NONLINEAR PUSHOVER ANALYSIS OF SEISMIC LOAD ON MULTI-STOREY REINFORCED CONCRETE HOSPITAL BUILDING

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A project report submitted in partial fulfillment of the requirements for the award of Bachelor of Engineering (Hons.) Environmental Engineering

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DECLARATION

I hereby declare that this project is based on my original work except for citations and quotations which have been duly acknowledged. I also declare that it had not been previously and concurrently submitted for any other degree or award at UTAR or other institutions.

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APPROVAL OF SUBMISSION

I certify that this project report entitled **"NONLINEAR PUSHOVER ANALYSIS OF SEISMIC LOAD ON MULTI-STOREY REINFORCED CONCRETE HOSPITAL BUILDING"** was prepared by **SIAH MENG ZHE** has met the required standard for submission in partial fulfillment of the requirements for the award of Bachelor of Engineering (Hons.) Environmental Engineering at Universiti Tunku Abdul Rahman.

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ABSTRACT

Nonlinear pushover analysis is a nonlinear static procedure which is a very useful tool to evaluate the seismic performance of a high-rise building. Malaysia is not situated on actively seismic fault zone, but it is close to plate boundaries and surrounded by highly seismic fault zone countries such as Indonesia and Philippines. Therefore, Malaysia is influenced by both local and global earthquake. However, 50% of selected buildings found are facing concrete deterioration problem caused by seismic force. Institution of Engineers (IEM) Malaysia recommended the adoption of Eurocode to substitute BS 8110 for concrete code of practice in local construction industries. In this research, a 7 story hospital building is modeled and the ground condition is assumed at Ranau, Sabah by SCIA Engineer 15. Nonlinear pushover analysis on SCIA Engineer allows the setting of design parameter by adopting the recommendation of European code. The pushover load is then applied onto the model until the structure reaches its failure point, a pushover curve is plotted by base shear vs. roof displacement to evaluate the seismic performance of this hospital buildings. Furthermore, the study of irregularities of model and the application of braced frame system has been conducted. The results show that negative X direction has the strongest seismic resistance with base shear value 127.90MN. In addition, the existence of irregularity reduces the seismic-resistant capacity of buildings and the braced frame system is capable to improve the seismic performance of buildings.

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LIST OF SYMBOLS/ABBREVATIONS

A_{Ek}	Actual Seismic Situation
a_g/g	Ground acceleration
E	Modulus Elasticity
f_{ck}	Cylinder crushing strength
f_{yk}	Characteristic Strength
H	Story Height
M_s	surface-wave magnitude
q	Behaviour factor
V_b	Base Shear
ß	Beta
γ_I	Important Factor
Δ_a	Allowable Story Displacement

ACI	American Concrete Institute
ACMC	Asian Concrete Model Code
AS	Australia Standard
ASCE	American Society of Civil Engineers
BS	British Standard
EC2	Eurocode 2

EC 8	Eurocode 8
ELF	Equivalent Lateral Force
IEM	The Institution of Engineers Malaysia
ISO	International Organization for Standardization
MRS	Modal Response Spectrum
NEHRP	National Earthquake Hazards Reduction Program
NZS	New Zealand Standard
POA	Pushover Analysis
PL	Permanent Load
RC	Reinforced Concrete
SL	Seismic Load
UBC	Uniform Building Code
VL	Variable Load

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CHAPTER 1

INTRODUCTION

1.1 Overview

High-rise buildings can be classified as residential or commercial. Nowadays, more and more complex high-rise buildings with various architectural feature and style are appearing. The degree of high-rise buildings indicates the economics and technological strength of a country. In South-East Asia countries like Malaysia, most of the cities are dominated by high-rise building because of the growth of economy and population density. The influence of its tallness creates different conditions and difficulties in design, construction and operation. Therefore, a proper understanding of methods and techniques is required of the planning, design, construction and operation. High-rise buildings should be designed to have a capacity to carry combined actions include permanent actions, variable actions and seismic actions at certain safety level and at certain degree of reliability. Therefore, proper account of actions, material properties, structural systems and method of analysis should be considered while designing the high-rise buildings (Safarizki, Kristiawan and Basuki, 2013). The distinct of high-rise buildings and low-rise buildings on the loadings is high-rise buildings is significantly influenced by both gravitational load and the lateral load. Therefore, high-rise buildings are strict to the design code and regulations.

1.2 Background

Earthquake is a sudden shaking of ground due to the interaction of tectonic plates. Three types of interaction cause earthquakes: divergence boundary, convergent boundary and transform boundary. Generally, the intensity of earthquake is determined by using Mercalli Intensity Scale or Richter Magnitude Scale.

Table 1.1: Richter Magnitude Scale and Mercalli Intensity Scale description

Type	Richter Mag.	Mercalli intensity	Average earthquake effects
Micro	< 2.0	I	Micro quakes not felt. Recorded by seismographs.
	2.0–2.9	I to II	Felt slightly by some people. No damage to buildings.
Minor	3.0–3.9	II to IV	Often felt by people, but rarely causes damage; shaking of indoor objects can be noticeable.
Light	4.0–4.9	IV to VI	Felt by most people in the affected area. Slightly felt outside. Noticeable shaking of indoor objects and rattling noises. Causes none to minimal damage. Some objects may fall off shelves or be knocked over.
Moderate	5.0–5.9	VI to VIII	Felt by everyone. Can cause damage of varying severity to poorly constructed buildings; none to slight damage to all other buildings. Casualties range from none to a few.
Strong	6.0–6.9	VII to X	Strong to violent shaking at epicenter. Damage to many buildings in populated areas. Earthquake-resistant structures survive with slight to moderate damage, while poorly-designed structures receive moderate to severe damage. Felt in wide area - up to hundreds of miles/kilometers from the epicenter. Damage can be result far from the epicenter. Death toll ranges from none to 25,000.
Major	7.0–7.9	VIII or greater	Felt over enormous areas. Causes damage to most buildings, some partially or completely collapse or receive severe damage. Well-designed structures receive damage. Death toll ranges from none to 250,000.
Great	8.0–8.9		Felt in extremely large regions. Major damage to buildings, structures destroyed. Moderate to heavy damage to sturdy or earthquake-resistant buildings. Large areas damaged, some totally destroyed. Death toll ranges from 100,000 to one million.
	9.0 and greater		Near or total destruction. Severe damage to or collapse of all buildings. Damage and shaking extends to distant locations. Permanent changes in ground topography. Death toll usually over one million. .

Malaysia is not situated on the high seismic fault zone. It is located close to two plate boundaries: (i) Inter plate boundary between Indo-Australian and Eurasian Plates on the west. (ii) Inter plate boundary between Eurasian and Philippines Plate on the east. West Malaysia is free from the threat of local seismicity and affect by the global seismicity generated from Sumatran Subduction zone and Sumatran fault. In Contrast, East Malaysia is influenced by both local and global seismicity generated from Southern Philippines and the Straits of Macassar, Sulu Sea and Celebes Sea (Rosaidi, 2001). The seismic map of Malaysia is shown in Figure 1.1.

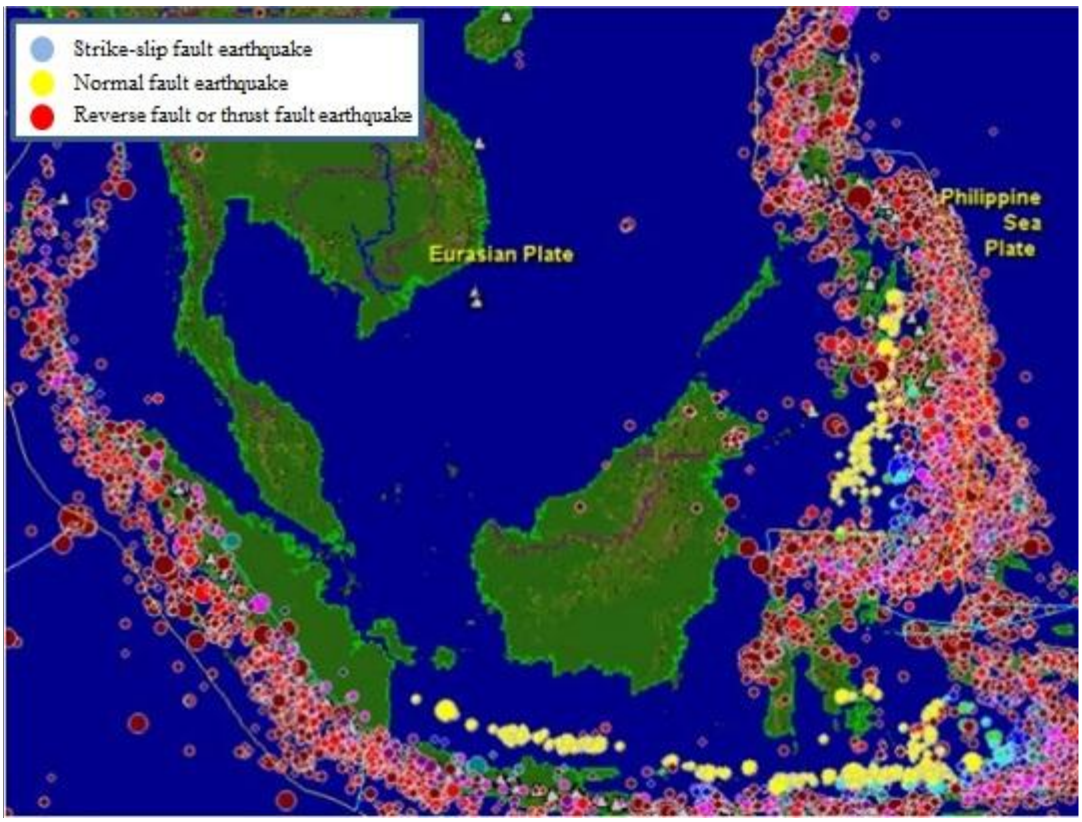


Figure 1.1: Map of seismic surrounding Malaysia

Earthquakes with the epicenter in Sumatra that caused aftershocks in Peninsular Malaysia listed as below:

Table 1.2: The latest Indonesia earthquakes with the epicenter in Sumatra (Syahrum, 2007)

No.	Location	Richter Magnitude Scale	Date of occurrence
1	West Sumatra	>7	March, 2007
2	North Sumatra	8.7	Dec, 2006
3	Sumatra-Andaman Island	9.0	26.Dec, 2006
4	Sumatra	7.4	2.Nov, 2002
5	Bengkulu	6.5	June, 2000

1.3 Problem Statement

In Malaysia, concrete code of practice in local construction industries was fully adopted BS 8110 guideline without any specification of seismic provision. BS 8110 was withdrawn by BSI group by year 2008, no further updates and maintenance for BS 8110 documents would be conducted.

West Malaysia is classified as seismically stable region. It is affected by global seismicity majorly originating from Sumatran plate margin, the maximum intensities observed were VI on the Modified Mercalli (MM) Scale. Nevertheless, West Malaysia is considered seismic vulnerable because vibrations generated from large seismic actions has the potential to threat western part of West Malaysia. In pervious investigation, 50 % of selected buildings in West Malaysia were detected that having concrete deterioration issues due to damages by the vibration during large seismic actions (MOSTI, 2009). In year 2004, a large earthquake which occurred in Sumatra with magnitude 9.0 also generated Indian Ocean tsunami hit along the northwest coastal areas of Peninsular Malaysia (Adiyanto and Majid, 2014).

East Malaysia includes Sabah and Sarawak is affected by both global and local seismicity and thence is classified as moderately active in seismicity. In Sarawak, the maximum intensities observed on MM scale were V, which was a distant seismic action originated from Southern Philippines. Besides, Sarawak has experienced earthquake from its local origin. Sabah is apt to seismic activities than Sarawak and West Malaysia, the maximum intensities observed were VII on MM scale for Sabah where the earthquake hit Lahad Datu and Kunak in year 1976. In 2015, Ranau was hit by an earthquake with magnitude 6.0. This earthquake caused mortality, structural damage of buildings and disruption of water supply in Ranau-Kundasang area (United States Geological Survey, 2015).

1.4 Aims and Objectives

The objectives of the study are shown as following:

- To determine force-displacement relationship (capacity curve) of RC building by using nonlinear pushover analysis.
- To evaluate the influence of structural irregularities towards seismic performance of RC building by nonlinear pushover analysis.
- To determine the enhancement of seismic performance of RC building by the application of frame bracing system.

1.5 Scope of Work

The main objective of this study is to carry out nonlinear static pushover analysis to determine pushover curve (a plot of force vs displacement curve) of the analyzed building. The scopes of work include the modeling and define properties of a hospital building, setting of parameter for nonlinear analysis in accordance of European code, UBC, ASCE standard. Besides, an evaluation of differences in structural irregularities and their influence on seismic vulnerability of the building is conducted.

1.6 Outline of Thesis

This report was organized as follows:

- Chapter 1 presents an introduction of this study.
- Chapter 2 deals with of the study of different type of seismic analysis, fundamental of design consideration for the seismic design of the buildings. The detail of seismic analysis is only focus only European code.
- Chapter 3 presents methodology of pushover analysis to a 7 storey reinforced concrete building to accordance of European code. This chapter defines the material properties of the structural members of the analyzed buildings. In addition, the seismic parameter such as ground acceleration has been set in accordance of European code and UBC, ASCE standards. Besides, different structural irregularities of buildings have been modeled to compare their seismic performance by using nonlinear pushover analysis.
- Chapter 4 presents the capacity curve of the buildings and the influence of structural irregularities upon building based on the result generated according to European code and other standard such as UBC and ASCE standard.

Furthermore, the comparison is made with both models with and without the application of wall bracing system to determine the improvement of the frame bracing system.

- Chapter 5 deals with conclusion and recommendation of this study.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Seismic consideration is required for the design of high-rise structures of seven or above had been mentioned in IEM position document (IEM, 2015). Therefore seismic analysis and design is important and should be considered in Malaysia for safety of structures. Most of the Malaysia buildings are designed according to BS8110 without any consideration of seismic effects. Nowadays, the buildings had been designed in accordance of European code for seismic design (Adiyanto and Majid, 2014).

2.2 Modeling by Finite Element Programs

Some international finite element programs such as ETABS, SAP2000 and SCIA engineer are used for structural analysis. The revision of seismic design codes is made due to the new findings in research. Hence, the result generated from seismic analysis among these programs might have some difference due to their respective limitation and setting.

2.2.1 ETABS

ETABS is developed specifically for building systems and structural analysis. Modeling of simple buildings through ETABS can be simple and time-saving. ETABS can also carry out the structural analysis of complex building model. In additions, many design codes are available in ETABS such as European code, Chinese code, U.S. code and so on. Users can specify the elements or parameters for desired structural analysis in accordance of desired code, response spectrum analysis, nonlinear static analysis, time history analysis are all available in ETABS. Output of analysis may be viewed graphically, tabular output and more. Since ETABS is mainly for designing structures, the analysis options for ETABS are limited (CSI, 2003).

2.2.2 SAP 2000

SAP 2000 provides structural system analysis for users with option to create modify design and analyze structure models. The latest version of SAP2000 is available in three different levels: SAP2000 Basic, SAP2000 PLUS & SAP2000 Advanced. All of these programs are capable for fast equation solvers, etc (CSI, 2003).

2.2.3 Scia Engineer

Scia Engineer is capable for structural modeling, analysis, drawing and design of structure in 1D, 2D and 3D format. This program supports U.S. code, Eurocode and other international standard. A wide variety of structural analysis can be performed such as static, dynamic, stability, linear, nonlinear and other types of analysis, optimization functions of this program helps to find the optimum design variant of structure.

2.3 Properties of Reinforced Concrete

2.3.1 Plain Concrete

Plain concrete is a constituent of aggregate, cement and water with a specific portion, it gains strength after curing. Plain concrete strength and durability is significantly influenced by water-cement ratio: low water-cement ratio leads to high durability and strength but decreases the workability of mix and form of concrete (Rebelo, 2014).

2.3.2 Reinforced Concrete (RC)

Plain concrete has a relatively low tensile strength: tensile strength in concrete is 10% - 20% of its compressive strength. Thus, when steel is reinforced into concrete enables the increasing of tensile strength and partially of the shear strength while the concrete is able to safeguard the steel by providing durability and fire resistance (Mosley, Bungey & Hulse, 2012).

Table 2.1: Properties of concrete and steel (Source: adapted from Mosley, Bungey & Hulse, 2012. p.6)

Material	Concrete	Steel
Tensile strength	Weak	Strong
Compressive strength	Strong	Strong, but slender steel will buckle
Shear strength	Fairly strong	Strong
Durability	Good	Corroded if unprotected
Fire resistance	Good	Poor, suffers rapid loss of strength at high temperatures.

2.3.3 Shrinkage and Creep

Shrinkage is the slowly deformation of concrete in time without any applied loads when the temperature is constant. Concrete has two sorts of shrinkage which are plastic shrinkage and drying shrinkage. Plastic shrinkage happens while the concrete is in plastic condition; it is because of the sudden loss of water from the concrete surface. Drying shrinkages occurs while concrete is hardened due to drying out persists over many months, drying shrinkage causes the volume reduction of concrete which is irreversible. A low w/c ratio helps to minimize the volume of moisture can be lost in hardened concrete in order to reducing of drying shrinkages (Bazant and Wittmann, 1982).

Creep occurs due to a successive deformation of concrete under sustained loading condition. Basically, creep is capable to reduce the tendency of cracking in restrained concrete members by relieving the stress due to shrinkage. However, the influence of creep is significant in beams, as the incrementing of deflection might create cracks' openings. Therefore, the reinforcement of compression zone of a flexural member can help to restrain the deflection by creeping (Mosley, Bungey & Hulse, 2012).

2.3.4 Durability

Durability indicates as the capability of concrete to resist any kinds of events such as fire, weathering, etc. The durability of concrete is affected by the conditions of exposure, concrete quality, type of cement used, cover of reinforcement and the crack width. Concrete has the possibility to be exposed to a wide variety of status such as subsurface, offshore, stored chemical or cold weather.

In order to enhance the durability of concrete, a densely, well compacted concrete is used because it has low permeability and fireproofing. In freeze condition, a concrete which has a water-cement ratio of 0.4 or below can withstand longer than concrete has a water-cement ratio of 0.5 or higher (Khoshakhlagh, 2011). In addition, adequate cover is provided for rebar is to prevent the corrosion of steel bar by the corrosive agents through the concrete cracks. The minimum concrete mix and cover is recommended in EC2 for different exposure conditions.

