ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES
INSTITUTE OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

Assessment of the Effect of Size of Diaphragm Discontinuity (Opening) on the Rigidity of Diaphragm and Distribution of Lateral Load to the Lateral Load Resisting Element

A Thesis Submitted to the Graduate School of the Addis Ababa University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Structural Engineering)

By

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This is to certify that the thesis prepared by Kassahun Memru, entitled: ***Assessment of the Effect of Size of Diaphragm Discontinuity (Opening) on the Rigidity of Diaphragm and Distribution of Lateral Load to the Lateral Load Resisting Element*** **and submitted in partial fulfillment of the requirements for the degree of Degree of Master of Science in Civil Engineering (Structural Engineering) complies with the regulations of the University and meets the accepted standards with respect to originality and quality.**

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ABSTRACT

Assessment of the Effect of Size of Diaphragm Discontinuity (Opening) on the Rigidity of Diaphragm and Distribution of Lateral Load to the Lateral Load Resisting Element

Kassahun Memru

Addis Ababa University, 2013

Earthquake - resistant structures are provided with lateral and vertical seismic force – resisting systems capable of transmitting inertial forces from the location of masses throughout the structure to the foundations. Floor and roof diaphragms play a key role in distributing earthquake-induced loads to the lateral load resisting systems. Continuity and regular transitions are essential requirements to achieve adequate load paths. Floor diaphragm that has a large opening is likely to be inefficient in distributing seismic loads to the vertical elements.

Design codes like UBC-97 specify that diaphragms having cut out or open areas greater than 50 percent of the gross enclosed area of the diaphragm are affected by diaphragm discontinuity[11]. However, there are different cases where this provision can be unsuccessful. In order to investigate the effect of diaphragm discontinuity on the response of a structure, different parametric studies have been carried out. In the parametric studies, story height, shear wall width, number of stories, number of bays, shape of diaphragm opening, diaphragm opening size, span length and opening location in stories are taken as parameters.

The study proceeded by assessing the effect of diaphragm discontinuity/opening on response of a structure like diaphragm rigidity, story drift, distribution of lateral load to lateral force resisting element and natural vibration period for each parametric study listed above. The study has shown that the rigidity of a slab does not only depend on the opening size, but rather depend on the stiffness of vertical elements, number of stories, aspect ratio of the slab dimension and shape of diaphragm opening.

The investigation has shown that the code provision relating diaphragm rigidity to opening size is not always satisfied.

Key words: diaphragm discontinuity, opening, diaphragm rigidity, actual diaphragm stiffness, rigid diaphragm, flexible diaphragm, stiffness, story drift, lateral force distribution, period

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My Family

And

My Advisor Dr.-Ing. Adil Zekaria

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A	Cross Sectional Area
a	Nodal Displacement Vector
ACI	American Concrete Institute
ATC	Applied Technology Council
BFs	Braced Frames
b_o	Critical Section
CBFs	Concentrically Braced Frames
Comb	Combination
DL	Dead Load
E	Elastic Young's Modulus
EBCS	Ethiopian Buildings Code Standard
EBFs	Eccentrically Braced Frames
EC	European Code
Ec	Modulus of Elasticity of Concrete
EQ	Earthquake
ETABS	Extended 3D Analysis of Building Systems
f	Force Vector
F_D	Damping Force Vector
F_E	Vector of Earthquake Loads
FEMA	Federal Emergency Management Agency
F_I	Inertia Force Vector

F_R	Vector of Restoring Forces
G	Elastic Shear Modulus
HFs	Hybrid Frames
H/L	Height to Width
I	Moment of Inertia
Ig	Moment of Inertia of Gross Concrete Section
J	Torsional Moment of Inertia
K	Global Stiffness Matrix
kPa	Kilo Pascal
LFRS	Lateral Force Resisting System
LLRS	Lateral Load Resisting System
LL	Live Load
Lmax, Lmin	Larger and Smaller in Plan Dimension of the Building Measured in Orthogonal Directions
MDOF	Multi Degree of Freedom
MRFs	Moment Resisting Frames
Psf	Pound Per Square Foot
P-Δ	Forces Deformation
RC	Reinforced Concrete
SDOF	Single Degree of Freedom
SWs	Structural Walls
T	Period
TSs	Tube Systems

UBC	Uniform Building Code
VLFR	Vertical Lateral Force Resisting
VLLR	Vertical Lateral Load Resisting
WD	With Diaphragm
WOD	Without Diaphragm
Δ(%)	Difference in Percent
Δav	Average Inter Story Drift
Δdiaph	In plane Diaphragm Displacement
Δdrr	Diaphragm Rigidity Ratio
Δstory	Inter Story Drift
δ	Local Deformation
λ	Ratio Between the Length of the Longer and the Length of the Smaller Side in Plan

Chapter One

1. Introduction

1.1. Background

Earthquake - resistant structures are provided with lateral and vertical seismic force – resisting systems capable of transmitting inertial forces from the location of masses throughout the structure to the foundations. Continuity and regular transitions are essential requirements to achieve adequate load paths.

Floor and roof systems act as horizontal diaphragms in building structures. They collect and transmit inertia forces to the vertical elements of lateral load resistant systems, i.e. columns and structural walls. They also ensure that vertical components act together under gravity and seismic loads. Diaphragm action is especially relevant in cases of complex and non - uniform layouts of vertical structural systems, or where systems with different horizontal deformation characteristics are used together (as in dual or mixed systems)[1].

Continuity between structural components is vital for the safe transfer of the seismic forces to the ground. Failure of buildings during earthquakes is often due to the inability of their parts to work together in resisting lateral forces. Floor diaphragms that have very elongated plan shapes, or a large opening, are likely to be inefficient in distributing seismic loads to the vertical elements. The assessment of the effect of diaphragm discontinuity on diaphragm rigidity and distribution of lateral load to lateral force resisting element is the main objective of this thesis.

1.2. Statement of the problem

Most of the time buildings or structures with floor plan have open down throughout the floor or in some section of the floor like mezzanine floors in a building. The existence of these openings has different architectural function or aesthetic value. When these openings are usually large, most of the time effect of the size of opening (diaphragm discontinuity) which affects rigidity of a diaphragm and distribution of lateral load to the lateral load resisting element are not given serious attention.

Design codes like UBC-97 specify that diaphragms having cut out 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 story to the next is affected by diaphragm discontinuity[11]. In this thesis, the effect of the size of diaphragm discontinuity (opening) on the diaphragm rigidity and lateral force distribution to is assessed.

1.3. Objectives

Floor diaphragms that have openings considerably weaken slab capacity and affect even distribution of seismic loads to the vertical lateral load resisting elements. The effect is governed by the rigidity of diaphragm and the size of openings. The objective of this thesis is to assess how a size of opening affects diaphragm rigidity and distribution of lateral load to the lateral load resisting elements.

1.4. Methodology

In order to assess the effects of diaphragm discontinuity (opening) on diaphragm rigidity and distribution of later force to the vertical element, number of building structures are modeled, analyzed and evaluated according to codes provision. To do this; software ETABS 9.7, NEHRP guidelines (FEMA 273)[4] and other additional material have been used to enhance and inspire the research, the materials used in this study are mentioned in the reference section.

Chapter Two

2. Literature Review

2.1. Floor diaphragm

The primary function of floor and roof systems is to support gravity loads and to transfer these loads to other structural members such as columns and walls. Furthermore, they play a central role in the distribution of wind and seismic forces to the vertical elements of the lateral load resisting system (such as frames and structural walls).

In the earthquake resistant design of building structures, the building is designed and detailed to act as a single unit under the action of seismic forces. Design of a building as a single unit helps to increase the redundancy and the integrity of the building. The horizontal forces generated by earthquake excitations are transferred to the ground by the vertical systems of the building, which are designed for lateral load resistance (e.g. frames, bracing, and walls). These vertical systems are generally tied together as a unit by means of the building floors and roof. In this sense, the floor/roof structural systems, used primarily to create enclosures and resist gravity (or out of plane) loads are also designed as horizontal diaphragms to resist and to transfer horizontal (or in-plane) loads to the appropriate vertical elements[3].

Diaphragms behave in-plane as horizontal continuous beams supported by vertical lateral resisting systems (also referred to as ‘ beam analogy ’). The deck or slab is the web of the beam carrying the shear and the perimeter spandrel or wall is the flange of the beam resisting bending as shown in Figure 2.0. Diaphragms should possess adequate shear and bending resistance to withstand in - plane seismic loads and out - of - plane gravity loads[1].

The various floor and roof systems that have evolved primarily for the purpose of supporting gravity loads do not lend themselves easily to analytical calculation of in-plane stiffness of the floor diaphragm. Therefore, in this thesis, the diaphragm rigidity (diaphragm in-plane stiffness) is evaluated according to FEMA 273 classification, which is discussed in section 2.4.3.4.

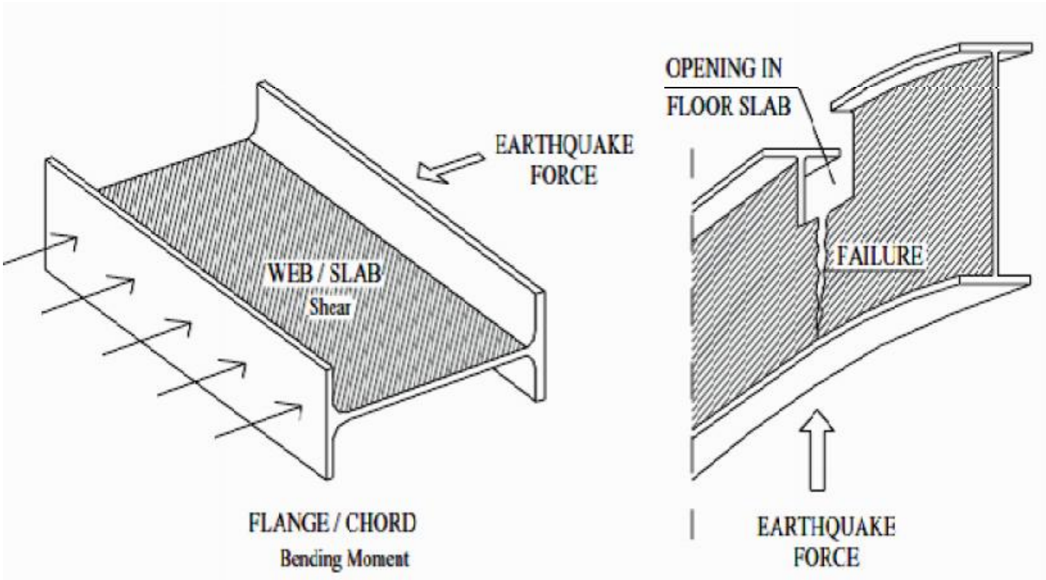
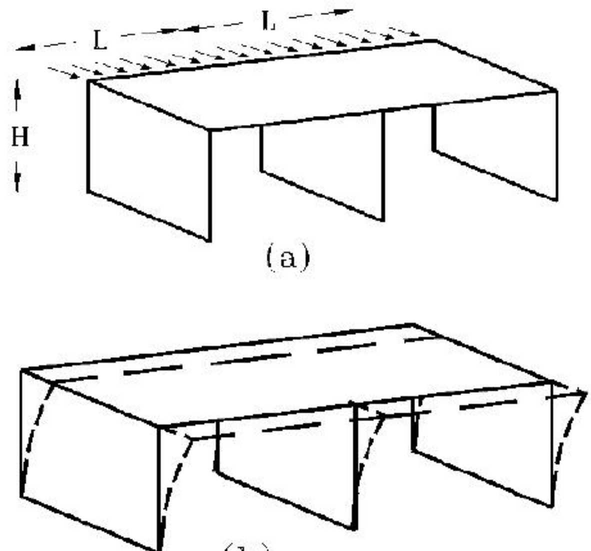


Figure 2.0: Beam analogy for horizontal diaphragm:
load distribution (left) and common failure (right)

2.2. Classification of diaphragm behavior

The distribution of horizontal forces by the horizontal diaphragm to the various vertical lateral load resisting (VLLR) elements depend on the relative rigidity of the horizontal diaphragm and the VLLR elements. According to FEMA 273, floor diaphragms shall be classified as rigid, stiff and flexible[1,3,4].



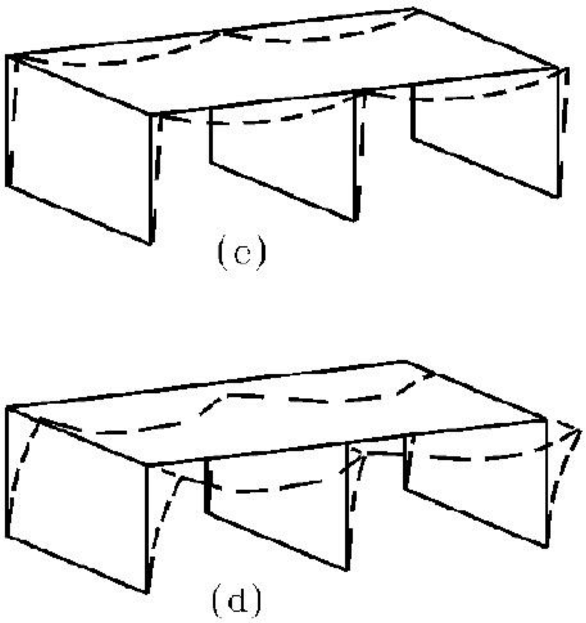


Figure 2.1: Diaphragm behavior

(a) Loading and building proportions. (b) Rigid diaphragm behavior.

(c) Flexible diaphragm behavior, (d) Semi rigid diaphragm behavior

2.2.1. Rigid diaphragm

Diaphragms shall be considered as rigid when the maximum lateral deformation of the diaphragm is less than half the average inter-story drift of the associated story. Rigid diaphragm distributes the horizontal forces to the VLLR elements in proportion to their relative stiffness. It is based on the assumption that the diaphragm does not deform itself and will cause each vertical element to deflect the same amount. Rigid diaphragms capable of transferring torsional and shear deflections and forces are also based on the assumption that the diaphragm and shear walls undergo rigid body rotation and this produces additional shear forces in the shear wall. In rigid diaphragms, the diaphragm deflection when compared to that of the VLLR elements will be insignificant. Rigid diaphragms consist of reinforced concrete diaphragms, precast concrete diaphragms, and composite steel deck[1,3,4].

2.2.2. Flexible diaphragm

Diaphragms shall be considered as flexible when the maximum lateral deformation of the diaphragm along its length is more than twice the average inter-story drift of the story immediately below the diaphragm. For diaphragms supported by basement walls, the average inter-story drift of the story above the diaphragm may be used in lieu of the basement story.

Flexible diaphragm distributes horizontal forces to the vertical lateral load resisting elements independent of relative stiffness of the VLLR element, and the lateral load distribution is according to the tributary area. In the case of a flexible diaphragm, the diaphragm deflection as compared to that of the VLLR elements will be significantly large. Flexible diaphragm distributes lateral loads to the VLLR elements as a series of simple beams spanning between these elements. Flexible diaphragm is not considered to be capable of distributing torsional and rotational forces. Flexible diaphragms are - roofs or floors, including but not necessarily limited to, those sheathed with plywood, wood decking, or metal decks without structural concrete topping slabs[1,3,4].

2.2.3. Stiff diaphragm

No diaphragm is perfectly rigid or perfectly flexible. Reasonable assumptions, however, can be made as to a diaphragm's rigidity or flexibility in order to simplify the analysis. If the diaphragm deflection and the deflection of the VLLR elements are of the same order of magnitude, then the diaphragm cannot reasonably be assumed as either rigid or flexible. Diaphragms that are neither flexible nor rigid shall be classified as stiff[1,3,4].

2.3. Significant factors affecting diaphragm behavior

Low-rise buildings and buildings with very stiff vertical elements such as shear walls are more susceptible to floor diaphragm flexibility problems than taller structures.

In buildings with long and narrow plans, if seismic resistance is provided either by the end walls alone, or if the shear walls are spaced far away from each other, floor diaphragms may exhibit the so-called bow action Figure 2.2. The bow action subjects the end walls to torsional deformation and stresses. If a sufficient bond is not provided between the walls and the diaphragm, the two will be separated from each other starting at the wall corners. This separation results in a dramatic increase in the wall torsion and might lead to collapse[3].

In addition, buildings having a long and narrow floor plan (slender plan) act like flexible beams, and bending deformation of the slabs becomes significant, referred to as the bowing action of the slab. In this type of structure, the actual distribution to vertical members could differ a great deal from the distribution obtained on the basis of the rigid assumption[17]. ASCE7 (2005) acknowledged that ignoring the in-plane flexibility of the diaphragms can result in considerable errors when predicting the seismic response of RC buildings with diaphragm plan aspect ratio greater than 3:1[19].

Euro code 8 states that the slenderness λ = Lmax/Lmin of the building in plan shall be not higher than 4, in order to attain plan regularity. Where Lmax and Lmin are respectively the larger and smaller in plan dimension of the building, measured in orthogonal directions[20].

Another potential problem in diaphragms can be due to any abrupt and significant changes in a wall stiffness below and above a diaphragm level, or any such changes in the relative stiffness of adjacent walls in passing through one floor level to another as shown in Figure 2.3. This can cause high shear stresses in the floor diaphragm and/or a redistribution of shear forces among the walls.

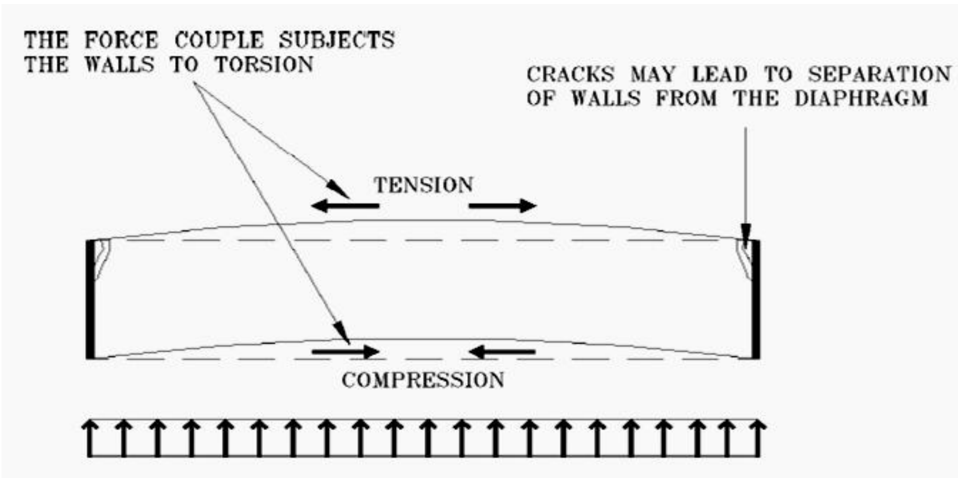


Figure 2.2: A plan showing bow action subjects the end walls to torsion

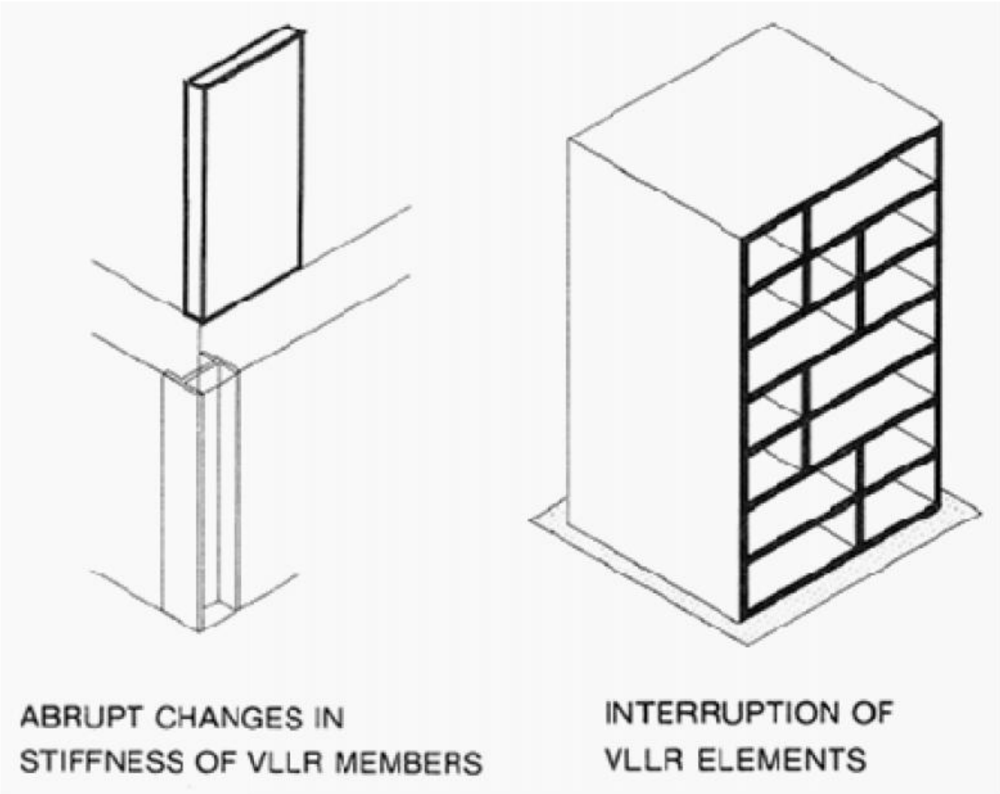


Figure 2.3: Abrupt changes in stiffness and location of VLLR elements can cause drastic redistribution of forces

In buildings with significant plan irregularities, such as multi-wing plans, L-shape, H-shape, V-shape plans, etc. (Figure 2.4). In this type of buildings, the fan-like deformations in the wings of a diaphragm can lead to a stress concentration at the junction of the diaphragms (see Figure 2.5).

Other classes of buildings include those with relatively large openings in one or more of the floor decks (Figure 2.6) and tall buildings resting on a significantly larger low-rise part (Figure 2.7). In the latter case, the action of the low-rise portion as the shear base and the corresponding redistribution of shear forces (kick-backs) may subject the diaphragm located at the junction of the low-rise and high-rise parts (and sometimes a number of floor diaphragms above and below the junction) to some significant in-plane shear deformations.

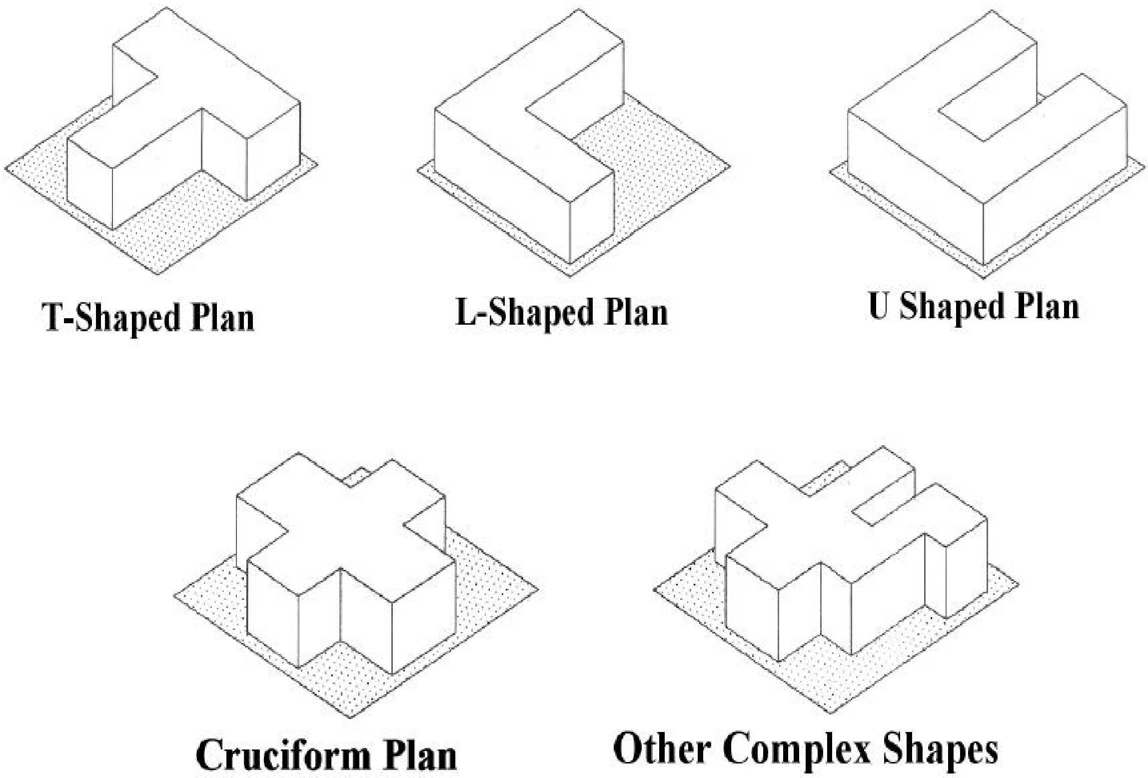


Figure 2.4: Typical plan irregularities

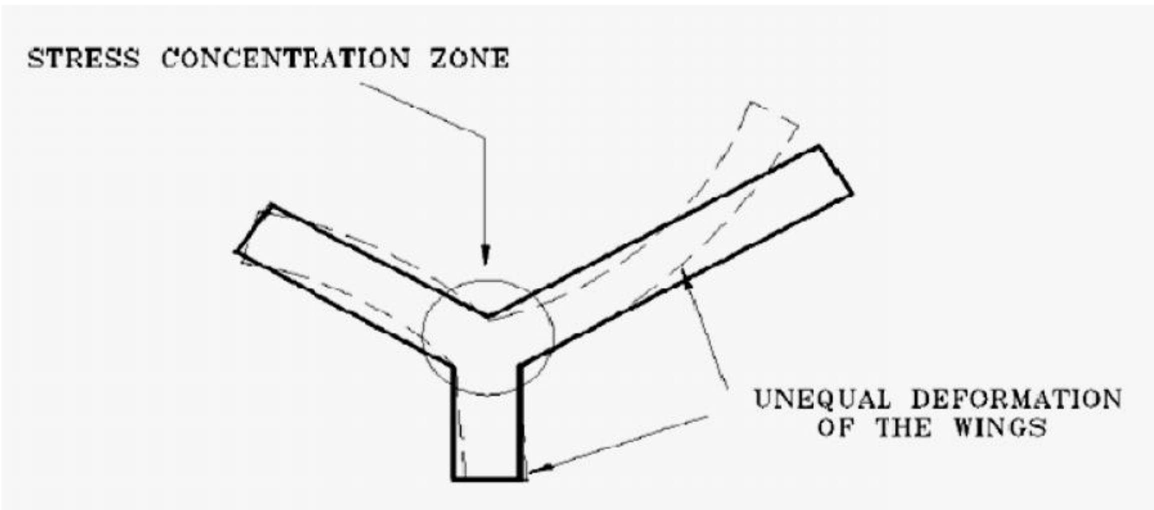


Figure 2.5: Fan-like deformation of wings causes stress concentration at the junction

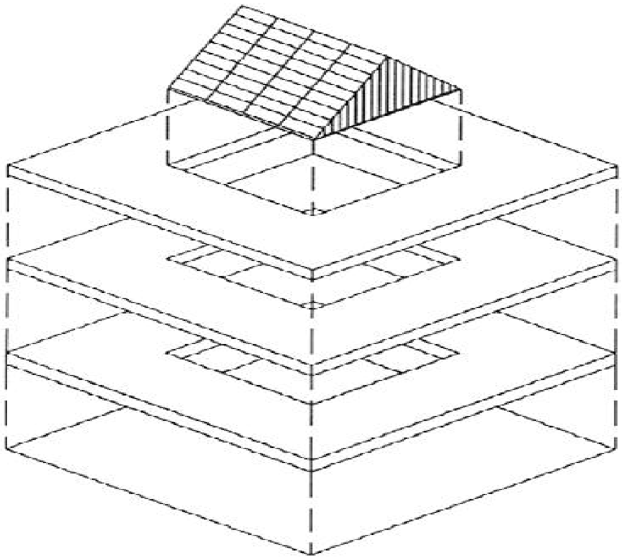


Figure 2.6: Significant floor openings

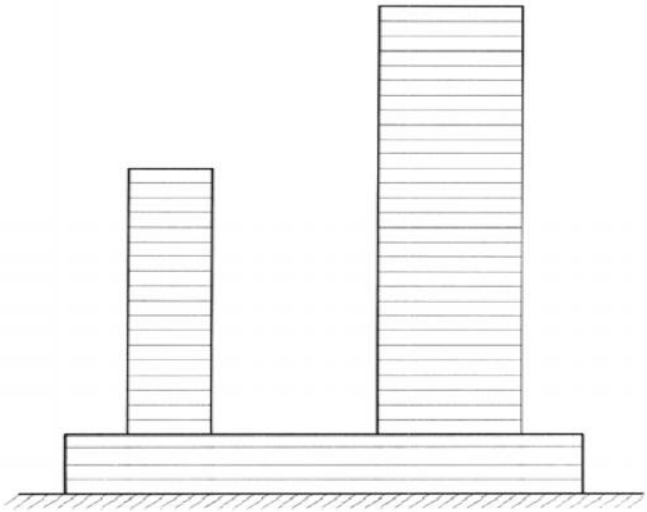


Figure 2.7: Elevation of towers on an expanded low-rise base

2.4. Opening in a diaphragm

2.4.1. Opening in two way concrete floor slab according to ACI-code

Although there are several different variations of two-way slabs, they can be generally described as one or a combination of three two-way systems: flat plates, flat slabs, and two-way beam-supported slabs. The selection of the most advantageous location for a floor opening depends on the type of two-way slab, which is designed and evaluated. The simplest type of two-way slab to construct is known as a flat plate. These slabs are supported directly by the columns and have a completely flat soffit. For live loads of about 50 psf (2.5 kPa), column spacing typically ranges from 15 to 25 ft (4.5 to 7.5 m) with minimum slab thicknesses of 6 to 10 in. (150 to 250 mm). For longer spans, drop panels (thickened portions of the slab) are added at the columns. This system is referred to as a flat slab and has an economical span range of 25 to 30 ft (7.5 to 9 m) with minimum slab thicknesses of 8.5 to 10 in. (200 to 250 mm). Two-way beam-supported slabs have beams spanning between columns in both directions that act with the slab to support gravity loads[8].

