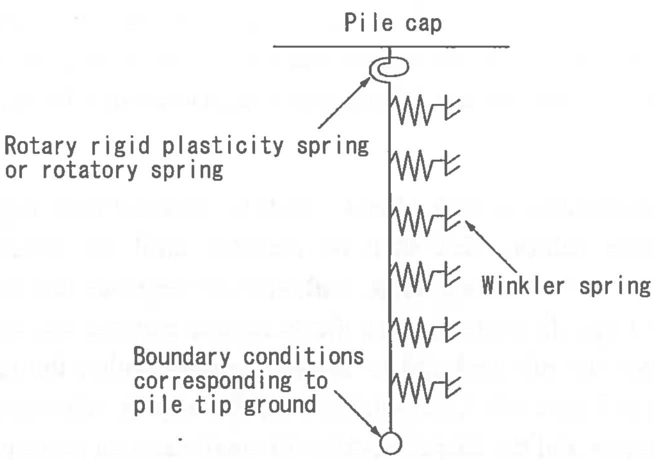


Figure 6.6.8 Boundary Condition Model for Fixed or Free and Others to be Considered

The method in b assumes a failure mode of a pile-ground system to obtain the ultimate lateral resistance from the balance of forces under the ultimate state and a simplified equation has been obtained for piles where lateral load is acting on its pile-head (refer to Tables 6.6.2 and 6.6.3). However, there are following restrictions in the application of this equation: For piles with long fixed pile-head, Broms assumes plastic hinges on two locations, the pile-head and the embedded part, meaning that the equation can only be applied to piles with deformation capacity that can guarantee that its pile-head will not reach the limit of plastic deformation after a hinge has been formed at the pile-head, before the hinge is formed on the embedded part. Since the failure mode of a pile-ground system taking into consideration the effects of ground displacement in an earthquake is different from the failure mode assumed by Broms, such effects cannot be taken into consideration. Nor can lateral displacement be obtained.

In addition to the above, there are calculation methods using the finite-element method and boundary-element methods, but they are seldom used in design and used more often in the research for behavioral analysis of piles.

In the publication "Dynamic bearing capacity and deformation performance in building seismic design[6.6.3]" by the organization, the deformation performance of piles are evaluated into three categories, and the method for evaluation of the ultimate lateral resistance limit of the overall pile foundation is indicated as follows. For details, kindly refer to the publication.

- Pile where ample deformation performance cannot be expected.

  The lateral resistance of each pile, when any of the piles that forms the pile foundation is in failure, shall be the sum of the values obtained by the method a(a) or (b), taking into consideration the consistency of pile-head lateral displacement of each pile, such as rigid floor assumptions. This shall apply to site-cast concrete piles with large axial force, steel pipe piles with local buckling, and ready-made concrete piles, etc. In this method, for the evaluation of lateral resistance in the non-linear region of the pile-body, the method of a(a) or (b) shall be extended and applied.

- Pile where ample deformation performance can be expected.

  When each pile has reached flexural ultimate lateral resistance limit, the lateral resistance is assumed to be sustained and the ultimate lateral resistance limit for each pile is obtained by the method indicated in b, and shall be the sum of these values. This can be applied to site-cast steel pipe concrete piles, ready-made concrete piles wound with steel pipe, steel pipe piles not subject to local buckling, and site-cast concrete piles with small axial force.

- Piles in the intermediate region of (a) and (b).

  The lateral resistance of each pile when any of the piles comprising the pile foundation reaches its ultimate plastic deformation capacity shall be the sum of the values obtained by elasticity analysis, such as a(c), taking into consideration the consistency of the lateral displacement of the pile-head. This shall apply to normal site-cast concrete piles where $\sigma/F_c \leq 0.35$ (σ: compressive unit stress, F_c: Design standard strength of concrete) with diameter thickness ratio of around 50.

In any deformation performance of the pile foundation, from the viewpoint of preventing fragile failure, when any of the piles that comprise the pile foundation should shear failure, the lateral resistance of each pile at that moment shall be obtained and the sum of their values shall be the ultimate lateral resistance limit.

The evaluation in the publication "Dynamic bearing capacity and deformation performance in building seismic design" as indicated above are all rough calculations with the exception of the evaluation method employing elasticity analysis, and may not always be on the safe-side of a more precise calculation method. Especially for lateral displacement, a more reliable value is difficult to obtain. For deformation performance of the pile-body, although it has been classified into three categories, as mentioned above, currently, it is still lacking in sufficient backing materials. Therefore, when these evaluation methods are to be adopted, the range of application for each method shall be fully considered and care should be taken not to design on the danger-side.