2.4 Structural Model & Configuration

2.4.1 Structural Irregularities

Seismic performance of buildings and non-building structures is influenced by its plan and elevation. The irregularities of a buildings or non-building structures can be classified to plan irregularity and vertical irregularity. Plan irregularity consists of torsional irregularity, re-entrant corner, diaphragm discontinuity, out-of-plane offsets and non-parallel systems. Vertical irregularity consists of mass irregularity, stiffness irregularity, vertical geometric irregularity, discontinuity in capacity and plan.

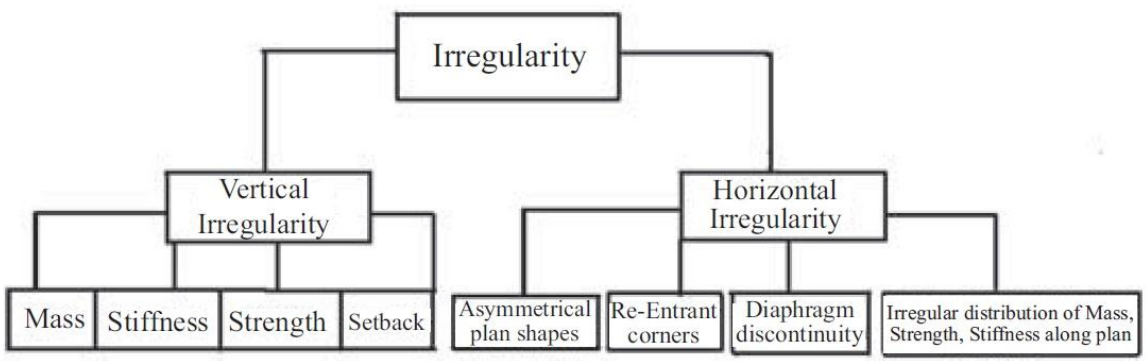


Figure 2.1: Type of Irregularity (Varadharajan, Sehgal and Saini, 2013)

Many researches regarding irregularities and its effects of seismic vulnerability have been carried out:

Sameer and Gore (2016) researched about the effects of plan irregularity by comparing symmetrical and asymmetrical models in terms of base shear, story drift and displacement, research found that the configuration of plan of model has the significant effects on the seismic response of model (Sameer and Gore, 2016).

Arun and Hemalatha (2013) researched about the limitation of irregular structure by two models: too tall model and too long model to study their seismic response. This research found that the aspect ratio (height/length for tall models or length/height for long models) of models that exceed than 5.6 does not meet the seismic performance limit (Arun and Hemalatha, 2013).

Bansal and Gagandeep (2014) researched about the effects of vertical mass irregularity of RC building frames by comparing both mass regular and mass irregular model. Results found that mass irregular RC building frame tends to experience more shear force than mass regular RC building frame (Bansal and Gagandeep, 2014).

Ahmed and Raza (2014) researched about the effects of plan irregularity towards the seismic response of a structure by three different models: Y-shape, rectangular shape and diaphragm discontinuity in terms of base shear and lateral displacement. Research found that regular structure (rectangular model) is more seismically vulnerable than irregular structure (Y-shape model and diaphragm discontinuity model),

2.4.2 Strengthening Model

Lateral forces such as wind and seismic force are developing high stresses, causing vibration and producing sway movement of a structure. Therefore, it is a crucial part for a building having adequate strength to resist lateral forces. The strengthening strategy can be either the application of braced frame system or shear walls (Kevadkar and Kodag, 2013).

2.4.2.1 Braced Frame System

Braced frame system has same functions with trusses which act as compression and tension member of a structure. This system enhances a structure to resist more lateral forces originated from wind or seismic force, a braced frame system can be RC braced frame or steel braced frame (Kevadkar and Kodag, 2013).

Braced frame system can be classified to diagonal bracing, K-bracing and eccentric bracing as shown in Figure 2.2. Diagonal bracing is the most efficient bracing system as it is fully resisting lateral loads while the columns and beams only resist gravity loads. K bracing and eccentric bracing are applied when the opening is necessary to be provided for windows or doors, these two bracing are resisting partially of the lateral loads which is less efficient as compared to diagonal bracing (Siddiqi, Hameed and Akmal, 2014).

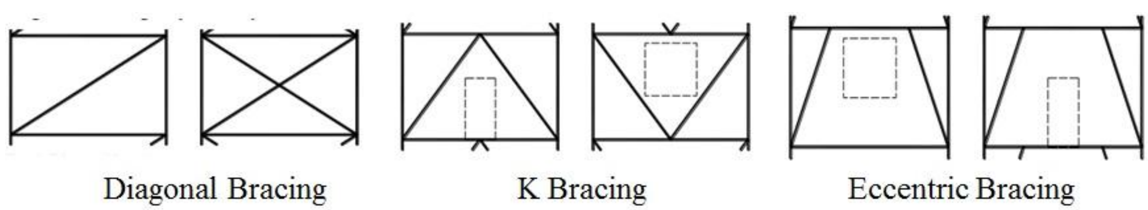


Figure 2.2: Types of Bracing System

Some researches regarding to the seismic response of buildings with braced frame system are listed as below:

Kulkarni, Kore and Tanawade (2013) researched about the seismic response among models with bare braced frame, fully braced frame, optimal braced frame and outrigger frame. The results are shown in Figure 2.3: braced frame system enables the reduction of lateral displacement when subjected to lateral load as compared to bare braced frame system.

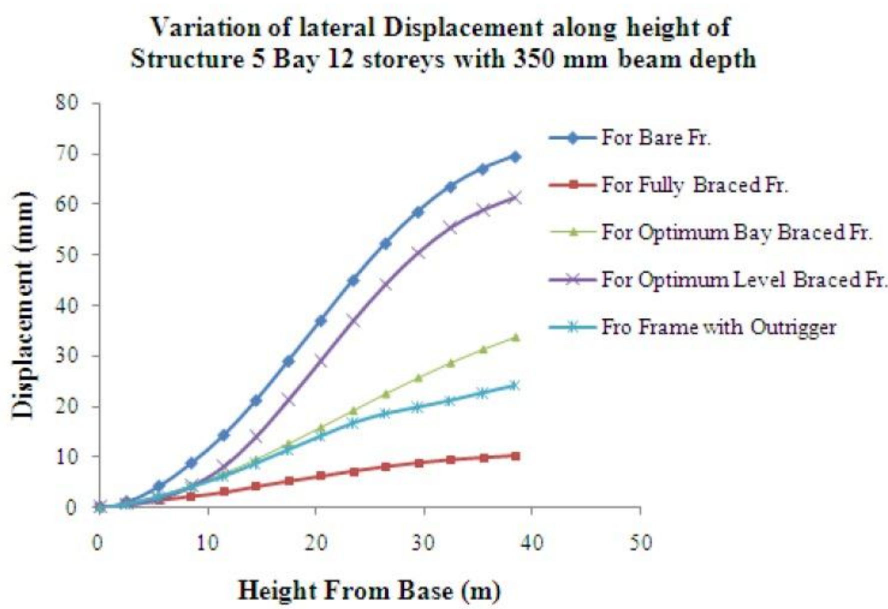


Figure 2.3: A comparison among bare braced frame and other type of braced frame in terms of lateral displacement of the model

Siddiqi, Hameed and Akmal (2014) researched about the efficiency of different types of braced frame system in resisting lateral loads. The types of braced frame include single & double diagonal bracing, K & V bracing and Eccentric bracing. Research found that double diagonal bracing is the most efficient in resisting lateral loads at the same time it yields minimum weight to the structure itself (Siddiqi, Hameed and Akmal, 2014).

2.4.2.2 Shear Wall

Shear Wall is a vertical plate of RC wall on tall buildings applied to resist lateral loads from wind and seismic action, lateral force will be resisted by beams and columns for buildings without shear wall. Hence, the primarily objective of shear wall is to increase the rigidity of the lateral load resistance of buildings. Designing of shear wall should be considering the center of mass, stiffness and strength (Itware and Kalwane, 2015).

Sardar and Karadi (2015) researched about the effect of changes location of shear wall with a subjected lateral load: a comparison among several model has made as shown in Figure 2.4. Research found that the presence of shear wall would increase the stiffness and strength of buildings to become a more seismic-resistant structure (Sardar and Karadi, 2015).

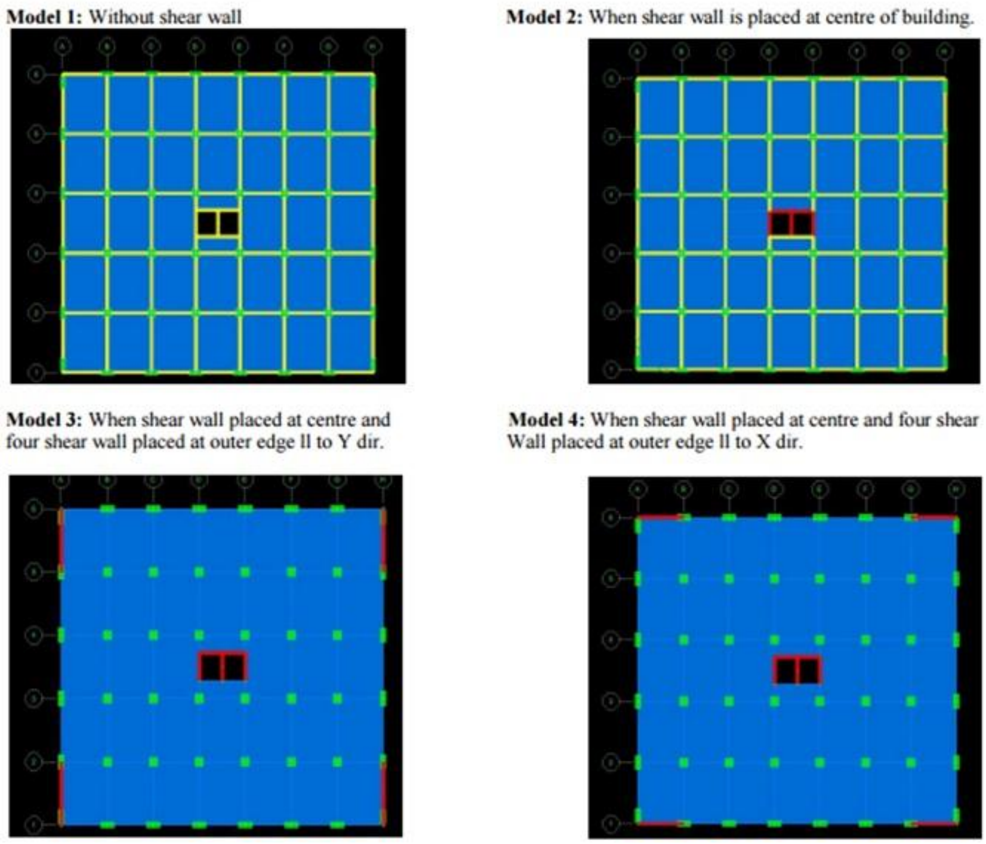


Figure 2.4: A study of changes in shear wall location on story drift subjected to a lateral load (Sardar and Karadi, 2015).

2.5 Current State of Structural Design in Malaysia

BS 8110 had been widely used in local construction industries in Malaysia before year 2008. In year 2008, the BS8110 had been withdrawn by BSI group. Therefore, The Institution of Engineers, Malaysia (IEM) recommended the adoption of Eurocode 2 (EC2) to substitute BS 8110 for the concrete code of practice in local construction industries (IEM, 2016).

The Committee has conducted a study of EC2, AS 3600, NZS 3110, ACMC 2001 and ACI 318. Besides EC2, other codes of practices are inappropriate for Malaysia to adopt because: (i) AS 3600's compliance to ISO is not confirmed and not widely adopted worldwide. (ii) ACI 318 is commonly used in North America only and many formulae are based on imperial units. (iii) ACMC 2001 not completed, still under development. (iv) NZS 3110 similar situation to AS 3600 (IEM, 2016).

EC2 consists of some features such as National Annex which includes the special considerations of other countries such as shrinkage or creep of concrete components especially in hot and humid Malaysian climates. Since EC2 documents would have regular maintenance, adopting EC2 would be able to get updates of latest concrete technology (IEM, 2016).

2.6 Seismic Performance Requirement

The primary purpose of all seismic codes is to protect life and human properties. However, different code provisions has its particular requirement shall be fulfilled. The main seismic codes consist of Eurocode 8 (EC8), Uniform Building Code (UBC), American Society of Civil Engineers 7 (ASCE 7) and National Earthquake Hazards Reduction Program –NEHRP (McIntosh and Pezeshk, 2016).

EC8 primary objective is to protect the human lives and minimize the damage from a seismic action, and structures are important for public remain operational. Thus, EC8 provides for a two level seismic design: No-collapse and Damage Limitation. For No-collapse requirement: the design and construction of buildings shall be able to prevent any local or global collapse. However, its repair might be uneconomical. The second level of design is damage limitation refer to the reduction of property loss. Structure retain itself full strength and stiffness under frequent seismic actions. The damage of structure can be repaired easily and economically (Fardis, et al., 2005).

UBC 1997 is the complete version of building code. The aim of seismic provision is to potect the life or limb, health, public welfare and property by controlling and regulating the design, construction, use and occupancy, materials used and maintenance of all buildings and structures. Additionally, the design and construction of buildings and structures shall fulfill the minimum requirement of the UBC to resist the effects of seismic ground motion (UBC, 1997).

ASCE 7-10 provides the design procedures of buildings structures and other components. Buildings and structures shall consist of lateral load and gravitational load resisting systems which contribute sufficient stiffness and strength to resist the design seismic ground motion. This provision states the specification of the current material design e.g. steel, wood, concrete, the other ordinary structural materials used for construction (ASCE 7-10, 2010).

NEHRP is imitated as a reference instead of a seismic code. The target of NEHRP is to effectively reduce the seismic effects of life and prevent the collapse for buildings, the improvement of seismic performance and strengthen the expected capabilities for all buildings and non-building facilities to withstand during seismic ground motion (NEHRP, 1994).

2.7 Methods of Analysis

2.7.1 Linear Seismic Analysis

Linear seismic analysis concerns the linearity of structures; structural linearity can be classified to material linearity and geometric linearity. Material linearity can be described by Hooke's Law: the linear relationship between stress and strain, which is only valid for a certain value called proportionality limit (Lautrup, 2011).

2.7.1.1 Equivalent Lateral Force Method

Equivalent Lateral Force **(**ELF) Method is a linear static analysis of structure and performed under a set of lateral forces. The set of lateral force applies separately in both direction X and Y of the structures. The peak inertia loads induced by horizontal component of seismic action through the set of lateral forces is simulated either in direction X or Y (Fardis et al, 2005).

The procedure of ELF method has three major procedures: (1) Determine the seismic base shear. (2) Distribute the shear vertically along the height of the structure. (3) Distribute the shear horizontally across the width and breadth of the structure. The seismic base shear for each horizontal direction can be expressed by the following equation:

$$F_b = \lambda m S_d(T_1) \tag{2.1}$$

λm refers the effective modal mass of the first mode, λ is fraction of total mass, m, of the buildings above the foundation or above the top of the rigid basement. The introduction of λ factor is to bring the result of ELF method closer to modal response spectrum analysis. $S_d(T_1)$ is the design spectrum value at the fundamental period T_1. The estimation of fundamental period T_1 is provided by Rayleigh quotient:

$$T_1 = 2\pi \sqrt{\frac{\sum_i m_i \delta_i^2}{\sum F_i \delta_i}} \tag{2.2}$$

δ_i represents the displacement of masses caused by horizontal force F_i, m_i is the mass of the storey i. EC8 allows the empirical expressions of value T_1:

$T_1 = 0.085H^{3/4}$	(For steel moment frame buildings less than 40 m tall)	(2.3)
$T_1 = 0.075H^{3/4}$	(For buildings less than 40m tall with concrete frames or with steel frames with eccentric bracings)	(2.4)
$T_1 = 0.05H^{3/4}$	(For buildings less than 40m tall with any other type of structural system including concrete wall buildings)	(2.5)

However, equation 2.2 provides a more accurate fundamental period T_1 than empirical equation.

The peak base shear, F_b is translated to a set of lateral inertia forces in the same direction by Eq.2.6:

$$F_i = F_b \frac{\Phi_i m_i}{\sum_j \Phi_j m_j} \tag{2.6}$$

m_i (m_j) are the storey masses, Φ_i (Φ_j) represent the elevation of masses above the basement level. Commonly, the lateral load pattern of F_i is termed by inverted triangle.

The accuracy of ELF methods is affected by structural irregularity and its inelastic behavior. Minor irregularity of mass or stiffness over the height provides high accuracy of the lateral force distribution by ELF procedure (BSSC 2003). In ELF method, the level of inelastic behavior is not accounted for in the distribution of lateral loads.

2.7.1.2 Modal Response Spectrum Analysis

Modal response spectrum (MRS) analysis is a linear dynamic analysis. The objective of MRS analysis is to estimate the peak value of the seismic action. Even though the structure is analyzed for two horizontal directions, X and Y. MRS Analysis are preferable on a completed 3D structural model. The first step of MRS analysis is the determination of natural frequencies and vibration (Fardis et al, 2005).