For the purposes of design, two-way slab systems are divided into column and middle strips in two perpendicular directions. The column strip width on each side of the column centerline is equal to 1/4 of the length of the shorter span in the two perpendicular directions. The middle strip is bounded by two column strips. Section 13.4.1 of ACI 318-052 permits openings of any size in any new slab system, provided you perform an analysis that demonstrates both strength and serviceability requirements are satisfied. As an alternative to detailed analysis for slabs with openings, ACI 318-05 gives the following guidelines for opening size in different locations for flat plates and flat slabs. These guidelines are illustrated in Figure 2.8 for slabs with $\ell_2 \geq \ell_1$[7]:

• In the area common to intersecting middle strips, openings of any size are permitted (Section 13.4.2.1);

• In the area common to intersecting column strips, the maximum permitted opening size is 1/8 the width of the column strip in either span (Section 13.4.2.2); and

• In the area common to one column strip and one middle strip, the maximum permitted opening size is limited such that only a maximum of 1/4 of the slab reinforcement in either strip may be interrupted (Section 13.4.2.3).

To apply this simplified approach, ACI 318-05 requires that the total amount of reinforcement calculated for the panel without openings, in both directions, must be maintained; thus, half of the reinforcement interrupted must be replaced on each side of the opening.

In addition to flexural requirements, the reduction in slab shear strength must also be considered when the opening is located anywhere within a column strip of a flat slab or within 10 times the slab thickness from a concentrated load or reaction area. The effect of the slab opening is evaluated by reducing the perimeter of the critical section b_o by a length equal to the projection of the opening enclosed by two lines extending from the centroid of the column and tangent to the opening, as shown in Figure 2.9a. For slabs with shear heads to assist in transferring slab shear to the column, the effect of the opening is reduced, and b_o is reduced by only half the length enclosed by the tangential lines, as shown in Figure 2.9b [8].

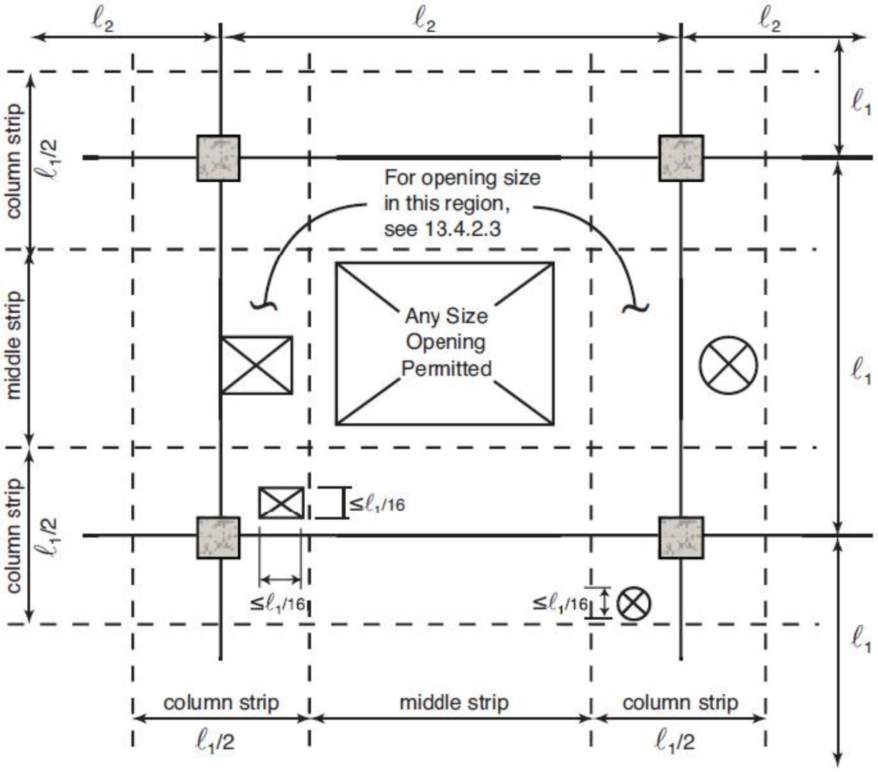


Figure 2.8: Suggested opening sizes and locations in flat plates with $l_2 \geq l_1$ [Refer. 7]

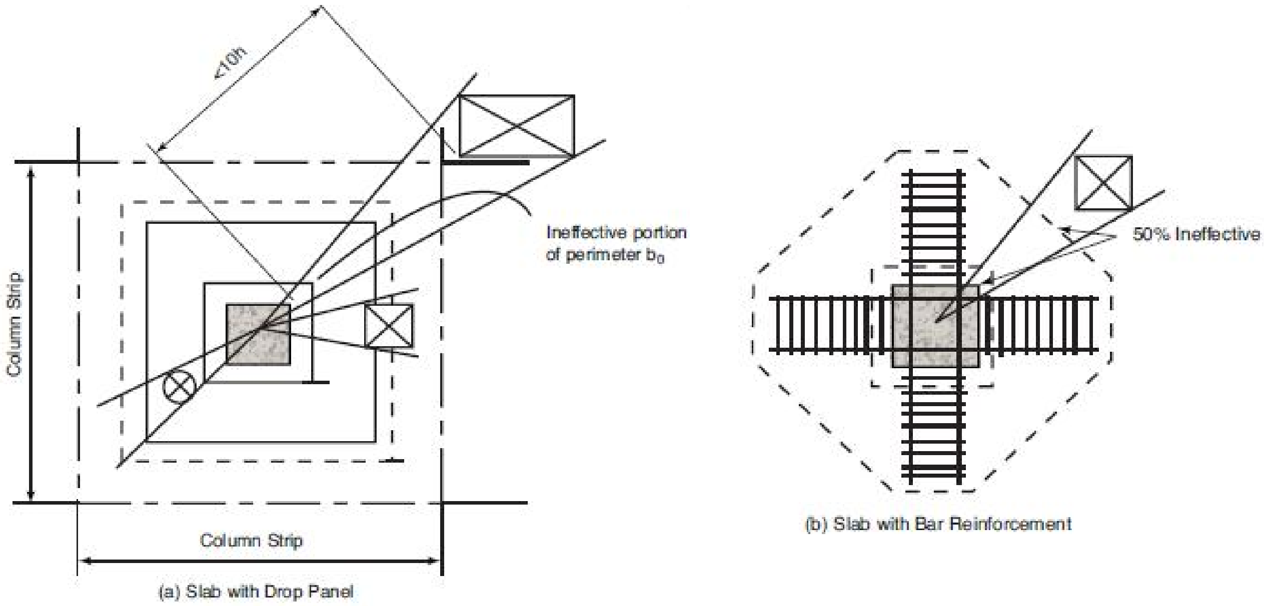


Figure 2.9: Reduction to perimeter of critical section [Refer. 7]

Note: Figure 2.8 Shows reduction to perimeter of critical section b_o for a flat plate or flat slab with openings in column strips or within a distance of 10 times the thickness of the slab from a column: (a) no shear heads; and (b) with shear heads[8].

2.4.2. Diaphragm rigidity and opening size

UBC-97 quantifies plan structural irregularities for diaphragm discontinuity. If 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 story to the next [11].

Floor diaphragms that have very elongated plan shapes, or large openings, are likely inefficient in distributing seismic loads to the vertical element[2].

Excessive openings in a diaphragm can result in a flexible diaphragm response along with force concentrations and load path deficiencies at the boundaries of the openings[12].

The responsibility of the designer in supplying appropriate parameters in diaphragm modeling using current available computational tools cannot be taken lightly. The description of a diaphragm as being flexible or rigid is subjective, and it is not defined by a single parameter. The relative importance of the geometry of the diaphragm including shape and openings; the floor system being composed of just a slab, a slab on girders, a joist system or a system using precast elements; the strength of connections between diaphragm elements and to the vertical members of the lateral force-resisting system; the relative stiffness of the diaphragm and the vertical structural elements (a diaphragm may be considered rigid if supported on columns, but the same diaphragms would be flexible if shear walls are present); and other considerations come into play. In general, most floor systems currently used in reinforced concrete structures would lead to rigid in-plane diaphragms, but this could be misleading if any of the limiting factors mentioned affects behavior[9].

Ethiopian buildings code standard, EBCS-8 (1995), do not have defined guidelines for diaphragm discontinuity size.

2.4.3. Effects of opening (diaphragm discontinuity)

2.4.3.1. Diaphragm capacity

Gravity and earthquake loads should flow in a continuous and smooth path through the horizontal and vertical elements of structures and be transferred to the supporting ground. Discontinuities are, however, frequently present in plan and elevation. Sidestepping and offsetting are common vertical discontinuities, which lead to unfavorable stress concentrations. In plan, openings in diaphragms may considerably weaken slab capacities. This reduction of resistance depends on the location, size and shape of the openings. Figure 2.10 depicts an example of stress concentrations caused by a large opening for stairwells in a floor slab. Conversely, small openings do not jeopardize the load transfer at a floor level; the diaphragm behaves like a continuous beam under uniform seismic forces. High stress concentrations may also exist at the connection between structural walls and slabs, as well as between columns and flat slabs[1].

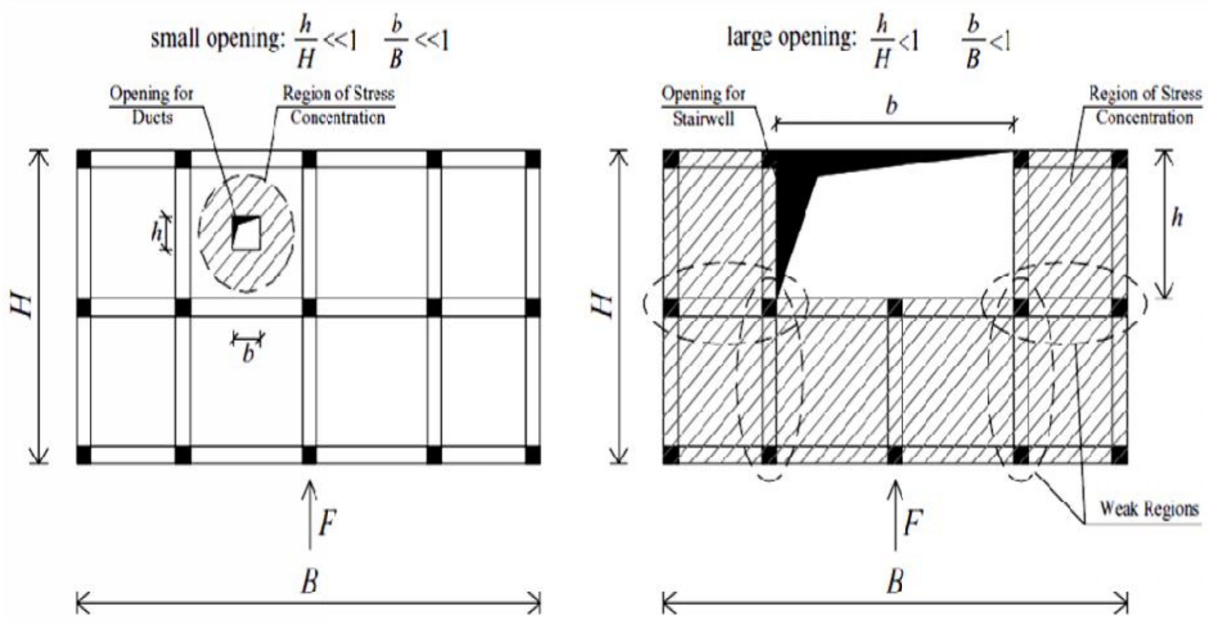


Figure 2.10: Stress concentrations caused by small (left) and large (right) openings in horizontal diaphragms

2.4.3.2. Vertical load path

Earthquake - resistant structures should be provided with lateral and vertical seismic force – resisting systems capable of transmitting inertial forces from the location of masses throughout the structure to the foundations. Structures designed for gravity loads have very limited capacity to withstand horizontal loads. Inadequate lateral resisting systems and connections interrupt the load path. Continuity and regular transitions are essential requirements to achieve adequate load paths as shown in Figure 2.11.

In framed structures, gravity and inertial loads generated at each storey are transmitted first to the beams by floor diaphragms (or slabs), then to columns and foundations as presented in Figure 2.12.

Mechanical and geometrical properties of beam - to - column and column - to - base connections may alter the load path. Continuity between structural components is vital for the safe transfer of the seismic forces to the ground. Failure of buildings during earthquakes is often due to the inability of their parts to work together in resisting lateral forces[1].

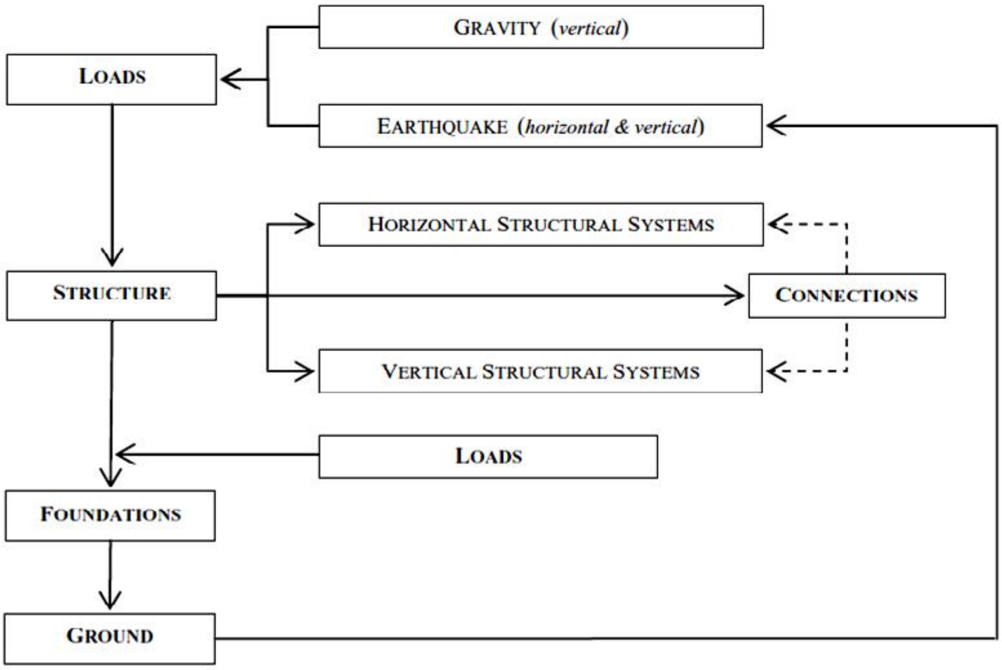


Figure 2.11: Path for vertical and horizontal loads

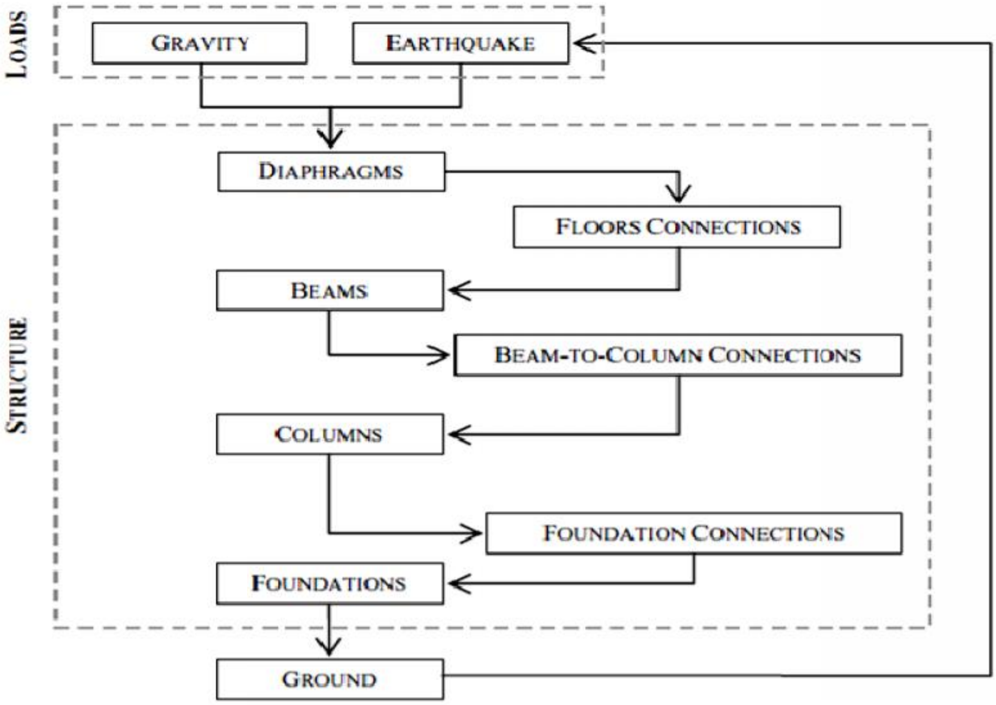


Figure 2.12: Load path in building structures

2.4.3.3. Story drift

Drift is defined as the relative lateral displacement between two adjacent floors, and the term drift index, is defined as the drift divided by the story height. The relative lateral displacement of buildings is sometimes measured by an overall drift ratio or index, which is the ratio of maximum lateral displacement to the height of the building. More commonly, however, an inter-story drift ratio, angle, or index is used, which is defined as the ratio of the relative displacement of a particular floor to the story height at that level see Figure 2.13.

The lateral displacement or drift of a structural system under wind or earthquake forces, is important in structural stability of a building. Excessive and uncontrolled lateral displacements can create severe structural problems. Empirical observations and theoretical dynamic response studies have indicated a strong correlation between the magnitude of inter-story drift and building damage potential[3].

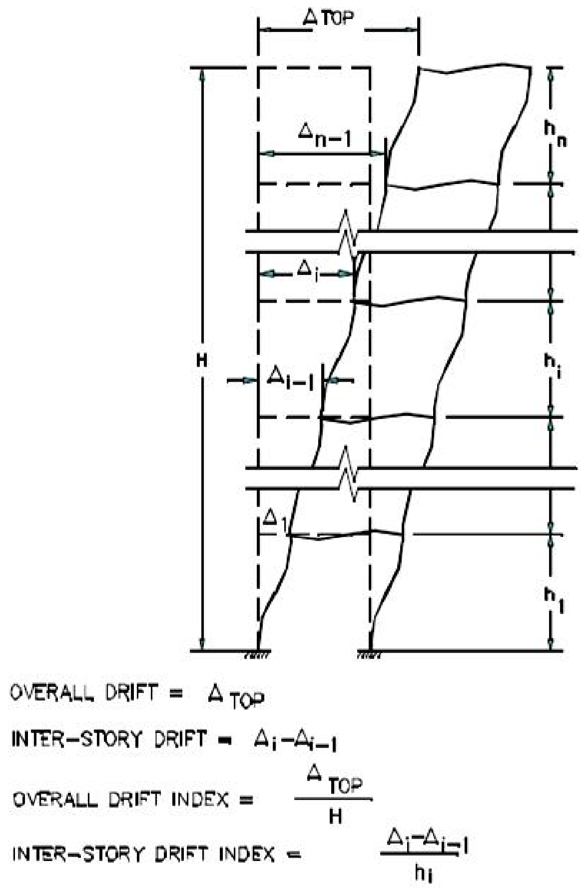


Figure *2.13: Load path in building structures*

2.4.3.4. Diaphragm rigidity

In order to estimate the diaphragm rigidity, it is necessary to predict the deflection of the diaphragm under the influence of lateral loads. The various floor and roof systems that have evolved primarily for the purpose of supporting gravity loads do not lend themselves easily to analytical calculation of lateral deflections. Different codes give different recommendation on diaphragm rigidity of a floor diaphragm. EBCS 8 recommends the rigid body condition may be considered valid if the in-plane deviations of all points of the diaphragm from their rigid body position are less than 5% of their respective absolute displacements under the seismic load combination. FEMA 273 classifies the rigidity of a floor diaphragm according to its behaviour, which can be computed as shown in the Table 2.1 below. [4,16].

Table 2.1: FEMA 273 diaphragm classification

FMA 273 Diaphragm classification	
Rigid	$\Delta_{Diaph} < 0.5\Delta_{Story}$
Stiff	$0.5\Delta_{Story} \leq \Delta_{Diaph} \leq 2\Delta_{Story}$
Flexible	$\Delta_{Diaph} > 2\Delta_{Story}$

Δ_{Diaph} – maximum diaphragm deformation; Δ_{Story} – average inter-story drift of story directly below the diaphragm

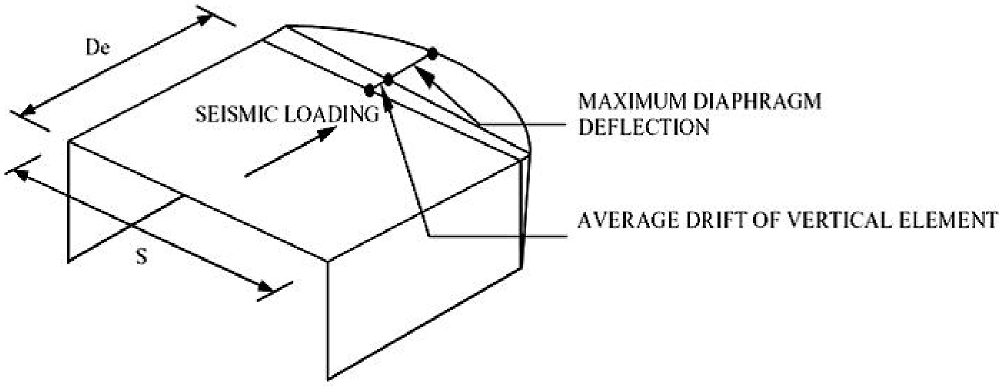


Figure 2.14: Maximum diaphragm deflection and average drift of vertical element

2.4.3.5. Lateral force distribution

Floor diaphragms in reinforced concrete (RC) buildings are typically modeled as rigid during the design phase and so the effect of in-plane diaphragm flexibility on the structure is often not considered. For the rigid diaphragm model, the diaphragm has equal in-plane displacements along its entire length under lateral load such that horizontal forces are transferred to the vertical LLRS proportional to the relative stiffness of each frame. A flexible diaphragm, however, exhibits in-plane bending due to lateral load, resulting in additional horizontal displacements along its length. This can lead to damage of the diaphragm due to high flexural stresses along its boundaries. This flexibility also increases the lateral load transfer to frames that were not designed to carry these additional lateral loads based on a rigid diaphragm model. If this effect is sizeable, it can lead to overloading of structural elements[6].

2.4.3.6. Natural period of vibration

The ground shaking during an earthquake contains a mixture of many sinusoidal waves of different frequencies, ranging from short to long periods. The time taken by the wave to complete one cycle of motion is called period of the earthquake wave. In general, earthquake shaking of the ground has waves whose periods vary in the range 0.03 - 33sec. Even within this range, some earthquake waves are stronger than the others. Intensity of earthquake waves at a particular building location depends on a number of factors, including the magnitude of the earthquake, the epicentral distance, the type of ground that the earthquake waves traveled through before reaching the location of interest and rigidity of the structure, flexible building undergoes larger relative horizontal displacements than rigid building[10].

Fundamental natural period T is an inherent property of a building. Any alterations made to the building will change its T. Value of T depends on the building flexibility and mass; more the flexibility, the longer is the T, and more the mass, the longer is the T. In general, taller buildings are more flexible and have larger mass, and therefore, have a longer T. On the contrary, low- to medium-rise buildings generally have shorter T[10].

2.5. Lateral force and lateral force resisting system

2.5.1. Lateral force

Lateral forces are typically considered to be those which act parallel to the ground plane and may occur at many angles other than perfectly horizontal. Generally lateral forces are considered to act transversely to the primary structural system.

Seismic Loads and wind are the most fundamental lateral forces. May be so small as to be unnoticed, or large enough to level cities. They occur simultaneously with gravity loads. Wind is really a very complex phenomenon with a complex interaction on a building structure. It is influenced greatly by local terrain. When contacting a building, it can produce pressures and suction forces on any surface of a building, plus internal pressures that tend to balloon the building outward. Seismic loads are forces generated by inertia of building mass as ground moves below the structure. It generates forces in direct proportion to the building's mass and stiffness. A massless building would in fact have no seismic forces with at all. By altering the building's stiffness, a substantial change to seismic force is possible[1].

2.5.2. Lateral force resisting system

2.5.2.1. Vertical system

Structural and non - structural damage under earthquakes is caused by inadequate stiffness and/or strength of vertical components of lateral structural systems used for buildings, bridges and other types of construction. Vertical components may also fail because of insufficiency or absence of ductility. To achieve satisfactory seismic performance, vertical components of lateral resisting systems should comply with the structural requirements. Seismic behaviour depends on materials of construction, system configurations and failure modes[1].

Earthquake resistance can be achieved through a wide range of vertical systems, which can range from free - standing columns to complex three - dimensional framed tubes and/or cores. Figure 2.15 shows basic structural systems, which have been ranked according to their lateral stiffness[1].

Columns are the simplest structural elements with lateral stiffness and strength. The relationship between applied actions and lateral deformations depends on their geometric and mechanical properties. The deformed shape of columns is generally characterized by double curvature, thus inelastic demand can be concentrated at both ends[1].