From the above, in this guideline, we recommend the evaluation of the lateral resistance limit to be made by the calculation method taking the non-linearity of the pile-body and ground into consideration, as indicated in a(c). When adopting this method, there is a need to establish the relationship between the flexural moment M of each element of the pile-body and the curvature ϕ, and the relationship between lateral subgrade reaction of each ground stratum p and lateral displacement.

The M- ϕ relationship can be calculated from the stress - strain relationship of the pile material, but will be significantly influenced by axial force (refer to Section 6.7 "Cross section design of pile-body"). Generally, a pile located in the central part of the building will be receiving small varying axial force in an earthquake, but those located on the outer perimeter tends to receive a much larger varying axial force. Especially for buildings with a large height-to-width ratio, where such influence cannot be ignored, the M- ϕ relationship should be established by taking the effects of the varying axial force into consideration in addition to the normal perpendicular load (refer to Section 6.1, 3(4)). However, the calculation of the behavior of a pile where perpendicular and lateral load will vary simultaneously under a seismic load will require high analytical skills, and even though it is possible, it has a larger meaning in research, and is not practical to apply it into design at this stage. Therefore, to what extent the effects of axial force, including the variance in axial force in the pile axis direction, are to be considered is left to the judgment of the designer. There is also an option to select the following rough calculation for the discussion of the ultimate limit load. The method is to set the $M - \phi$ relationship by grouping the piles that support the building into the central part of the building, the outer perimeter on the compression-side and the outer perimeter on the tension-side, and setting up the M- ϕ relationship under a constant perpendicular load corresponding to the axial force for the central part piles, the maximum axial force under earthquake for the compression-side piles and the minimum axial force under earthquake for the tension-side piles.

For p - y relationship, although it is possible to directly establish the relationship, here, we shall indicate the method to establish lateral modulus of subgrade reaction k_h corresponding to the secant modulus. Regarding the lateral modulus of subgrade reaction k_h of the ground, in the publication "Dynamic bearing capacity and deformation performance in building seismic design" by the organization, when the pile-body is within its elastic range, k_h-value calculated backwards from the lateral loading test results of an actual-sized pile can be expressed as a product of the reference lateral modulus of subgrade reaction k_{h0} (the lateral modulus of subgrade reaction for lateral displacement of 1 (cm)) with the non-dimensional lateral displacement y to the 1/2 power. Here the non-dimensional lateral displacement is the non-dimensioned lateral displacement indicated in units of cm.

From the above results, for the k_h-value in actual design, we recommend the use of Eq. (6.6.3). However, the following has been taken into consideration regarding the lateral modulus of subgrade reaction k_h, since its value will dramatically increase and accuracy will fall when lateral displacement y becomes very small, and when y becomes large, the ground will indicate plastic failure and turn into a plastic lateral subgrade reaction p_y (Figure 6.6.9).

$$0.0 \leq \bar{y} \leq 0.1 \quad : \quad k_h = 3.16 \cdot k_{h0}$$

$$0.1 \leq \bar{y} \quad : \quad k_h = k_{h0} \cdot \bar{y}^{-1/2} \qquad (6.6.3)$$

Where, $p = k_h \cdot \bar{y} \leq p_y$ Therefore, $k_h \leq p_y \cdot \bar{y}^{I}$

Here, for the reference lateral modulus of subgrade reaction (k_{h0}), there is a method to obtain this value from the lateral loading test of the pile at the work-site and another method to base it from existing research results, but there is a difference in the accuracy of the coefficients figures from both. Therefore, when calculating the lateral resistance limit for design and lateral displacement limit for design, the value of k_{h0} obtained directly from the lateral load test of the pile may be used but when k_{h0} based on research results are to be used, a discounted value considering the variance of the value must be used.