The result of eigenmode-eigenvalue analysis is for subsequent estimation of the peak elastic response on the basis response spectra in three direction, X,Y and Z. The parameter for each normal mode n is shown as below:

- The natural circular frequency, ω_n, and the corresponding natural period

$$T_n = 2\pi/\omega_n \tag{2.7}$$

- The mode shape, represented by vector Φ_n.
- The modal participation factor represented by Γ_{Xn}, Γ_{Yn}, Γ_{Zn} in response to the component of the seismic action in direction X,Y or Z, computed as:

$$\Gamma_{Xn} = \frac{\Phi_n^T M I_X}{\Phi_n^T M \Phi_n} = \frac{\sum_i \varphi_{Xi,n}\, m_{Xi}}{\sum_i (\varphi_{Xi,n}^2 m_{Xi} + \varphi_{Yi,n}^2 m_{Yi} + \varphi_{Zi,n}^2 m_{Zi})} \tag{2.8}$$

$$\Gamma_{Yn} = \frac{\Phi_n^T M I_Y}{\Phi_n^T M \Phi_n} = \frac{\sum_i \varphi_{Yi,n}\, m_{Yi}}{\sum_i (\varphi_{Xi,n}^2 m_{Xi} + \varphi_{Yi,n}^2 m_{Yi} + \varphi_{Zi,n}^2 m_{Zi})} \tag{2.9}$$

$$\Gamma_{Zn} = \frac{\Phi_n^T M I_Z}{\Phi_n^T M \Phi_n} = \frac{\sum_i \varphi_{Zi,n}\, m_{Zi}}{\sum_i (\varphi_{Xi,n}^2 m_{Xi} + \varphi_{Yi,n}^2 m_{Yi} + \varphi_{Zi,n}^2 m_{Zi})} \tag{2.10}$$

i The nodes of the structure associated with dynamic degrees of freedom

M : Mass matrix

I_X : A vector with element equal to 1 for translational degrees of degrees of freedom parallel ro direction X and with all element equal to zero, similarly to

I_y and I_z

$\varphi_{Xi,n}$: The element of Φ_n corresponding to the translational degree of freedom of node I parallel to direction X, similarly to $\varphi_{Yi,n}$ & $\varphi_{Zi,n}$

m_{Xi} : Associated element of mass matrix, similarly to m_{Yi}& m_{Zi}

The effective modal masses in direction X, Y and Z represented by M_{Xn}, M_{Yn}, M_{zn} respectively, computed as

$$M_{Xn} = \frac{(\Phi_n^T M I_X)^2}{\Phi_n^T M \Phi_n} = \frac{(\sum_i \varphi_{Xi,n}\, m_{Xi})^2}{\sum_i(\varphi_{Xi,n}^2 m_{Xi} + \varphi_{Yi,n}^2 m_{Yi} + \varphi_{Zi,n}^2 m_{Zi})} \qquad (2.11)$$

$$M_{Yn} = \frac{(\Phi_n^T M I_Y)^2}{\Phi_n^T M \Phi_n} = \frac{(\sum_i \varphi_{Yi,n}\, m_{Yi})^2}{\sum_i(\varphi_{Xi,n}^2 m_{Xi} + \varphi_{Yi,n}^2 m_{Yi} + \varphi_{Zi,n}^2 m_{Zi})} \qquad (2.12)$$

$$M_{Zn} = \frac{(\Phi_n^T M I_Z)^2}{\Phi_n^T M \Phi_n} = \frac{(\sum_i \varphi_{Zi,n}\, m_{Zi})^2}{\sum_i(\varphi_{Xi,n}^2 m_{Xi} + \varphi_{Yi,n}^2 m_{Yi} + \varphi_{Zi,n}^2 m_{Zi})} \qquad (2.13)$$

These are essentially base-shear-effective modal masses because the reaction force (base shear) in direction X,Y or Z due to mode n are equal to:

$$F_{bX,n} = S_a(T_n) M_{Xn} \qquad (2.14)$$

$$F_{bY,n} = S_a(T_n) M_{Yn} \qquad (2.15)$$

$$F_{bZ,n} = S_a(T_n) M_{Zn} \qquad (2.16)$$

The minimum number n of modes to be taken account should be at least equal to $3\sqrt{n_{st}}$ where n_{st} is the number of storey. The combination of modal responses in all directions, the maximum value of E_E of seismic action effect may be taken as

$$E_E = \sqrt{\sum_N E_{Ei}^2} \ (Square\ Root\ of\ Sum\ of\ Square(SRSS)\ rule) \qquad (2.17)$$

Where the summation extends over the N modes taken into account and E_i is the peak value of seismic action effect due to vibration mode i.

However, EC8 recommend the complete quadratic combination (CQC rule) as an example, the maximum value E_E of a seismic action effect may be taken as

$$E_E = \sqrt{\sum_{i=1}^{N} \sum_{j=1}^{N} r_{ij} E_{Ei} E_{Ej}} \ (CQC\ rule) \tag{2.18}$$

r_{ij} is the correlation coefficient of mode i & j. In comparison, CQC rule provides more accurate result.

2.7.2 Nonlinear Seismic Analysis

The inelastic seismic response is quite important when structure is subjected to huge seismic action, are expected to result in deformation beyond the limit point of elastic behavior. The primary application of non-linear methods of analysis is to evaluate the seismic performance of new designs, assess existing or retrofitted buildings (Stana, 2014). Nonlinear structural behavior may consist of geometry nonlinearity, material nonlinearity and a combination of both.

Geometry Nonlinearity concerns the P-Δ effects: the 2nd order effect of gravity load acting on a laterally deformed structure. Gravity loading may influence the structural response under a significant lateral displacement. For a well-designed structure, the changes in displacements and member forces with the consideration P-Δ effects compared to the changes that not including P-Δ effects must be less than 10%. In severe case, P-Δ effects can contribute to dynamic instability and loss of lateral resistance (Deierlein et al. 2010).

Material Nonlinearity concerns the inelastic behavior of structure. Inelastic behavior may described by a force-deformation (F-D) relationship (the measurement of strength vs. translational or rotational deformation. Once structure reaches its yield strength, nonlinear response may increase to an ultimate point before degrading to a residual strength value (Napier, 2014).

2.7.2.1 Pushover Analysis

Pushover analysis (POA) is a nonlinear-static approach carried out under constant gravity loads and monotonically increasing lateral forces, applied at the location of masses in the structural model to simulate the inertia force induced by a single horizontal component of the seismic action. POA can describe the plastic mechanism(s) and structural damage because the applied lateral force increase monotonically but are not fixed. POA is the extension of ELF method of linear analysis into the non-linear regime. Thus, it addresses only horizontal component of seismic action.

The first step of POA is to suppose a certain lateral load pattern. This purpose is to represent all the forces which are produced when the structured model is subjected to seismic action. For POA the following lateral load patterns have been used:

1. Uniform Pattern: Lateral load proportional to the masses at all elevations.

$$F_i = W_i \tag{2.19}$$

2. Modal Pattern: lateral forces proportional to the product of the mass matrix by the relevant modal vector.

$$F_i = W_i \phi_{ij} \tag{2.20}$$

W_i : Weight of the 'i' storey

Φ_{ij} : The *ith* element of the mode shape vector corresponding to the 'i' storey for mode.

3. Inverted Triangular Pattern

$$F_i = \frac{W_i h_i}{\sum_{j=1}^{n} W_j h_j} V_b \tag{2.21}$$

$$V_b = S_d(T_n)W \tag{2.22}$$

$S_d(T_n)$: The acceleration ordinate of the design spectrum at the fundamental period T_n.

W: Total weight of the structure.

4. FEMA Load Pattern: Lateral load pattern suggested by FEMA is similar to inverted triangular pattern. FEMA introduced coefficient k to the formula (FEMA, 2000).

$$F_i = \frac{W_i h_i^k}{\sum_{j=1}^{n} W_j h_j^k} V_b \tag{2.23}$$

k: Coefficient dependent on the fundamental period T_n of the structure. The value of k is between 1 and 2. The effect of value k is shown as below:

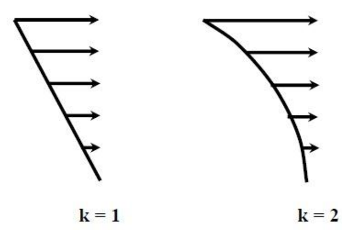


Figure 2.5: k value of lateral load pattern

5. Kunnath's Load Pattern (Kunnath, 2004)

$$F_i = \sum_{j=1}^{n} a_{mr} \Gamma_j M_i \varphi_{ij} S_a(\zeta_j, T_j) \tag{2.24}$$

a_{mr}: Modification factor that control relative effects of each included mode, the value can be positive or negative.

Γ_j: Participation factor for mode j.

M_i: Mass of the i*th*-storey

j_{ij}: Mode shape of the i*th*-storey for mode j

$S_a(\zeta_j,T_j)$: Spectral acceleration for a given earthquake loading at frequency corresponding to the period *T* and damping ratio z for mode *j*.

EC8 recommended adopting both of the standard lateral force patterns: uniform pattern and modal pattern to perform POA, each lateral force should be applied in both positive and negative direction and the POA result to be used should be the most unfavorable one from the two analyzes. A key element of POA is the "capacity curve". Capacity curve shows the relationship between base shear, F_b, and a representative lateral displacement of the structure, d_n, it shows a determination of collapse load and ductility capacity of the structure (Fardis et al, 2005).

2.7.2.2 Nonlinear Time-history Analysis

Time-history analysis is nonlinear dynamic analysis method was developed for research, code calibration or other special purposes. The limitation of nonlinear dynamic analysis lack of wide availability of some reliable and numerically stable computer programs with non-linear dynamic analysis capacities, thus this time-history method needs the others approaches such as the conventional force-based design that uses q factor and linear analysis to complete the analysis (Filippou, D'Ambrisi and Issa,1992).

The selection of time step size is a crucial step of time-history analysis. Time step size affects the accuracy, stability and the rate of convergence of the analysis. 3 dimensional models are preferred for the accurate description of the time history response. Dynamic equilibrium is applied which take into account of mass, damping and stiffness of model.

$$[M]\ddot{r} + [C]\dot{r} + [K]r = P \tag{2.25}$$

$\ddot{r}$ is the relative acceleration, $\dot{r}$ is the relative velocity, r is the relative displacement, P represents the external loading. In the case of seismic loading, the external loading can be expressed as:

$$P = -[M]\ddot{a_g} \tag{2.26}$$

$\ddot{a_g}$ is the ground acceleration of. [M], [C], [K] represent mass matrix, damping matrix and stiffness matrix respectively.

The incremental equations of motion can be expressed by the equilibrium of force increments during a time step:

$$[M]\Delta\ddot{r} + [C]\Delta\dot{r} + [K]\Delta r = \Delta P \tag{2.27}$$

$\Delta\ddot{r}, \Delta\dot{r}, \Delta r$ represent the increment of acceleration, velocity and displacement vector during the time step , Δt respectively.

2.7.3 Seismic Study in Malaysia

Seismic study in Malaysia has become more popular to both Malaysian and foreign researchers. Those studies is conducted in accordance of Malaysia seismicity include evaluation of seismic analysis, the seismic performance of buildings and non-building structure such as bridge.

The following literatures are completed according to Malaysia seismic situation:

Ramli and Adnan (2014) studied the seismic performance of a bridge structure in East Malaysia under shallow crustal zone and Sulawesi subduction zone by using MRS analysis to simulate the seismic motion: the study concluded that MRS analysis is the most appropriate method for the seismic analysis of bridge structure (Ramli and Adnan, 2014).

Syahrum (2007) researched about the seismic analysis and design of residential buildings by adoption of Indonesian Code. The seismic force is calculated by using ELF method, the seismic analysis and design consider 1, 2, 3 and 4 story low rise building (Syahrum, 2007).

Sooria, Sawada and Goto (2012) proposed for the seismic resistant design according to Malaysian seismicity. The study is only focus about the understanding of attenuation characteristic of seismic ground motion in Malaysia by determine the peak ground acceleration and peak ground velocity (Sooria, Sawada and Goto, 2012).

Zulkefli (2010) researched about the seismic analysis of a bridge structure by Time History analysis. The comparison between conventional bridge and integral bridge has been done by the observation by the girder response in term of shear force and moment capacities (Zulkefli, 2010).

Adiyanto and Majid (2014) studied the seismic design of two storey building with low class ductility by comparing two design codes: BS 8110 and EC 2. Nonlinear time history analysis is used to analyze the seismic response of the buildings. This study also compares the total cost required for the seismic design of low ductility of buildings: re-designed the frame but considering other ductility class specified in EC 8 (Adiyanto and Majid, 2014).

2.8 Critical Review

Malaysia is located at seismically stable zone. However, severe damage incurred not necessary caused by the physical size of an earthquake but also on other factors such as the location and time of an earthquake occur, the population density of specified area and secondary event such as fire. Some important infrastructure must be designed and constructed as earthquake-resistant structure in order to function well during seismic events.

In this research, a 7 storey hospital building is assumed to be located at Ranau, Sabah, East Malaysia. Nonlinear pushover analysis is carried out to determine the failure mechanism of this hospital building. The design parameter is according to the site condition and the recommendation by Institution of Engineers, Malaysia (IEM). Besides, a study of the effects of plan and elevation irregularities has been conducted to understand its influences to seismic response of hospital buildings. Additionally, the improvement of seismic performance of buildings by the application of bracing frame system has been carried out to compare the results of the plain buildings (without bracing system).

2.9 Summary

Linear or nonlinear analysis may be static or dynamic. The analysis methods are presented as below:

Table 2.2: Summary of seismic analysis

Type of Analysis	Static	Dynamic
Linear	Equivalent Lateral Force Analysis	Modal Response Spectrum Analysis
Nonlinear	Pushover Analysis	Time-history Analysis

Wind or seismic force acts laterally on buildings, the lateral load is insignificant in low to medium-rise buildings. Therefore, linear analysis such as ELF analysis or MRS analysis is adequate for low to medium-rise buildings. However, the lateral force becomes significance on high-rise buildings become of its tallness. Nonlinear analysis such as pushover analysis and nonlinear time-history analysis are preferable to be carried out when analyzing high-rise buildings because of the accountable for nonlinear properties and failure mechanism can be achieved with its important parameters for designing a high-rise building.

CHAPTER 3

RESEARCH METHODOLOGY

3.1 Introduction

Seismic analysis can be conducted by finite element based structural programs. These programs are capable for modeling, defining materials and section properties, setting load cases and load combinations and design. Most of the programs are updated for a wide availability of national and international design code and standard such as U.S. code, British Standard (BS), Eurocode (EC), etc.

3.2 Modeling

A 3D model of 7 storey hospital building is considered in this study, ground support condition of the model is hinged support and soil condition is assumed as very dense sand and gravel with slightly silt.

Table 3.1: *In situ* soil characteristic

Soil Type	very dense sand and gravel with slightly silt
Specific Weight (kg/m^3)	1800
Friction angle (degree)	35
Gamma unsaturated (kN/m^3)	19
Gamma saturated (kN/m^3)	21

The height of ground floor is 5m from the base and height for each storey is 3m. The dimension of main block is 35m x 35m, a 5m x 10m opening can be observed through to plan of building for air ventilation. Additionally, an extension of block starts from 1st floor and it is supported by external columns. The dimension of extension block is 15m x 15m.