Frames show higher stiffness, strength and ductility than free – standing columns because of their deflected shape. Frame behaviour significantly depends on the relative rigidity of structural members (beams and columns) and connections (beam - to - columns and base columns)[1].

Frames with diagonal braces exhibit higher lateral stiffness and strength than moment frames; the ductility of braced systems is generally endangered by the occurrence of member (diagonal) buckling. Moment frames can be stiffened by infill panels. Infilled frames exhibit higher stiffness, strength and ductility than bare frames. Under lateral seismic loads, infills behave like one diagonal compression brace. Infill panels are often made of brittle materials, such as masonry or concrete, which crack due to their low tensile strength. Lateral stiffness of braced and infilled frames can be enhanced by employing structural walls. These elements usually exhibit high in - plane stiffness and resistance; their ductility depends primarily on the detailing of the foundation connection and their shape. Walls can be arranged to form rigid core systems. The latter possess high resistance but, as for structural walls, their inelastic behaviour can be impaired by seismic details with low ductility[1].

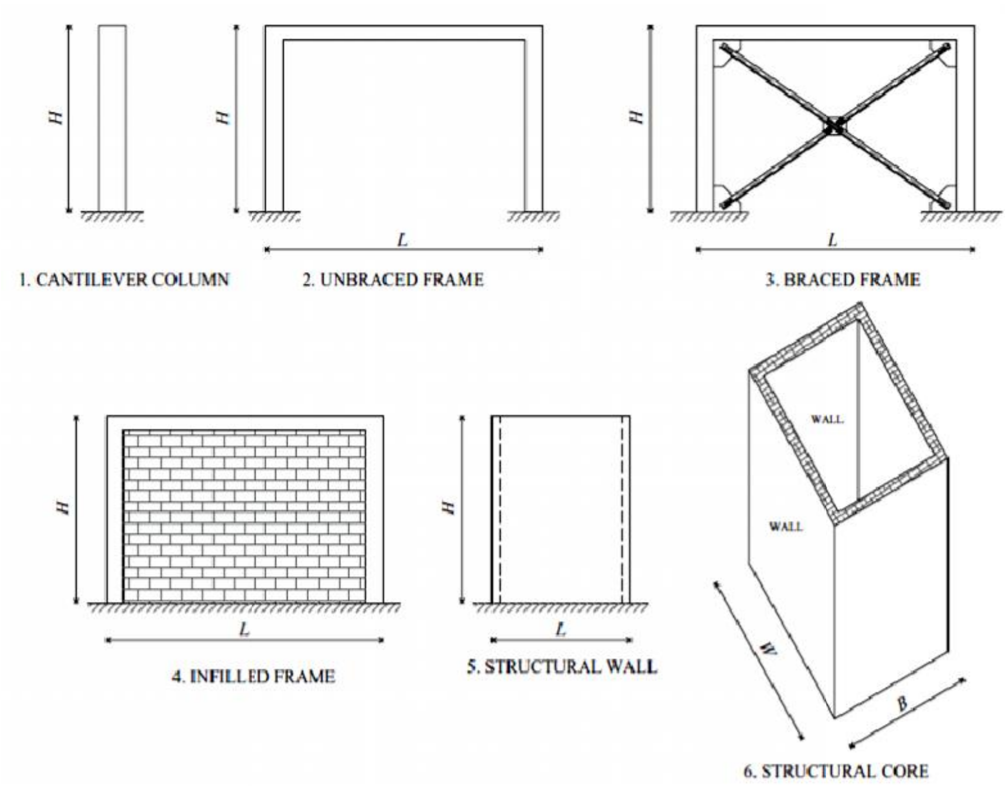


Figure 2.15: Basic vertical structural systems with increasing lateral stiffness (from top left to bottom right)

2.5.2.2. Typical lateral force resisting system

Typical lateral force - resisting systems include the following:

(i) Moment - Resisting Frames, (ii) Braced Frames, (iii) Structural Walls

(iv) Hybrid Systems and (v) Tube Systems

(i) Moment - Resisting Frames

Moment - resisting frames (MRFs) are structural systems consisting of beams, columns and joints. These systems are frequently used as structural skeletons in RC, steel and composite buildings and bridges. Metal and composite MRFs can be classified according to the stiffness and strength of the beam-to-column connections or the sensitivity to second - order effects. Where stiffness is the response characteristic employed, frames can be 'rigid' or 'semi - rigid'. Where, in turn, resistance is used, frames can be 'full strength' or 'partial strength'; the strength is quantified through the bending moment capacity. 'Sway' frames are those with lateral stiffness inadequate to prevent secondary effects, e.g. P - Δ effects; in turn, if these effects are negligible, the frames are described as 'non - sway'[1].

(ii) Braced Frames

Braced frames (BFs) are lateral force - resisting systems, which consist of beams, columns, diagonal braces and joints. Many brace configurations may be efficiently employed to withstand earthquake loads. Braced frames are often grouped into two categories, i.e. concentrically braced frames (CBFs) and eccentrically braced frames (EBFs), depending on the layout of the diagonals employed[1].

(iii) Structural Walls

Structural walls (SWs) are vertical systems, which are frequently combined with RC, steel and composite framed structures to control lateral deflections. These systems are often classified according to their height - to - width (H/L) ratio (also known as vertical aspect ratio) in 'squat' and 'slender' (or 'cantilever') walls. Squat walls have low slenderness: their H/L ratios vary between 1 and 3. Slender or cantilever walls are those with H/L > 6. Under horizontal loads, the ratio of bending - to - shear deflections of structural walls increases with the system aspect ratio H/L[1].

(iv) Hybrid Frames

Rigid moment - resisting frames are ductile systems with high resistance, but their lateral stiffness is often inadequate to prevent large drifts under earthquake forces. To reduce storey and roof drifts, MRFs are often connected to bracing systems or structural walls (also known as ' hybrid frames or dual systems'). It is generally cost - effective for hybrid frames (HFs) to employ frames that are designed for gravity loads only, while horizontal forces are resisted by bracing systems, e.g. braced frames, or structural walls. Typical hybrid frame system is shown below in Figure 2.16[1].

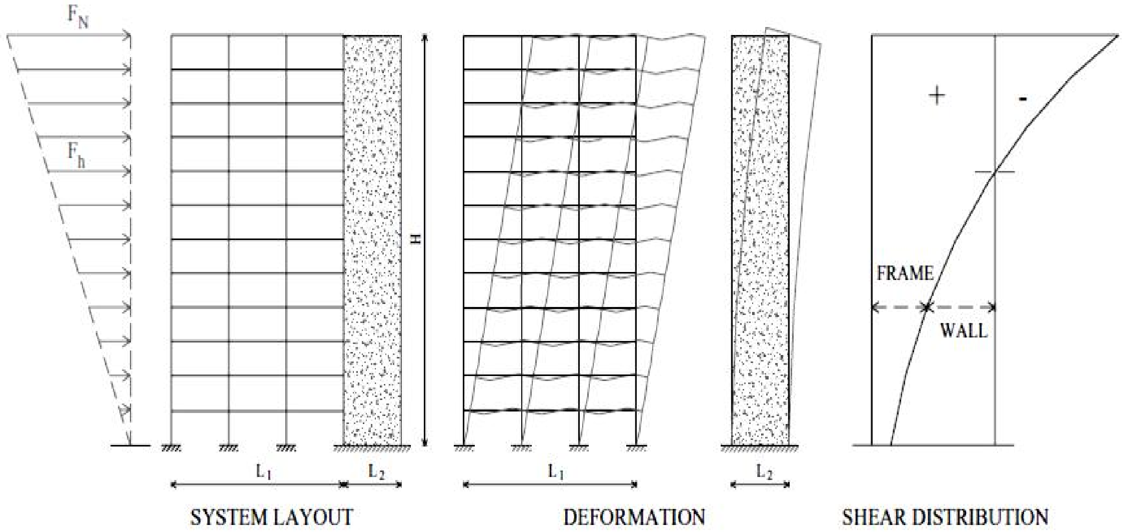


Figure 2.16: Typical hybrid frame, interaction between frame and structural wall

(v) Tube Systems

Tube systems (TSs) are structural systems in which lateral stiffness and strength are provided by MRFs, BFs, SWs or hybrid systems that form either a single tube around the perimeter of the structure, or nested tubes around the perimeter and core of the structure. Tube systems are frequently used for high - rise structures[1].

2.6. Structural response characteristics

2.6.1. Stiffness

Stiffness defines the relationship between actions and deformations of a structure and its components. Whereas member stiffness is a function of section properties, length and boundary conditions, system stiffness is primarily a function of the lateral resisting mechanisms utilized, e.g. moment – resisting frames, braced frames, walls or dual systems, as illustrated in section 2.5[1].

Several types of stiffness may be defined, depending on the nature of applied loads. Structures designed for vertical (gravity) loads generally possess sufficient vertical stiffness. Earthquakes generate inertial forces due to vibration of masses. Horizontal components of these inertial forces are often dominant; hence lateral (or horizontal) stiffness is of primary importance for structural earthquake engineers[1].

2.6.1.1. Factors influencing stiffness

i. Material Properties

Material properties that influence the structural stiffness are the elastic Young ' s modulus E and the elastic shear modulus G.

ii. Section Properties

Section properties that affect the structural stiffness are the cross - sectional area A , the flexural moment of inertia I and the torsional moment of inertia J.

iii. Member Properties

The lateral stiffness also depends on the type of structural members utilized to withstand earthquake loads. Structural walls are much stiffer in their strong axis than columns. Geometrical properties of structural components, such as section dimensions, height and aspect ratio, influence significantly their horizontal shear and flexural stiffness values.

iv. Connection Properties

Connection behaviour influences significantly the lateral deformation of structural systems. For example, in multi - storey steel frames, 20 – 30% of the relative horizontal displacement between adjacent floors is caused by the deformability of the panel zone of beam - to - column connections.

v. System Properties

The lateral stiffness of a structure depends on the type of system utilized to withstand horizontal earthquake loads, the distribution of the member stiffness and the type of horizontal diaphragms connecting vertical members. For example, moment - resisting frames (MRFs) are generally more flexible than braced frames. The latter class includes concentrically (CBFs) and eccentrically (EBFs) braced frames. Structural walls are stiffer than all types of frames. Frames with rigid connections exhibit higher stiffness than those with semi - rigid connections. A vertical structural system for earthquake resistance is provided in section 2.5. It suffices here to state that uniform distribution of stiffness in plan and elevation is necessary to prevent localization of high seismic demand.

Other theoretical backgrounds of structural response like diaphragm rigidity - in section 2.4.3.4, story drift in section - 2.4.3.3, lateral force distribution in section - 2.4.3.5 and period in section - 2.4.3.6 are presented.

2.7. Method of analysis

The use of seismic analysis both in research and practice has increased substantially in recent years due to the proliferation of verified and user - friendly software and the availability of fast computers. The methods reviewed are grouped into static or dynamic methods, which are applied in elastic and inelastic response analysis. Dynamic analysis is the most natural approach towards the assessment of earthquake response, but is significantly more demanding than static analysis in terms of computational effort and interpretation of results[1].

Table 2.2: Comparisons of requirements for static and dynamic analyses

Properties	Static analysis	Dynamic analysis
Detailed models	✓	✓
Stiffness and strength representation	✓	✓
Mass representation	✗*	✓
Damping representation	✗	✓
Additional operators	✗	✓
Input motion	✗	✓
Target displacement	✓	✗
Action distribution fixed	✓*	✗
Short analysis time	✓	✗

Key: ✓ = Yes; ✗ = No; * = not necessarily for adaptive pushover analysis.

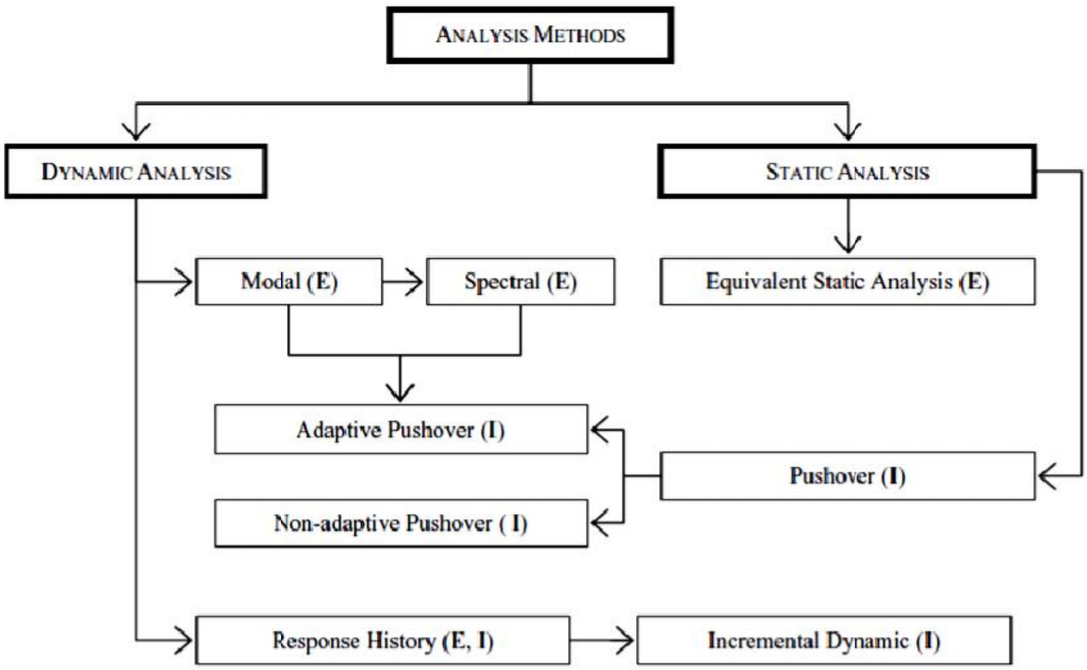


Figure 2.17: Common methods of structural analysis used in earthquake engineering

Key : E = elastic analysis; I = inelastic analysis

2.7.1. Dynamic analysis

The equation of equilibrium for a multi - degree of freedom (MDOF) system subjected to earthquake action is as follows[1]:

$F_I + F_D + F_R = F_E$ ………………………………………………………….. 2.6.1

Where

F_I - is the inertia force vector,

F_D - the damping force vector,

F_R - the vector of restoring forces and

F_E - the vector of earthquake loads.

Equation 2.6.1 may be expressed as:

$$\underline{M}x + \underline{C}x + F_R = -\underline{M}Ix_g \text{ ......} 2.6.2$$

Where the inertia, damping and earthquake forces are expressed, respectively, as:

$$F_I = \underline{M}x \text{ ......} 2.6.3$$

$$F_D = \underline{C}x \text{ ......} 2.6.4$$

$$F_E = -\underline{M}Ix_g \text{ ......} 2.6.5$$

In which M and C are the mass and damping matrices, x_g the acceleration of the ground, x is the vector of (absolute) accelerations of the masses and x is the vector of velocity relative to the base of the structure, respectively. I is a vector of influence coefficients, i.e. the i^{th} component represents the acceleration at the i^{th} degree of freedom due to a unit ground acceleration at the base. For simple structural models with degrees of freedom corresponding to the horizontal displacements at storey level, I is a unity vector. In this case, it represents the rigid body acceleration of the structure due to a unit base acceleration. The use of MDOF lumped systems for dynamic analyses results in a diagonal mass matrix M in which translational and rotational masses are located along the main diagonal. The use of consistent mass representations leads to a fully populated mass matrix. If the MDOF system behaves linearly, the vector of the restoring forces in equation 2.6.1 can be expressed as follows:

$$F_R = \underline{K}x \text{ ......} 2.6.6$$

In which k is the stiffness matrix and x the vector of displacements.

The matrix form of the dynamic equilibrium of motion given in equation 2.6.2 is identical to the equation of motion for single - degree of freedom (SDOF) systems given by equation 2.6.7. However, mass, damping and restoring forces (or stiffness for linearly elastic structures) for MDOF systems are expressed by matrices of coefficients representing the additional degrees of freedom.

$$m\ddot{u} + c\dot{u} + fu = m\ddot{u}_g \text{ ......} 2.6.7$$

Several methods of dynamic analysis of structures exist as shown in Figures 2.18. These methods can be employed either in the time or the frequency domain. The most commonly used methods for dynamic analysis of structures subjected to earthquake loads are modal, spectral and response history. Among these methods modal and spectral analysis are presented hereafter.

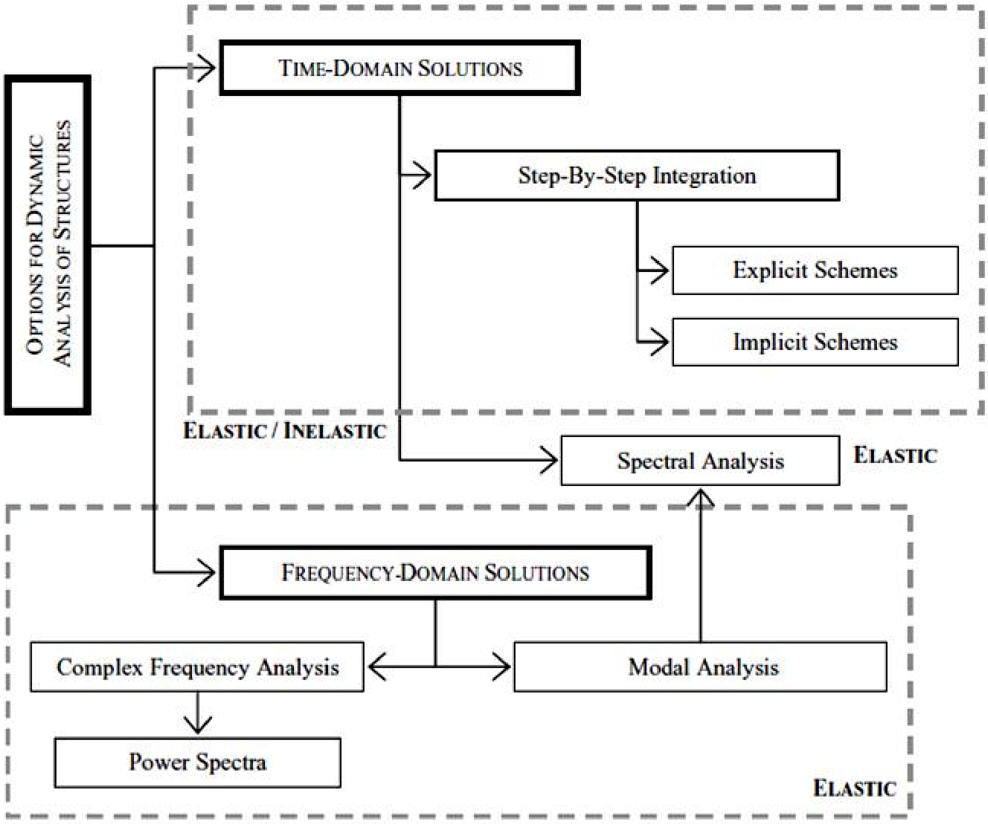


Figure 2.18: Common methods of structural analysis used in earthquake engineering

2.7.1.1. Modal and spectral analysis

The response of MDOF systems to a transient signal may be calculated by decomposing the system into series of SDOF systems, calculating the response of each in the time domain and then algebraically combining the response history to obtain the response of the MDOF system.

If the analysis is only focused on the maximum response quantities, then the various modal maxima are calculated under the effect of a response spectrum representing the transient signal, and the maxima are combined to give an upper bound of the maximum response of the MDOF. This is modal spectral analysis, or spectral analysis for short.

Both the above methods are applicable only to linear elastic systems, since they employ superposition. Modal analysis may be considered a time - domain solution, whereas it can be argued that modal - spectral analysis is a frequency - domain solution.

Two concepts are needed for the development of modal analysis. These are the principle of superposition and the convolution integral. Selection of earthquake spectra (input) and adequate combinations of modes are essential to perform modal spectral analysis. For a SDOF system, it can be shown that the displacement at time t is given by the solution of equation 2.6.7. The coupled equation of motion for MDOF structures given in matrix form in equation 2.6.2 can be rewritten for linearly elastic systems as follows[1]:

$$\underline{M}x + \underline{C}x + \underline{K}x = -\underline{M}Ix_g \quad \text{2.6.8}$$

2.7.2. Static analysis

Static methods are generally used to assess the capacity or ' supply ' of the structural system in terms of actions and deformations at different limit states or performance objectives. Static analysis may be viewed as a special case of dynamic analysis when damping and inertia effects are zero or negligible. The equation of static equilibrium for a lumped MDOF system can be derived from equation 2.6.1 by setting inertia F_I and damping F_D forces equal to zero, leading to equation 2.6.9:

$$R = F(t) \quad \text{2.6.9}$$

where

R - is the vector of restoring forces and

F (t) - the vector of the applied earthquake loads.

The most commonly used static analysis method in earthquake engineering is outlined below. Static methods can accommodate material inelasticity and geometric non - linearity. They, however, provide reliable results only for regular structural systems[1].

2.7.2.1. Equivalent static analysis

Equivalent static analysis (also referred to as equivalent lateral force, ELF method) is the simplest type of analysis that is used to assess the seismic response of structures. It is assumed that the behavior is linear elastic (which corresponds to material linearity), while geometrical non - linearity, i.e. second - order (P-Δ) effects, can be accounted for implicitly. The horizontal loads considered equivalent to the earthquake forces are applied along the height of the structure and are combined with vertical (gravity) loads. Methods of structural analysis are used to solve the equilibrium equations for a MDOF system, e.g. equation 2.6.2 in which the vector of restoring forces can be assumed proportional to the vector of nodal displacements of the structure.

The critical issue in equation 2.6.2 is often the load magnitude and distribution. With regard to magnitude, the elastic forces are obtained from the mass of the structure and its predominant period of vibration, and the earthquake spectrum is scaled by a response modification factor. This factor is supposed to represent the ability of the structure to absorb energy by inelastic deformation and damage. With regard to load distribution, the most common is a code – type pattern corresponding to the predominant (usually fundamental) mode of vibration. For buildings, inverted triangular or parabolic load patterns are often used, depending on the period of the building. The magnitude of the force at each storey level is also calculated from the predominant mode shape. A triangular distribution provides a good approximation of horizontal forces for structures, which vibrate predominantly in the first mode[1].

2.7.2.2. Second order P-delta effects

Typically, design codes require that second order P-Delta effects be considered when designing concrete frames. These effects are the global lateral translation of the frame and the local deformation of members within the frame.

Consider the frame object shown in Figure 2.11, which is extracted from a story level of a larger structure. The overall global translation of this frame object is indicated by Δ. The local deformation of the member is shown as δ. The total second order P-Delta effects on this frame object are those caused by both Δ and δ .

ETABS program has an option to consider P-Delta effects in the analysis. When P-Delta effects are considered in the analysis, the program does a good job of capturing the effect due to the Δ deformation shown in Figure 2.11, but it does not typically capture the effect of the δ deformation (unless, in the model, the frame object is broken into multiple elements over its length)[13].

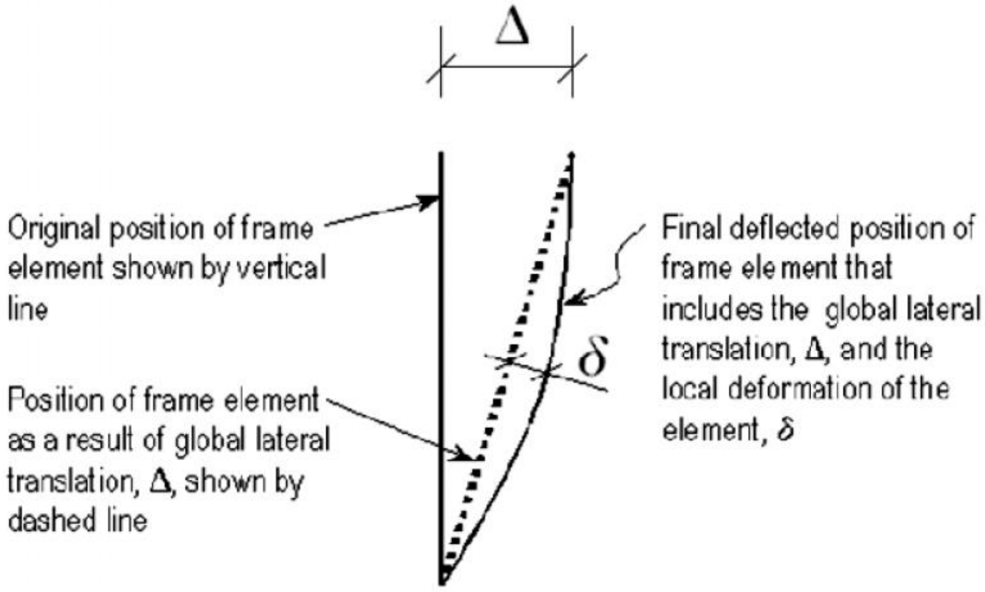


Figure 2.19: The total second order P-delta effects on a frame element caused by both Δ and δ

2.7.3. Elastic second-order analysis

The stiffnesses EI used in an analysis for strength design should represent the stiffnesses of the members immediately prior to failure. This is particularly true for a second-order analysis that should predict the lateral deflections at loads approaching ultimate. The EI values should not be based totally on the moment-curvature relationship for the most highly loaded section along the length of each member. Instead, they should correspond to the moment-end rotation relationship for a complete member.

Elastic second-order analysis consider section properties determined taking into account the influence of axial loads, the presence of cracked regions along the length of the member, and the effects of load duration[7].

ACI recommends the following properties for the members in the structure:

Table 2.3: ACI recommendation of stiffness modifiers for elastic second-order analysis

Compression members	Columns		$0.70Ig$
	Walls	Uncracked	$0.70Ig$
		Cracked	$0.35Ig$
Flexural members	Beams		$0.35Ig$
	Flat plates and flat slabs		$0.25Ig$

Where

Ec - Modulus of elasticity concrete

Ig - Moment of inertia of gross concrete section

2.8. ETABS software

ETABS (Extended Three dimensional Analysis of Building System) is a sophisticated, yet easy to use, special purpose analysis and design program developed specifically for building systems. ETABS offers the widest assortment of analysis and design tools available for the structural engineer working on building structures. The following list represents just a portion of the types of systems and analyses that ETABS can handle easily:

- Buildings with steel, concrete, composite or joist floor framing
- Complex shear walls with arbitrary openings
- Flat and waffle slab concrete buildings
- Buildings subjected to any number of vertical and lateral load cases and combinations, including automated wind and seismic loads
- Floor modeling with rigid or semi-rigid diaphragms

- Rigid and semi rigid options affect only the analysis of the model. If the rigid option is selected, a fully rigid diaphragm is assumed, which causes all of its constrained joints to move together as a planar diaphragm that is rigid against membrane deformation. Effectively, all constrained joints are connected to each other by links that are rigid in the plane but do not affect out-of-plane (plate) deformation. If the semi rigid option is selected, the in-plane rigidity of the diaphragm comes from the stiffness of the objects that are part of the diaphragm. Semi rigid option gives the building the ability to behave as its actual behaviour[13].

2.9. Finite element modeling

The finite element method was introduced in the early 1960s by scientists lik Argyris, Clough and Zienkiewicz. Since then the method has been developed to be one of the most powerful methods to solve engineering problems. Finite element method is based on matrix algebra, and its efficiency depends directly on the performance of the computer. Nowadays, when powerful computers are available, new methods of non-linear analysis are being developed. The finite element method is today widely used especially in mechanical and civil engineering.

Both analytical and finite element solutions are based on the governing differential equations. The largest difficulty using analytical methods is to find a function that fulfils the differential equation and the boundary condition over the entire body. Therefore, analytical methods are limited to solving simple problems. Finite element method is an approximation method that leads to division a structure into finite number of connected elements. The set of elements is called finite element mesh and loads and boundary conditions are in the form of concentrated forces in element nodes. The accuracy of finite element analysis depends on size of elements and the order of so-called shape function. The latter is in most cases either linear or quadratic[5].