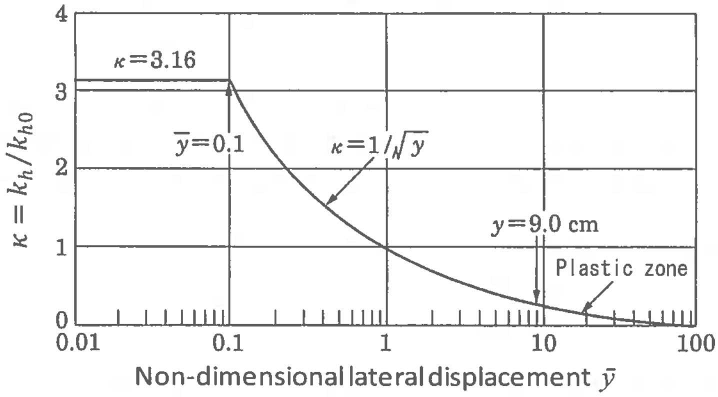


Figure 6.6.9 Relationship Between Lateral Modulus of Subgrade Reaction and Pile Lateral Displacement (Example of the Case Where Plastic Displacement = 9.0 Cm)

In the recommended analysis method, the reference lateral modulus of subgrade reaction k_{h0} for each stratum will be necessary, as indicated in Figure 6.6.5, and assuming the same lateral modulus of subgrade reaction to apply in the depth direction in all the cases, k_{h0} based on existing research is obtained as the lateral modulus of subgrade reaction when lateral displacement at the ground surface is 1 cm. Therefore, currently, the most appropriate way would be to obtain the relationship between lateral subgrade reaction and lateral displacement from the on-site test, such as lateral load test inside the borehole, etc. and provide the numbers for design. However, in the case where on-site test results cannot be obtained, under the present circumstances, existing evaluation equations must be invoked as the lateral modulus of subgrade reaction for design for each stratum.

As evaluation equations concerning the reference lateral modulus of subgrade reaction, there are various proposed equations[6.6.4)] based on gathered information of on-site lateral load test results treated statistically. However, almost all of them are expressed as the relationship between the k_{h0} -value, calculated backwards from the pile-head lateral load - pile-head lateral displacement relationship from the lateral load test results of the pile, and the N-value of the standard penetration test conducted at the subject site. Here, to grasp the mechanical properties of the ground, the N-value of the standard penetration test is said to be more suitable for sandy soil ground than for cohesive soil ground, and in literatures[6.6.5)] the relationship between the k_{h0}-value and N-value for cohesive soil ground is reported to be with a larger variance than for sandy soil ground, therefore, note should be taken that the evaluation accuracy results for cohesive soil ground will indicate lower results.

Combining the above results of research mentioned above, and taking practicality into consideration, this guideline recommends the use of the following evaluation method, indicated in the former guideline[6.6.1)] as the reference lateral modulus of subgrade reaction k_{h0}.

$$k_{h0} = \alpha \cdot \xi \cdot E_0 \cdot \bar{B}^{-3/4} \qquad (6.6.4)$$

Where,

k_{h0}: Reference lateral modulus of subgrade reaction (kN/m^3)

α: Constant determined by the evaluation method (m^{-1})

ξ: Coefficient considering the effects of pile groups, as per Eq. (6.6.9) and (6.6.10). $\xi = 1.0$ in the case of a single pile.

E_0: Deformation coefficient

$\bar{B}$: Non-dimensioned pile diameter (non-dimensional value expressed in pile diameter, for example, pile diameter of 50 cm shall be 50).

The deformation coefficient E_0 in Eq. (6.6.4) is obtained by either of the following methods, but as mentioned above, method suitable for the soil property of the subject land is to be adopted and the values for constant α (m^{-1}) for each method, with the above taken into consideration, are given as follows:

i) Deformation coefficient for ground measured inside the borehole:
   For cohesive soil: $\alpha = 80$
   For sandy soil: $\alpha = 80$

ii) Deformation coefficient for ground obtained from a single axis or triple axis compression test:
   For cohesive soil: $\alpha = 80$

iii) Deformation coefficient for ground estimated at $E_0 = 700 \cdot N$ from the average N-value of the subject ground stratum:
   For cohesive soil: $\alpha = 60$
   For sandy soil: $\alpha = 80$

As for plastic lateral subgrade reaction p_y, the plastic region-I, where the slip lines are directed towards the ground surface, and the plastic region-II, where plasticization of the ground occurs to surround the sides of the pile, are generated as the plastic destruction state of the ground in front on the pile, as indicated in Figure 6.6.10. A plastic lateral subgrade reaction taking this into consideration has been proposed by Broms (refer to former guideline[6.6.1)]).

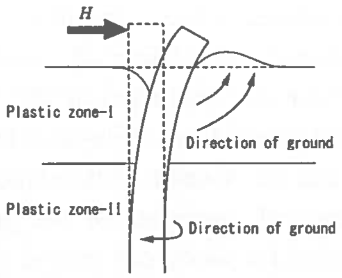


Figure 6.6.10 Plasticization State of Ground in front of Pile for Pile with Lateral Resistance

In this guideline, where the plastic lateral subgrade reaction is p_y in Eq. (6.6.3), we will recommend the use of the equation (6.6.5) proposed by Broms[6.6.6)] (refer to Figure 6.6.11) for sandy soil ground and a revised Ep. (6.6.6) proposed by Broms, (refer to Figure 6.6.12) for cohesive soil ground.