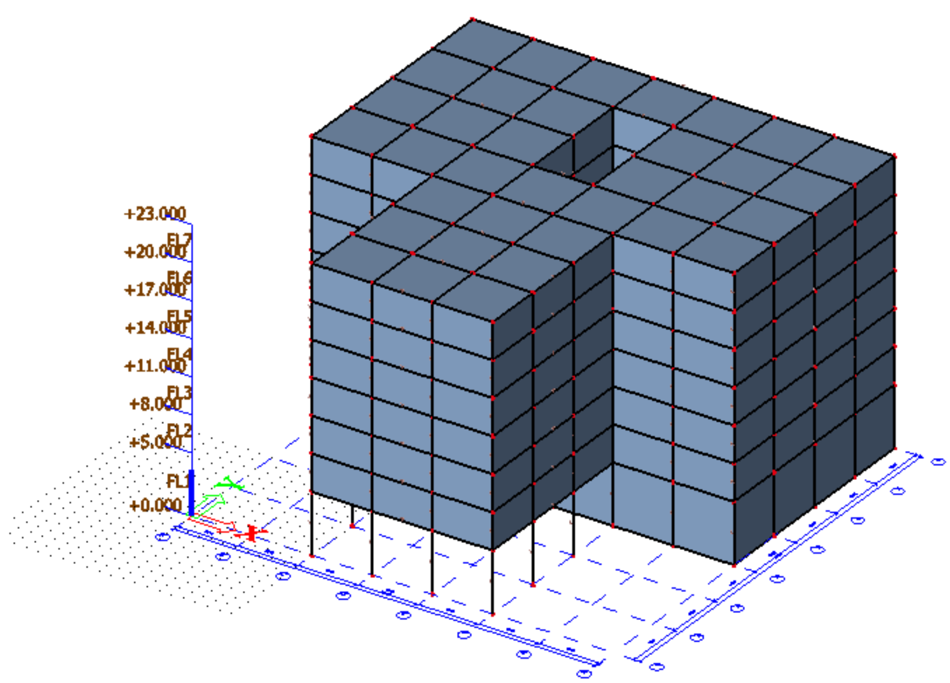


Figure 3.1: Three dimensional view of model

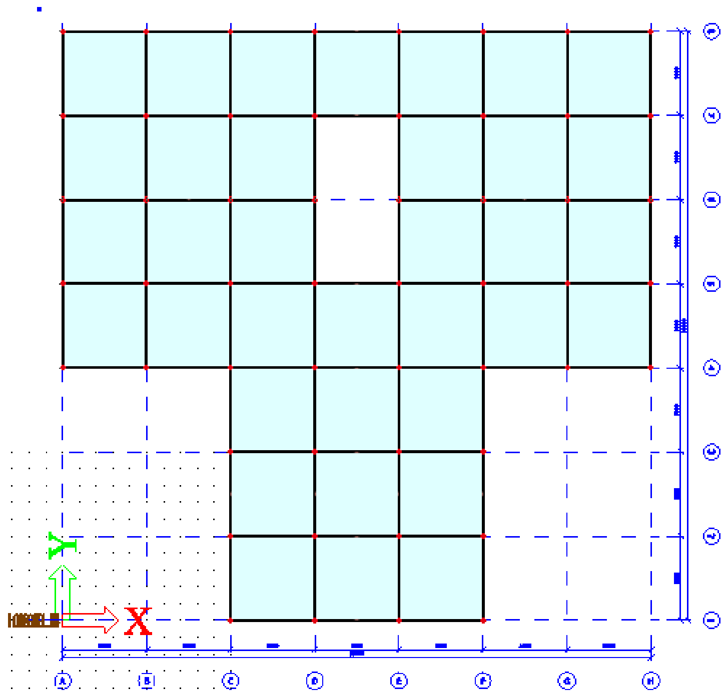


Figure 3.2: Plan view of model

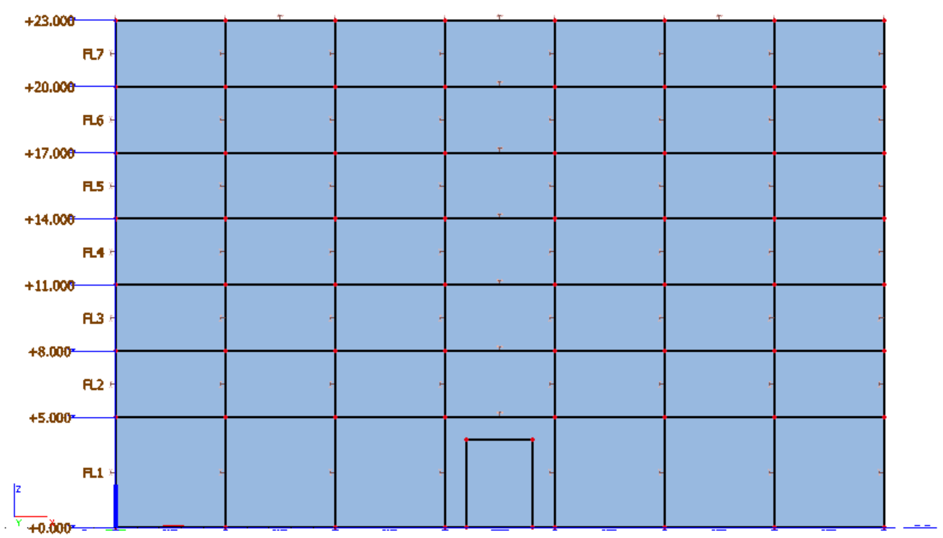


Figure 3.3: Front elevation of model

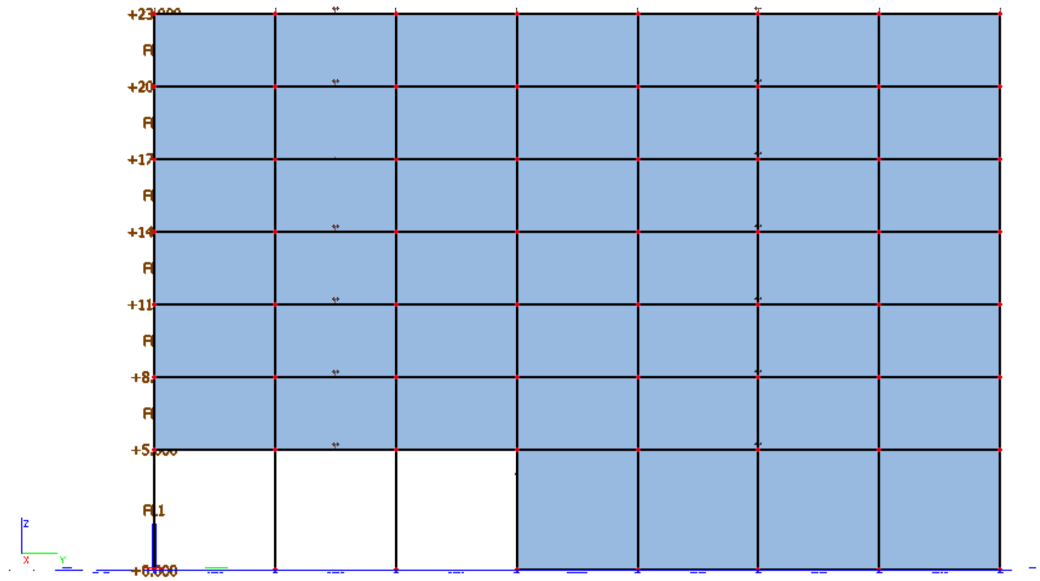


Figure 3.4: Side elevation of model

3.3 Properties

3.3.1 Materials

The structural frame is a reinforced concrete building; C25/30 grade concrete is applied for structural elements include slabs and beams. C30/37 grade concrete is applied for columns and walls. The rebar adopted for reinforcement is grade 500. The Poisson's ratio of reinforced concrete is 0.2 (Mosley, Hulse and Bungey, 2012).

Table 3.2: Material properties

1. Concrete	
Modulus elasticity (E)	31000kN/mm^2
Specific weight	25kN/m^3
Poisson's ratio	0.2
i. Grade 25/30 concrete	
Cylinder crushing strength (f_{ck})	25N/mm^2
Cube strength	30N/mm^2
ii. Grade 30/37 concrete	
Cylinder crushing strength (f_{ck})	30N/mm^2
Cube strength	37N/mm^2
2. Rebar (Grade 500)	
Modulus of elasticity (E)	200000kN/mm^2
Specific weight	77kN/mm^2
Characteristic strength (f_{yk})	500N/mm^2

3.3.2 Sections

The structural element of structure consists of beams, columns, slabs and walls. Specifications of structural elements are shown in Table 3.3.

Table 3.3: Section properties

Beams		
Material	C25/30 grade concrete	
Dimension (mm)	300 (width) x 600 (depth)	
Section shape	Rectangular	
Columns		
Material	C30/37 grade concrete	
Dimension (mm)	Main block	Extension block
	600(width) x 600(depth)	1000(width) x 1000 (depth)
Section shape	Rectangular	
Slabs		
Material	C25/30 grade concrete	
Thickness (mm)	300	
Walls		
Material	C30/37 grade concrete	
Thickness (mm)	300	

3.3.3 Nonlinearity

Static Pushover Method requires the nonlinear properties of structural member. In SCIA Engineer, nonlinear properties of member can be readily set up by user.

Table 3.4: Member nonlinearity

Member	Nonlinearity
Beam	Tension & Bending
Column	Compression

3.4 Load Cases

All individual load cases should be defined for the needs of pushover analysis. Programs adapt many types of load cases, mainly categorized into linear or nonlinear depends on response of structure towards the loading. In load pattern definition, modification of lateral load is available when load types specified as seismic or wind. Once the EC 8 is selected, specify parameter that consistent with the code selected could be defined.

3.4.1 Permanent Load & Variable Load

The design of loads is considering permanent loads and variable loads. Permanent loads represent the self-weight of structure and all fixed component such as ceilings, partition, etc. Whereas Variable loads represent the occupants, movable machinery, furniture, etc.

Wind load, seismic load and snow load are categorized as variable load but the design of these loads is separated from the variable loads when come to the consideration of load combination, these loads have their specified partial safety factor in load combination design .

Table 3.5: Permanent load (Mosley, Hulse and Bungey, 2012)

Case(s)	Description	Reference
Self-weight	Automatically calculated by SCIA Engineer 15.	-
Floor	Floor finishing and waterproof cover.	2.0kN/m^2

Table 3.6: Variable load (Mosley, Hulse and Bungey, 2012)

Story	Description	Variable load (kN/m^2)
Ground floor	Administration	2.5
1st floor	Office for general uses	2.5
2nd floor	Medical Equipment	5.0
3rd floor	Hospital wards	2.0
4th floor	Hospital wards	2.0

Story	Description	Variable load (kN/m^2)
5th floor	Surgery room	3.0
6th floor	Surgery room	3.0
Roof	With access	1.5

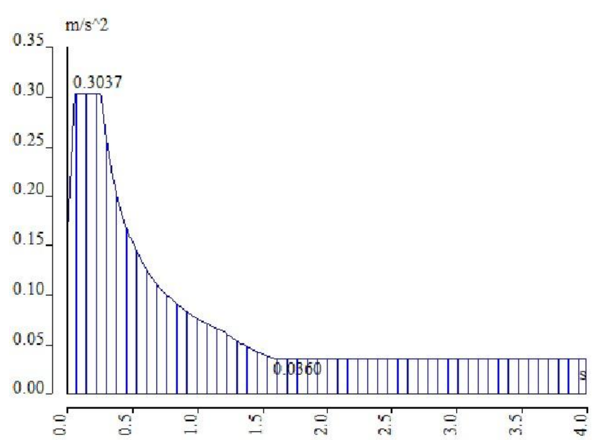
3.4.2 Seismic load

The nature of vibrations and the forces induced by a seismic action are complex phenomena. The estimation of seismic load is very difficult by manual mean, therefore the analysis need to be carried out by program. A simple approach is the equivalent static analysis in which base shear at the foot of the structure is calculated and distributed as horizontal force at each floor level according to certain defined criteria.

The input data of seismic spectrum is required to carry out analysis. Spectrum type refers to the 2 types of horizontal elastic response spectrum. The spectra are classified as function of the magnitude of the earthquakes that contribute most to the seismicity at the given site. Type 1 spectrum adopted to earthquakes with surface-wave magnitude (M_s) greater than 5.5. If the magnitude of earthquake is not greater than 5.5, type 2 should be adopted.

Malaysian adopts EC 8: Design of Structures for Earthquake Resistance by minor modification of some value. The whole of Malaysia include Peninsular Malaysia, Sabah and Sarawak is classified as low seismicity. The analyzed hospital building is assumed to be located at Sabah, therefore all the parameters in Table 3.7 adopt the value recommended by Malaysia decision on EC 8.

Table 3.7: Seismic spectrum input (IEM,2016)

Parameter	**Value**	**Note**
Damping	0.05	-
Spectrum Type	2	surface-wave magnitude (M_s) lesser than 5.5
Ground Type	B	Very dense sand or gravel or very stiff clay
Ground acceleration, a_g/g	0.18g	Lifeline built facilities
Behavior factor, q	2	-
Beta, ß	0.2	-
Spectrum	 **Figure 3.5: Response Spectrum Diagram**	

3.5 Load Combinations

Permanent load (PL) and variable load (VL) will occur in different combinations,

Table 3.8: Load combinations (Biasioli, et al., 2014)

No.	Situation	Equation
1	Checking for static equilibrium	1.10PL+1.50VL
2	For the design of structural member excluding geotechnical actions	1.35PL+1.50VL
3	Design Seismic Situation	1.0PL+0.6VL+SL

Seismic Load (SL) is the design seismic action which is equal to $\gamma_I A_{Ek}$. γ_I refers to the important factor of structure, the recommended value for γ_I is slightly difference as shown in Table 3.9. The effects of seismic action can be classified as local effects and global effects (Biasioli, et al. 2014).

Table 3.9: Importance classes and recommended values for γ_I for buildings (IEM,2016)

Importance class	**Importance factor, γ_I (recommended by EC8)**	**Importance factor, γ_I (recommended value by Malaysia Standard)**	**Recommended building categories**
I	0.8	0.8	Minor constructions
II	1.0	1.0	Ordinary buildings (individual dwellings or shops in low rise buildings)
III	1.2	1.2	Buildings of large occupancies (condominiums, shopping centres, schools and public buildings)
IV	1.4	1.5	Lifeline built facilities (hospitals, emergency services, power plants and communication facilities)

3.6 Pushover Analysis

Pushover analysis is conducted when all definitions has set. The magnitude of lateral pushover load is start from 1kN/m^2 on one direction of the model, the result based on 1kN/m^2 is generated after running the manual pushover analysis: the base shear and roof displacement of the model. Subsequently, the magnitude of lateral load is increased to 2kN/m^2, same procedure is mentioned on above to get the data of base shear and roof displacement of model. The increased interval of lateral pushover load is 1kN/m^2 until reaches the failure point of model, the failure condition is discussed in Chapter 4.

Pushover analysis is conducted on 4 different directions: Positive X, Negative X, Positive Y and Negative Y. Each direction has their respective maximum pushover load: the lateral pushover load which causes failure of model. Maximum pushover load is used as a reference for further study regards to irregularity of model and braced frame system.

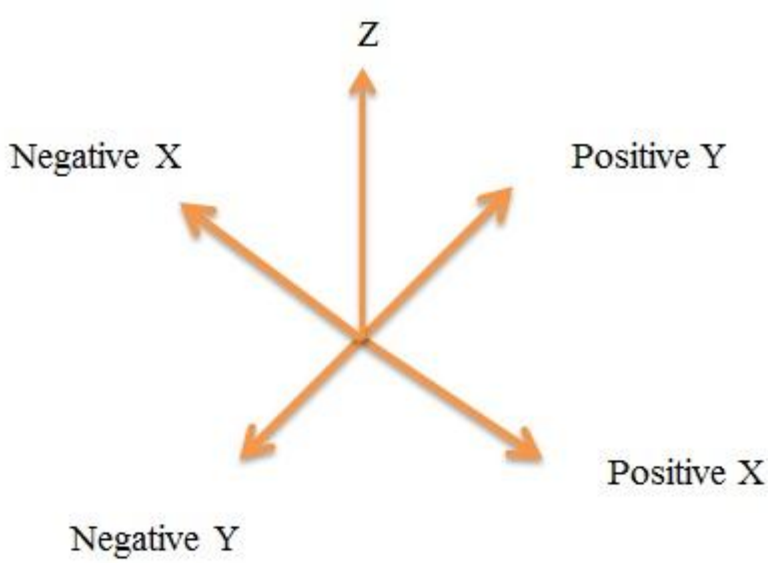


Figure 3.6: Direction of pushover analysis

3.7 Capacity Curve

The methodology of manual pushover analysis is mentioned in clause 3.6. The results of base shear and roof displacement of model in one direction from 1kN/m^2 lateral pushover load until the maximum pushover load are using to plot the capacity curve (pushover curve): a plot of base shear versus roof displacement of model (Hassaballa, et al, 2014). The failure point on capacity curve is spotted by using the polynomial equation generated from Microsoft Excel.

3.8 Irregularity Modeling

In this study, the irregularity of mass is only considering vertical mass irregularity and diaphragm discontinuity. Mass irregularity exists when the effective mass of any story exceed 150% of the effective mass of an adjacent story. Alternatively, a roof which is lighter than the floor below need not be considered the presence of mass irregularity (UBC, 1997). The calculation of mass irregularity is shown as below:

$Percentage\ of\ mass\ exceed\ at\ 1st\ floor$:

$$= \frac{Total\ Mass\ of\ 1st\ floor}{Total\ Mass\ of\ ground\ floor} \times 100\%$$

$$= \frac{55860\, kg/m^3}{55860\, kg/m^3} \times 100\%$$

$$= 100\%\ (adsence\ of\ mass\ irregularity)$$

For mass irregularity study, a comparison of mass regular hospital and mass irregular hospital has been is studied. The detail of these two models is shown in Table

3.10 and Table 3.11. Pushover analysis is applied to evaluate the influence of mass irregularity of a building.

Table 3.10: Total mass of each story for mass irregular model

Story	**Variable load (expressed in kg/m^2)**	**Permanent load (expressed in kg/m^2)**	**Total Mass (expressed in kg/m^2)**	**Percentage of Mass Exceed (Adjacent Storey)**
Ground floor	24500	31360	55860	-
1st floor	24500	31360	55860	100
2nd floor	49000	31360	83790	150
3rd floor	19600	31360	50960	63.41
4th floor	19600	31360	50960	100
5th floor	29400	31360	60760	119.23
6th floor	29400	31360	60760	100
Roof	14700	28420	43120	70.97

Table 3.11: Total mass of each story for mass regular model

Story	**Variable load (expressed in kg/m^2)**	**Permanent load (expressed in kg/m^2)**	**Total Mass (expressed in kg/m^2)**	**Percentage of Mass Exceed (Adjacent Storey)**
Ground floor	24500	31360	55860	-
1st floor	24500	31360	55860	100
2nd floor	49000	31360	67032	120
3rd floor	19600	31360	50960	76
4th floor	19600	31360	50960	100
5th floor	29400	31360	60760	119.23
6th floor	29400	31360	60760	100
Roof	14700	28420	43120	70.97

Diaphragm discontinuity including those having opening area for a structure. Opening is unavoidable for modeling or constructing a building because the purpose of creating opening is for air ventilation, stairways, shafts or other functions (Kumar and Gundakalle, 2015). The modeling of diaphragm discontinuity with 5%, 10% and 15% opening area in plan view is shown in Figure 3.7, Figure 3.8 and Figure 3.9 respectively. The effects of different diaphragm opening are compared by pushover analysis

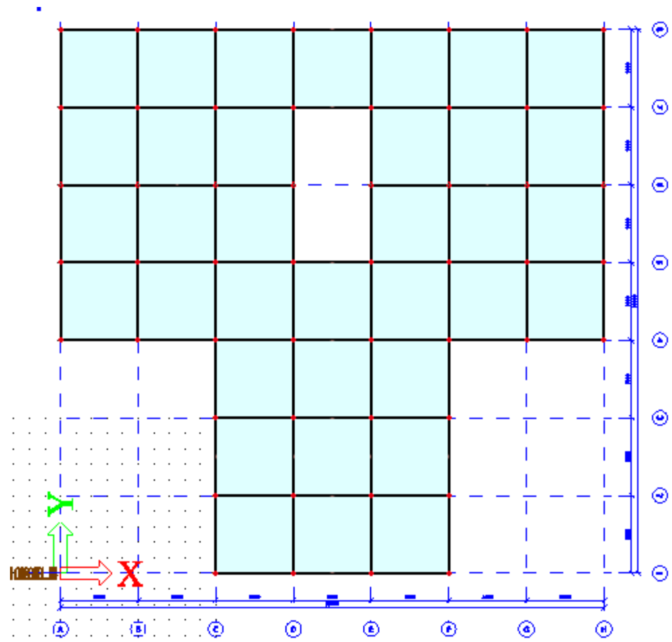


Figure 3.7: Diaphragm discontinuity with 5% opening area

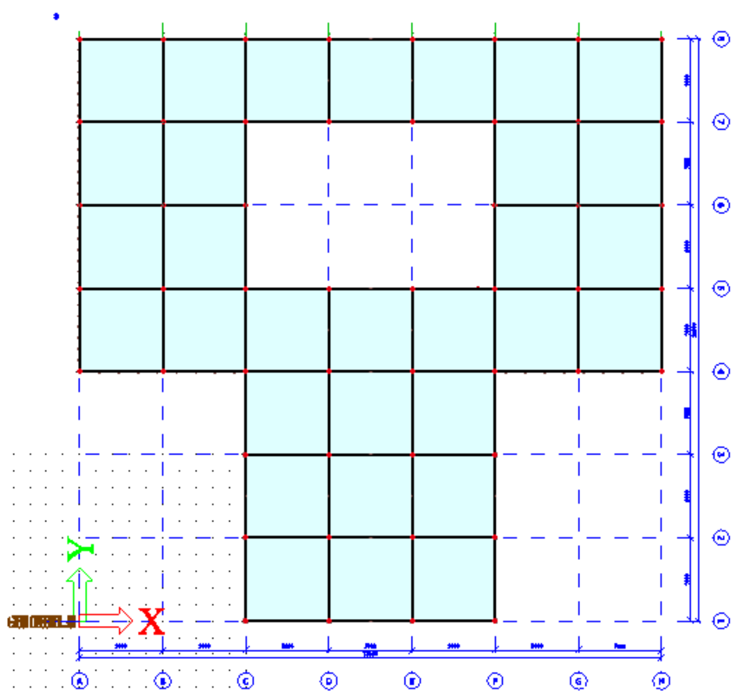


Figure 3.8: Diaphragm discontinuity with 20% opening area

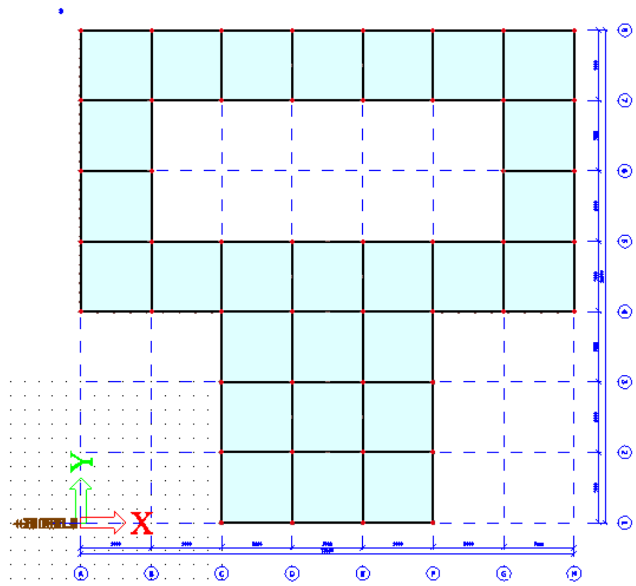


Figure 3.9: Diaphragm discontinuity with 40% opening area

3.9 Wall Bracing System Modeling

A braced frame of a building is designed to resist lateral force primarily are wind force and seismic force. A bracing has similar function with a truss which is designed as a tension and compression member of the buildings (Khan, Narayana and Raza, 2015).