The fundamental equation of equilibrium of the finite element method has the form of a system of linear equations:

$$[K]\{a\} = \{f\}$$

where

K - is global stiffness matrix

a - is nodal displacement vector

f - is force vector

2.10. Damage related to diaphragm

In a number of buildings, there has been evidence of roof diaphragms, which is caused by tearing of the diaphragm. The following figure shows failure resulting from diaphragm flexibility in Loma Prieta earthquake, 1989 (EERI, 1990), Figure 2.20[18].

Similarly, damage related to diaphragm response and behavior was observed in concrete structures following the 1994 Northridge earthquake, primarily for precast construction. A department store in the Northridge Fashion Center experienced damage to the roof diaphragm (concrete fill over metal deck supported on steel beams) and to the floor diaphragms (topping slab over precast elements) (Hamburger 1996)[6].

The Santa Monica College precast concrete parking structure experienced chord failure in the diaphragm (Phillips 1996). Precast concrete parking structures at the Northridge Fashion Center had diaphragm movement large enough to cause failure in some of the interior frame columns designed for gravity loads and the Glendale Civic Center garage (cast-in place, post-tensioned construction) had collector failures between the topping slab and a shear wall (Corley et al. 1996) [6].

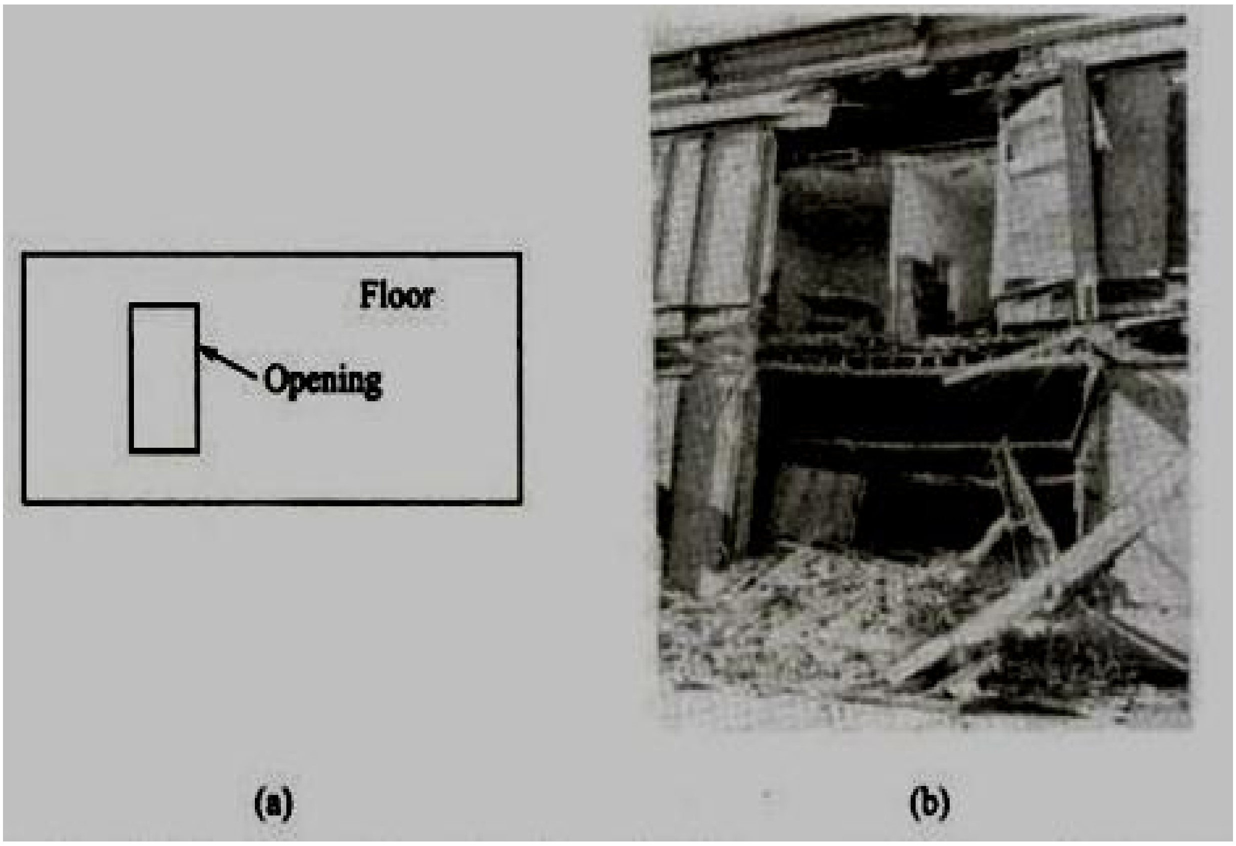


Figure 2.20: (a) Diaphragm discontinuity; (b) Failure resulting from diaphragm flexibility in Loma Prieta earthquake, 1989 (EERI, 1990)

Chapter Three

3. Parametric Study

3.1. General

In this chapter, parametric studies of different cases are presented based on the theoretical background acquired from previous chapters; to demonstrate the effect of diaphragm discontinuity on the diaphragm rigidity and lateral load distribution to vertical element.

3.2. Description of the parametric study

Parametric studies are carried out for eight different cases, which are believed to verify response of a structure with diaphragm discontinuity/opening. The parameters are story height, shear wall width, number of stories, number of bays, shape of diaphragm opening, size of diaphragm opening, span length and opening location in stories. In each parametric study, four responses of structures; diaphragm rigidity, story drift, lateral force distribution to vertical element and natural vibration period are evaluated. The response of story drift and natural vibration period are not as such significant. Therefore, analysis results of diaphragm rigidity and lateral force distribution to vertical element are presented and discussed.

The structures are analyzed for two key assumptions; actual diaphragm rigidity stiffness (without diaphragm, WOD) and rigid diaphragm assumption (with diaphragm, WD). WOD and WD terms are the method diaphragm analysis used to get actual diaphragm stiffness and rigid diaphragm assumption using ETABS software according to section 2.8.

Analyses are carried out for 123 different model structures. Since the analyses are for two major assumptions, a total of 246 structures are analyzed. From these structures representative 35 /for two assumptions 70/ model structures are chosen and discussed in detail.

3.3. Modeling

3.3.1. Description of structures

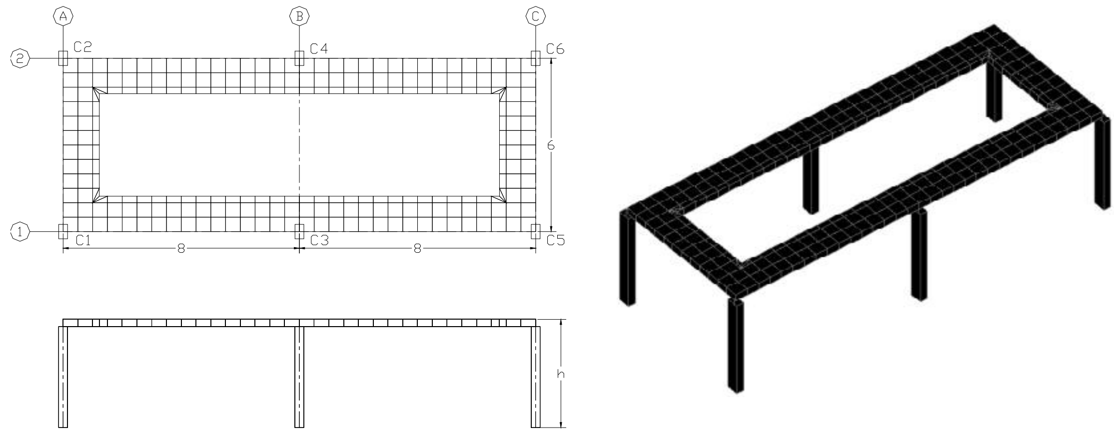
A rectangular floor plan is selected for the parametric study. This floor plan shape was chosen intentionally so that the selected case study structures can provide a critical structural layout for evaluating diaphragm discontinuity/opening for actual diaphragm stiffness and rigid diaphragm assumption.

Detailed descriptions of the structures are summarized in Table 3.1 below, which consists structure codes, number of stories, number of bays, story height, opening size, shear wall width and aspect ratio. From 123 model structures analyzed, 35 typical structures, which are chosen for the discussion are tabulated below. The remaining structures are documented in Appendix Table app1. Sample drawings of floor plan, section and 3D of three structures are shown in Figure 3.1.

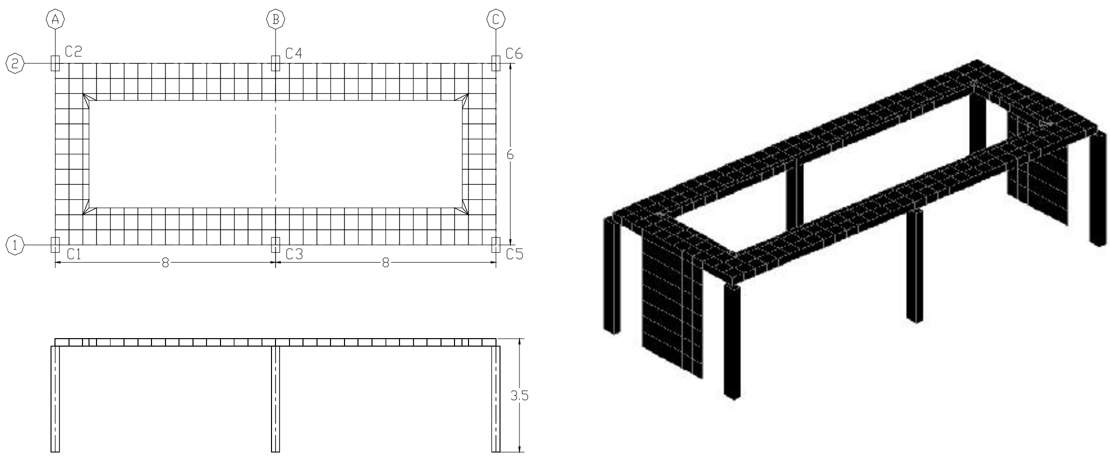
Table 3.1: Structures for parametric study

Structure type - code	No of stories	No of bays	Story height, h (m)	Opening size (%)	Shear wall width, w (m)	Aspect ratio (LxW)	Remark
S1B2H25O50W2	1	2	2.5	50	2	2.66	Double bay
S1B2H45O50W2	1	2	4.5	50	2	2.66	"
S1B2H55O50W2	1	2	5.5	50	2	2.66	"
S1B2H65O50W2	1	2	6.5	50	2	2.66	"
S1B2H75O50W2	1	2	7.5	50	2	2.66	"
S1B2H85O50W2	1	2	8.5	50	2	2.66	"
S1B2H35O50	1	2	3.5	50	-	2.66	Without shear wall
S1B2H35O50W1	1	2	3.5	50	1	2.66	Double bay
S1B2H35O50W1.5	1	2	3.5	50	1.5	2.66	"
S1B2H35O50W2	1	2	3.5	50	2	2.66	"
S1B2H35O50W3	1	2	3.5	50	3	2.66	"
S1B2H35O50W4	1	2	3.5	50	4	2.66	"
S1B2H35O50W5	1	2	3.5	50	5	2.66	"
S1B2H35O50W6	1	2	3.5	50	6	2.66	"
S2B2H35O50W3	2	2	3.5	50	3	2.66	Same height
S3B2H35O50W3	3	2	3.5	50	3	2.66	"

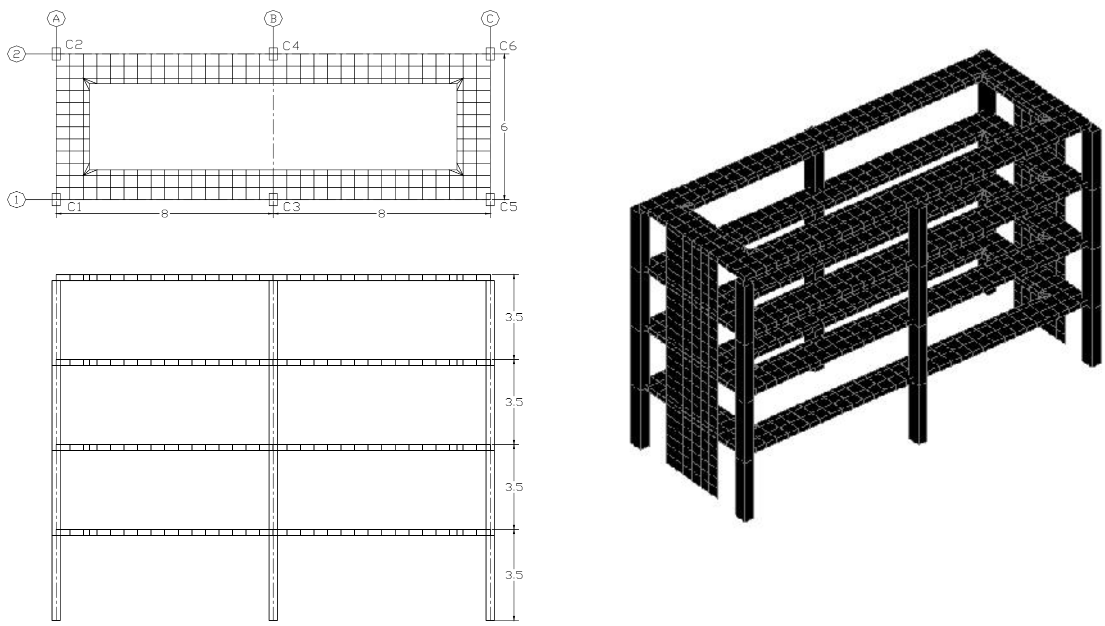
Structure type - code	No of stories	No of bays	Story height, h (m)	Opening size (%)	Shear wall width, w (m)	Aspect ratio (LxW)	Remark
S4B2H35O50W3	4	2	3.5	50	3	2.66	"
S5B2H35O50W3	5	2	3.5	50	3	2.66	"
S6B2H35O50W3	6	2	3.5	50	3	2.66	"
S1B1H35O50W1.5	1	1	3.5	50	1.5	1.33	Single bay
S1B3H35O50W1.5	1	3	3.5	50	1.5	4.00	Three bays
S1B4H35O50W1.5	1	4	3.5	50	1.5	5.33	Four bays
S1B2H35O50W0.5R	1	2	3.5	50	0.5	2.66	Rectang ular
S1B2H35O50W0.5C	1	2	3.5	50	0.5	2.66	With corridor at center
S1B2H35O50W0.5IR	1	2	3.5	50	0.5	2.66	Irregular opening shape
S1B2H35O20W1.25	1	2	3.5	20	1.25	2.66	20% opening
S1B2H35O30W1.25	1	2	3.5	30	1.25	2.66	30% opening
S1B2H35O40W1.25	1	2	3.5	40	1.25	2.66	40% opening
S1B2H35O60W1.25	1	2	3.5	60	1.25	2.66	60% opening
S1B2H35O50W2-6x6	1	2	3.5	50	2	1.00	2@3m span, x-direc.
S1B2H35O50W2-8x6	1	2	3.5	50	2	1.33	2@4m span, x-direc.
S1B2H35O50W2-10x6	1	2	3.5	50	2	1.66	2@5m span, x-direc.
S1B2H35O50W2-12x6	1	2	3.5	50	2	2.00	2@6m span, x-direc.
S1B2H35O50W2-14x6	1	2	3.5	50	2	2.33	2@7m span, x-direc.
S6B2H35O50-1FW3	6	2	3.5	50	3	2.66	Opening at 1st floor
S6B2H35O50-3FW3	6	2	3.5	50	3	2.66	Opening at 3rd floor
S6B2H35O50-6FW3	6	2	3.5	50	3	2.66	Opening at 6th floor



(a)



(b)



(c)

Figure 3.1: Sample drawings for S1B2H35O50 (a), S1B2H35O50W3 (b) and S4B2H35O50W3 (c) structures

3.3.2. Material properties

Normal-weight concrete with a characteristic cube compressive strength of 25MPa and characteristic yield strength of 400MPa for reinforcement is used for all members according to EBCS-1[14].

A horizontal diaphragm of flat plate having a thickness of 250mm, which satisfy a serviceability requirement according to EBCS-2[15] is modeled by finite-element method using ETABS software.

3.3.3. Vertical element system

Main vertical element systems used for the parametric study are classified into two; moment-resisting frame system, which consists only column as vertical element and hybrid frame system (dual system), which consists of columns and shear walls as stated in section 2.5.2. The end shear wall is used to make the dual system, which is located in the center of the shorter span of the rectangular floor plan. Based on the parametric study cases, one of the above two vertical element systems is used at a time.

3.3.4. Loading

Loading on the structures consists of uniform live load of 5kN/m^2 and imposed dead load of 3kN/m^2. Permanent dead load of the structure is computed by the software using unit weight of the concrete.

Lateral earthquake force for a bed rock acceleration ratio of 0.3g applied to the structure. It is computed by equivalent static analysis method according to Ethiopian Building Code Standard, EBCS-8[16].

A lateral load equal to the weight of the building is applied to the building in accordance with the weight distribution, and the average inter-story drift, Δ_{av}, diaphragm deformation, Δ_{diaph}, and shear force distribution to vertical element is computed.

3.4. Analysis

Analyses are carried out using ETABS software, which is discussed in section 2.8. A total of 123 model structures are analyzed having various variables. Typical 35 structures are selected and discussed below. Results for each case are presented below in tabular and graphical format according to its category. The results of both diaphragm rigidity and lateral load distribution to vertical element are recorded for the worst seismic force direction of the floor plan. From section 2.4.3.4; EBCS 8 and FEMA 273 diaphragm rigidity classification, FEMA 273 recommendation is selected due to its detail classification. In the analysis elastic second-order effect is considered according to section 2.7.3.

3.5. Parametric studies of cases and discussions

Analysis result and discussion of typical 35 structures that are categorized into eight parametric studies are presented hereafter. Table 3.2 shows lists of the parameters of each case.

Table 3.2: Lists of parameters in eight cases

Cases	Parametric study
Case – 1	Story height as a parameter
Case – 2	Shear wall width as a parameter
Case – 3	Number of stories as a parameter
Case – 4	Number of bays as a parameter
Case – 5	Shape of opening as a parameter
Case – 6	Size of opening as a parameter
Case – 7	Span length as a parameter
Case – 8	Opening location in story as a parameter

3.5.1. Case – 1: Story height as a parameter

The first parametric study is carried out by considering the variation of story height as a variable making the other constant. This is done by taking seven structures having different story height. The structures are labeled by codes; S1B2H25O50W2, S1B2H35O50W2, S1B2H45O50W2, S1B2H55O50W2, S1B2H65O50W2, S1B2H75O50W2 and S1B2H85O50W2 as shown in Table 3.1. These structures have the same floor plan configuration with 50 percent opening size. Vertical element systems are dual systems (hybrid/wall frame), which have

300mmx500mm column size and 200mm thick and 2m wide shear wall. The only difference is story height. The structures have 2.5m, 3.5m, 4.5m, 5.5m, 6.5m, 7.5m and 8.5m height respectively. The analysis results for each response shown as below in a tabular and graphical format and discussion follows.

i. Case – 1, Response – 1: Diaphragm rigidity

Table 3.3: Case – 1, Diaphragm rigidity summary for actual diaphragm stiffness

Structure type - Code	Height (m)	Δdiaph (mm)	Δav (mm)	$\Delta drr = \frac{\Delta diaph}{\Delta av}$	FEMA 273 classification
S1B2H25O50W2	2.5	1.635	0.210	7.79	Flexible
S1B2H35O50W2	3.5	2.379	0.599	3.97	Flexible
S1B2H45O50W2	4.5	2.798	1.319	2.12	Flexible
S1B2H55O50W2	5.5	3.042	2.464	1.23	Stiff
S1B2H65O50W2	6.5	3.202	4.139	0.77	Stiff
S1B2H75O50W2	7.5	3.320	6.454	0.51	Stiff
S1B2H85O50W2	8.5	3.416	9.529	0.36	Rigid

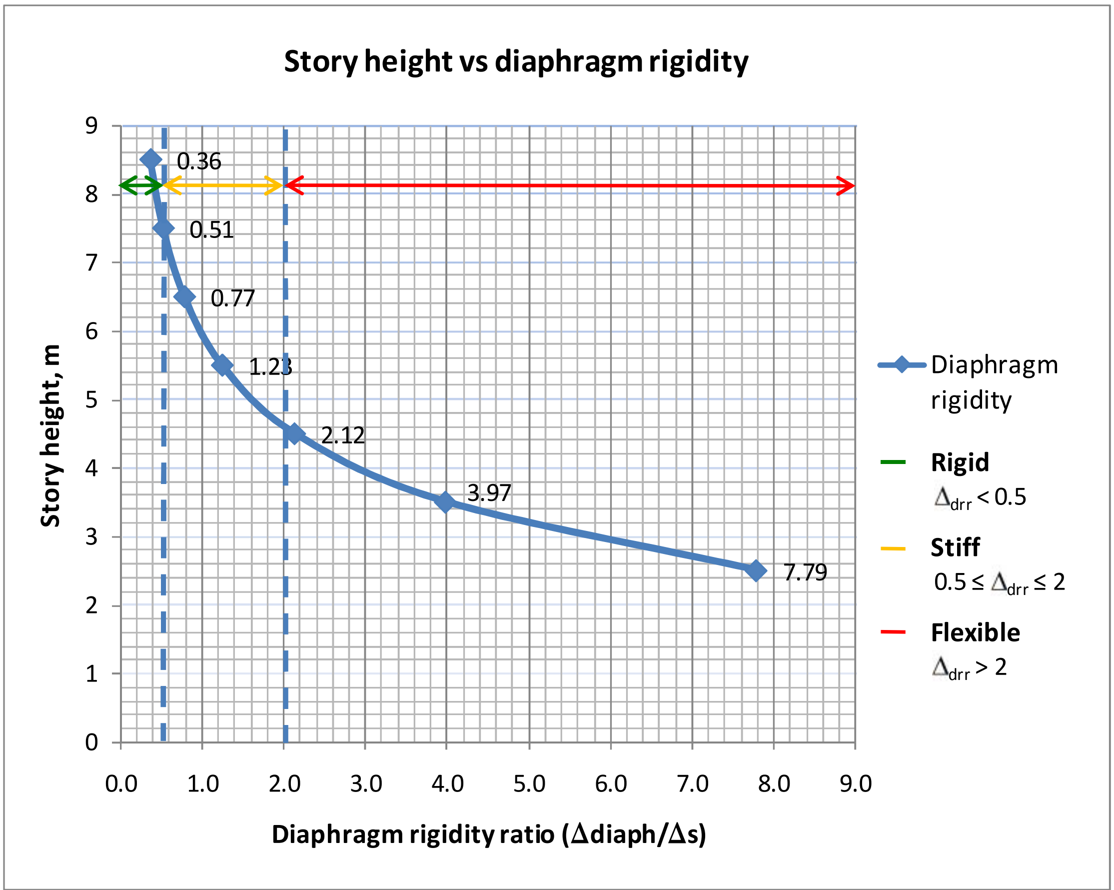


Figure 3.2: Case – 1, Story height versus diaphragm rigidity

ii. Case – 1, Response – 2: Lateral force distribution to vertical element

Table 3.4: Case – 1, Shear force distribution in column for WOD and WD assumption

Structure type - Code	Height	Column, C1			Column, C2		
		wod	wd	Δ(%)	wod	wd	Δ(%)
S1B2H25O50W2	2.5	3.23	4.51	**-39.6**	3.23	4.51	**-39.6**
S1B2H35O50W2	3.5	3.70	4.42	**-19.5**	3.70	4.42	**-19.5**
S1B2H45O50W2	4.5	4.15	4.57	**-10.1**	4.15	4.57	**-10.1**
S1B2H55O50W2	5.5	4.54	4.80	**-5.7**	4.54	4.80	**-5.7**
S1B2H65O50W2	6.5	4.89	5.07	**-3.7**	4.89	5.07	**-3.7**
S1B2H75O50W2	7.5	5.23	5.35	**-2.3**	5.23	5.35	**-2.3**
S1B2H85O50W2	8.5	5.55	5.64	**-1.6**	5.55	5.64	**-1.6**

Column, C3			Column, C4			Column, C5		
wod	wd	Δ(%)	wod	wd	Δ(%)	wod	wd	Δ(%)
24.41	4.03	**83.5**	24.41	4.03	**83.5**	3.23	4.51	**-39.6**
15.06	3.79	**74.8**	15.06	3.79	**74.8**	3.70	4.42	**-19.5**
10.21	3.77	**63.1**	10.21	3.77	**63.1**	4.15	4.57	**-10.1**
7.77	3.84	**50.6**	7.77	3.84	**50.6**	4.54	4.80	**-5.7**
6.50	3.94	**39.4**	6.50	3.94	**39.4**	4.89	5.07	**-3.7**
5.81	4.06	**30.1**	5.81	4.06	**30.1**	5.23	5.35	**-2.3**
5.42	4.17	**23.1**	5.42	4.17	**23.1**	5.55	5.64	**-1.6**

Column, C6		
wod	wd	Δ(%)
3.23	4.51	**-39.6**
3.70	4.42	**-19.5**
4.15	4.57	**-10.1**
4.54	4.80	**-5.7**
4.89	5.07	**-3.7**
5.23	5.35	**-2.3**
5.55	5.64	**-1.6**

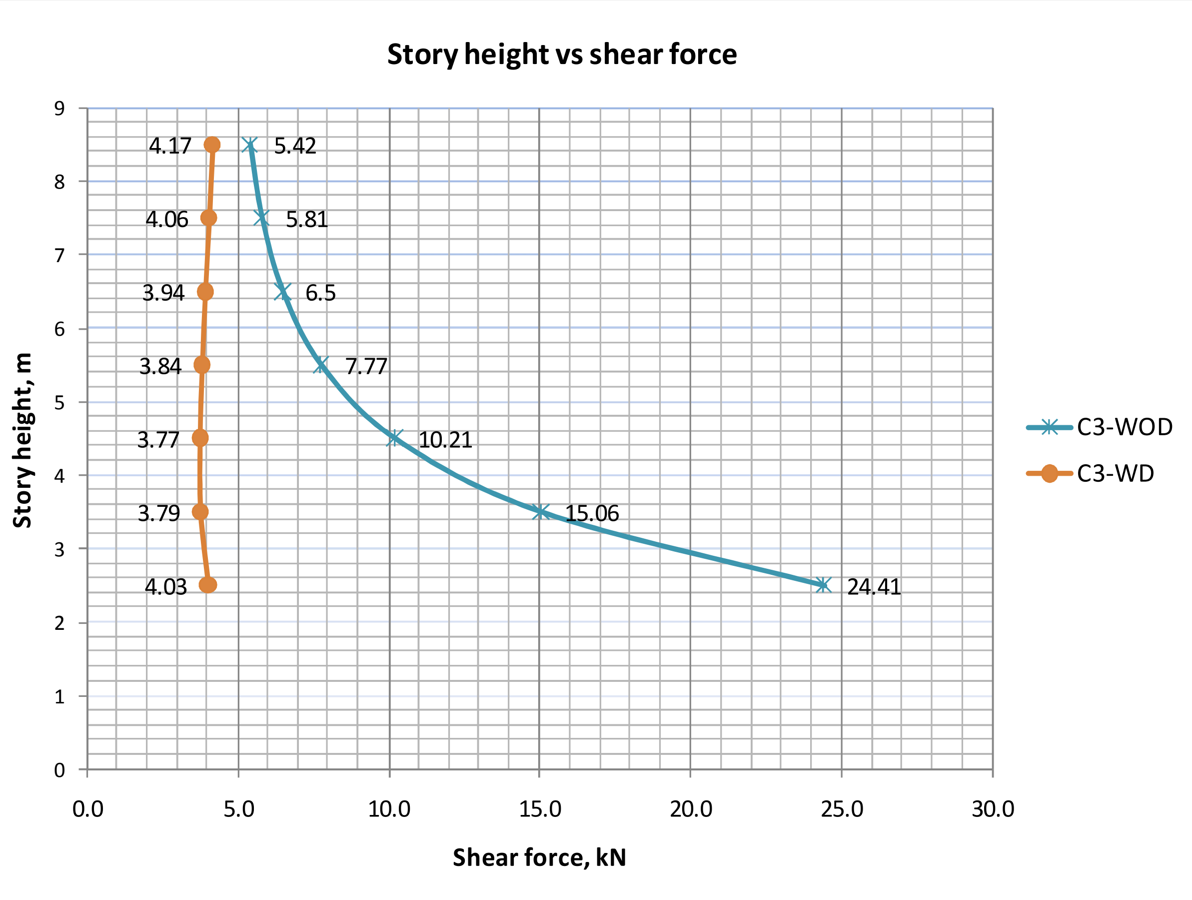


Figure 3.3: Case – 1, Story height versus shear force distribution for WOD and WD assumption

Discussion on case – 1 results */Story height as a parameter/*

The discussion of the above two responses; diaphragm rigidity and lateral force distribution to vertical element of the first parametric study are presented hereafter.