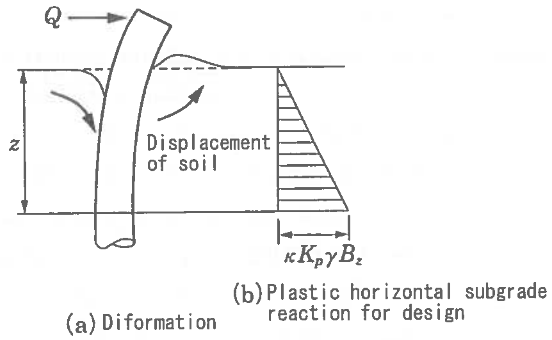


Figure 6.6.11 Plastic Lateral Subgrade Reaction of Sandy Soil Ground

1) Sandy soil ground:

$$\frac{p_y}{\gamma B} = \kappa K_p \frac{z}{B} \qquad (6.6.5)$$

Where,

γ = Unit volume weight of ground (kN/m^3)

B =Pile diameter (m)

κ = Coefficient considering the influence of group piles by Eq. (6.6.11), κ = 3.0 in case of a single pile

K_p= Coefficient of passive earth pressure

z = Depth (m)

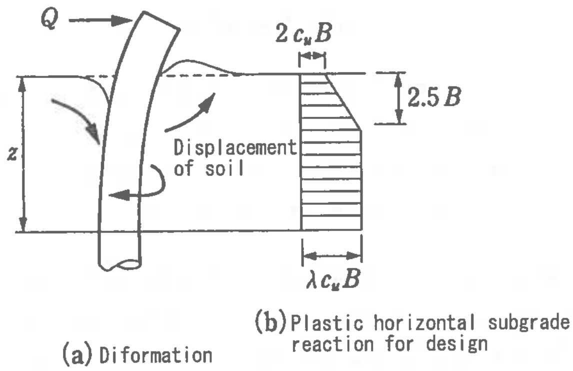


Figure 6.6.12 Plastic Lateral Subgrade Reaction of Coherent Soil Ground

2) Cohesive soil ground:

When $\frac{z}{B} \leqq 2.5 : \frac{p_y}{\gamma B} = 2\left[1 + \mu \frac{z}{B}\right] \frac{c_u}{\gamma B}$

When $\frac{z}{B} \geqq 2.5 : \frac{p_y}{\gamma B} = \lambda \frac{c_u}{\gamma B}$ (6.6.6)

Where,

μ, λ: Coefficient considering the influence of group piles by Eq. (6.6.12) and (6.6.13), μ = 1.4 and λ= 9.0 in case of a single pile

C_u:Undrained shear strength (kN/m^3)

In order to know the accuracy of the design values in the recommended equations for lateral modulus of subgrade reaction and the plastic lateral subgrade reaction mentioned above, a comparison of the pile-head lateral displacement obtained from 38 cases of actual, on-site lateral load test with the calculated values has been made and indicated in Figure 6.6.13. It should be added that the results of this figure are for (a) the elastic region of the pile-body in consideration of the plastic subgrade reaction, and (b) the pile-head lateral displacement ratio (tested value/calculated value) has been calculated for each load stage in each test and the average value has been indicated in the chart as one sample (in other words, 1 sample for 1 test). The standard deviation of the chart is approximately 0.542, indicating a rather large dispersion, but the overall average is approximately 0.962 and it can be seen that there is a tendency to slightly over evaluate the pile-head lateral displacement.

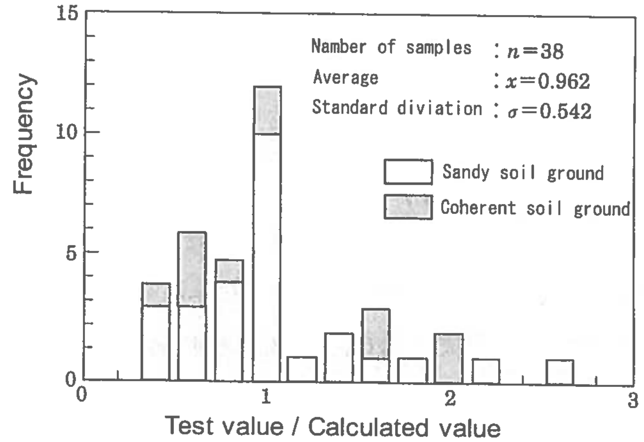


Figure 6.6.13 Design Accuracy for Recommended Lateral Modulus of Subgrade Reaction and Test Valure/ Calculated Value

In the calculation method of a (a), lateral displacement of pile-head $y_0$is used as the representative value to obtain the lateral modulus of subgrade reaction k_h (refer to Eq. (6.6.3)). This being the case, it must be understood that it will be different from the calculation results using k_h-value obtained, corresponding the lateral displacement of the pile in depth orientation distribution.