The diagonal wall bracing system has been applied on the model to compare the lateral displacement of both models: with wall bracing system and without wall bracing system. The wall bracing system is shown in Figure 3.10 (in X direction) and Figure 3.11 (in Y direction). Pushover analysis is used to determine the improvement can be carried out by diagonal wall bracing system.

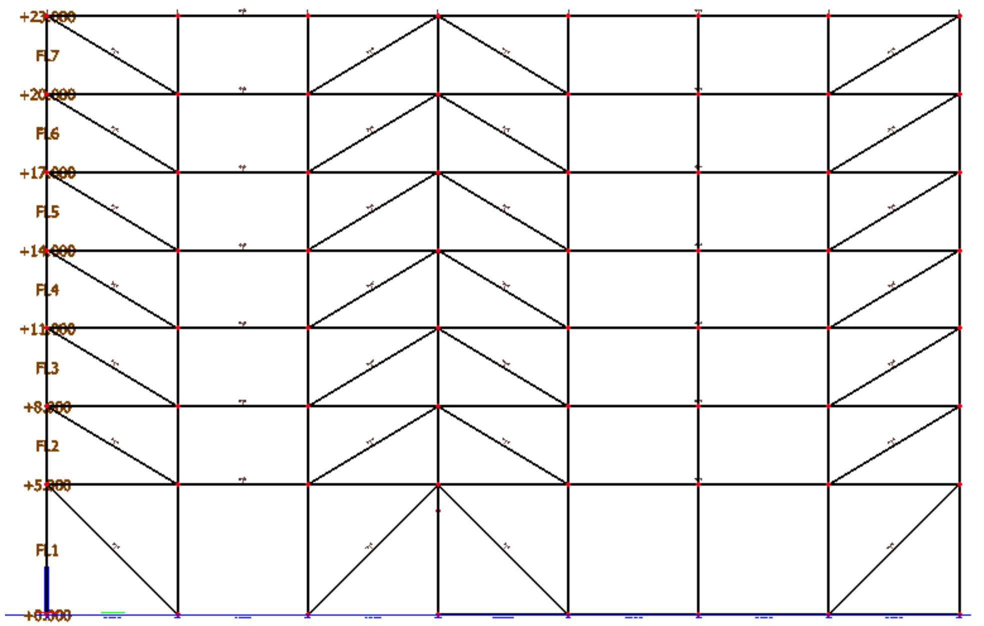


Figure 3.10: Model with RC wall bracing system in X direction

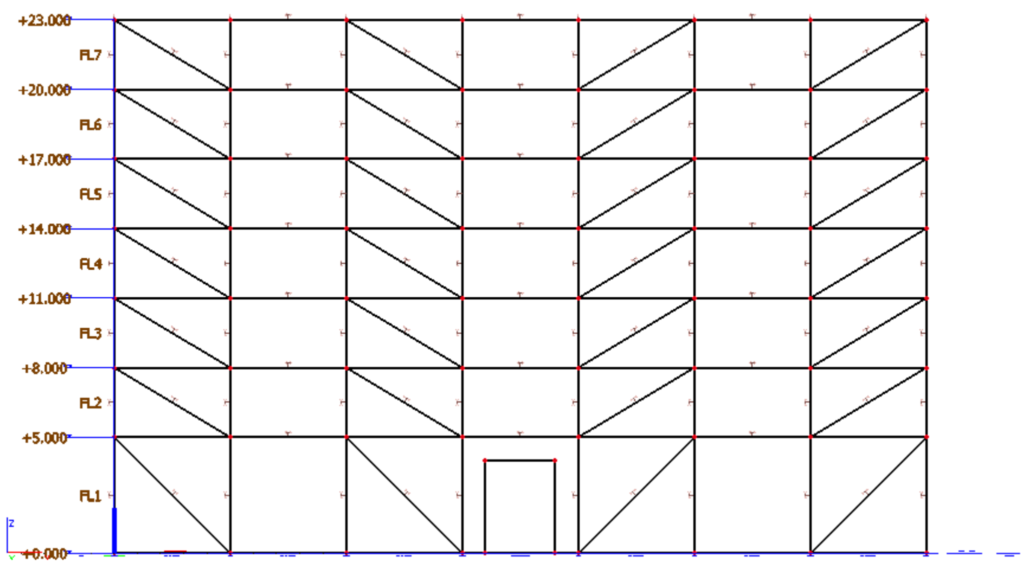


Figure 3.11: Model with RC wall bracing system in Y direction

CHAPTER 4

RESULT & DISCUSSION

4.1 Introduction

By using manually nonlinear pushover via SCIA Engineer 15 enables to obtain a series of data with roof displacement and base shear. Capacity curve (Pushover curve) is able to be plotted (roof displacement vs. base shear) by using Mircosoft Excel. The maximum allowable story displacement Δ_a shall not exceed 2% of the story height h for the structures which are five stories or more in height (ASCE 7-98, 2002). The maximum allowable for 7 story hospital building, Δ_a is 60mm.

Table 4.1: Allowable displacement of each story

Level	**Story height (m), *h***	**Max. allowable displacement (mm), Δ_a**
Roof	3	60
Level 6	3	60
Level 5	3	60

4.2 Pushover Analysis and Capacity Curve

Pushover analysis is applied in 4 directions: Positive X, Positive Y, Negative X and Negative Y as shown in Figure 4.1. The pushover load is incrementally increasing until the displacement limit of model has reached its allowable displacement Δ_a. The pushover analysis stopped after one node among the nodes has exceeded the allowable displacement Δ_a as shown in Figure 4.2.

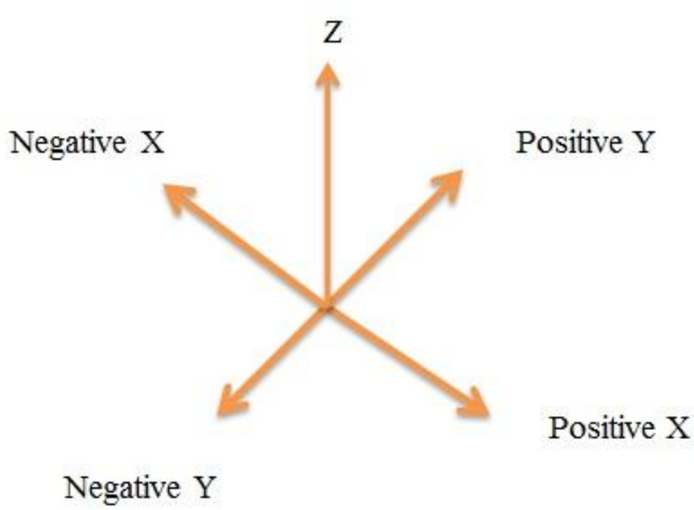


Figure 4.1: Direction of pushover analysis

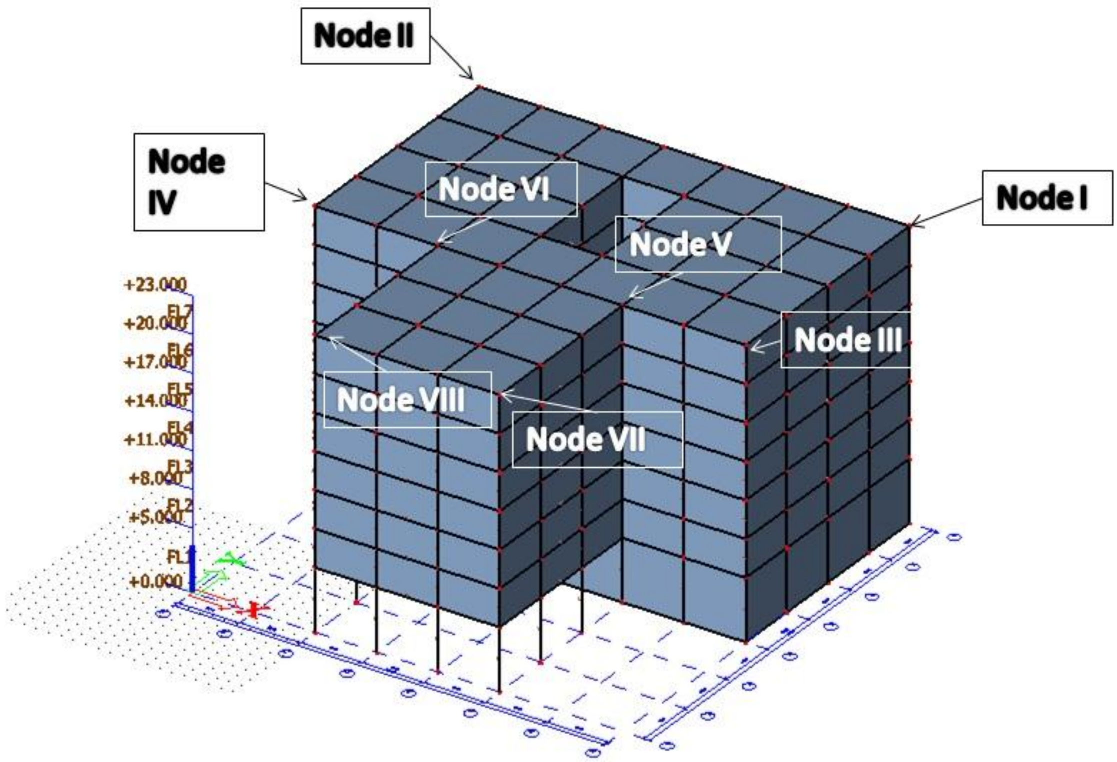


Figure 4.2: Location of node I until node VIII

4.2.1 Ductility of Pushover Curve

Pushover curve in 4 directions can be shown in Figure 4.3, Figure 4.5, Figure 4.7 and Figure 4.9. Typical pushover curves consist of two major parts: elastic region and inelastic region. Inelastic region is the key determinant of the failure point of the model in pushover curve. The failure point of 4 respective directions is obtained where the allowable displacement Δ_a has reached by pushover analysis (Taïeb and Sofiane, 2014).

4.2.2 Pushover analysis in X direction

Table 4.2 shows the maximum base shear of model for direction X is 87.60×10^3 kN roof displacement of 8 nodes are recorded; node VII and node VIII exceed the displacement limit (60mm) where the displacement of these two nodes are 61.6mm and 62.1mm respectively, these 2 nodes are located at extension block. Roof displacement of main block is within the displacement limit. A pushover curve is plotted by base shear versus roof displacement. Failure point of the model can be originated by using the interpolation of such equation has been generated by Microsoft Excel, by applying displacement limit equals to 60mm (ASCE 7-98, 2002). In a nutshell, the base shear of the model is 84.64×10^3kN at a displacement limit 60mm which causes the model to failure, which is shown in Figure 4.3; the deformed pattern of the model when pushover load is applied in positive X direction is shown in Figure 4.4.

Table 4.2: Base shear & roof displacement by pushover analysis in positive X direction

Pushover Direction		Positive X							
Displacement Limit		60mm							
Pushover Load (kN/m^2)	Base Shear (10^3kN)	Roof Displacement (mm)							
		Node I	Node II	Node III	Node IV	Node V	Node VI	Node VII	Node VIII
0	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	2.19	-1.0	2.7	0.7	1.0	1.0	1.5	1.4	1.7
2	4.38	-0.2	3.5	1.9	3.1	2.2	2.8	2.9	3.2
3	6.57	0.6	4.3	3.2	4.3	3.5	4.0	4.4	4.7
4	8.67	1.4	5.2	4.4	5.6	4.7	5.3	5.9	6.2
5	10.95	2.3	6.8	5.6	6.8	5.9	6.5	7.4	6.9
6	13.14	3.1	6.8	6.9	8.4	7.2	7.8	8.9	9.0

Pushover Direction		Positive X							
Displacement Limit		60mm							
Pushover Load (kN/m^2)	Base Shear (10^3kN)	Roof Displacement (mm)							
		Node I	Node II	Node III	Node IV	Node V	Node VI	Node VII	Node VIII
8	17.25	4.7	8.5	9.3	10.6	9.7	10.2	12.0	12.3
10	21.90	6.4	10.2	11.8	13.1	12.2	12.7	15.0	15.4
12	26.28	8.0	11.9	14.3	15.0	14.7	15.3	18.1	18.1
14	30.66	9.7	13.6	16.8	18.2	17.2	17.0	21.2	21.6
16	33.98	11.0	14.9	18.8	20.2	19.2	19.8	23.6	24.6
18	39.42	13.1	17.0	21.9	23.9	23.3	22.9	27.4	27.6
20	43.80	14.8	18.7	24.4	25.9	24.8	25.5	30.5	30.9
24	52.56	18.1	23.1	29.5	31.0	29.9	30.6	36.7	37.2
28	61.32	21.5	25.6	34.6	36.1	35.0	35.7	43.0	43.4
32	70.08	24.9	29.9	39.6	41.3	40.2	40.7	49.2	49.7
36	78.84	28.3	32.4	44.7	46.4	45.2	46.8	55.4	55.9

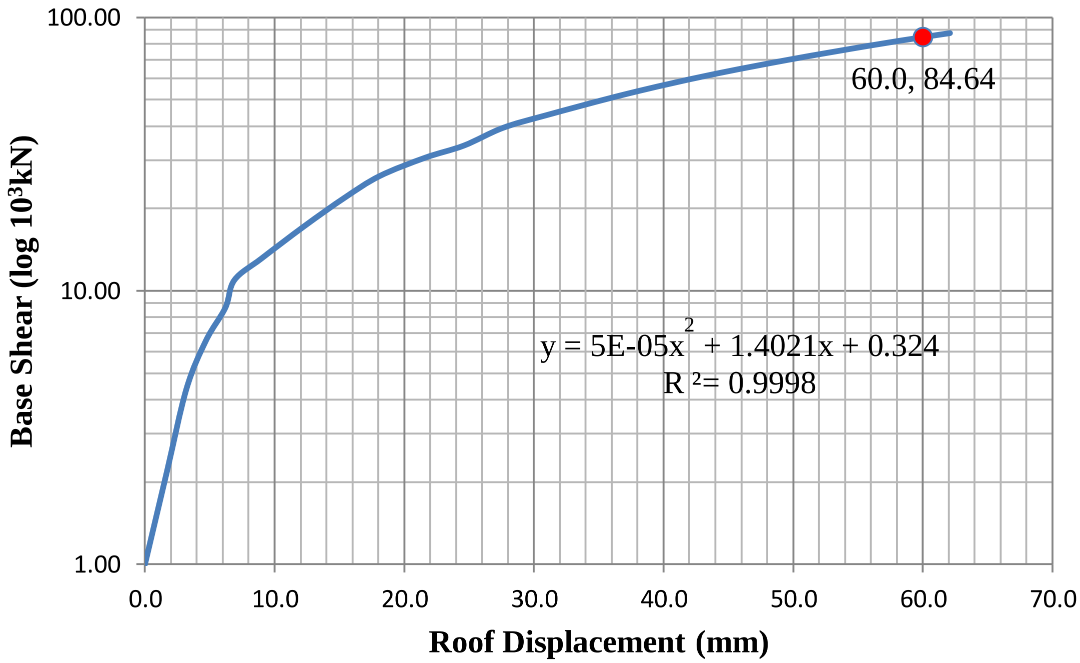


Figure 4.3: Pushover curve in positive X direction

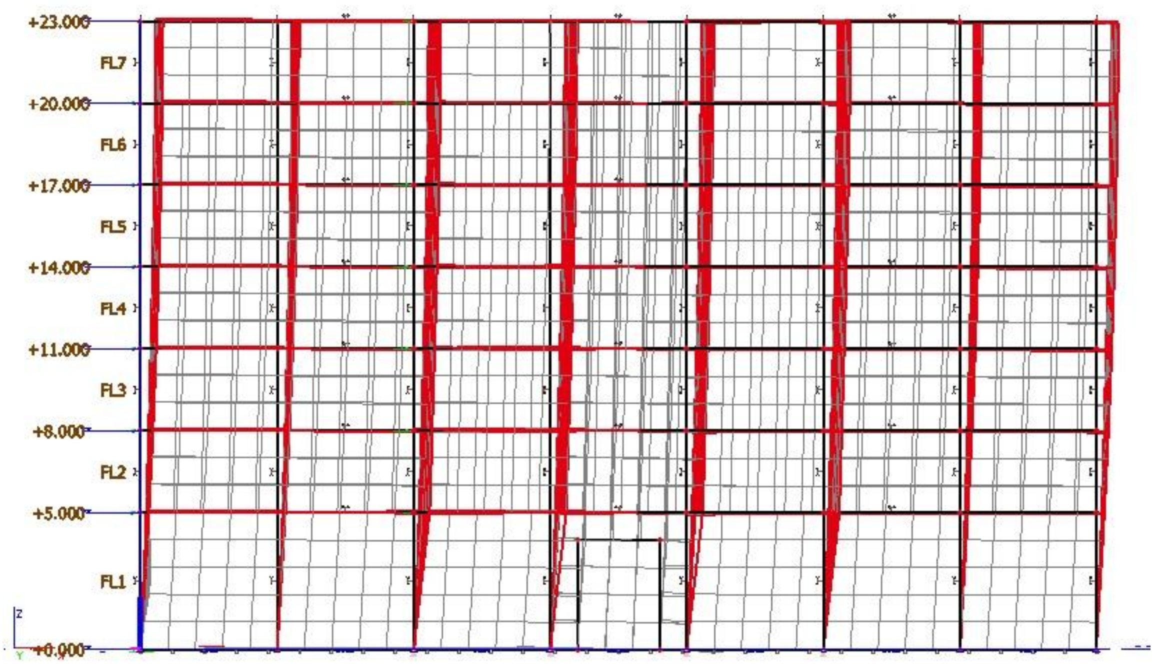


Figure 4.4: Deformed pattern in positive X direction

Table 4.3 shows the maximum base shear of model for direction X is 85.41×10^3kN, roof displacement of 8 nodes are recorded; node VII and node VIII exceed the allowable displacement (60mm) where the displacement of these two nodes are 60.6mm and 60.1mm respectively, these 2 nodes are located at extension block. Roof displacement of main block is within the allowable displacement. Failure point of the model: base shear is 84.34×10^3kN at 60mm displacement limit which is shown in Figure 4.5; the deformed pattern of the model when pushover load is applied in negative X direction is shown in Figure 4.6.