Analyses results shown in a tabular and graphical format above indicate diaphragm discontinuity resulted in significant floor diaphragm rigidity difference between actual diaphragm stiffness and rigid diaphragm assumption. As observed from the graph, decreasing story height reduces the diaphragm rigidity. A 2.5m, 3.5m and a 4.5m height floor has a flexible diaphragm; 5.5m, 6.5m and 7.5m height floor has a stiff diaphragm; and 8.5m height floor has a rigid floor. This is due to change in stiffness of a vertical element system, which is resulted from the story height variation.

The relationship between vertical element height and stiffness is inverse; as the height reduced the vertical element gets stiffer. Stiffness of a vertical element means resistance to deformation. The larger the stiffness, the larger is the force required to deform it. For example; a short column is stiffer as compared to a tall column, and it attracts larger earthquake force. From this concept when the vertical element gets stiffer the diaphragm tendency to attract force reduced, therefore, it easily deforms and gets flexible.

Lateral force distribution to vertical element is the second response of this parametric study. The analysis result of actual and rigid diaphragm assumption shows a diaphragm discontinuity result in substantial change between the two analyses, especially in the intermediate column, since the floor diaphragm experiences larger in-plane diaphragm deformation in the middle due to the diaphragm discontinuity. The change is more significant when the height is reduced, which result flexible diaphragm according to response one discussion. In shear force distribution to vertical element, a maximum of 83.5 percent difference is observed for a flexible 2.5m high floor; and a minimum of 23.1 percent difference is observed for a rigid 8.5m high floor diaphragm in actual diaphragm stiffness and rigid diaphragm assumption. As discussed above this is mainly because when height reduced the vertical element gets stiffer; the larger the stiffness, the larger is the force required to deform it; therefore, it attracts larger earthquake force. In addition, flexible nature of the diaphragm that is resulted from the diaphragm discontinuity with height reduction, gave tributary area force distribution that escalates the change in force distribution between actual diaphragm stiffness and rigid diaphragm assumption.

3.5.2. Case – 2: Shear wall width as a parameter

The second parametric study is carried out by considering the variation of a vertical element system as a parameter making the other constant. This is done by taking structures having different shear wall width. The structures are labeled by codes; S1B2H35O50, S1B2H35O50W1, S1B2H35O50W1.5, S1B2H35O50W2, S1B2H35O50W3 and S1B2H35O50W4, as shown in Table 3.1. These structures have the same floor plan configuration with 50 percent diaphragm opening size and 300mmx500mm columns size. The difference between the structures is shear wall width. They have a shear wall width of 0.0m (without shear wall), 1.0m, 1.5m, 2.0m, 3.0m and 4.0m respectively. The transverse shear walls located in the end frames provided a layout that maximizes the in-plane deformation of the diaphragms because the interior frames are flexible relative to the stiffer end frames with shear walls.

Analysis results are shown as below in a tabular and graphical format and discussion follows.

i. Case – 2, Response – 1: Diaphragm rigidity

Table 3.5: Case – 2, Diaphragm rigidity summary for actual diaphragm stiffness

Structure type - Code	Shear wall width, (m)	Δdiaph (mm)	Δav (mm)	$\Delta drr = \frac{\Delta diaph}{\Delta av}$	FEMA 273 classification
S1B2H35O50	0	0.683	8.151	0.08	Rigid
S1B2H35O50W1	1	1.902	2.600	0.73	Stiff
S1B2H35O50W1.5	1.5	2.243	1.176	1.91	Stiff
S1B2H35O50W2	2	2.379	0.599	3.97	Flexible
S1B2H35O50W3	3	2.463	0.218	11.29	Flexible
S1B2H35O50W4	4	2.464	0.114	21.54	Flexible

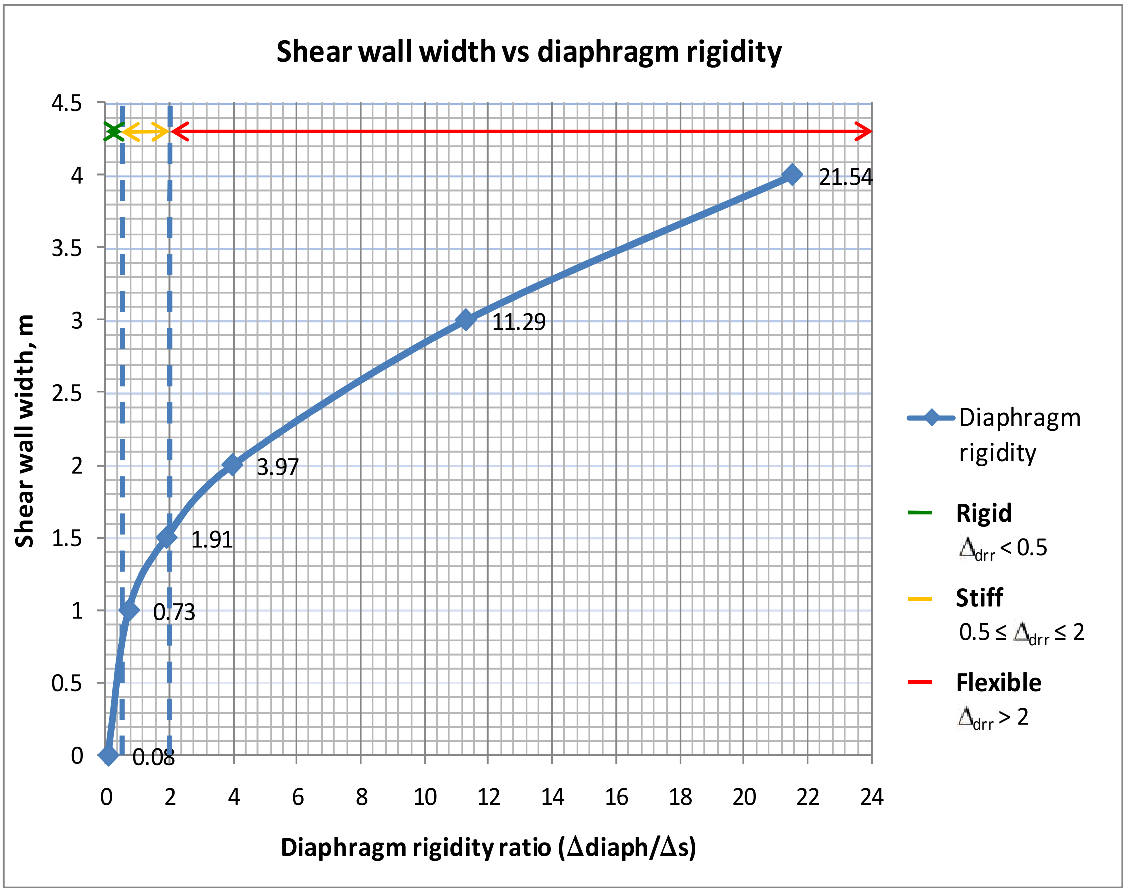


Figure 3.4: Case – 2, Shear wall width versus diaphragm rigidity

ii. Case – 2, Response – 2: Lateral force distribution to vertical element

Table 3.6: Case – 2, Shear force distribution in column for WOD and WD assumption

Structure type - Code	Shear wall width	Column, C1			Column, C2		
		wod	wd	Δ(%)	wod	wd	Δ(%)
S1B2H35O50	0	41.63	42.79	**-2.8**	41.63	42.79	**-2.8**
S1B2H35O50W1	1	14.92	16.47	**-10.4**	14.92	16.47	**-10.4**
S1B2H35O50W1.5	1.5	7.00	8.02	**-14.6**	7.00	8.02	**-14.6**
S1B2H35O50W2	2	3.70	4.42	**-19.5**	3.70	4.42	**-19.5**
S1B2H35O50W3	3	1.49	1.94	**-30.2**	1.49	1.94	**-30.2**
S1B2H35O50W4	4	0.90	1.22	**-35.6**	0.90	1.22	**-35.6**

Column, C3			Column, C4			Column, C5		
wod	wd	Δ(%)	wod	wd	Δ(%)	wod	wd	Δ(%)
43.38	41.06	**5.3**	43.38	41.06	**5.3**	41.63	42.79	**-2.8**
22.87	14.75	**35.5**	22.87	14.75	**35.5**	14.92	16.47	**-10.4**
17.34	7.05	**59.3**	17.34	7.05	**59.3**	7.00	8.02	**-14.6**
15.06	3.79	**74.8**	15.06	3.79	**74.8**	3.70	4.42	**-19.5**
13.50	1.55	**88.5**	13.50	1.55	**88.5**	1.49	1.94	**-30.2**
12.96	0.86	**93.4**	12.96	0.86	**93.4**	0.90	1.22	**-35.6**

Column, C6		
wod	wd	Δ(%)
41.63	42.79	**-2.8**
14.92	16.47	**-10.4**
7.00	8.02	**-14.6**
3.70	4.42	**-19.5**
1.49	1.94	**-30.2**
0.90	1.22	**-35.6**

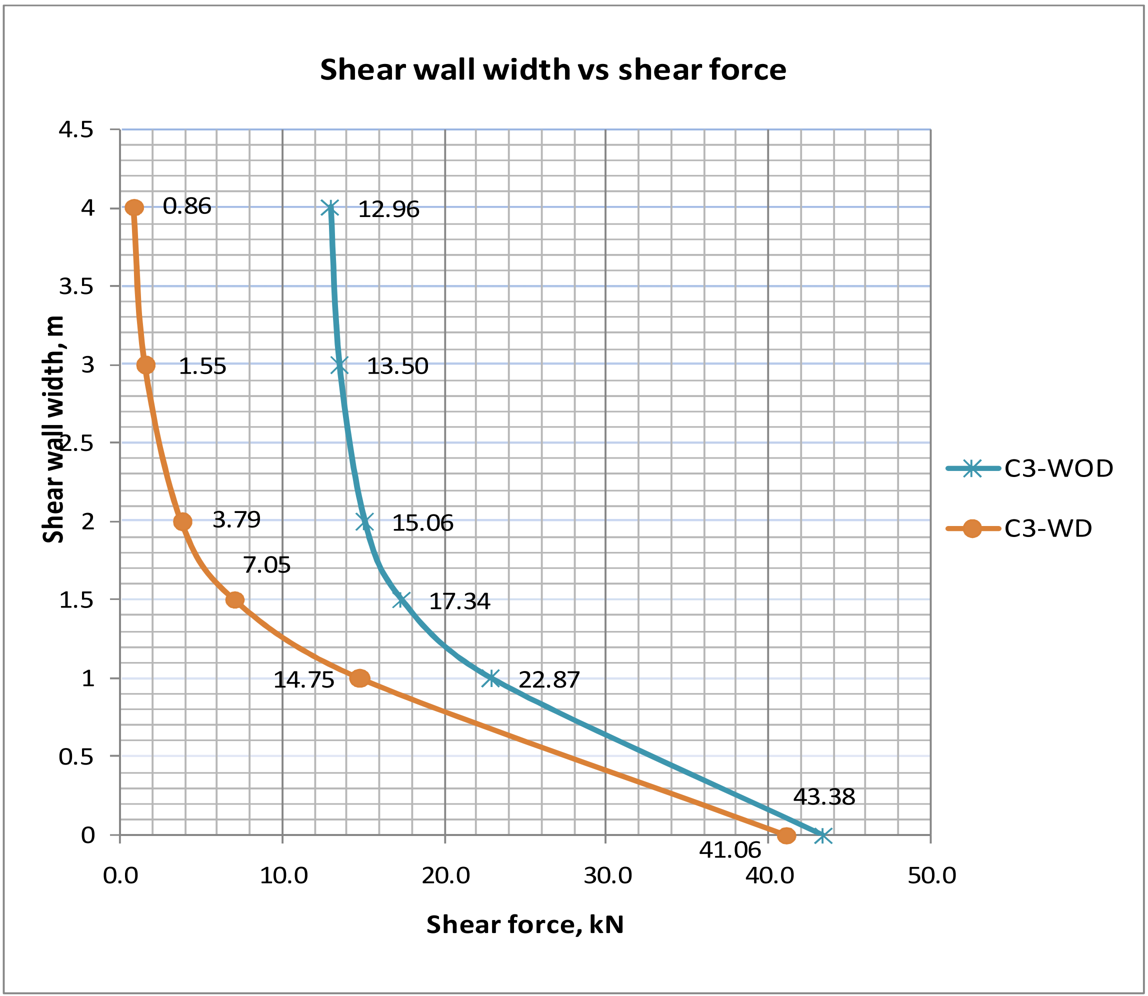


Figure 3.5: Case – 2, Shear wall width versus shear force distribution for WOD and WD assumption

Discussion on case – 2 results */Shear wall width as a parameter/*

In case – 2 parametric study, from the observation of the table and the graph, diaphragm discontinuity became a serious concern; in diaphragm rigidity and lateral force distribution to vertical element; in actual diaphragm stiffness and rigid diaphragm assumption as discussed below.

The first discussion is on the diaphragm rigidity of the floor. Observation from the table and the graph indicates that the change in dimension of the width of a shear wall plays a major role on the relative rigidity of a diaphragm. 0, 1, 1.5, 2, 3 and 4 meter dimensions shear wall width yields, rigid for 0 meter, stiff for 1 and 1.5 meter, flexible for 2, 3 and 4 meter according to FEMA diaphragm classification. The major reason for this diaphragm rigidity variation is, as the shear wall width dimension increases the vertical lateral load resisting (VLLR) element (shear wall) gets stiffer. As discussed in case – 1 parametric study the stiffer the VLLR element, the larger is the force required to deform it; therefore, it attracts larger earthquake force. Therefore, when VLLR element attracts more forces, the diaphragm tendency of force attraction reduced, which resulted changed in its rigidity behavior.

The second discussion of this parametric study is the distribution of shear force to vertical element. Change in dimension of shear wall width seriously affects the distribution of the lateral force. The table shows more than 50 percent difference is seen between actual diaphragm stiffness and rigid diaphragm assumption for intermediate columns for a shear wall width of 1.5m and above. The difference rises up to 93.4 percent for a shear wall width of 4m. The basic reason for this variation is the change in rigidity of a floor diaphragm that is influenced by diaphragm discontinuity.

The effects of diaphragm discontinuity influenced by stiffness of the vertical element that can result relatively rigid, stiff or flexible diaphragm. In a condition where the diaphragm rigidity is affected by flexibility, the variation in force distribution became large and significant as shown in the table. As discussed in detail in case – 1 parametric study the flexibility of the diaphragm resulted in tributary area load distribution in a floor. Therefore, the effect of diaphragm discontinuity on lateral load distribution to vertical element when the stiffness of the vertical elements varies, become a serious issue.

3.5.3. Case – 3: Number of stories as a parameter

In case 3 number of variations of the stories are taken as a parameter. The structures have a dual systems (hybrid/wall frame) vertical element, with column size 300mmx500mm and a shear wall having 200mm thick and 3m width. The structures are labeled by codes; S1B2H35O50W3, S2B2H35O50W3, S3B2H35O50W3, S4B2H35O50W3, S5B2H35O50W3 and S6B2H35O50W3. All structures have the same floor plan and vertical element configuration and 50 percent diaphragm opening size. The difference is number of stories. The structures have 1-story, 2-story, 3-story, 4-story, 5-story and 6-story floors respectively. Results are shown as below in a table and graphical format.

i. Case – 3, Response – 1: Diaphragm rigidity

Table 3.7: Case – 3, Diaphragm rigidity summary for actual diaphragm stiffness

Structure type - Code	Story	Δdiaph (mm)	Δav (mm)	$\Delta drr = \frac{\Delta diaph}{\Delta av}$	FEMA 273 classification
S1B2H35O50W3	1	2.463	0.218	11.29	Flexible
S2B2H35O50W3	2	4.448	1.586	2.80	Flexible
	1	1.800	0.957	1.88	Stiff
S3B2H35O50W3	3	5.064	3.809	1.33	Stiff
	2	3.495	3.406	1.03	Stiff
	1	1.402	1.685	0.83	Stiff
S4B2H35O50W3	4	5.320	8.474	0.63	Stiff
	3	4.334	8.416	0.52	Stiff
	2	2.865	6.835	0.42	Rigid
	1	1.053	3.175	0.33	Rigid
S5B2H35O50W3	5	5.446	12.386	0.44	Rigid
	4	4.834	12.666	0.38	Rigid
	3	3.718	11.680	0.32	Rigid
	2	2.351	8.984	0.26	Rigid
	1	0.829	3.957	0.21	Rigid
S6B2H35O50W3	6	5.512	18.141	0.30	Rigid
	5	5.201	18.852	0.28	Rigid
	4	4.268	18.451	0.23	Rigid
	3	3.150	16.496	0.19	Rigid
	2	1.970	12.290	0.16	Rigid
	1	0.640	5.263	0.12	Rigid

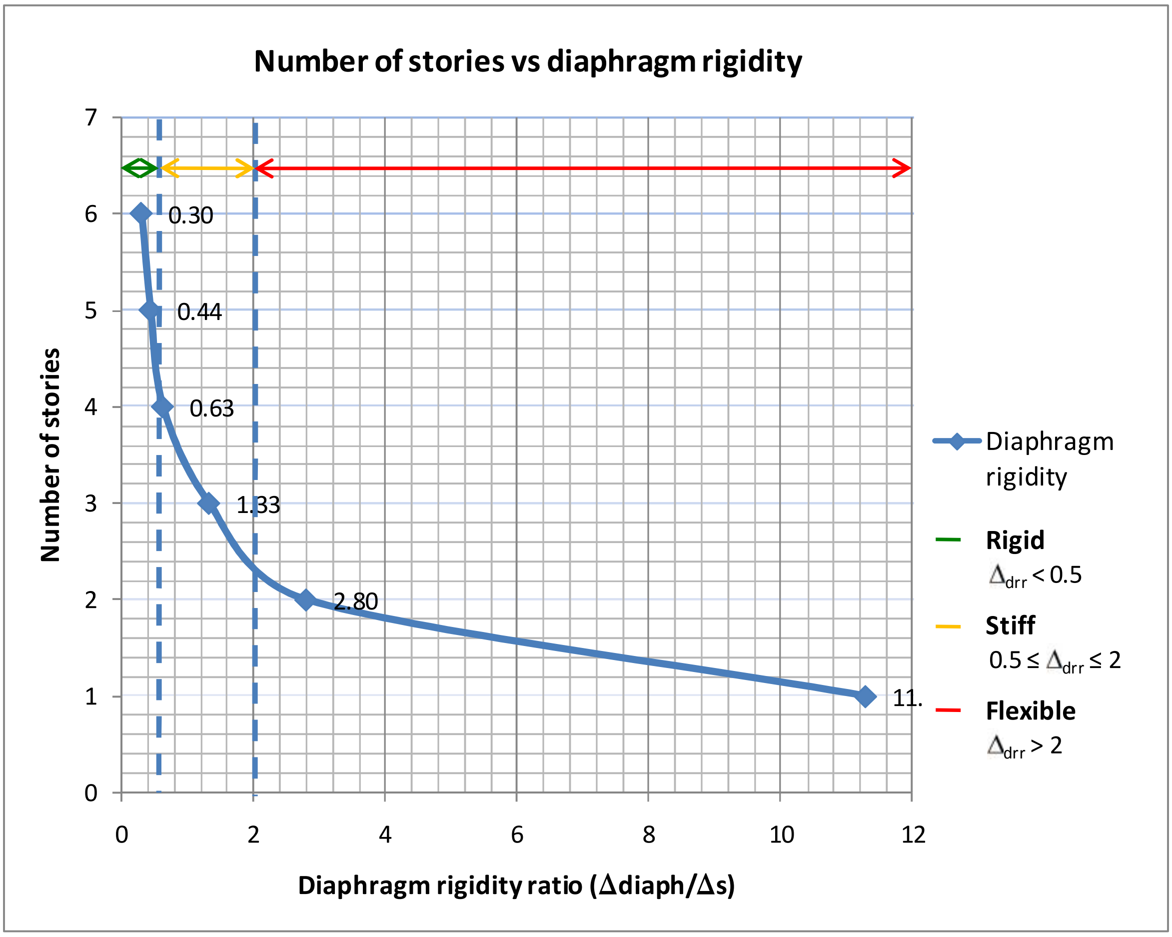


Figure 3.6: Case – 3, Number of stories versus diaphragm rigidity

Note: - The graph is for roof floor rigidity only

ii. Case – 3, Response – 2: Lateral force distribution to vertical element

Table 3.8: Case – 3, Shear force distribution in column for WOD and WD assumption

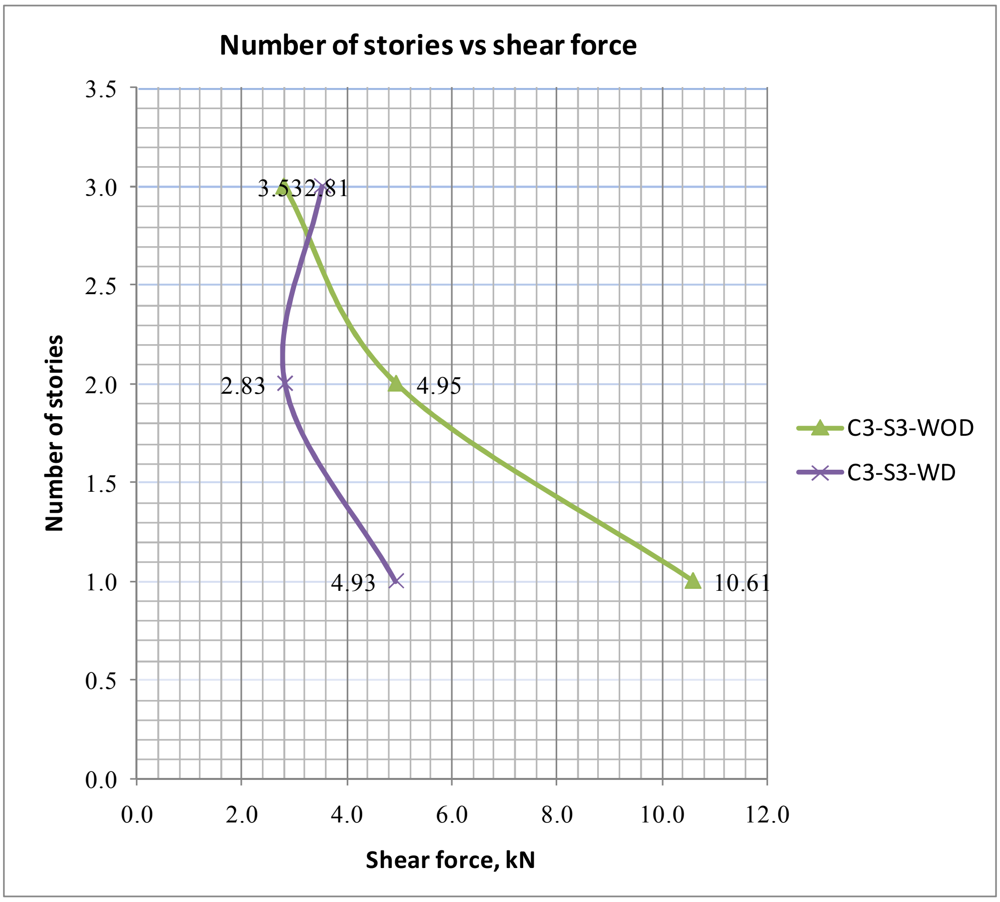
Structure type - Code	Story	Column, C1			Column, C2		
		wod	wd	Δ(%)	wod	wd	Δ(%)
S1B2H35O50W3	1	1.49	1.94	**-30.2**	1.49	1.94	**-30.2**
S2B2H35O50W3	2	3.79	4.03	**-6.3**	3.79	4.03	**-6.3**
	1	4.34	4.38	**-0.9**	4.34	4.38	**-0.9**
S3B2H35O50W3	3	8.12	8.36	**-3.0**	8.12	8.36	**-3.0**
	2	5.59	5.48	**2.0**	5.59	5.48	**2.0**
	1	6.40	6.51	**-1.7**	6.40	6.51	**-1.7**
S4B2H35O50W3	4	15.90	16.29	**-2.5**	15.90	16.29	**-2.5**
	3	11.96	11.54	**3.5**	11.96	11.54	**3.5**
	2	11.06	11.39	**-3.0**	11.06	11.39	**-3.0**
	1	10.58	10.35	**2.2**	10.58	10.35	**2.2**

Structure type - Code	Story	Column, C1			Column, C2		
		wod	wd	Δ(%)	wod	wd	Δ(%)
S5B2H35O50W3	5	22.26	22.77	**-2.3**	22.26	22.77	**-2.3**
	4	15.70	15.09	**3.9**	15.70	15.09	**3.9**
	3	17.12	17.34	**-1.3**	17.12	17.34	**-1.3**
	2	13.53	13.65	**-0.9**	13.53	13.65	**-0.9**
	1	11.95	11.64	**2.6**	11.95	11.64	**2.6**
S6B2H35O50W3	6	30.59	31.26	**-2.2**	30.59	31.26	**-2.2**
	5	21.30	20.49	**3.8**	21.30	20.49	**3.8**
	4	23.99	24.26	**-1.1**	23.99	24.26	**-1.1**
	3	22.65	22.52	**0.6**	22.65	22.52	**0.6**
	2	17.53	17.82	**-1.7**	17.53	17.82	**-1.7**
	1	14.81	14.27	**3.6**	14.81	14.27	**3.6**

Structure type - Code	Story	C3			C4		
		wod	wd	Δ(%)	wod	wd	Δ(%)
S1B2H35O50W3	1	13.50	1.55	**88.5**	13.50	1.55	**88.5**
S2B2H35O50W3	2	2.90	1.87	**35.5**	2.90	1.87	**35.5**
	1	11.92	3.81	**68.0**	11.92	3.81	**68.0**
S3B2H35O50W3	3	2.81	3.53	**-25.6**	2.81	3.53	**-25.6**
	2	4.95	2.83	**42.8**	4.95	2.83	**42.8**
	1	10.61	4.93	**53.5**	10.61	4.93	**53.5**
S4B2H35O50W3	4	5.38	6.13	**-13.9**	5.38	6.13	**-13.9**
	3	6.07	5.27	**13.2**	6.07	5.27	**13.2**
	2	6.49	4.96	**23.6**	6.49	4.96	**23.6**
	1	11.26	7.57	**32.8**	11.26	7.57	**32.8**
S5B2H35O50W3	5	7.28	8.50	**-16.8**	7.28	8.50	**-16.8**
	4	7.57	5.91	**21.9**	7.57	5.91	**21.9**
	3	6.53	6.87	**-5.2**	6.53	6.87	**-5.2**
	2	6.67	5.30	**20.5**	6.67	5.30	**20.5**
	1	10.59	7.93	**25.1**	10.59	7.93	**25.1**
S6B2H35O50W3	6	9.72	11.50	**-18.3**	9.72	11.50	**-18.3**
	5	9.63	7.43	**22.8**	9.63	7.43	**22.8**
	4	8.42	8.43	**-0.1**	8.42	8.43	**-0.1**
	3	7.48	7.88	**-5.3**	7.48	7.88	**-5.3**
	2	7.43	5.99	**19.4**	7.43	5.99	**19.4**
	1	10.86	9.22	**15.1**	10.86	9.22	**15.1**

Structure type - Code	Story	C5			C6		
		wod	wd	Δ(%)	wod	wd	Δ(%)
S1B2H35O50W3	1	1.49	1.94	**-30.2**	1.49	1.94	**-30.2**
S2B2H35O50W3	2	3.79	4.03	**-6.3**	3.79	4.03	**-6.3**
	1	4.34	4.38	**-0.9**	4.34	4.38	**-0.9**
S3B2H35O50W3	3	8.12	8.36	**-3.0**	8.12	8.36	**-3.0**
	2	5.59	5.48	**2.0**	5.59	5.48	**2.0**
	1	6.40	6.51	**-1.7**	6.40	6.51	**-1.7**
S4B2H35O50W3	4	15.90	16.29	**-2.5**	15.90	16.29	**-2.5**
	3	11.96	11.54	**3.5**	11.96	11.54	**3.5**
	2	11.06	11.39	**-3.0**	11.06	11.39	**-3.0**
	1	10.58	10.35	**2.2**	10.58	10.35	**2.2**
S5B2H35O50W3	5	22.26	22.77	**-2.3**	22.26	22.77	**-2.3**
	4	15.70	15.09	**3.9**	15.70	15.09	**3.9**
	3	17.12	17.34	**-1.3**	17.12	17.34	**-1.3**
	2	13.53	13.65	**-0.9**	13.53	13.65	**-0.9**
	1	11.95	11.64	**2.6**	11.95	11.64	**2.6**
S6B2H35O50W3	6	30.59	31.26	**-2.2**	30.59	31.26	**-2.2**
	5	21.30	20.49	**3.8**	21.30	20.49	**3.8**
	4	23.99	24.26	**-1.1**	23.99	24.26	**-1.1**
	3	22.65	22.52	**0.6**	22.65	22.52	**0.6**
	2	17.53	17.82	**-1.7**	17.53	17.82	**-1.7**
	1	14.81	14.27	**3.6**	14.81	14.27	**3.6**

Figure 3.7: Case – 3, Number of stories versus shear force distribution for WOD and WD assumption



Discussion on case – 3 results /Number of stories as a parameter/

The result for diaphragm rigidity and lateral load distribution to vertical element shows how the effects of diaphragm discontinuity vary when number of stories increases. The discussion presented as follows.