The reference lateral modulus of subgrade reaction and plastic lateral subgrade reaction indicated above are all for piles with a circular cross-section, but in the future, designs using piles as a consecutive underground wall (refer to Section 6.2 “Types and performance of piles”) featuring the properties of pile cross-sections could be considered. In other words, as indicated in literature[6.6.8)], these piles possess both a strong axis and a weak axis from their cross-section shapes and their resistance characteristics will differ for each direction, and design must be made taking these effects into consideration. Furthermore, there are only very little evaluation methods available for the design of such piles, however, in actual design, the evaluation method indicated in literatures[6.6.8)] can be used as a reference.

The calculation method of a(c) that takes into account the non-linearity of the pile-body and the ground is rather complex and the use of a computer is a must. Therefore, a rough calculation method is indicated as an exemplar calculation taking into consideration the effects of non-linearity of the pile-body and the ground by applying the calculation method a (a) for reference (refer to calculation example 8).

APPENDIX II ARCHITECT LICENSE IN JAPAN

The education curriculum at Architectural University in Japan is a four-year undergraduate course and five years graduate course (two years Mater and three years Doctoral). The first two years of undergraduate curriculum includes architectural planning and design, structural, mechanical and electrical engineering. The last two years of undergraduate is for specialized fields. In the graduate course, students study in higher levels of specialized fields including research works. All undergraduate or graduate must have to obtain an architectural license through passing national examination and gaining experience in respective fields as given in Table II.1 and II.2. Type of structures, size and categories of buildings versus class of architects are given in Table II.3.

Table II.1 Requirements of Examination for First Class Architect License

	Educational Background			Work Experience (Year)
	Attended School	Years	Course	
1	Univeristy	4	Architecture/Civil Eng	≥ 2
2	College	2		≥ 4
3	College	3		≥ 3
4	Technical Junior College	5		≥ 4
5	Others whom Ministry of Land, Infracture, Transport and Tourism approves			

Table II.2 Requirements of Examination for Second Class Architect License

	Catagories	Field	Work Experience (Year)
1	First Class License Holders	Architecture	No Need
		Civil Eng	≥ 1
2	Techical High School	Architecture	≥ 3
		Civil Eng	
3	No educational background as architect		≥ 7
4	Others whom Governor of Tokyo, Osaka, Kyoto, Hokkaido and 43 prefectures approves		

Notes: Source- The Japan Architectural Education and Information Center, Institutional System of Architects and as of 30th March 2011.

Table II.3 Type of Structures, Size and Catagorises of Buildings vs Class of Architects

Floor Area	Timber Structures (*1)			RC, Steel Encased RC, Stone/Brick Masonry, Concrete Block, Unreinforced Concrete			
				H ≤ 13m and Eaves H ≤ 9m		H > 13m	Eaves H > 9m
	Number of Storey						
(sq-m)	1	2	3	≤ 2	≥ 3		
30	No license required			No license required			
100				First/Second Class Architect			
200	First/Second Class Architect or Timber Structure Architect						
300							
500						First Class Architect	
1000	(*2)	(*2)	(*2)				

Notes:

(*1) Only the First Class Architect is permitted to perform design or construction supervision of a new building if height exceeds 13m or eaves height exceeds 9m.

(*2) Only the First Class Architect is permitted to perform design or construction supervision of the building catagories like School, Hospital, Theater, Cinema Hall, Auditorium, Public Hall, Departmental Store, Shopping Center.

II.1 Structural Engineer License in Japan

Structural engineer license can be obtained under following conditions:

(1) Must have five years practical experience while holding first class architect license as per Table II.1.

(2) Must have to pass national examination for first class license of structural design.

II.2 Design Check System in Japan

Official design check is done in two steps:

(a) General check by the city government

(b) Special check by the licensed structural engineers belonging to the government recognized companies specialized in building plan and design check.