Table 4.3: Base shear & roof displacement by pushover analysis in negative X direction

Pushover Direction		**Negative X**							
Displacement Limit		**60mm**							
Pushover Load (kN/m^2)	**Base Shear (10^3kN)**	**Roof Displacement (mm)**							
		Node I	**Node II**	**Node III**	**Node IV**	**Node V**	**Node VI**	**Node VII**	**Node VIII**
0	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	2.10	1.1	0.5	1.6	0.9	1.4	1.0	1.6	1.4
2	4.40	1.9	1.2	2.8	2.1	2.7	2.2	3.1	2.9
3	6.40	2.7	2.0	4.0	3.3	3.9	3.4	4.6	4.4
4	8.80	3.5	2.8	5.3	4.5	5.1	4.6	6.1	5.9
5	10.80	4.3	3.6	6.5	5.7	6.3	5.9	7.6	7.4
6	13.10	5.1	4.4	7.7	6.9	7.6	7.1	9.1	8.9
7	15.30	5.9	5.2	8.9	8.1	8.8	8.3	10.6	10.4
8	18.00	6.7	6.0	10.2	9.3	10.0	9.5	12.1	11.9
9	19.70	7.6	6.8	11.4	10.6	11.2	10.7	13.6	13.5
10	22.00	8.4	7.6	12.6	11.8	12.5	11.9	15.1	14.9
11	24.10	9.0	8.3	13.8	13.0	13.7	13.2	16.6	16.4
12	26.30	10.0	9.1	15.1	14.2	14.0	14.4	18.1	17.9

Pushover Direction		Negative X							
Displacement Limit		60mm							
Pushover Load (kN/m^2)	Base Shear ($x10^3$kN)	Roof Displacement (mm)							
		Node I	Node II	Node III	Node IV	Node V	Node VI	Node VII	Node VIII
13	28.47	12.7	8.9	16.9	15.6	16.5	15.9	20.0	19.6
14	30.70	13.6	9.7	18.2	16.8	17.8	17.2	21.5	21.2
16	35.04	15.3	11.4	20.7	19.4	20.4	19.7	24.7	24.3
18	39.42	17.0	13.1	23.3	21.9	22.9	22.3	27.8	27.4
20	43.80	18.7	14.8	25.9	24.4	25.5	24.8	30.9	30.5
22	48.18	20.4	16.4	28.4	27.0	28.1	27.4	34.0	33.6
24	52.56	22.1	18.1	31.0	19.5	30.6	29.9	37.2	36.7
26	56.94	23.9	19.8	33.6	32.0	33.2	32.5	40.3	39.8
28	61.32	26.6	21.5	36.1	34.6	35.0	35.3	43.4	43.0
30	65.68	27.9	23.2	38.7	37.1	38.3	37.6	46.5	46.1
32	70.08	29.9	24.9	41.3	39.6	40.0	40.1	49.7	49.2
34	74.46	30.7	26.6	43.8	42.2	43.4	42.7	52.8	52.3
36	78.82	32.4	28.3	46.4	44.7	46.0	45.2	55.9	55.4
38	83.22	34.1	29.9	49.0	47.3	48.5	47.8	59.0	58.5
39	85.41	35.9	30.8	49.8	49.0	49.0	49.8	60.6	60.1

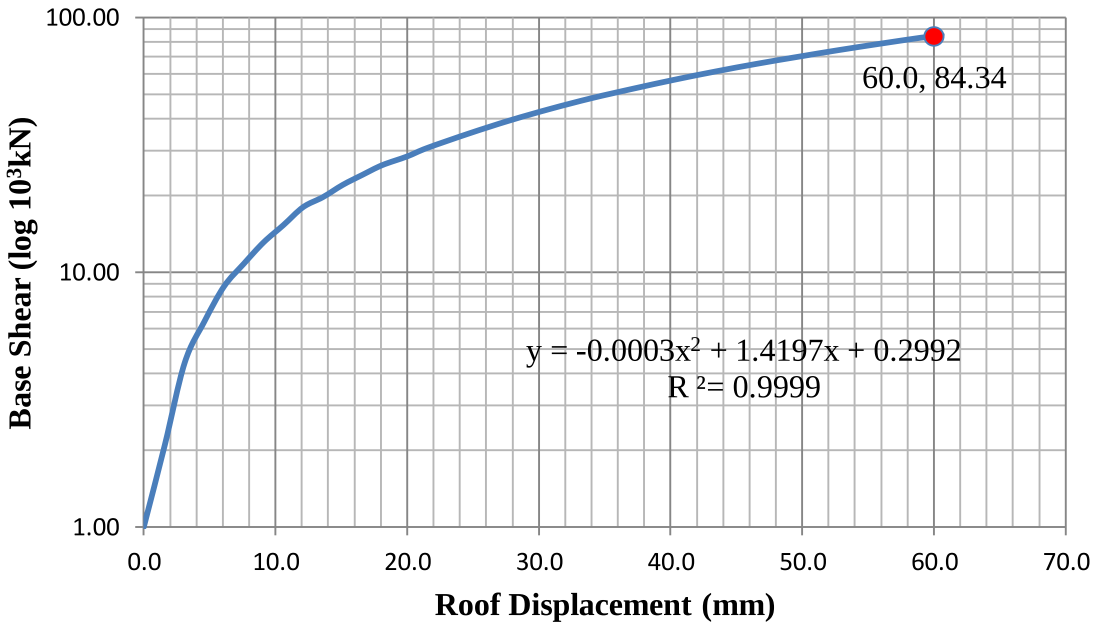


Figure 4.5: Pushover curve in negative X direction

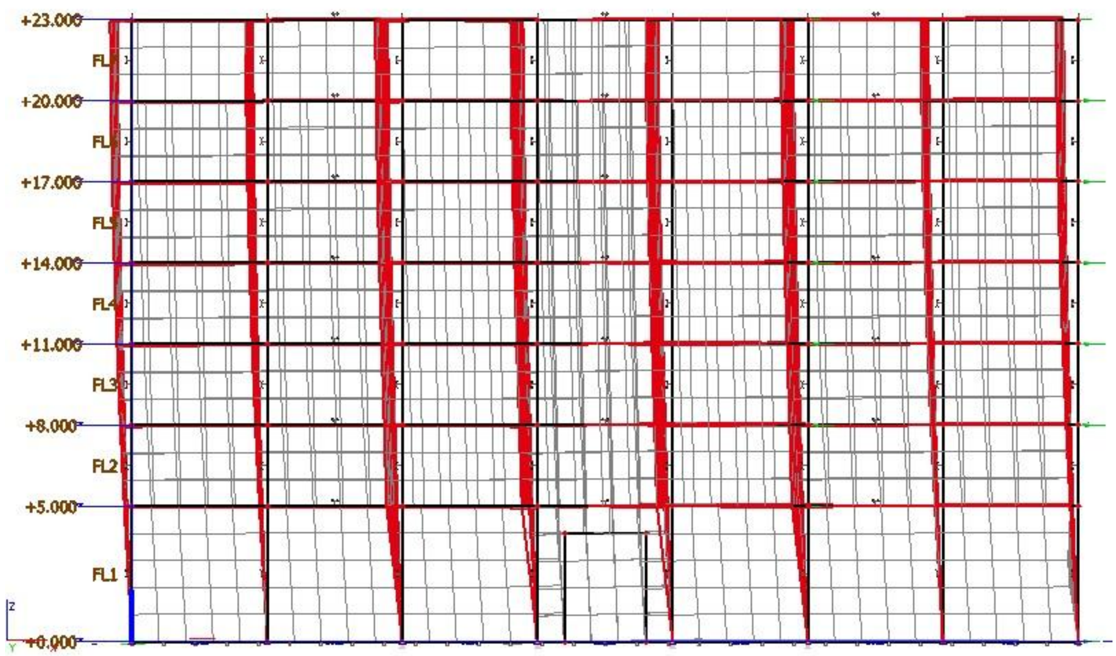


Figure 4.6: Deformed pattern in negative X direction

4.2.3 Pushover analysis in Y direction

Table 4.4 shows the maximum base shear of model in positive Y direction is 88.96×10^3 kN, roof displacement of 8 nodes are recorded; all of these nodes are exceeding the displacement limit which is 60mm. Failure point of the model: base shear is 89.43×10^3 kN at 60mm allowable displacement which is shown in Figure 4.7: the pattern of pushover curve in positive Y direction is different with other directions: After linear region, the model has results in minor displacement while the base shear is increasing; the deformed pattern on the model when pushover load is applied in positive Y direction is shown in Figure 4.8.

Table 4.4: Base shear & roof displacement by pushover analysis in positive Y direction

Pushover Direction		Positive Y							
Displacement Limit		60mm							
Pushover Load (kN/m^2)	Base Shear (10^3kN)	Roof Displacement (mm)							
		Node I	Node II	Node III	Node IV	Node V	Node VI	Node VII	Node VIII
0	0.00	0	0	0	0	0	0	0	0
1	2.34	10.7	10.7	10.2	10.2	10.1	10.1	9.8	9.8
2	4.69	12.0	12.0	11.5	11.5	11.4	11.4	11.2	11.2
3	7.03	13.2	13.2	12.7	12.7	12.6	12.6	12.3	12.3
4	9.37	14.5	14.5	13.9	13.9	13.9	13.9	13.6	13.6
5	11.72	15.7	15.7	15.2	15.2	15.1	15.1	14.8	14.8
6	13.97	16.9	16.9	16.4	16.4	16.3	16.3	16.0	16.0
8	18.74	19.5	19.5	18.8	18.8	18.9	18.9	18.6	18.6
10	23.42	22.0	22.0	21.6	21.6	21.5	21.5	21.2	21.2
12	28.06	24.6	24.6	24.8	24.8	24.8	24.8	23.9	23.9
14	32.78	27.2	27.2	26.7	26.7	26.7	26.7	26.7	26.7

Pushover Direction		Positive Y							
Displacement Limit		60mm							
Pushover Load (kN/m^2)	Base Shear (10^3kN)	Roof Displacement (mm)							
		Node I	Node II	Node III	Node IV	Node V	Node VI	Node VII	Node VIII
18	42.15	32.5	32.5	31.9	31.9	32.0	32.0	31.9	31.9
22	51.51	37.8	37.8	37.2	37.2	37.3	37.3	37.0	37.0
26	60.85	43.2	43.2	42.5	42.5	42.7	42.8	42.4	42.5
30	70.23	48.9	48.9	48.3	48.3	48.5	48.5	48.3	48.3
34	79.33	54.7	54.7	54.9	54.9	54.4	54.4	54.2	54.2
38	88.96	61.2	61.2	60.5	60.5	61.8	61.8	60.9	60.9

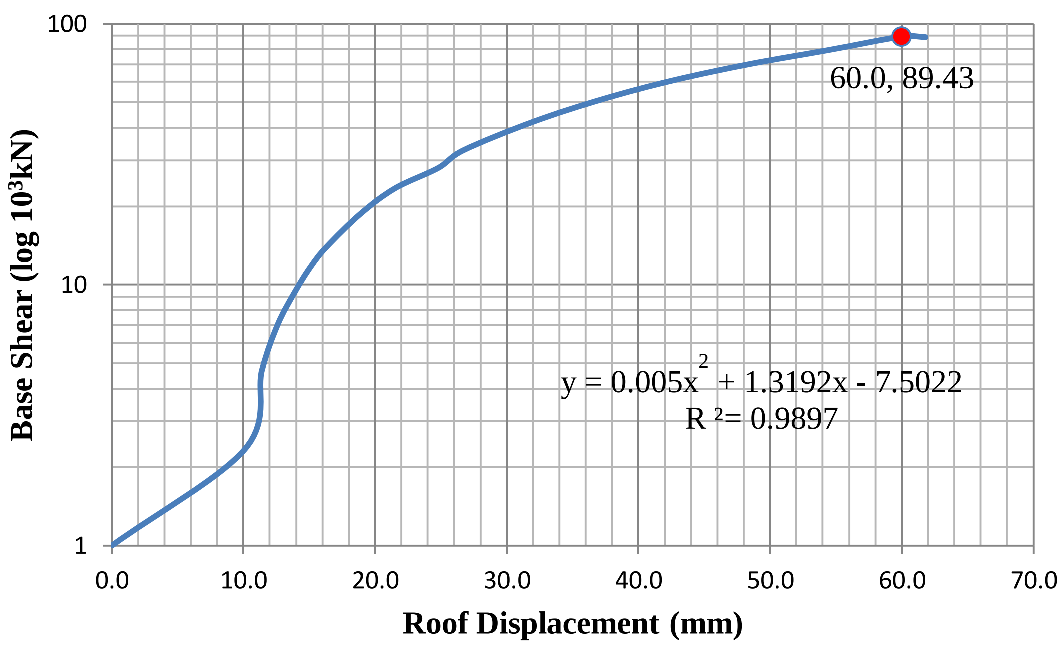


Figure 4.7: Pushover curve in positive Y direction

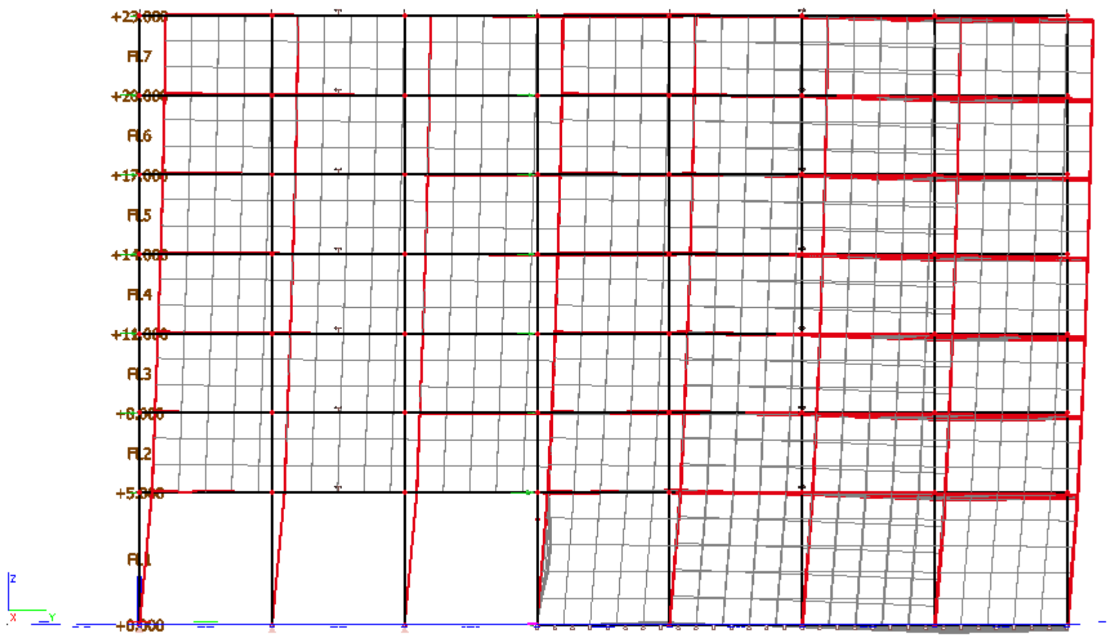


Figure 4.8: Deformed pattern in positive Y direction

Table 4.5 shows the maximum base shear of model for direction Y is 130.41×10^{3} kN , roof displacement of 8 nodes are recorded; all of these nodes are exceeding the allowable displacement which is 60mm. Failure point of the model: base shear is 127.90×10^{3}kN at 60mm allowable displacement which is shown in Figure 4.9; the deformed pattern of the model when pushover load is applied in positive Y direction is shown in Figure 4.10.