The computation for diaphragm rigidity according to FEMA diaphragm classification shows; increasing number of stories increases the floor diaphragm rigidity. This is because; when number of stories increases the building get flexible[10]. The flexibility of a building is due to its vertical element system. Flexible building undergoes large displacement because it cannot resist large force. A large displacement results in a larger inter story drift. Large inter-story drift increases the relative diaphragm rigidity computed. From the table, a single and a two story floor has a diaphragm rigidity ratio of 11.29 and 2.8 respectively with a flexible diaphragm classification. When the number of stories rises to three and four the diaphragm gets stiffer. Further increasing the stories result in a rigid diaphragm. This indicates that, buildings with a low number of stories are more susceptible for diaphragm rigidity; mainly, because of increment of rigidity of the building.

The second response of a case – 3 parametric study is lateral force distribution to vertical element. As observed from the graph, the variation of the distribution of lateral force to vertical element between actual and rigid diaphragm assumption is decreasing while number of stories increases. For the intermediate column, a single story floor has 88.5 percent difference between the two assumptions. This difference falls to -18.3 percent for a six story floor. This is mainly due to the increase in relative rigidity of the diaphragm which is increased when number of stories increases. Even though the difference is decreasing, the value of differences are not negligible; it is a value that can bring serious change in vertical element strength. From this observation and discussion, the effect of diaphragm discontinuity, in lateral force distribution to vertical element, will become serious in low rise building; because the lower the building the higher the vertical lateral load resisting element stiffness and the higher the attraction of more forces, which result in a flexible diaphragm. As discussed in the previous parametric study, flexibility of the diaphragm yields tributary area distribution of lateral force that brings significant change between actual diaphragm stiffness and rigid diaphragm assumption.

3.5.4. Case – 4: Number of bays as a parameter

In case 4 number of variations of bays are taken as a parameter. The structures are labeled by codes; S1B1H35O50W1.5, S1B2H35O50W1.5, S1B3H35O50W1.5 and S1B4H35O50W1.5. All structures have the same floor plan and vertical element configuration and 50 percent diaphragm opening size and 300mmx500mm columns size and 200mm thick and 1.5m wide shear wall, which make dual systems (hybrid/wall frame) vertical element. The only variation is number of bays. Bays considered in this study are 1-bay, 2-bay, 3-bay and 4-bay building structure in x-direction. All structures have one bay in y-direction. Analysis results for the responses are shown in a tabular and graphical format below.

i. Case – 4, Response – 1: Diaphragm rigidity

Table 3.9: Case – 4, Diaphragm rigidity summary for actual diaphragm stiffness

Structure type - Code	Bays	Δdiaph (mm)	Δav (mm)	$\Delta drr = \frac{\Delta diaph}{\Delta av}$	FEMA 273 classification
S1B1H35O50W1.5	1	0.297	0.768	0.39	Rigid
S1B2H35O50W1.5	2	2.237	1.184	1.89	Stiff
S1B3H35O50W1.5	3	6.825	1.316	5.19	Flexible
S1B4H35O50W1.5	4	10.253	1.319	7.77	Flexible

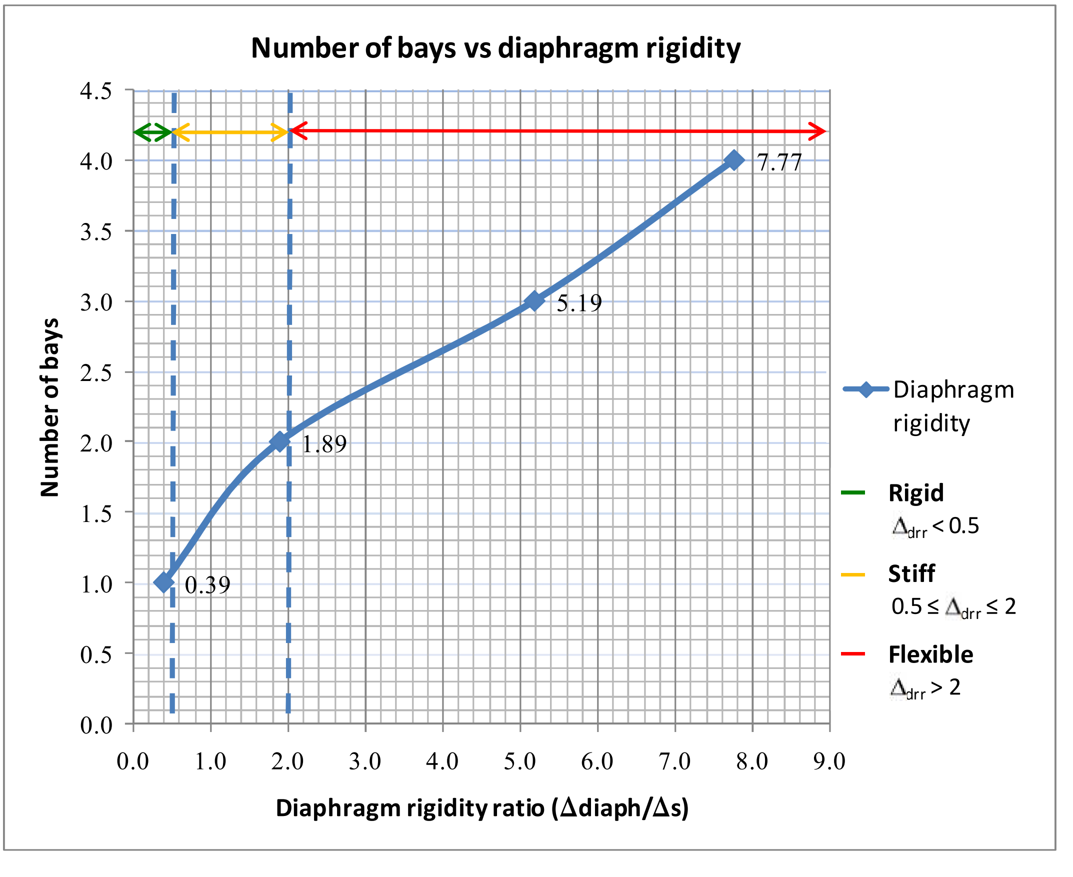


Figure 3.8: Case – 4, Number of bays versus diaphragm rigidity

i.

ase – 4, Response – 2: Lateral force distribution to vertical element

Table 3.10: Case – 4, Shear force distribution in column for WOD and WD assumption

Structure type - Code	Bays	Column, C1			Column, C2		
		wod	wd	Δ(%)	wod	Wd	Δ(%)
S1B1H35O50W1.5	1	4.58	4.61	**-0.7**	4.58	4.61	**-0.7**
S1B2H35O50W1.5	2	7.05	8.07	**-14.5**	7.05	8.07	**-14.5**
S1B3H35O50W1.5	3	7.82	11.30	**-44.5**	7.82	11.30	**-44.5**
S1B4H35O50W1.5	4	7.83	14.28	**-82.4**	7.83	14.28	**-82.4**

Column, C3			Column, C4			Column, C5		
wod	wd	Δ(%)	wod	wd	Δ(%)	wod	wd	Δ(%)
4.58	4.61	**-0.7**	4.58	4.61	**-0.7**			
17.36	7.07	**59.3**	17.36	7.07	**59.3**	7.05	8.07	**-14.5**
31.31	9.25	**70.5**	31.31	9.25	**70.5**	31.31	9.25	**70.5**
35.08	11.58	**67.0**	35.08	11.58	**67.0**	53.18	10.74	**79.8**

Column, C6			Column, C7			Column, C8		
wod	wd	Δ(%)	wod	wd	Δ(%)	wod	wd	Δ(%)
7.05	8.07	**-14.5**						
31.31	9.25	**70.5**	7.82	11.30	**-44.5**	7.82	11.30	**-44.5**
53.18	10.74	**79.8**	35.08	11.58	**67.0**	35.08	11.58	**67.0**

Column, C9			Column, C10		
wod	wd	Δ(%)	wod	wd	Δ(%)
7.83	14.28	**-82.4**	7.83	14.28	**-82.4**

Discussion on case – 4 results */Number of bays as a parameter/*

The number of bays as a parameter is the fourth parametric study. From the analysis result, here also, diaphragm discontinuity is the big deal. Both diaphragm rigidity and lateral load distribution to vertical element are affected by floor opening.

From the analysis results shown in table and graph, increasing number of bays affect seriously the rigidity of the diaphragm. It is obviously known that according to a rigid diaphragm assumption, when lateral force is applied in the direction perpendicular to the longer horizontal direction of the structure, change in horizontal displacement of parts of the diaphragm is almost zero. Because the diaphragm is infinitely rigid. However, in actual diaphragm stiffness analysis, a result of a floor diaphragm with diaphragm discontinuity shows a change in horizontal displacement at the edge and center of the diaphragm. Diaphragm deformation of (Δdiaph) 0.2965, 2.2372, 6.8254 and 10.2525 for one, two, three and four bays respectively observed. According to FEMA classification, these diaphragms are rigid, stiff, flexible and flexible respectively.

Basically, when number of bays increases the diaphragm gets flexible. This is because increasing number of bays in one direction make the building long and narrow. As stated in section 2.3, a building having a long and narrow floor diaphragms act like flexible beam and bending deflection of the diaphragm becomes significant. The long and narrow floor plan with diaphragm discontinuity reduces in plane stiffness of the diaphragm. If the stiffness is reduced it can easily deform and get flexible. Therefore, in this particular response of this parametric study, diaphragm discontinuity is a significant factor which causes the bending or bow action of the diaphragm that results in a flexible diaphragm.

Lateral force distribution to vertical element is the second response of this parametric study. As clearly seen from the table the effect of diaphragm discontinuity on lateral force distribution as number of bays increases is large and significant. 59.3 and 79.8 percent lateral force distribution difference to vertical element between the two assumption for two and four bays building structures are observed. This is mainly due to the flexible nature of the diaphragm, which is resulted from increasing number of bays in one direction. The flexibility of the diaphragm causes tributary area distribution of force in the floor; which cause the middle column to take large shear force that results in considerable differences between actual diaphragm stiffness and rigid diaphragm assumption.

3.5.5. Case – 5: Shape of opening as a parameter

In case 5 variations of shape of openings are taken as a parameter. The structures are labeled by codes; S1B2H35O50W0.5R, S1B2H35O50W0.5C and S1B2H35O50W0.5IR. All structures have the same vertical element configuration and 50 percent diaphragm opening, and 300mmx500mm column size. The shear wall has 200mm thickness and 0.5m width. The only variation is opening shape. Opening shapes taken in this study are; rectangular (S1B2H35O50W0.5R), opening with corridor in the center (S1B2H35O50W0.5C) and irregular shape opening (S1B2H35O50W0.5IR). The drawings are shown in Appendix B. Results for each response shown as below in a tabular format.

i. Case – 5, Response – 1: Diaphragm rigidity

Table 3.11: Case – 5, Diaphragm rigidity summary for actual diaphragm stiffness

Structure type - Code	Opening shape	Δdiaph (mm)	Δav (mm)	$\Delta drr = \frac{\Delta diaph}{\Delta av}$	FEMA 273 classification
S1B2H35O50W0.5R	Rectangular	1.2458	5.511	0.23	Rigid
S1B2H35O50W0.5C	With corridor	1.6977	5.213	0.33	Rigid
S1B2H35O50W0.5IR	Irregualr	2.7087	5.239	0.52	Stiff

ii. Case – 5, Response – 2: Lateral force distribution to vertical element

Table 3.12: Case – 5, Shear force distribution in column for WOD and WD assumption

Structure type - Code	Opening shape	Column, C1			Column, C2		
		wod	wd	Δ(%)	wod	wd	Δ(%)
S1B2H35O50W0.5R	Rectangular	30.08	31.84	**-5.9**	30.08	31.84	**-5.9**
S1B2H35O50W0.5C	With corridor	28.56	31.11	**-8.9**	28.56	31.11	**-8.9**
S1B2H35O50W0.5IR	Irregualr	28.39	32.13	**-13.2**	28.39	32.13	**-13.2**

Column, C3			Column, C4			Column, C5		
wod	wd	Δ(%)	wod	wd	Δ(%)	wod	wd	Δ(%)
33.96	29.36	**13.5**	33.96	29.36	**13.5**	30.08	31.84	**-5.9**
39.49	32.51	**17.7**	39.49	32.51	**17.7**	28.56	31.11	**-8.9**
38.92	28.98	**25.5**	38.92	28.98	**25.5**	28.39	32.13	**-13.2**

Column, C6		
wod	wd	Δ(%)
30.08	31.84	**-5.9**
28.56	31.11	**-8.9**
28.39	32.13	**-13.2**

Discussion on case – 5 results */Shape of opening as a parameter/*

The fifth parametric study is a shape of opening. Various opening shape results in different floor diaphragm rigidity behavior and lateral load distribution to vertical element due to the presence of diaphragm discontinuity.

The first discussion is about diaphragm rigidity. The diaphragm rigidity computed for each structure, according to FEMA classification, is shown in a table above. Observation from the table shows different opening shape result in different diaphragm rigidity. A regular opening shape with a rectangular shape results in a rigid diaphragm with diaphragm rigidity ratio (Δdiaph/Δav) equal to 0.23. But, an opening with irregular opening shape, with the same size to rectangular opening shape, yields a stiff diaphragm, Δdiaph/Δav is equal to 0.52. The reason for the variation of diaphragm rigidity is the result of tributary area load distribution to the intermediate section of the diaphragm that contributes to in-plane diaphragm deformation. The larger the tributary area load distribution, the larger the in-plane diaphragm deformation and the flexible the floor diaphragm. Therefore, if the shape of the opening is more convenient for the tributary area load distribution, the diaphragm rigidity behavior of the floor is easily affected.

The second response of this parametric study is the distribution of lateral force to vertical element. According to the rigid diaphragm assumption, a diaphragm having 50 percent diaphragm opening is rigid. From this statement, we expect the same result of lateral force distribution to vertical element for actual diaphragm stiffness and rigid diaphragm assumption. However, the analysis result shows a large difference between the two analysis, especially in cases where contribution of tributary area load distribution is large. The reason is mainly because of flexible nature of the diaphragm that is resulted from the shape of the opening as discussed above. S1B2H35O50W0.5IR structure has larger tributary area load distribution than the other two structures, due to its shape. It has 25.5 percent variation in shear force distribution to intermediate column between actual and rigid diaphragm assumption. This indicates that, the shape of opening seriously affects effect of diaphragm discontinuity on distribution of lateral load to the vertical element.

3.5.6. Case – 6: Size of opening as a parameter

In case 6 variations of size of openings are taken as a parameter. The structures are labeled by codes; S1B2H35O20W1.25, S1B2H35O30W1.25, S1B2H35O40W1.25, S1B2H35O50W1.25 and S1B2H35O60W1.25. All structures have the same vertical element configuration. The vertical element is a dual system (hybrid/wall frame). The column has 300mmx500mm section size. The shear wall has 200mm thickness and 1.25m width. The variation between the structures is opening size. Sizes of opening taken are; 20, 30, 40, 50 and 60 percent of the total floor plan area. Analysis results are shown as below in a graphical and tabular format.

i. Case – 6, Response – 1: Diaphragm rigidity

Table 3.13: Case – 6, Diaphragm rigidity summary for actual diaphragm stiffness

Structure type - Code	Opening size	Δdiaph (mm)	Δav (mm)	$\Delta drr = \frac{\Delta diaph}{\Delta av}$	FEMA 273 classification
S1B2H35O20W1.25	20	0.9910	2.894	0.34	Rigid
S1B2H35O30W1.25	30	1.1932	2.523	0.47	Rigid
S1B2H35O40W1.25	40	1.5959	2.132	0.75	Stiff
S1B2H35O50W1.25	50	2.1061	1.737	1.21	Stiff
S1B2H35O60W1.25	60	2.9620	1.319	2.25	Flexible

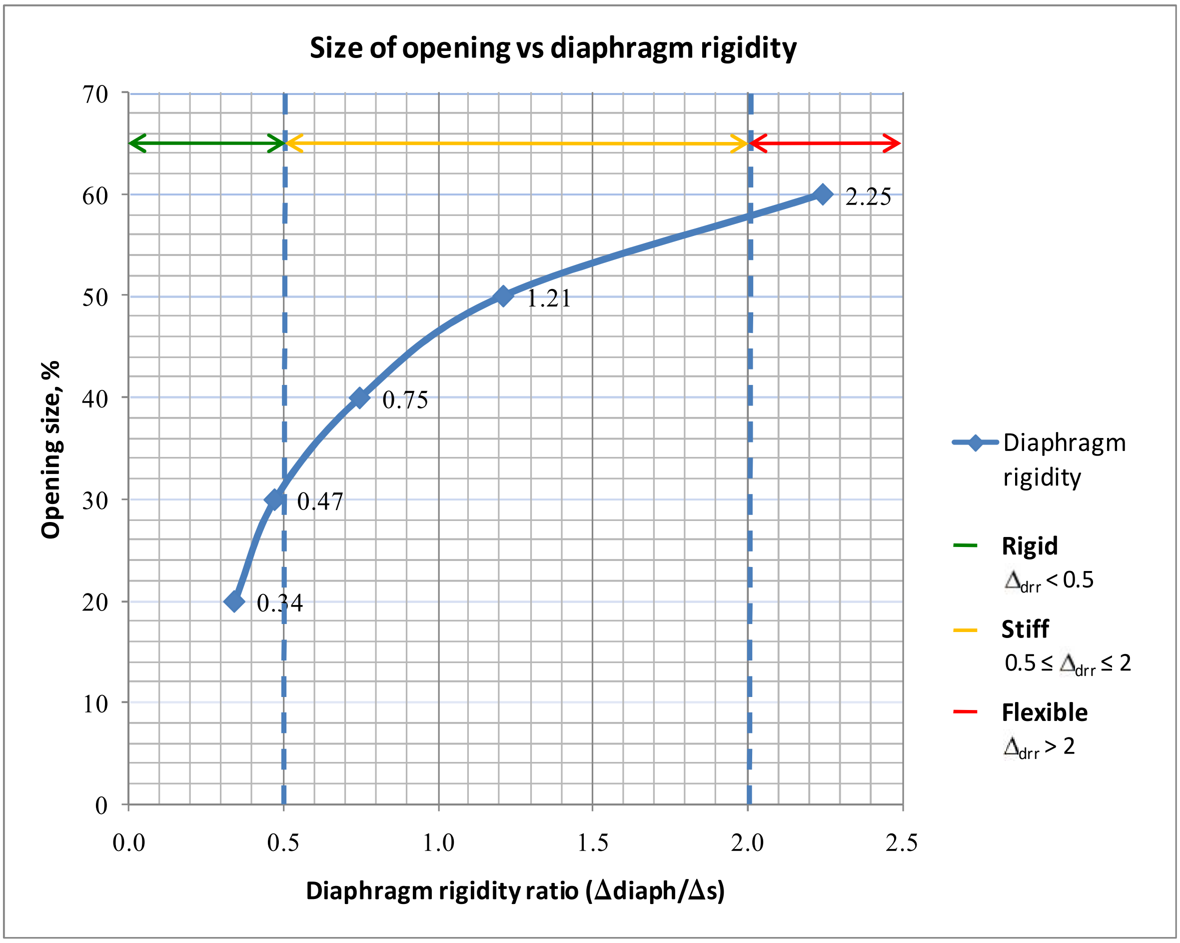


Figure 3.9: Case – 6, Size of opening versus diaphragm rigidity

ii. Case – 6, Response – 2: Lateral force distribution to vertical element

Table 3.14: Case – 6, Shear force distribution in column for WOD and WD assumption

Structure type - Code	Opening size	Column, C1			Column, C2		
		Wod	Wd	Δ(%)	wod	wd	Δ(%)
S1B2H35O20W1.25	20	16.89	17.34	**-2.7**	16.89	17.34	**-2.7**
S1B2H35O30W1.25	30	14.74	15.36	**-4.2**	14.74	15.36	**-4.2**
S1B2H35O40W1.25	40	12.47	13.40	**-7.5**	12.47	13.40	**-7.5**
S1B2H35O50W1.25	50	10.16	11.42	**-12.4**	10.16	11.42	**-12.4**
S1B2H35O60W1.25	60	7.69	9.50	**-23.5**	7.69	9.50	**-23.5**

Column, C3			Column, C4			Column, C5		
wod	wd	Δ(%)	wod	wd	Δ(%)	wod	wd	Δ(%)
20.75	15.90	**23.4**	20.75	15.90	**23.4**	16.89	17.34	**-2.7**
19.58	13.94	**28.8**	19.58	13.94	**28.8**	14.74	15.36	**-4.2**
19.33	12.04	**37.7**	19.33	12.04	**37.7**	12.47	13.40	**-7.5**
19.52	10.12	**48.2**	19.52	10.12	**48.2**	10.16	11.42	**-12.4**
21.18	8.23	**61.1**	21.18	8.23	**61.1**	7.69	9.50	**-23.5**

Column, C6		
wod	wd	Δ(%)
16.89	17.34	**-2.7**
14.74	15.36	**-4.2**
12.47	13.40	**-7.5**
10.16	11.42	**-12.4**
7.69	9.50	**-23.5**

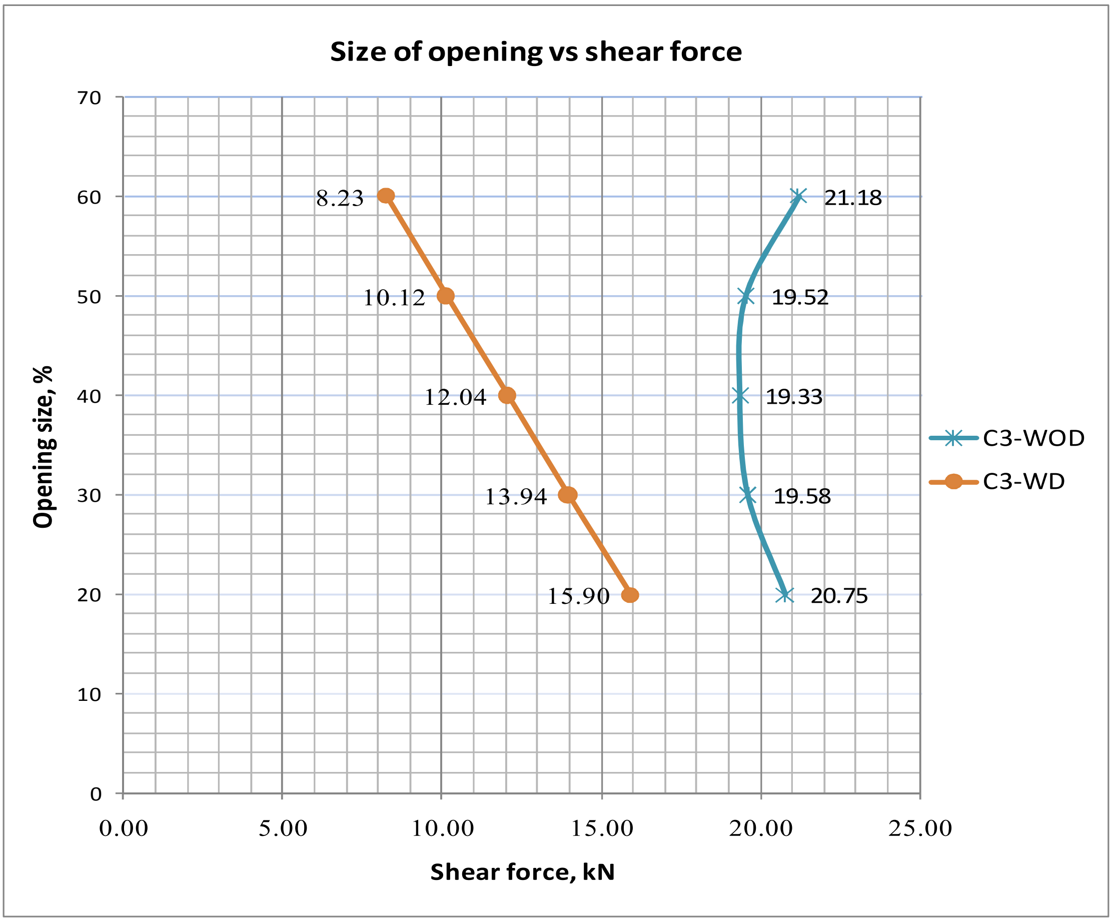


Figure 3.10: Case – 6, Size of opening versus shear force distribution for WOD and WD assumption

Discussion on case – 6 results */Size of opening as a parameter/*

In the study of the effect of diaphragm discontinuity, the size of the opening is a major factor that affects diaphragm rigidity and lateral load distribution to vertical element. The discussion is presented as follows.