APPENDIX III CALCULATION DOCUMENTS IN JAPAN

Infrastructure of Document

In general, infrastructure means roads, bridges, airports, dams etc., which are the basic civil engineering structures. Engineering documents are also regarded as important infrastructure. Illustrative examples for the seismic design of structures shall be composed of stepwise essential matters and they will be retained as an archive for the retrofitting of buildings. They are very important property materials for building owner and will be indispensable if the building is to be retrofitted in future.

The contents of the calculation sheets for buildings in Japan are introduced below.

CONTENTS

Soil investigation reports and other technical reports, if any, shall be attached..

I. OUTLINE OF STRUCTURAL DESIGN

I-1 OUTLINE OF STRUCTURAL DESIGN

Section 1. Outline of building

(1) Name of building
(2) Name of Structural Engineer
(3) Name of affiliated department
(4) Building Site
(5) Building Use
(6) Building Size
- Total floor area :
- Building floor area :
- Structural System :
- Storey :
- Building Height :
- Depth of the Foundation mat

(7) Structural characteristic matters of the building
–Structural design philosophy –
(8) Referenced Building Code and Standard
BNBC-2006
ACI-318-08
ASCE 7-05 and etc.
(9) Structural materials
- Concrete
- Steel reinforcement
- Other materials

(10) Soil report
(11) Building Shape Data

- Floor Plan
- Elevation

§2. LOADS

(1) Dead Load
(2) Live Load
(3) Snow Load
(4) Wind Load
(5) Seismic Load
(6) Other Load to be considered, if any.

§3. Stress of members

(1) Modeling of structural frames
(2) Stress by vertical load
(3) Stress by lateral load

§4. Proportioning of members
§5. Design of Pile Foundation or Mat Foundation
§6. Storey drift, Eccentricity and Vertical Stiffness

I-2 Overall Structural Design View

II. INDIVIDUAL CALCULATION SHEETS

§1. Secondary-Structural Members
§2. (Pile or Mat)Foundation
§3. Foundation Beams
§4. Other Miscellaneous Members

III. COMPUTER OUTPUT DATA

A File Book constructed by extracted calculation sheets from computer data.

Postscript

Computer is a vital key machine in any business office nowadays. Nobody likes to work without the use of a computer. About 40 years ago, even engineering calculations were done manually or by using slide rules and abacuses. Gradually the slide rules are replaced by digital calculator.

Though the structural analysis of buildings is completely and precisely done by means of computer nowadays, engineers still should have the capability to solve fundamental problems like critical load path, controlling load combination and assumptions such as support conditions, joint behavior etc. in an intuitive and technological way. The computer is only a tool for the analysis of buildings.

Here are the standard contents of the structural calculation sheets being used in Japan. The shaded items in the contents are the important parts where the structural engineer needs to describe his design policy. The structural engineer should make complete calculation sheets not only by means of the computer but also his own. Good calculations inspire good engineering.

APPENDIX IV LIVE LOADS AND LOAD COMBINATIONS

The regulated live loads by the Japanese Building Standard Law are shown in below. The live load for the building with apparel manufacturing machines is calculated at the clause of "Practical example for Industry Building" as an example.

1. Live load at each part of a building shall be calculated according to the actual conditions of the building concerned. Provided, that imposed loads for the floor of rooms as mentioned in the following table (**Table IV.1**) may be calculated by multiplying a value as shown in each columns (a), (b) and (c).

Table IV. 1 Live Loads

<table>
<tr><td colspan="3" rowspan="2">Items to be calculated
Kinds of rooms</td><td>(a)</td><td>(b)</td><td>(c)</td></tr>
<tr><td>Structural calculation for floors
(unit: N/sqm)</td><td>Structural calculation for beams, columns or foundations
(unit: N/sq m)</td><td>Calculation of seismic force
(unit: N/sq m)</td></tr>
<tr><td>(1)</td><td colspan="2">Habitable rooms of houses, bedrooms or sickrooms of buildings other than houses</td><td>1800</td><td>1300</td><td>600</td></tr>
<tr><td>(2)</td><td colspan="2">Offices</td><td>2900</td><td>1800</td><td>800</td></tr>
<tr><td>(3)</td><td colspan="2">Classrooms</td><td>2300</td><td>2100</td><td>1100</td></tr>
<tr><td>(4)</td><td colspan="2">Sales area in department stores or other stores</td><td>2900</td><td>2400</td><td>1300</td></tr>
<tr><td rowspan="2">(5)</td><td rowspan="2">Seating space or meeting rooms of theaters, movie theaters, entertainment halls, grand-stands public halls, assembly halls or other buildings for similar use</td><td>In the case of fixed seats</td><td>2900</td><td>2600</td><td>1600</td></tr>
<tr><td>In the case of other seats</td><td>3500</td><td>3200</td><td>2100</td></tr>
<tr><td>(6)</td><td colspan="2">Automobile garages or passageways for automobiles</td><td>5400</td><td>3900</td><td>2000</td></tr>
<tr><td>(7)</td><td colspan="2">Corridors, vestibules or stairs</td><td colspan="3">For those connected to rooms as mentioned in (3) through (5), the value of "In the case of other seats" of (5) is to be taken.</td></tr>
<tr><td>(8)</td><td colspan="2">Open space on roof or balconies</td><td colspan="3">Value of (1) is to be used. Provided, that in the case of buildings for use as schools or department stores, the value of (4) is to be taken.</td></tr>
</table>