Table 4.5: Base shear & roof displacement by pushover analysis in negative Y direction

Pushover Direction		Negative Y							
Displacement Limit		60mm							
Pushover Load (kN/m^2)	Base Shear (10^3kN)	Roof Displacement (mm)							
		Node I	Node II	Node III	Node IV	Node V	Node VI	Node VII	Node VIII
0	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	2.42	8.2	8.2	7.7	7.7	7.6	7.6	7.4	7.4
2	4.83	7.0	7.0	6.5	6.5	6.4	6.4	6.1	6.1
3	7.25	5.7	5.7	5.2	5.2	5.1	5.1	4.9	4.9
4	9.66	4.4	4.4	4.9	4.9	3.9	3.9	3.6	3.6
5	12.08	3.2	3.2	2.7	2.7	2.6	2.6	2.4	2.4
6	14.49	1.9	1.9	1.5	1.5	1.3	1.3	1.1	1.1
7	16.91	0.7	0.7	0.1	0.1	0.1	0.1	-0.1	-0.1
8	19.32	-0.6	-0.6	-1.0	-1.0	-1.2	-1.2	-1.3	-1.3
9	21.74	-1.8	-1.8	-2.2	-2.2	-2.4	-2.4	-2.6	-2.6
10	24.15	-3.1	-3.1	-3.5	-3.5	-3.7	-3.7	-3.8	-3.8
11	26.57	-4.4	-4.4	-4.7	-4.7	-4.9	-4.9	-5.1	-5.1
12	28.98	-5.6	-5.6	-6.0	-6.0	-6.2	-6.2	-6.3	-6.3
13	31.40	-6.9	-6.9	-7.2	-7.2	-7.4	-7.4	-7.5	-7.5
14	33.81	-8.1	-8.1	-8.5	-8.5	-8.7	-8.7	-8.8	-8.8
15	36.23	-9.4	-9.4	-9.7	-9.7	-9.9	-9.9	-10.0	-10.0
16	38.64	-10.7	-10.7	-11.0	-11.0	-11.2	-11.2	-11.3	-11.3
17	39.85	-11.3	-11.3	-11.6	-11.6	-11.8	-11.8	-11.9	-11.9
18	43.47	-13.2	-13.2	-13.5	-13.5	-13.7	-13.7	-13.7	-13.7
20	45.89	-14.4	-14.4	-14.7	-14.7	-14.9	-14.9	-15.0	-15.0
22	53.13	-18.2	-18.2	-18.4	-18.4	-18.7	-18.7	-18.7	-18.7
24	58.00	-20.7	-20.7	-20.9	-20.9	-21.2	-21.2	-21.2	-21.2
26	62.76	-23.2	-23.2	-23.4	-23.4	-23.7	-23.7	-23.7	-23.7

Pushover Direction		Negative Y							
Displacement Limit		60mm							
Pushover Load (kN/m^2)	Base Shear (10^3kN)	Roof Displacement (mm)							
		Node I	Node II	Node III	Node IV	Node V	Node VI	Node VII	Node VIII
28	67.62	-25.8	-25.8	-25.9	-25.9	-26.2	-26.2	-26.1	-26.1
30	72.45	-28.3	-28.3	-28.4	-28.4	-28.7	-28.7	-28.6	-28.6
32	77.28	-30.8	-30.8	-30.9	-30.9	-31.2	-31.2	-31.1	-31.1
34	82.11	-33.3	-33.3	-33.4	-33.4	-33.7	-33.7	-33.6	-33.6
36	86.94	-35.8	-35.8	-35.9	-35.9	-36.2	-36.2	-36.1	-36.1
38	91.77	-38.5	-38.5	-38.5	-38.5	-38.9	-38.9	-38.7	-38.7
40	96.60	-41.3	-41.3	-41.2	-41.2	-41.6	-41.6	-41.4	-41.4
44	106.26	-46.9	-46.9	-46.7	-46.7	-47.1	-47.1	-46.8	-46.8
48	111.09	-49.7	-49.7	-49.5	-49.5	-49.9	-49.9	-49.6	-49.6
52	125.58	-58.2	-58.2	-57.9	-57.9	-58.2	-58.2	-57.9	-57.9
56	130.41	-61.0	-61.0	-60.2	-60.2	-61.1	-61.1	-60.6	-60.6

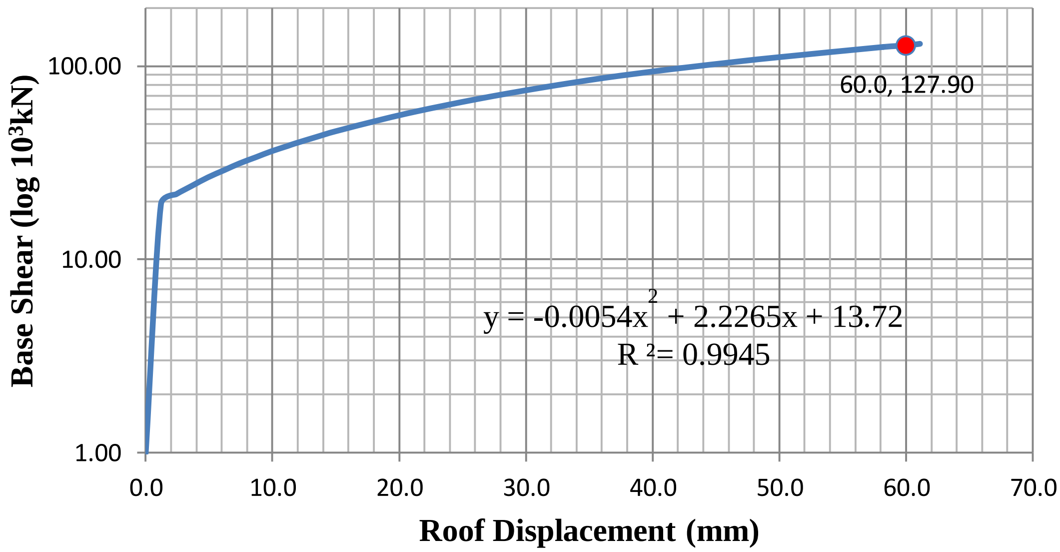


Figure 4.9: Pushover curve in positive Y direction

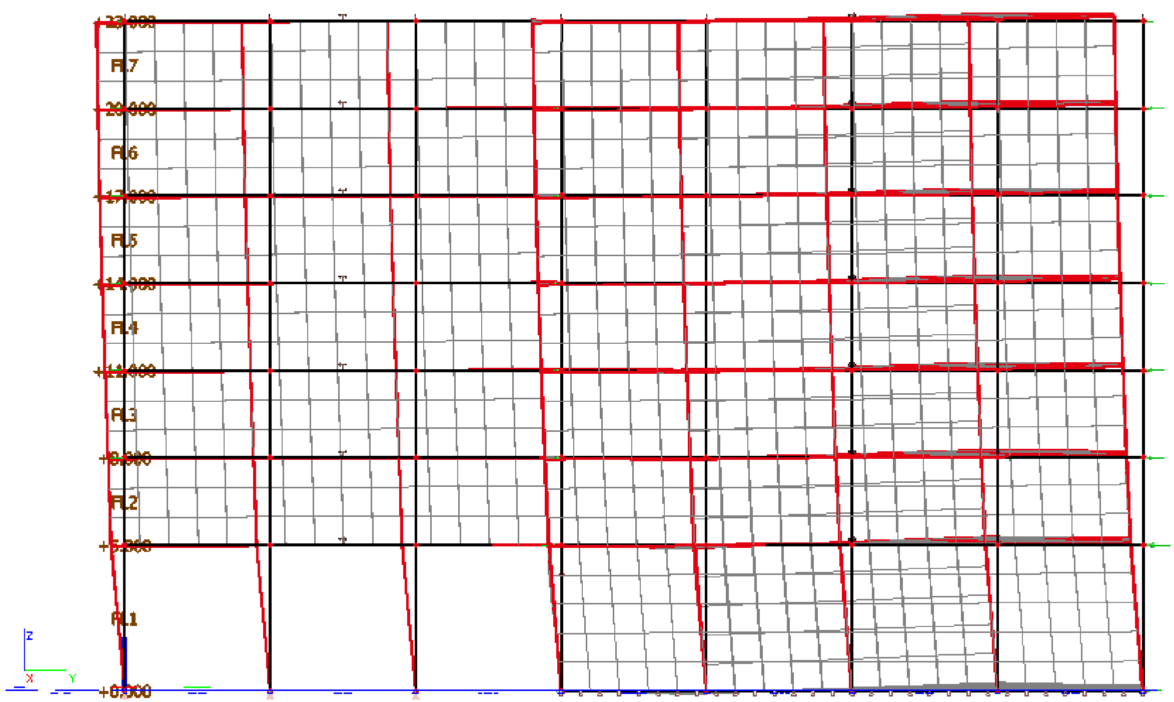


Figure 4.10: Deformed pattern in negative Y direction

4.2.4 Overall Effects of Pushover Analysis

The overall effects of pushover analysis of the model can be concluded: when the pushover analysis has applied to its respective direction and the allowable displacement has reached, negative Y direction has the highest base shear value than other directions, which is 127.90×10^3kN. Whereas, base shear value for positive X direction, negative X direction and positive Y direction are slightly different, which are 84.64×10^3kN, 84.34×10^3 kN and 89.43×10^3 kN respectively.

It is found that negative X direction is the most seismically vulnerable direction because it results in the lowest base shear among other direction which is more sensitive to a seismic action. Whereas Negative Y direction is the strongest among the other directions when the model is subjected to a seismic ground motion, this is because the extension block acts as a support member when pushover load is applied at negative Y direction (Sahu, 2016).

4.3 Structural Irregularity of Model

A structure with complex and irregular configuration in plan and elevation is more seismic vulnerable than a simple and regular configuration's structure in term of lateral strength, stiffness and ductility. The plan configuration of structure has significant impacts on base shear and displacement of a structure (Sameer and Gore, 2016).

Most of seismic codes emphasize the structural configuration to classify them as regular or irregular structures. The degree of irregularity influences the seismic vulnerability of buildings; irregularities of a structure are influenced by size and shape of the structures, uniformity of the mass, stiffness and strength distribution or combination with other properties in any direction (Sadashiva, MacRae & Deam, 2009). Irregularity can be divided to horizontal irregularities and vertical irregularities as shown in Figure 4.11.

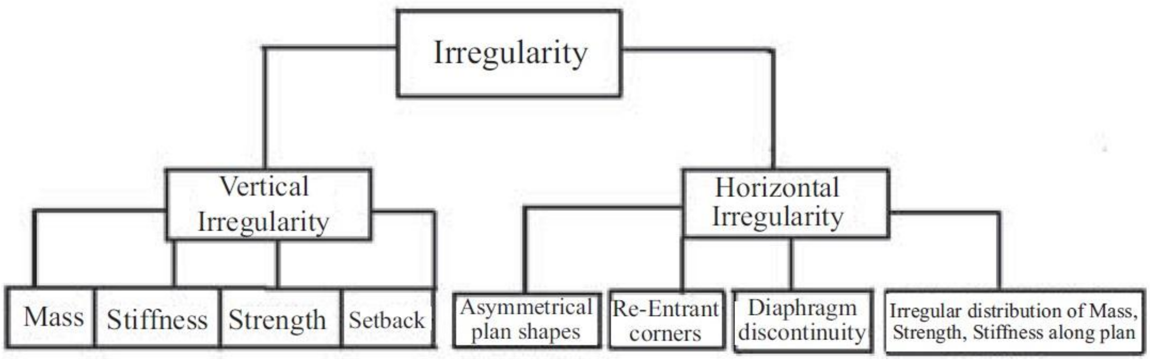


Figure 4.11: Classification of irregularity (Varadharajan, Sehgal and Saini, 2013)

4.3.1 Vertical Mass irregularity

Vertical mass irregularity is caused by the irregular distribution of mass along the building height: it is one of the vertical irregularities which contribute to the vulnerability of structures. Mass irregularity exists when the effective mass of any story exceed 150% of the effective mass of an adjacent story. Alternatively, a roof which is lighter than the floor below need not be considered mass irregularity (UBC, 1997).

Table 4.6 shows the presence of mass irregularity at 2nd floor of the hospital model, which the effective mass of 2nd story exceed 150% of the effective mass of an adjacent story, the rest story which the effective mass is less than 150% of the effective mass of an adjacent story is considered absence of mass irregularity (UBC, 1997).

Table 4.6: Total mass of each story

Story	Variable load (expressed in kg/m^2)	Permanent load (expressed in kg/m^2)	Total Mass (expressed in kg/m^2)	Percentage of Mass Exceed (Adjacent Storey)
Ground floor	24500	31360	55860	-
1st floor	24500	31360	55860	100
2nd floor	49000	31360	83790	150
3rd floor	19600	31360	50960	63.41
4th floor	19600	31360	50960	100
5th floor	29400	31360	60760	119.23
6th floor	29400	31360	60760	100
Flat roof	14700	28420	43120	70.97

A comparison of mass regular and mass irregular hospital models was made in another study to investigate the effects of mass irregularity on seismic vulnerability of a building (Bansal and Gagandeep, 2014). The value of shear force and moment of each story is shown in Table 4.7 and Table 4.8 where failure pushover load is applied at the four different directions on the mass irregular hospital model and mass regular hospital model respectively.

Table 4.7: Shear Force and moment for each story of mass irregular hospital model under failure pushover load

Mass Irregular Hospital Model				
Direction	**Pushover load (kN/m^2)**	**Story**	**Shear Force (10^3 kN)**	**Moment (10^3 kN.m)**
Positive X	40	7	6.586	15.875
		6	21.873	104.594
		5	34.280	171.425
		4	46.692	304.422
		3	59.107	475.351
		2	71.481	686.506
		1	58.419	1066.888
Negative X	39	7	6.423	15.848
		6	21.326	74.514
		5	33.421	167.138
		4	45.524	296.810
		3	57.628	463.467
		2	69.694	668.890
		1	56.961	1040.215
Positive Y	38	7	8.194	20.124
		6	20.603	71.178
		5	32.449	161.506
		4	44.303	287.816
		3	56.173	450.185
		2	67.242	648.367
		1	81.534	900.967

Mass Irregular Hospital Model				
Direction	**Pushover load (kN/m^2)**	**Story**	**Shear Force (10^3 kN)**	**Moment (10^3 kN.m)**
Negative Y	56	7	9.317	27.274
		6	30.490	104.594
		5	47.907	235.787
		4	65.327	420.226
		3	82.740	657.770
		2	100.400	949.940
		1	120.870	1624.850

Table 4.8 Shear Force and moment for each story of mass regular hospital model under failure pushover load

Mass Regular Hospital Model				
Direction	**Pushover load (kN/m^2)**	**Story**	**Shear Force (10^3 kN)**	**Moment (10^3 kN.m)**
Positive X	40	7	5.774	13.909
		6	19.136	66.877
		5	29.989	149.988
		4	40.85	266.366
		3	51.714	415.928
		2	62.544	600.267
		1	51.126	933.527

Mass Regular Hospital Model				
Direction	**Pushover load (kN/m^2)**	**Story**	**Shear Force (10^3 kN)**	**Moment (10^3 kN.m)**
Negative X	39	7	5.611	13.516
		6	18.589	64.968
		5	29.131	145.701
		4	39.628	258.754
		3	50.235	404.043
		2	60.757	583.116
		1	49.668	906.885
Positive Y	38	7	7.505	18.785
		6	18.288	64.33
		5	28.573	143.949
		4	38.864	254.906
		3	49.165	397.089
		2	59.513	572.427
		1	72.226	796.879
Negative Y	56	7	8.727	25.427
		6	28.527	97.394
		5	44.77	219.389
		4	60.87	390.735
		3	77.039	611.405
		2	93.402	883.003
		1	112.298	1514.894

The peak shear force and moment value of each story for mass regular and mass irregular hospital models are plotted and contrasted in Figure 4.12 and Figure 4.13, when the $40kN/m^2$ pushover load is applied in positive X direction. Similar trend can be found in negative X direction as shown in Figure 4.14 and Figure 4.15. Mass regular model tends to result in less shear force and moment than mass irregular model while same magnitude of pushover load is applied.

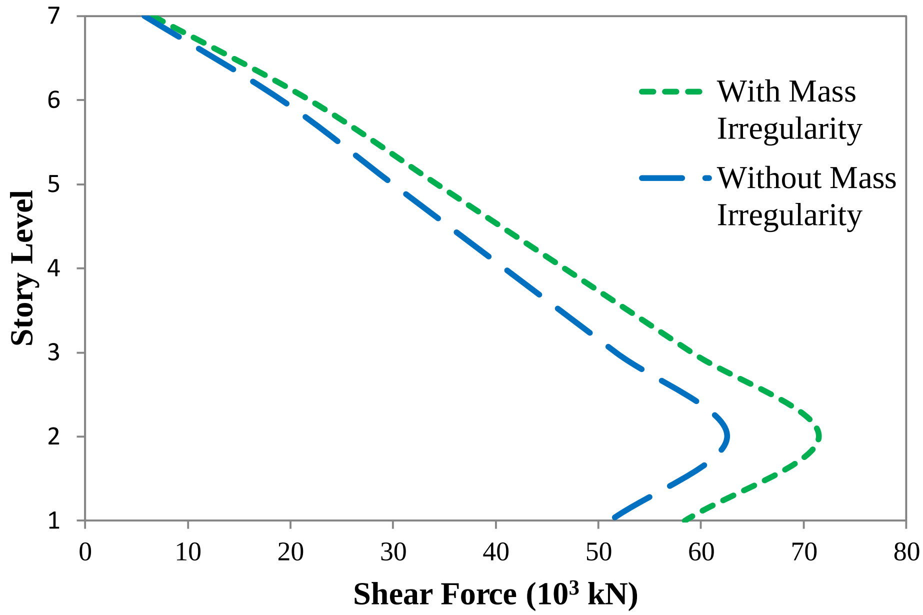


Figure 4.12: Comparison of peak shear force of mass regular and mass irregular models in positive X direction

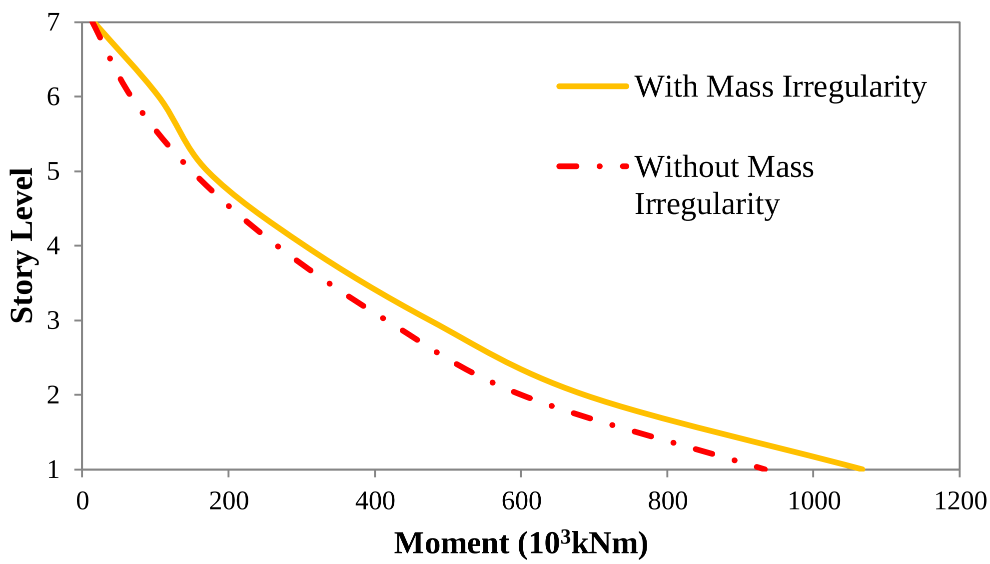


Figure 4.13: Comparison of peak moment of mass regular and mass irregular models in positive X direction