The primary discussion is on how diaphragm discontinuity affects a rigidity of a diaphragm, when different sizes of diaphragm openings are existed on a floor. Observation of results from the table and the graph shows increasing size of opening changes rigidity of the diaphragm. For opening size 20, 30, 40, 50 and 60 diaphragm rigidity ratios are 0.34, 0.47, 0.75, 1.21 and 2.25 respectively. FEMA classification yields; rigid diaphragm for 20 and 30 percent opening size, stiff diaphragm for 40 and 50 percent opening size and rigid diaphragm for 60 percent opening size.

The basic reason for diaphragm rigidity variation is increasing the opening size reduces parts of the diaphragm that contribute to the rigidity of the diaphragm. When opening size get larger, obviously in-plane membrane deformation of the diaphragm increases and the diaphragm rigidity behaviour changes.

Lateral load distribution to vertical element is the second response of this parametric study. As observed from the graph and the table when opening size increases, there is a significant change between actual and rigid diaphragm assumptions on lateral load distribution. Opening size 20, 30, 40, 50 and 60 have lateral load distribution difference; 23.4, 28.8, 37.7, 48.2 and 61.1 percent respectively, to the middle column between the two assumptions. The basic reason for this variation is the change in the behavior of the diaphragm. As discussed above when the opening size increases the diaphragm get stiff or flexible based on the size of the opening. The flexibility of the diaphragm gives tributary area load distribution that affects the middle column to take larger lateral forces unlike from the rigid diaphragm assumption.

3.5.7. Case – 7: Span length as a parameter

The 7th case considers the variations of span length dimension as a parameter. The structures are labeled by codes; S1B2H35O50W2-6x6, S1B2H35O50W2-8x6, S1B2H35O50W2-10x6, S1B2H35O50W2-12x6, S1B2H35O50W2-14x6 and S1B2H35O50W2-16x6. All structures have the same vertical element configuration. The shear wall has 200mm thickness and 2m width. The variations between the structures are the dimension of the two span in x-direction. Different span lengths that are taken in x-direction are; 2@3m, 2@4m, 2@5m, 2@6m, 2@7m and 2@8m. The y-direction span has a constant 6m length. Analysis results are shown as below in a graphical and tabular format.

i. Case – 7, Response – 1: Diaphragm rigidity

Table 3.15: Case – 7, Diaphragm rigidity summary for actual diaphragm stiffness

Structure type - Code	Span (m)	Δdiaph (mm)	Δav (mm)	$\Delta drr = \dfrac{\Delta diaph}{\Delta av}$	FEMA 273 classification
S1B2H35O50W2-6x6	6	0.1000	0.337	0.30	Rigid
S1B2H35O50W2-8x6	8	0.2775	0.398	0.70	Stiff
S1B2H35O50W2-10x6	10	0.5310	0.459	1.16	Stiff
S1B2H35O50W2-12x6	12	0.9470	0.513	1.85	Stiff
S1B2H35O50W2-14x6	14	1.5542	0.560	2.78	Flexible
S1B2H35O50W2-16x6	16	2.3792	0.599	3.97	Flexible

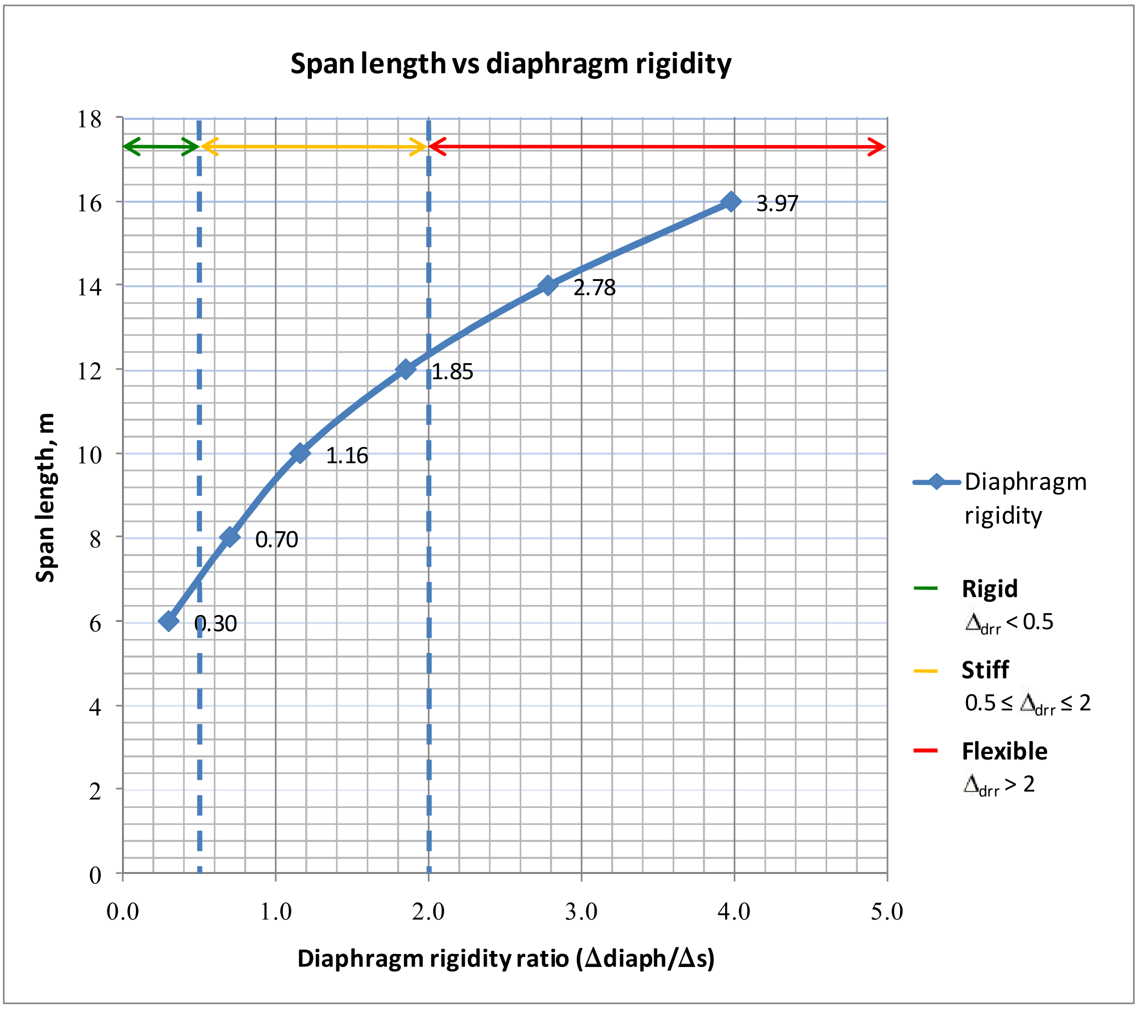


Figure 3.11: Case – 7, Span length versus diaphragm rigidity

ii. Case – 7, Response – 2: Lateral force distribution to vertical element

Table 3.16: Case – 7, Shear force distribution in column for WOD and WD assumption

Structure type - Code	Span	Column, C1			Column, C2		
		wod	wd	Δ(%)	wod	wd	Δ(%)
S1B2H35O50W2-6x6	6	2.04	2.00	**2.0**	2.04	2.00	**2.0**
S1B2H35O50W2-8x6	8	2.43	2.49	**-2.5**	2.43	2.49	**-2.5**
S1B2H35O50W2-10x6	10	2.82	2.98	**-5.7**	2.82	2.98	**-5.7**
S1B2H35O50W2-12x6	12	3.15	3.46	**-9.8**	3.15	3.46	**-9.8**
S1B2H35O50W2-14x6	14	3.45	3.94	**-14.2**	3.45	3.94	**-14.2**
S1B2H35O50W2-16x6	16	3.70	4.42	**-19.5**	3.70	4.42	**-19.5**

Column, C3			Column, C4			Column, C5		
wod	wd	Δ(%)	wod	wd	Δ(%)	wod	wd	Δ(%)
2.59	1.99	**23.2**	2.59	1.99	**23.2**	2.04	2.00	**2.0**
3.81	2.38	**37.5**	3.81	2.38	**37.5**	2.43	2.49	**-2.5**
5.39	2.74	**49.2**	5.39	2.74	**49.2**	2.82	2.98	**-5.7**
7.71	3.10	**59.8**	7.71	3.10	**59.8**	3.15	3.46	**-9.8**
10.90	3.45	**68.3**	10.90	3.45	**68.3**	3.45	3.94	**-14.2**
15.06	3.79	**74.8**	15.06	3.79	**74.8**	3.70	4.42	**-19.5**

Column, C6		
wod	wd	Δ(%)
2.04	2.00	**2.0**
2.43	2.49	**-2.5**
2.82	2.98	**-5.7**
3.15	3.46	**-9.8**
3.45	3.94	**-14.2**
3.70	4.42	**-19.5**

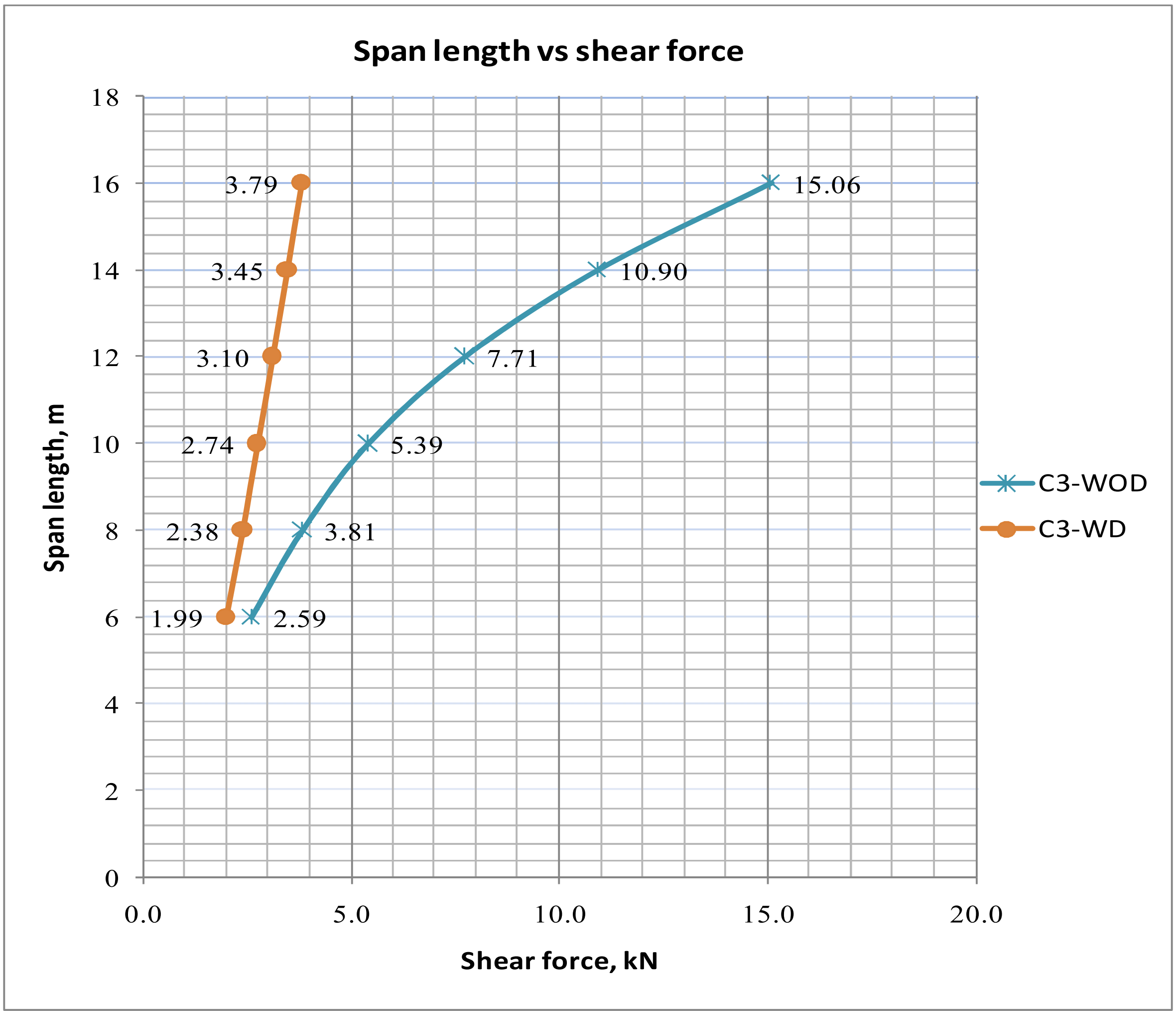


Figure 3.12: Case – 7, Span length versus shear force distribution for WOD and WD assumption

Discussion on case – 7 results */Span length as a parameter/*

The seventh parametric study is the span length as a parameter. Span length is one of the factors that affect diaphragm discontinuity effect on diaphragm rigidity and lateral load distribution to vertical element.

The first discussion is on the diaphragm rigidity of a floor. FEMA classification of the diaphragm rigidity of the structures indicates; when the dimension of the span changes the rigidity of the diaphragm also changes. In this parametric study, six structures with different span lengths are analyzed and checked for diaphragm rigidity behavior. The result in the table shows 2@3mx1@6m has a rigid diaphragm, 2@4mx1@6m, 2@5mx1@6m and 2@6mx1@6m has a stiff and 2@7mx1@6m and 2@8mx1@6m has a flexible diaphragm. When the dimension of the span increases, the diaphragm rigidity is changed. Because, increasing the span in one direction mean making the building structure narrow and long.

Narrow and long building structure experiences stiff or flexible floor diaphragm based on the dimension of the span as discussed in case – 4 parametric study. Therefore, from this discussion, we can deduce that the span dimension is one of the factors that can change the effects of diaphragm discontinuity on diaphragm rigidity.

Lateral load distribution to vertical element is the second discussion of this parametric study. A dimension of span alters lateral load distribution to the vertical element comparing actual and rigid diaphragm assumption. As discussed in detail in the above parametric study, the difference is mainly due to the change in diaphragm rigidity of the floor that is resulted from the change in span length. The larger the span length the flexible the diaphragm and the higher the tributary area load distribution, which affects the intermediate column to take large lateral forces.

3.5.8. Case – 8: Opening location in stories as a parameter

The 8th case considers the variations of location of opening in stories as a parameter, which are labeled by S6B2H35O50-1FW3, S6B2H35O50-3FW3 and S6B2H35O50-6FW3. All structures have the same vertical element configuration. The shear wall has 200mm thickness and 3m width. The variations between the structures are the location of opening in a six story building; the opening is located in 1st, 3rd and 6th floor. Analysis results are shown as below in a graphical and tabular format.

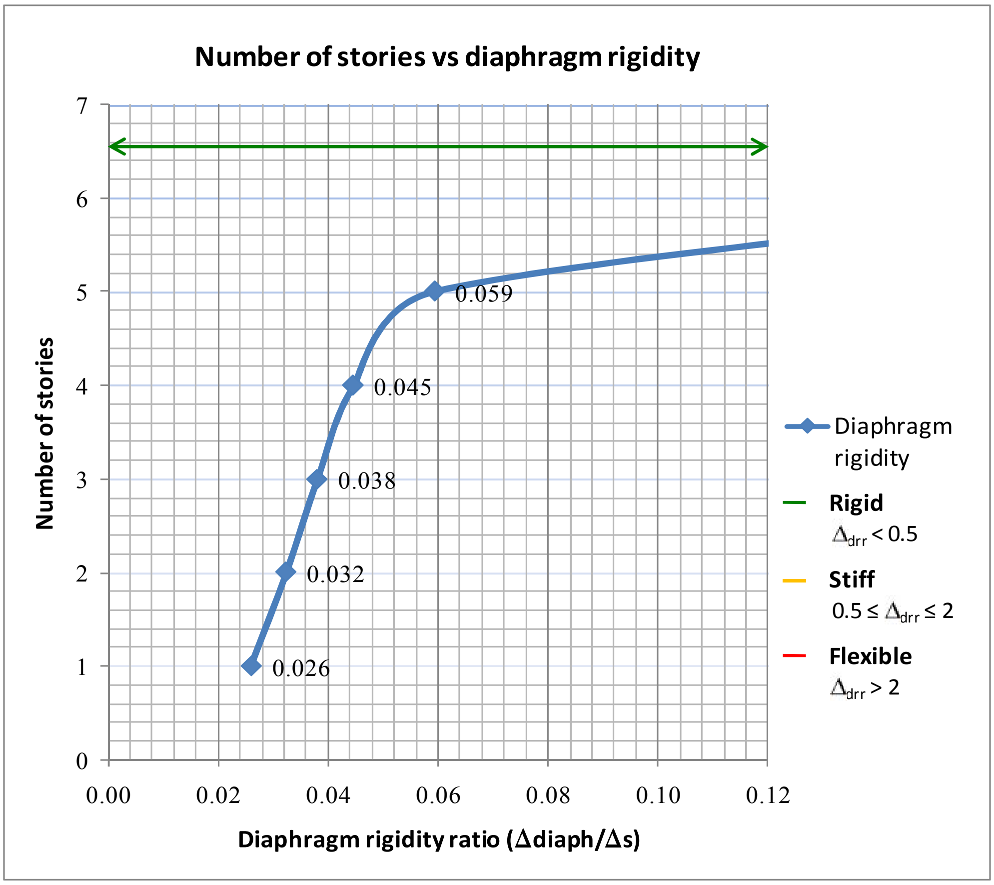
i. Case – 8, Response – 1: Diaphragm rigidity

Table 3.17: Case – 8, Diaphragm rigidity summary for actual diaphragm stiffness

Structure type - Code	Story	Δdiaph (mm)	Δav (mm)	$\Delta drr = \frac{\Delta diaph}{\Delta av}$	FEMA 273 classification
S6B2H35O50-1FW3	6	1.542	28.390	0.054	Rigid
	5	1.395	29.564	0.047	Rigid
	4	1.098	29.027	0.038	Rigid
	3	0.809	26.041	0.031	Rigid
	2	0.547	19.492	0.028	Rigid
	1	0.257	8.340	0.031	Rigid
S6B2H35O50-3FW3	6	1.609	29.067	0.055	Rigid
	5	1.420	30.247	0.047	Rigid
	4	1.265	29.693	0.043	Rigid
	3	1.910	26.248	0.073	Rigid
	2	0.584	19.531	0.030	Rigid
	1	0.164	8.336	0.020	Rigid
S6B2H35O50-6FW3	6	4.773	24.825	0.192	Rigid
	5	1.563	26.358	0.059	Rigid
	4	1.171	26.294	0.045	Rigid
	3	0.911	23.945	0.038	Rigid
	2	0.586	18.146	0.032	Rigid
	1	0.205	7.877	0.026	Rigid

Figure 3.13: Case – 8, Number of stories versus diaphragm rigidity

Note: - The graph is for roof floor rigidity only



ii. Case – 8, Response – 2: Lateral force distribution to vertical element

Table 3.18: Case – 8, Shear force distribution in column for WOD and WD assumption

Structure type - Code	Story	Column, C1			Column, C2		
		wod	wd	Δ(%)	wod	wd	Δ(%)
S6B2H35O50-1FW3	6	46.97	47.85	**-1.9**	46.97	47.85	**-1.9**
	5	32.82	31.64	**3.6**	32.82	31.64	**3.6**
	4	37.15	37.50	**-0.9**	37.15	37.50	**-0.9**
	3	35.09	34.92	**0.5**	35.09	34.92	**0.5**
	2	27.70	27.96	**-0.9**	27.70	27.96	**-0.9**
	1	23.16	22.42	**3.2**	23.16	22.42	**3.2**
S6B2H35O50-3FW3	6	48.31	49.14	**-1.7**	48.31	49.14	**-1.7**
	5	33.29	32.41	**2.6**	33.29	32.41	**2.6**
	4	39.23	38.85	**1.0**	39.23	38.85	**1.0**
	3	34.56	35.13	**-1.6**	34.56	35.13	**-1.6**
	2	27.68	27.60	**0.3**	27.68	27.60	**0.3**
	1	22.99	22.34	**2.8**	22.99	22.34	**2.8**
S6B2H35O50-6FW3	6	40.39	41.93	**-3.8**	40.39	41.93	**-3.8**
	5	29.75	28.05	**5.7**	29.75	28.05	**5.7**
	4	33.74	34.21	**-1.4**	33.74	34.21	**-1.4**
	3	32.80	32.61	**0.6**	32.80	32.61	**0.6**
	2	26.02	26.28	**-1.0**	26.02	26.28	**-1.0**
	1	22.61	21.90	**3.1**	22.61	21.90	**3.1**

Structure type - Code	Story	Column, C3			Column, C4		
		wod	wd	Δ(%)	wod	wd	Δ(%)
S6B2H35O50-1FW3	6	29.57	30.70	**-3.8**	29.57	30.70	**-3.8**
	5	17.92	16.21	**9.5**	17.92	16.21	**9.5**
	4	16.16	16.49	**-2.0**	16.16	16.49	**-2.0**
	3	13.83	13.69	**1.0**	13.83	13.69	**1.0**
	2	5.05	5.60	**-10.9**	5.05	5.60	**-10.9**
	1	11.89	9.93	**16.5**	11.89	9.93	**16.5**
S6B2H35O50-3FW3	6	29.93	31.30	**-4.6**	29.93	31.30	**-4.6**
	5	22.91	18.08	**21.1**	22.91	18.08	**21.1**
	4	2.15	11.29	**-425.1**	2.15	11.29	**-425.1**
	3	17.70	8.71	**50.8**	17.70	8.71	**50.8**
	2	7.64	11.33	**-48.3**	7.64	11.33	**-48.3**
	1	15.34	13.38	**12.8**	15.34	13.38	**12.8**

Structure type - Code	Story	Column, C3			Column, C4		
		wod	wd	Δ(%)	wod	wd	Δ(%)
S6B2H35O50-6FW3	6	25.76	18.58	**27.9**	25.76	18.58	**27.9**
	5	13.00	18.82	**-44.8**	13.00	18.82	**-44.8**
	4	17.84	16.80	**5.8**	17.84	16.80	**5.8**
	3	14.19	14.36	**-1.2**	14.19	14.36	**-1.2**
	2	9.88	9.59	**2.9**	9.88	9.59	**2.9**
	1	15.05	13.86	**7.9**	15.05	13.86	**7.9**

Structure type - Code	Story	Column, C5			Column, C6		
		wod	wd	Δ(%)	wod	wd	Δ(%)
S6B2H35O50-1FW3	6	46.97	47.85	**-1.9**	46.97	47.85	**-1.9**
	5	32.82	31.64	**3.6**	32.82	31.64	**3.6**
	4	37.15	37.50	**-0.9**	37.15	37.50	**-0.9**
	3	35.09	34.92	**0.5**	35.09	34.92	**0.5**
	2	27.70	27.96	**-0.9**	27.70	27.96	**-0.9**
	1	23.16	22.42	**3.2**	23.16	22.42	**3.2**
S6B2H35O50-3FW3	6	48.31	49.14	**-1.7**	48.31	49.14	**-1.7**
	5	33.29	32.41	**2.6**	33.29	32.41	**2.6**
	4	39.23	38.85	**1.0**	39.23	38.85	**1.0**
	3	34.56	35.13	**-1.6**	34.56	35.13	**-1.6**
	2	27.68	27.60	**0.3**	27.68	27.60	**0.3**
	1	22.99	22.34	**2.8**	22.99	22.34	**2.8**
S6B2H35O50-6FW3	6	40.39	41.93	**-3.8**	40.39	41.93	**-3.8**
	5	29.75	28.05	**5.7**	29.75	28.05	**5.7**
	4	33.74	34.21	**-1.4**	33.74	34.21	**-1.4**
	3	32.80	32.61	**0.6**	32.80	32.61	**0.6**
	2	26.02	26.28	**-1.0**	26.02	26.28	**-1.0**
	1	22.61	21.90	**3.1**	22.61	21.90	**3.1**

- ***Discussion on case – 8 results*** */Opening location in stories as a parameter/*

The last parametric study is opening location in stories as a parameter. Similar to the above discussion opening location in stories is one of the diaphragm discontinuity problems. The diaphragm rigidity and lateral load distribution variation observed in the analysis is discussed below.

To evaluate the effect of opening location in different story levels, three building structures having 50 percent opening in 1st, 3rd and 6th story levels are analyzed. According to FEMA classification, the result in the table shows the three structures fall in rigid diaphragm classification. This is mainly because the structures under study are a six story building, which can be affected by building flexibility as discussed in detail in case – 3 parametric study. High-rise building experiences relatively low vertical element stiffness, which reduces its lateral load resisting capacity and resulting relatively a rigid floor diaphragm. However, there is a slight variation of a diaphragm rigidity ratio between the three structures where opening location varies, which mainly result change in lateral force distribution to vertical element.

The second discussion is on results of lateral force distribution to vertical element. The result in the table shows, there is a change in actual and rigid diaphragm assumption. An intermediate column of a building structure with 2nd story-level opening has +50.8 percent difference, between the two assumptions, which is the largest of the three structures. The change in lateral force distribution is observed for a rigid diaphragm floor that is classified as 'rigid' according to FEMA. First of all, the change is observed because, in rigid diaphragm assumption, the diaphragm is infinitely rigid; but in FEMA rigidity classification, the diaphragm will be classified as 'rigid' for a diaphragm rigidity ratio 0 to 0.5. Therefore, we can expect variation in the lateral force distribution of the two assumptions for diaphragm rigidity ratio greater than 0.00, because the diaphragm discontinuity affects in-plane stiffness of the diaphragm. Whenever in-plane rigidity of the diaphragm is affected, it results in a tributary area load distribution.

3.6. Summary of discussion

A total of 123 reinforced concrete structures, 246 for two assumptions, were analyzed and a typical 35, 70 for two assumptions, model structures were evaluated and discussed in detail to assess the effect of size of diaphragm discontinuity on diaphragm rigidity and lateral load distribution to vertical element. A basic structural layout of a building was selected to maximize the in-plane deformation of the diaphragm to allow an evaluation of the potential impact of diaphragm discontinuity on reinforced concrete structures. The analyses were carried out for actual diaphragm stiffness and rigid diaphragm assumption in order to investigate the effect of diaphragm discontinuity. For the analysis, ETABS 9.7 software is used. The rigidity of the diaphragm is classified according to NEHRP guidelines (FEMA 273 diaphragm classification). Mainly, diaphragm discontinuity with 50 percent floor diaphragm opening taken for the investigation as per the code provision, which is considered as a rigid diaphragm.