2. In calculating compressive force for columns or foundations due to vertical loads, the values of column (b) of the table of the preceding paragraph may be reduced down to those obtained by multiplying the said values by the figures shown in the following table according to the number of floors supported by such columns or foundations. Provided, that this shall not apply to the live loads for the floors of rooms as mentioned in item (5) of the table in the said paragraph.

Number of floors supported	2	3	4	5	6	7	8	9 or more
Multiplier for reducing live load	0.95	0.9	0.85	0.8	0.75	0.7	0.65	0.6

3. Even if the live load for the floors of commercial warehouses calculated according to the actual conditions under paragraph 1 is less than 3,900 Newton/sq m, the value shall be deemed to be 3,900 Newton/sq m.

(Load Combinations)

Load combinations regulated by the Japanese Building Standard Law is shown in following Table IV.2:

Table IV.2 Load combination

Stress type	Load condition	Ordinary district	Heavy snow district
Permanent	Normal	G + P	G + P
	Snow		G +P + 0.7S
Temporary	Snow	G + P + S	G + P + S
	Wind	G + P + W	G + P* + W
			G + P* + 0.35S + W
	Earthquake	G + P + K	G + P + 0.35S + K
Permanent allowable stress is used to verify permanent stress, temporary allowable stress to verify temporary stress.	G: stress caused by dead load P: stress caused by live load S: stress caused by snow load W: stress caused by wind load K: stress caused by seismic load		P* Stress caused by the live load shall be reduced appropriately for verify-cation of overturning of building and pulling-out of columns.

Live loads are the weights of people, furniture, supplies, machines, stores, and so on, borne by the building during its use and occupancy.

Live loads are distinguished from dead loads which are the weights of the building itself, the secondary members and the finishing materials. Live loads are movable and variable during the use and occupancy of the building, and sometimes cause dynamic effects. Therefore, they are easily affected by social transitions, such as the rapid advances in building services equipment and mechanization.

The loads of small or movable pieces of equipment are considered as live loads. But equipment that belongs to the building, fixed and heavy is regarded as bead load. For example, in case of designing garments factory buildings, it is very important for structural engineers to decide if the machines (such as sewing machines) are dead load or live load. The structural engineer needs to obtain the correct data of machine loads from the clients.

(Practical example for Industry Building)

Sewing machines (weight is 10kN/set) are assumed to be placed on the floor bed of 4.0m×5.0m area.

1) For slab design

The slab shall be designed for the distributed load of "ω" as below.

$$\omega = 10\,\text{kN/set} \times 10\,\text{set}/4.0\text{m} \times 5.0\text{m} = 5\,\text{kN}/\text{m}^2$$

Which is loaded to the slab directly.