In this chapter, eight parametric studies were carried out; story height, vertical element system, number of stories, number of bays, shape of opening, size of opening, span length and opening location in stories as a parameter. For each parametric study, four responses were evaluated; diaphragm rigidity, story drift, lateral load distribution to vertical element and natural vibration period. The responses of story drift and natural vibration period were not as such significant. Therefore, analysis results of diaphragm rigidity and lateral force distribution to vertical element were presented and discussed in detail. All the discussions made are summarized as follows.

i. The stiffness of the vertical element system affect the effect of diaphragm discontinuity on the responses of a structure based on case 1 and case 2 studies. When a vertical element gets stiffer the relative in-plane stiffness of the diaphragm reduced, which affects mainly diaphragm rigidity and lateral force distribution to vertical element.
ii. Low-rise building structures are more susceptible to diaphragm discontinuity than high-rise building, case 3 and 8 studies. Because the lower the building, the rigid the building system and the stiffer the vertical element. When a vertical element gets stiffer, the relative in-plane rigidity of the diaphragm affected, which can change diaphragm behaviour and lateral force distribution.

iii. The effect of diaphragm discontinuity in long and narrow building structure is more serious than a building with short rectangular plan, case 4 and 7 studies. Long and narrow building structure experiences flexible floor diaphragm, which affect in-plane diaphragm stiffness. Change in in-plane diaphragm stiffness affects diaphragm rigidity behavior and lateral load distribution.

iv. The shape of opening affects the effects of diaphragm discontinuity on diaphragm rigidity and lateral load distribution to vertical element by varying tributary area load distribution, case 5 study. When a shape contributing a larger tributary area exists, the effect of diaphragm discontinuity on diaphragm rigidity and lateral load distribution to vertical element get larger.

v. Opening size is one of the major factors that affects the effect of diaphragm discontinuity on diaphragm rigidity and lateral load distribution to vertical element, case 6 study. Increasing opening size reduces parts of the diaphragm that contribute to the rigidity of the diaphragm. When opening size get larger, obviously in-plane membrane deformation of the diaphragm increases and the diaphragm rigidity behavior and lateral load distribution to vertical element changes.

vi. For a building structure with moment-resisting frame, effects of diaphragm discontinuity on diaphragm rigidity and lateral load distribution to vertical element when actual diaphragm stiffness and rigid diaphragm assumption model is used, the difference between the two assumptions is not appreciable. This is due to the fact that the in-plane stiffness of the diaphragm is much larger than the out-of-plane column stiffness.

vii. For a building structure with dual system, the effect of diaphragm discontinuity between the two assumptions is large and significant. Because the dual system consists a shear wall that gives large lateral stiffness to the vertical element, which reduces the relative in-plane stiffness of the diaphragm.

viii. ASCE7 (2005) and Euro code 8 (2003) acknowledged that a diaphragm plan aspect ratio greater than 3 and 4 respectively can result in considerable errors when predicting the seismic response of building structures. However, it has been observed that the aspect ratio of a diaphragm alone is not sufficient to determine its rigidity. Mainly, the relative stiffness between the floor diaphragm and the adjoining vertical lateral force resisting element systems should be checked.

Chapter Four

4. Conclusion and Recommendation

4.1. Conclusion

A total of 123 reinforced concrete structures were analyzed, and typical 35 model structures were evaluated and discussed in detail to assess the effect of size of diaphragm discontinuity on diaphragm rigidity and lateral load distribution to vertical element. In order to investigate the effect, eight parametric studies were carried out; story height, vertical element system, number of stories, number of bays, shape of opening, size of opening, span length and opening location in stories as a parameter. For each parametric study, four responses were evaluated; diaphragm rigidity, story drift, lateral load distribution to vertical element and natural vibration period.

From the study, it is observed that effects diaphragm discontinuity on the response of a structure cannot be overlooked. It results in changing the response of a structure significantly differing from the same structure analyzed with rigid diaphragm assumption. The difference in response mainly observed in diaphragm rigidity and lateral load distribution to vertical element. The basic problem here is that the structure is subjected to additional force and stress that are not considered in rigid diaphragm assumption.

Based on the analysis result, evaluation and discussion made in the previous chapter, the following conclusions were made:

1. Effects of diaphragm discontinuity on diaphragm rigidity and lateral load distribution to vertical element is mainly influenced by; vertical element stiffness, number of stories, aspect ratio of the slab dimension, shape and size of the diaphragm opening.
2. Stiffness of the vertical element is one of the most governing factors that determine the effects of discontinuity on diaphragm rigidity and lateral load distribution to vertical element. Because, the larger the stiffness, the larger is the force required to deform it and the more the attraction of lateral force than the floor diaphragm, which reduces the floor diaphragm lateral force resistance. This results in the increase in the in-plane deformation of the diaphragm that changes

diaphragm rigidity and lateral load deformation. Those results computed for different parametric study; whether number of stories or bays, opening size or shape mainly affected by the change in stiffness of the vertical element.

3. It has been observed that assessing the aspect ratio of a diaphragm alone is not sufficient to determine its rigidity. Mainly, the relative stiffness between the floor diaphragm and the adjoining vertical lateral force resisting element systems should be checked.
4. For a building structure with dual system (hybrid/wall frame) vertical elements, effects of discontinuity on diaphragm rigidity and lateral load distribution to vertical element is larger and more significant than moment-resisting frame system. This is mainly because the diaphragm attracts more lateral force for moment-resisting frame than the stiffer dual systems.
5. Code provision of diaphragm discontinuity, which states that ***diaphragms having cut out 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 story to the next*** are affected by diaphragm discontinuity. However, it is proved in this paper that, a diaphragm having not only 50 percent diaphragm opening size but also a floor diaphragm with 30 and 40 percent opening size is affected by diaphragm discontinuity. Therefore, the investigation has shown that the code provision relating diaphragm rigidity to opening size is not always satisfied.

4.2. Recommendation

From all the above discussion and evaluation, it is concluded that diaphragm discontinuity is a serious problem that can alter responses of a structure. The change in responses can affect internal forces and stresses of a floor diaphragm as well as vertical element systems that were not designed to carry those variations based on a rigid diaphragm assumption. Therefore, the following recommendation is given regarding the size of opening of a floor diaphragm to minimize the effect diaphragm discontinuity.

Code provision of 50 percent opening size, regarding its rigidity, is erratic, which is affected by various factors discussed in chapter three. Therefore, generally, a small diaphragm opening size is recommended in a floor diaphragm. Whenever opening size 30 percent and above exists effects of diaphragm discontinuity on diaphragm rigidity and lateral load distribution to vertical element should be checked, in structures, which have similar property of parameters that are discussed in this paper.

4.3. Further research

This thesis focuses on certain parametric studies and responses of a structure to assess effects of diaphragm discontinuity. However, further research can be made to investigate the diaphragm discontinuity effect in varying horizontal and vertical plan configuration. This can be done by taking number of parametric studies like different diaphragm opening location, unsymmetrical opening shape, floor diaphragm shape with U, L and T. And additional responses can also be checked like torsion and stress distribution in the diaphragm for each parametric study.

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APPENDICES

Appendix A: All the Structures Analyzed for Parametric Study

Table app1: A total of 123-structures analyzed for parametric study

Structure type - code	No of stories	No of bays	Story height (m)	Opening size (%)	Shear wall width (m)	Aspect ratio (LxW)	Column size (cm)
S1B2H25O50W2 C30X50	1	2	2.5	50	2	2.66	30x50
S1B2H45O50W2 C30X50	1	2	4.5	50	2	2.66	30x50
S1B2H55O50W2 C30X50	1	2	5.5	50	2	2.66	30x50
S1B2H65O50W2 C30X50	1	2	6.5	50	2	2.66	30x50
S1B2H75O50W2 C30X50	1	2	7.5	50	2	2.66	30x50
S1B2H85O50W2 C30X50	1	2	8.5	50	2	2.66	30x50
S1B2H35O50 C30X50	1	2	3.5	50	-	2.66	30x50
S1B2H35O50W1 C30X50	1	2	3.5	50	1	2.66	30x50
S1B2H35O50W2 C30X50	1	2	3.5	50	2	2.66	30x50
S1B2H35O50W3 C30X50	1	2	3.5	50	3	2.66	30x50
S1B2H35O50W4 C30X50	1	2	3.5	50	4	2.66	30x50
S1B2H35O50W5 C30X50	1	2	3.5	50	5	2.66	30x50
S1B2H35O50W6 C30X50	1	2	3.5	50	6	2.66	30x50
S2B2H35O50W3 C30X50	2	2	3.5	50	3	2.66	30x50
S3B2H35O50W3 C30X50	3	2	3.5	50	3	2.66	30x50
S4B2H35O50W3 C30X50	4	2	3.5	50	3	2.66	30x50
S5B2H35O50W3 C30X50	5	2	3.5	50	3	2.66	30x50
S6B2H35O50W3 C30X50	6	2	3.5	50	3	2.66	30x50
S1B1H35O50W2 C30X50	1	1	3.5	50	2	1.33	30x50
S1B3H35O50W2 C30X50	1	3	3.5	50	2	4.00	30x50
S1B4H35O50W2 C30X50	1	4	3.5	50	2	5.33	30x50
S1B2H35O50W1C C30X50	1	2	3.5	50	1	2.66	30x50
S1B2H35O50W1IR C30X50	1	2	3.5	50	1	2.66	30x50
S1B2H35O20W2 C30X50	1	2	3.5	20	2	2.66	30x50
S1B2H35O30W2 C30X50	1	2	3.5	30	2	2.66	30x50
S1B2H35O40W2 C30X50	1	2	3.5	40	2	2.66	30x50
S1B2H35O60W2 C30X50	1	2	3.5	60	2	2.66	30x50
S1B2H35O50W3-6x6 C30X50	1	2	3.5	50	3	1.00	30x50
S1B2H35O50W3-8x6 C30X50	1	2	3.5	50	3	1.33	30x50
S1B2H35O50W3-10x6 C30X50	1	2	3.5	50	3	1.66	30x50
S1B2H35O50W3-12x6 C30X50	1	2	3.5	50	3	2.00	30x50
S1B2H35O50W3-14x6 C30X50	1	2	3.5	50	3	2.33	30x50

Structure type - code	No of stories	No of bays	Story height (m)	Opening size (%)	Shear wall width (m)	Aspect ratio (LxW)	Column size (cm)
S6B2H35O50-1FW3 C30X50	6	2	3.5	50	3	2.66	30x50
S6B2H35O50-3FW3 C30X50	6	2	3.5	50	3	2.66	30x50
S6B2H35O50-6FW3 C30X50	6	2	3.5	50	3	2.66	30x50
S1B1H35O50C30X50	1	1	3.5	50	-	1.33	30x50
S1B2H25O50C30X50	1	2	2.5	50	-	2.66	30x50
S1B2H45O50C30X50	1	2	4.5	50	-	2.66	30x50
S1B2H55O50C30X50	1	2	5.5	50	-	2.66	30x50
S1B2H70O50C30X50	1	2	7.0	50	-	2.66	30x50
S1B2H35O50C30X50	1	2	3.5	50	-	2.66	30x50
S3B2H35O50C30X50	3	2	3.5	50	-	2.66	30x50
S4B2H35O50C30X50	4	2	3.5	50	-	2.66	30x50
S5B2H35O50C30X50	5	2	3.5	50	-	2.66	30x50
S6B2H35O50C30X50	6	2	3.5	50	-	2.66	30x50
S1B2H35O50CC30X50	1	2	3.5	50	-	2.66	30x50
S1B2H35O50IRC30X50	1	2	3.5	50	-	2.66	30x50
S1B2H35O20C30X50	1	2	3.5	20	-	2.66	30x50
S1B2H35O30C30X50	1	2	3.5	30	-	2.66	30x50
S1B2H35O40C30X50	1	2	3.5	40	-	2.66	30x50
S1B2H35O60C30X50	1	2	3.5	60	-	2.66	30x50
S1B2H70O50W1C30X50	1	2	7.0	50	1	2.66	30x50
S1B2H70O50W2C30X50	1	2	7.0	50	2	2.66	30x50
S1B2H70O50W3C30X50	1	2	7.0	50	3	2.66	30x50
S1B2H35O30C40X60	1	2	3.5	30	-	2.66	30x50
S1B2H35O40C40X60	1	2	3.5	40	-	2.66	40x60
S1B2H35O50C40X60	1	2	3.5	50	-	2.66	40x60
S2B2H25O50C30X50	2	2	2.5	50	-	2.66	30x50
S2B2H25O50C40X60	2	2	2.5	50	-	2.66	40x60
S2B2H25O50W1C40X60	2	2	2.5	50	1	2.66	40x60
S2B2H25O50W3C40X60	2	2	2.5	50	3	2.66	40x60
S2B2H30O50W1C40X60	2	2	3.0	50	1	2.66	40x60
S2B2H30O50W3C40X60	2	2	3.0	50	3	2.66	40x60
S4B2H30O50C30X50	4	2	3.0	50	-	2.66	30x50
S4B2H30O50C40X60	4	2	3.0	50	-	2.66	40x60
S4B2H30O50W1C40X60	4	2	3.0	50	1	2.66	40x60
S4B2H30O50W3C40X60	4	2	3.0	50	3	2.66	40x60
S6B2H25O50C30X50	6	2	2.5	50	-	2.66	30x50
S6B2H35O30C30X50	6	2	3.5	30	-	2.66	30x50
S6B2H35O30W1C30X50	6	2	3.5	30	1	2.66	30x50
S6B2H35O30W3C30X50	6	2	3.5	30	3	2.66	30x50
S6B2H35O30W6C30X50	6	2	3.5	30	6	2.66	30x50
S6B2H35O40C30X50	6	2	3.5	40	-	2.66	30x50
S6B2H35O40W1C30X50	6	2	3.5	40	1	2.66	30x50
S6B2H35O40W3C30X50	6	2	3.5	40	3	2.66	30x50

Structure type - code	No of stories	No of bays	Story height (m)	Opening size (%)	Shear wall width (m)	Aspect ratio (LxW)	Column size (cm)
S6B2H35O40W6C30X50	6	2	3.5	40	6	2.66	30x50
S1B3H35O30C30X50	1	3	3.5	30	-	4.00	30x50
S1B3H35O40C30X50	1	3	3.5	40	-	4.00	30x50
S1B3H35O50C30X50	1	3	3.5	50	-	4.00	30x50
S1B3H35O30C40X60	1	3	3.5	30	-	4.00	40x60
S1B3H35O40C40X60	1	3	3.5	40	-	4.00	40x60
S1B3H35O50C40X60	1	3	3.5	50	-	4.00	40x60
S2B3H25O50C30X50	2	3	2.5	50	-	4.00	30x50
S2B3H25O50C40X60	2	3	2.5	50	-	4.00	40x60
S2B3H30O50C40X60	2	3	3.0	50	-	4.00	40x60
S2B3H30O50W1C40X60	2	3	3.0	50	1	4.00	40x60
S4B3H30O50C30X50	4	3	3.0	50	-	4.00	30x50
S4B3H30O50C40X60	4	3	3.0	50	-	4.00	40x60
S4B3H30O50W1C40X60	4	3	3.0	50	1	4.00	40x60
S6B3H35O30C30X50	6	3	3.5	30	-	4.00	30x50
S6B3H35O40C30X50	6	3	3.5	40	-	4.00	30x50
S6B3H35O50C30X50	6	3	3.5	50	-	4.00	30x50
S6B3H35O30C40X60	6	3	3.5	30	-	4.00	40x60
S6B3H35O40C40X60	6	3	3.5	40	-	4.00	40x60
S6B3H35O50C40X60	6	3	3.5	50	-	4.00	40x60
S1B4H35O30C30X50	1	4	3.5	30	-	5.33	30x50
S1B4H35O50C30X50	1	4	3.5	50	-	5.33	30x50
S1B4H35O40C30X50	1	4	3.5	40	-	5.33	30x50
S2B4H25O50C30X50	2	4	2.5	50	-	5.33	30x50
S2B4H30O50C30X50	2	4	3.0	50	-	5.33	30x50
S4B4H30O50C30X50	4	4	3.0	50	-	5.33	30x50
S6B4H35O30C30X50	6	4	3.0	30	-	5.33	30x50
S6B4H35O40C30X50	6	4	3.5	40	-	5.33	30x50
S6B4H35O50C30X50	6	4	3.5	50	-	5.33	30x50
S6B4H35O30C40X60	6	4	3.5	30	-	5.33	40x60
S6B4H35O40C40X60	6	4	3.5	40	-	5.33	40x60
S6B4H35O50C40X60	6	4	3.5	50	-	5.33	40x60
S1B2H25O50W3C30X50	1	2	2.5	50	3	2.66	30x50
S1B2H45O50W3C30X50	1	2	4.5	50	3	2.66	30x50
S1B2H55O50W3C30X50	1	2	5.5	50	3	2.66	30x50
S1B2H65O50W3C30X50	1	2	6.5	50	3	2.66	30x50
S1B2H75O50W3C30X50	1	2	7.5	50	3	2.66	30x50
S1B2H85O50W3C30X50	1	2	8.5	50	3	2.66	30x50
S1B1H35O50W3C30X50	1	1	3.5	50	3	1.33	30x50
S1B3H35O50W3C30X50	1	3	3.5	50	3	4.00	30x50
S1B4H35O50W3C30X50	1	4	3.5	50	3	5.33	30x50
S1B2H35O50W3RC30X50	1	2	3.5	50	3	2.66	30x50
S1B2H35O50W3CC30X50	1	2	3.5	50	3	2.66	30x50

Structure type - Code	No of Stories	No of bays	Story Height (m)	Opening size (%)	Shear wall width (m)	Aspect ratio (LxW)	Column size (cm)
S1B2H35O50W3IRC30X50	1	2	3.5	50	3	2.66	30x50
S1B2H35O20W3C30X50	1	2	3.5	20	3	2.66	30x50
S1B2H35O30W3C30X50	1	2	3.5	30	3	2.66	30x50
S1B2H35O40W3C30X50	1	2	3.5	40	3	2.66	30x50
S1B2H35O60W3C30X50	1	2	3.5	60	3	2.66	30x50

Appendix B: Sample and Representative Drawings

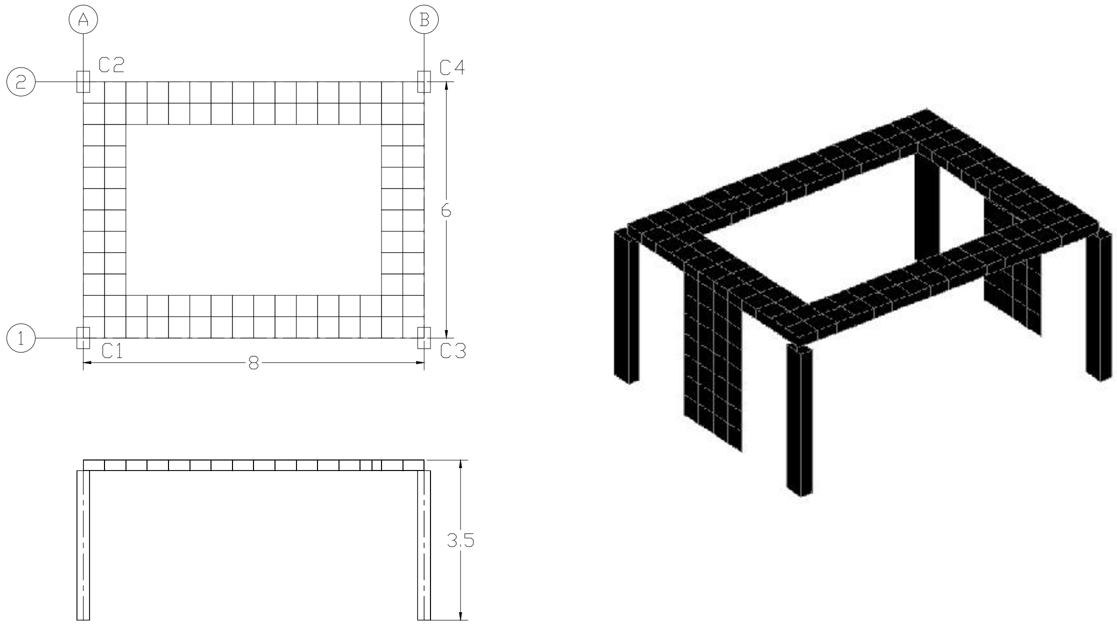

Figure B1: Drawings for S1B1H35O50W2

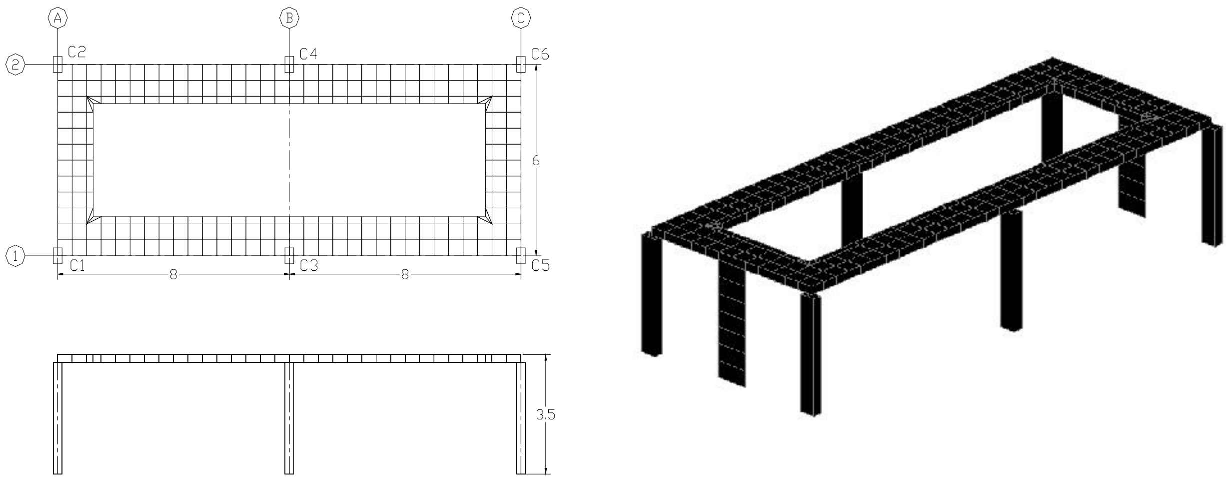

Figure B2: Drawings for S1B2H35O50W1R

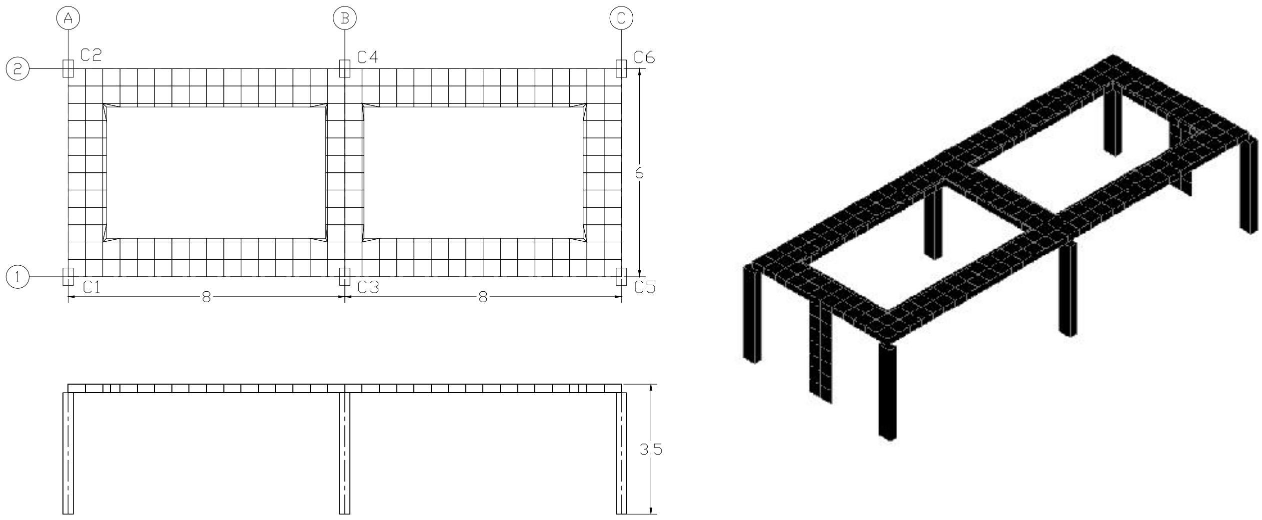

Figure B3: Drawings for S1B2H35O50W1C

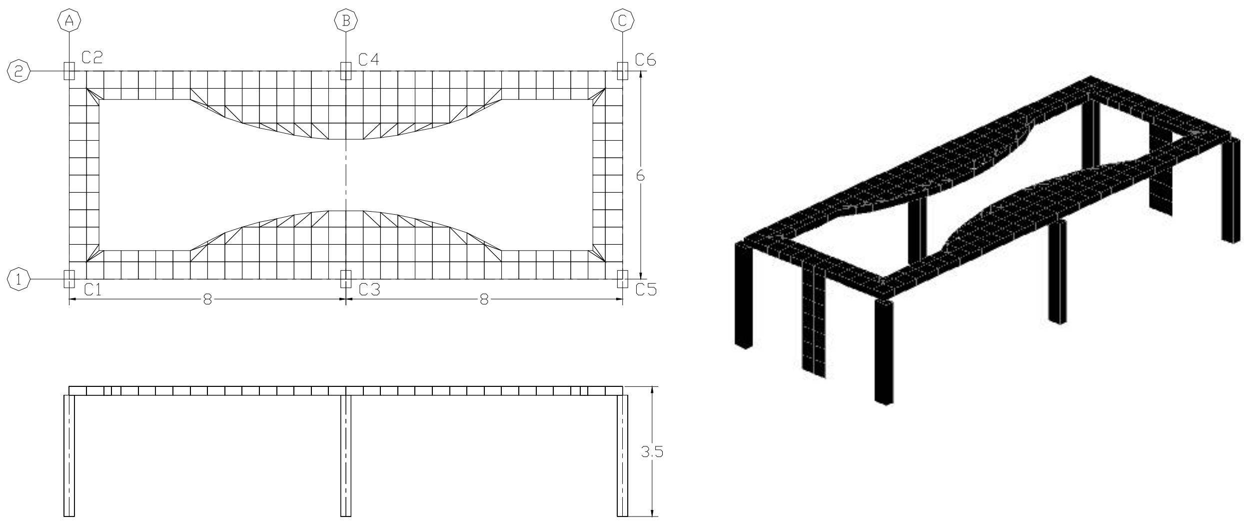

Figure B4: Drawings for S1B2H35O50W1IR

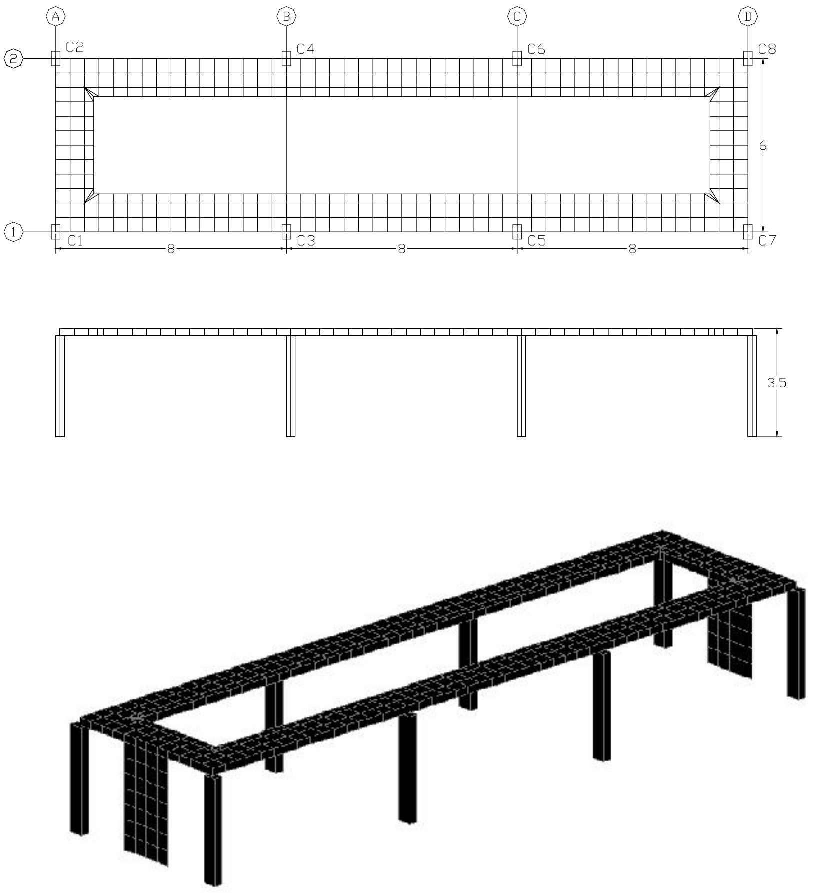

Figure B5: Drawings for S1B3H35O50W2

DECLARATION

I, the undersigned, declare that this thesis is my work and all sources of materials used for the thesis has been duly acknowledged.

Name Kassahun Memru

Signature _______________

Place Addis Ababa University

Institute of Technology

Date of submission July, 2013