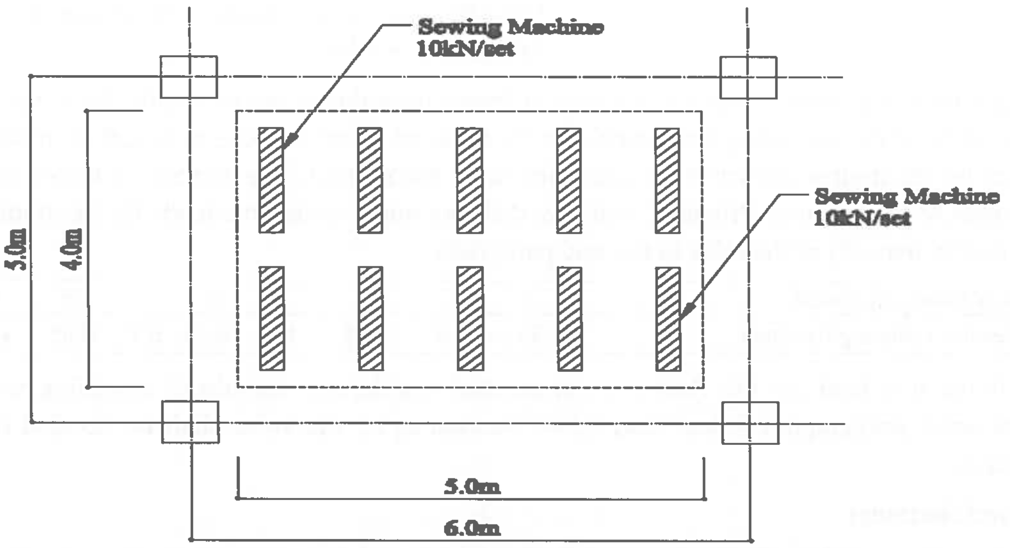


Figure IV.1 Displacement of Sewing Machine on Floor

2) For frames (columns, beams and foundations) design

Design load of 5kN/m^2 for frames can be reduced to 3.5 kN/m^2 that is 70 percent of 5kN/m^2. The approximate value of 70 percent is obtained from the statistical estimation.

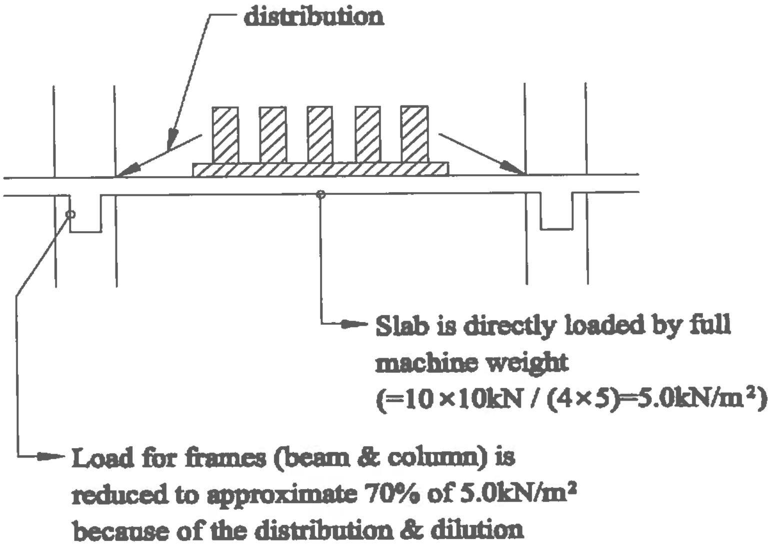


Figure IV.2 Distribution of Machine Loads

3) For live Load applied to seismic design

The live load (ω') applied to the seismic design shall be obtained by dividing the actual machine weight by the floor area.

$$\omega' = 10\frac{kN}{set} \times \frac{10set}{5m \times 6m} = 3.33kN/m^2$$

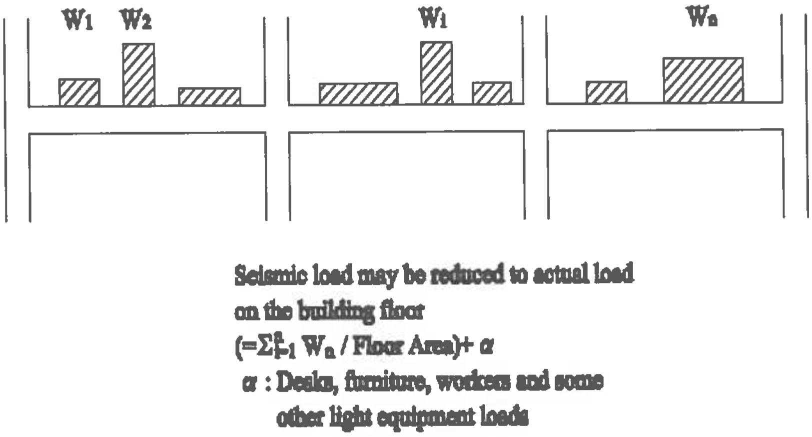


Figure IV.3 Seismic Load of Machine

LIST OF FIGURERS

Part-I

Appendix A.

Appendix B.

Appendix C.

Appendix I. Pile Foundation Design in Liquefaction Soil

Appendix-IV Live Loads and Load Combinations

LIST OF TABLES

Part-I

Prepared Under

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

A Technical Cooperation Project between PWD and JICA

2015