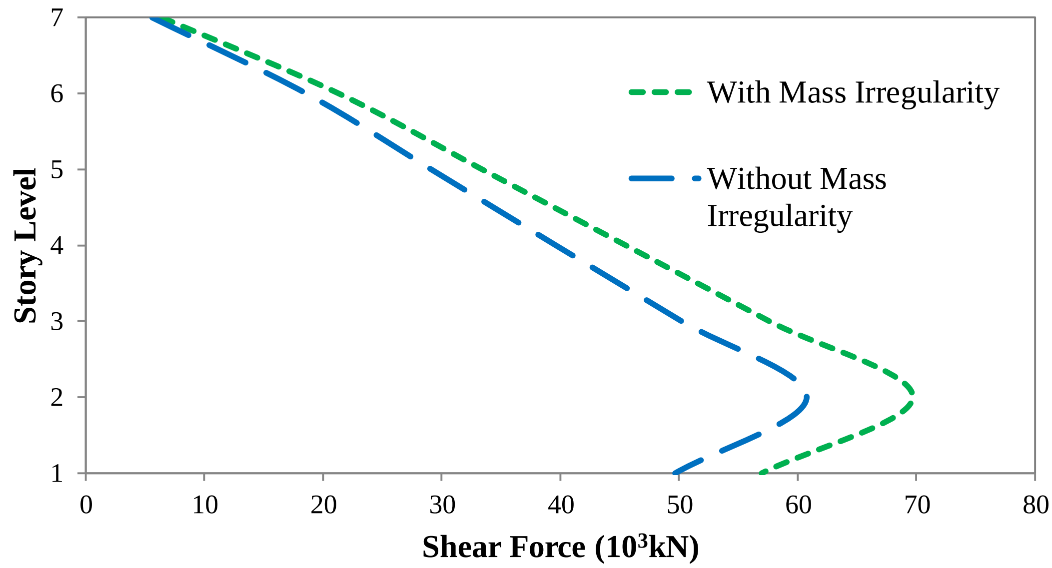


Figure 4.14: Comparison of peak shear force of mass regular and mass irregular models in negative X direction

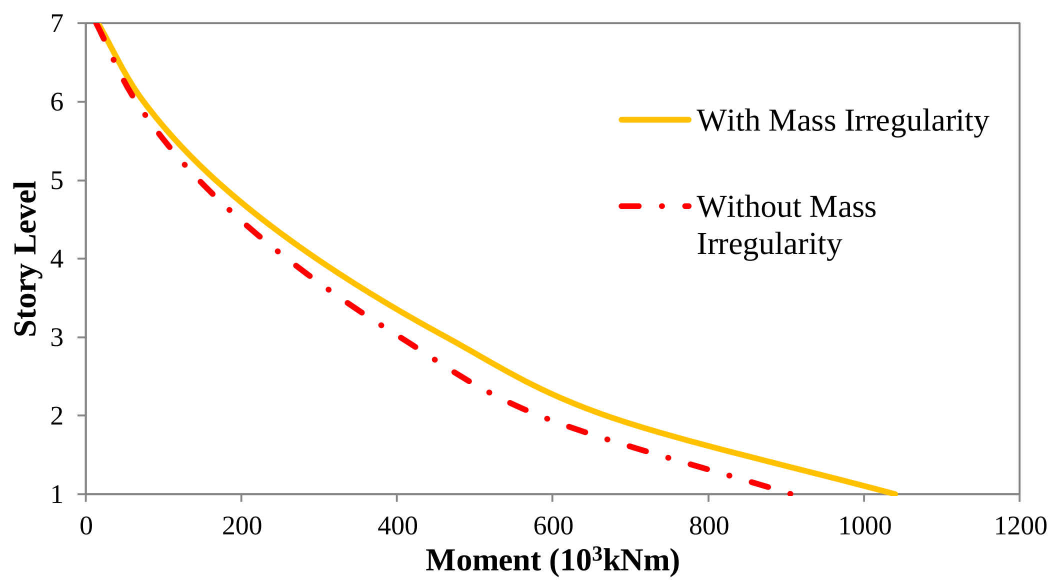


Figure 4.15: Comparison of peak moment of mass regular and mass irregular models in negative X direction

A similar procedure carries on in both positive and negative Y directions with applying pushover load 38kN/m^2 and 56kN/m^2 on respective directions. The comparison of both mass regular and mass irregular hospital model are illustrated by: the peak shear force and moment of positive Y direction are shown in Figure 4.16 and Figure 4.17. While Figure 4.18 and Figure 4.19 show the shear force and moment of negative Y direction.

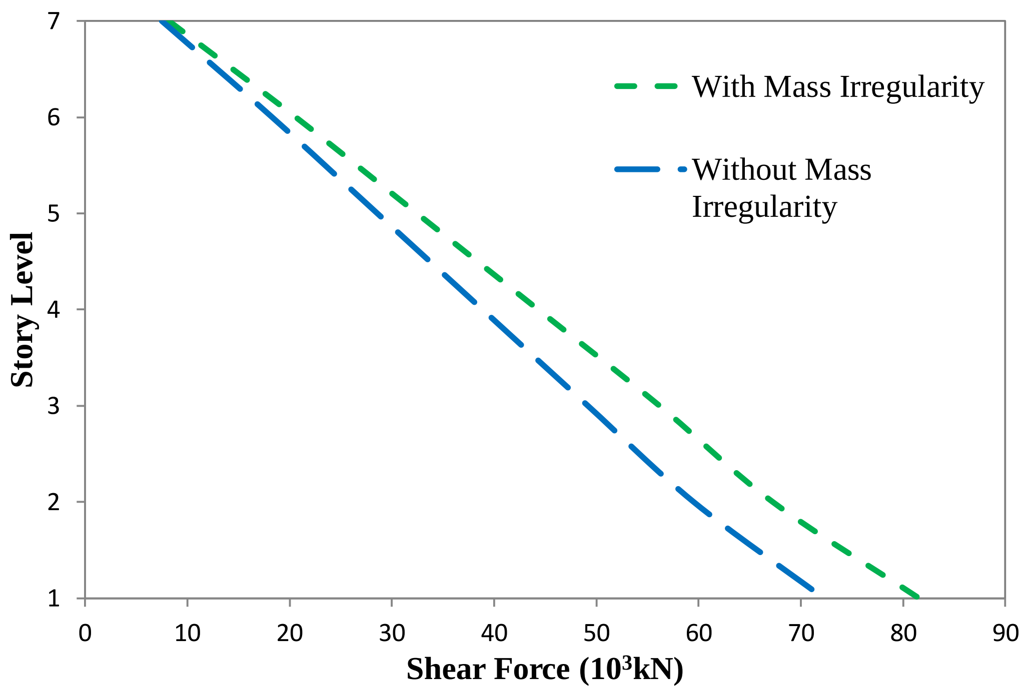


Figure 4.16: Comparison of peak shear force of mass regular and mass irregular models in positive Y direction

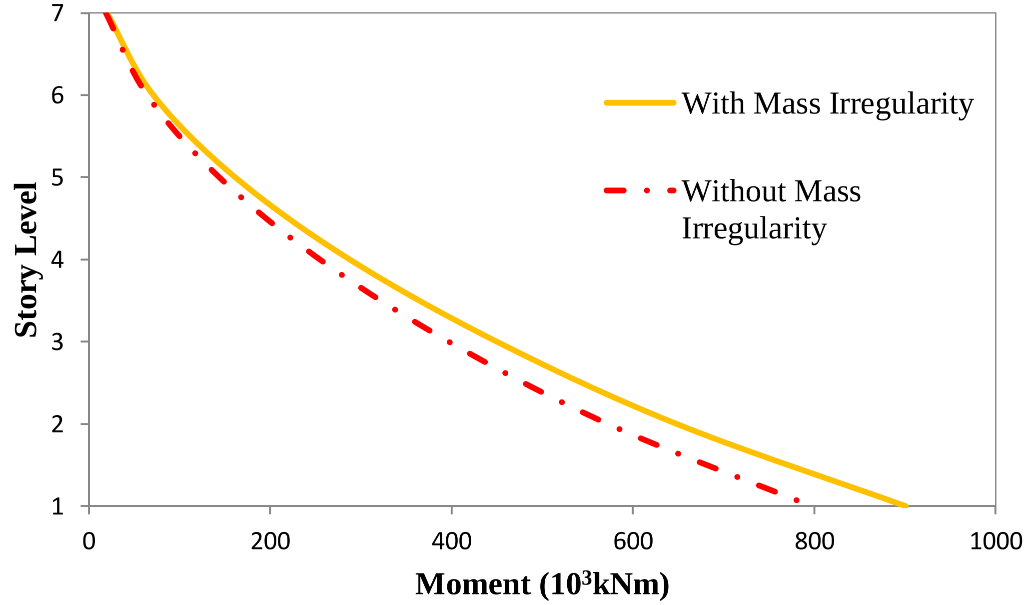


Figure 4.17: Comparison of peak moment of mass regular and mass irregular models in positive Y direction

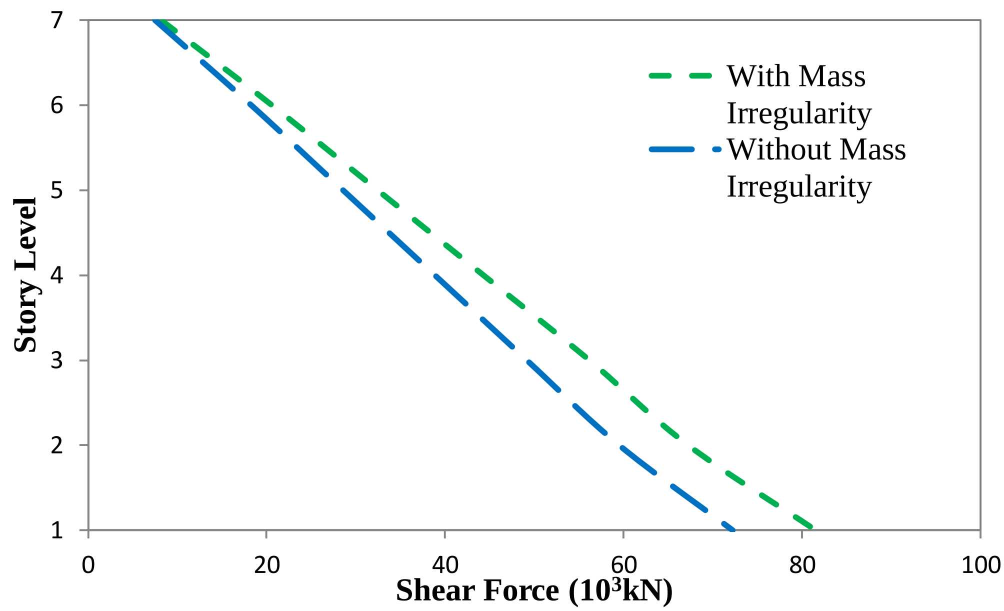


Figure 4.18: Comparison of peak shear force of mass regular and mass irregular models in negative Y direction

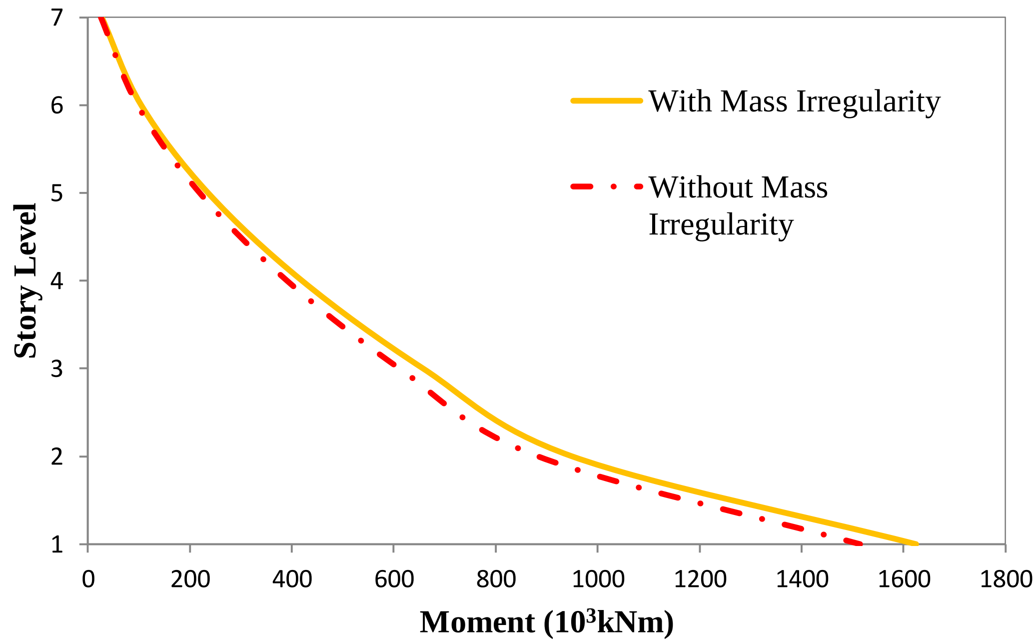


Figure 4.19: Comparison of peak moment of mass regular and mass irregular models in negative Y direction

A similar study is found that the model which has mass irregular frames experience larger structure shear than the model with similar regular building frames (Bansal and Gagandeep, 2014). When pushover load is applied in respectively direction, the peak shear force and moment of story is in ground story and decreasing of shear force while moving up in the building. A trend can be observed through the comparison of mass regular and mass irregular buildings in terms of shear force and moment. Mass regular building results in lower shear force and moment comparing to mass irregular building. In a nutshell, the existence mass irregularity creates weakness of a building by resulting on more shear force and moment during seismic action (Bansal and Gagandeep, 2014).

4.3.2 Plan Irregularity

Diaphragm discontinuity indicates a ratio of opening area of structures to total diaphragm area, the diaphragm openings are for the purpose of stairways, shafts or architectural features. Diaphragm discontinuity influences the seismic vulnerability of high-rise buildings by reducing the base shear which leads a structure to withstand lesser seismic force during earthquake (Kumar and Gundakalle, 2015). A comparison between three different diaphragm openings: 5%, 20% and 40% opening areas of the total diaphragm area of the hospital model are purposed. The base shear in four different directions are shown in Table 4.9. The greater the diaphragm opening results in lesser base shear which can be compared in Figure 4.20, diaphragm discontinuity influences the seismic performance by attracting lesser seismic force (Ahmed and Raza, 2014).

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Table 4.9: Base shear in different directions with 5%, 20% and 40% diaphragm opening

Opening	Direction	Base Shear (10^3 kN)
5%	Positive X	87.91
	Negative X	85.41
	Positive Y	88.96
	Negative Y	130.41
20%	Positive X	85.41
	Negative X	55.48
	Positive Y	73.87
	Negative Y	125.58
40%	Positive X	43.80
	Negative X	42.71
	Positive Y	43.92
	Negative Y	76.44

As the comparison shown in Figure 4.20 the greater diaphragm opening area results in lesser base shear, provision of diaphragm opening alters the seismic performance of the structures (Kumar and Gundakalle, 2015).

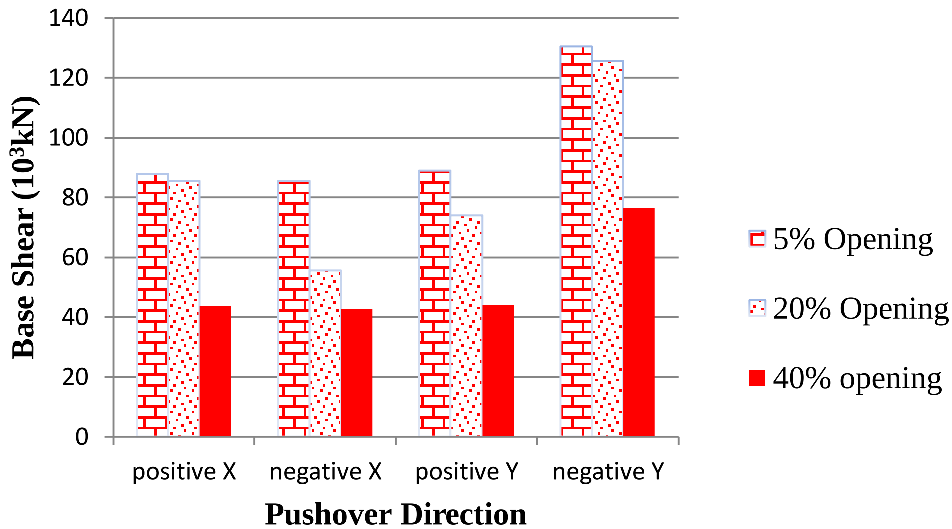

Figure 4.20: Base shear value for different diaphragm discontinuity

4.4 Strengthening RC building by Reinforced Concrete Braced Frame

Buildings are designed to transfer vertical load effectively such as permanent loads, variable loads, imposed loads, snow loads, etc. Besides vertical loads, buildings are subjected to lateral loads such as seismic and wind loads which cannot be negligible during design phase. In order to increase the resistance of lateral load- seismic load, the strengthening of RC buildings by shear walls or RC braced frame system is applied (Kevadkar and Kodag, 2013).

A contrast of both normal RC hospital building and strengthened RC hospital building with RC braced frame has been conducted. The lateral displacement of each story is shown in Table 4.10 and Table 4.11 which the pushover load is applied on 4 respective directions of the buildings.

Table 4.10: Story displacement of hospital building without wall bracing system under failure pushover load

Hospital Building without Wall Bracing System			
Direction	**pushover load (kN/m^2)**	**Story**	**Lateral Displacement (mm)**
Positive X	40	7	62.6
		6	60.3
		5	57.2
		4	53
		3	47.9
		2	42.2
		1	36.1
Negative X	39	7	60.6
		6	58.7
		5	55.8
		4	51.7
		3	46.7
		2	41.1
		1	35.2

Direction	pushover load (kN/m^2)	Story	Lateral Displacement (mm)
Positive Y	38	7	61.2
		6	56.7
		5	51.6
		4	45.8
		3	39
		2	31.2
		1	22.1
Negative Y	56	7	61.1
		6	59.1
		5	55.5
		4	50.5
		3	44
		2	35.6
		1	24.7

Table 4.11: Story displacement of hospital building with wall bracing system under failure pushover load

Hospital Building with Wall Bracing System			
Direction	**Pushover load (kN/m^2)**	**Story**	**Lateral Displacement (mm)**
Positive X	40	7	54.8
		6	52.9
		5	49.7
		4	45.5
		3	40.2
		2	34.3
		1	28.3

Hospital Building with Wall Bracing System			
Direction	**Pushover load (kN/m^2)**	**Story**	**Lateral Displacement (mm)**
Negative X	39	7	55.1
		6	53.0
		5	49.8
		4	45.6
		3	40.4
		2	34.5
		1	27.2
Positive Y	38	7	59.4
		6	54.6
		5	49.2
		4	43
		3	35.8
		2	27.7
		1	18.5
Negative Y	56	7	56.5
		6	54.4
		5	50.7
		4	45.7
		3	39.4
		2	32.3
		1	25.5

The lateral displacement of RC bracing framed model is reduced by 6.39% to 20.50% in positive X direction as compared to the lateral displacement of RC model without wall bracing system. In negative X direction, the lateral displacement of RC bracing framed model is reduced by 6.77% to 22.16%. Both Figure 4.21 and Figure 4.22

show the comparison of story displacement between with and without wall bracing system under pushover load in both positive X and negative X direction.

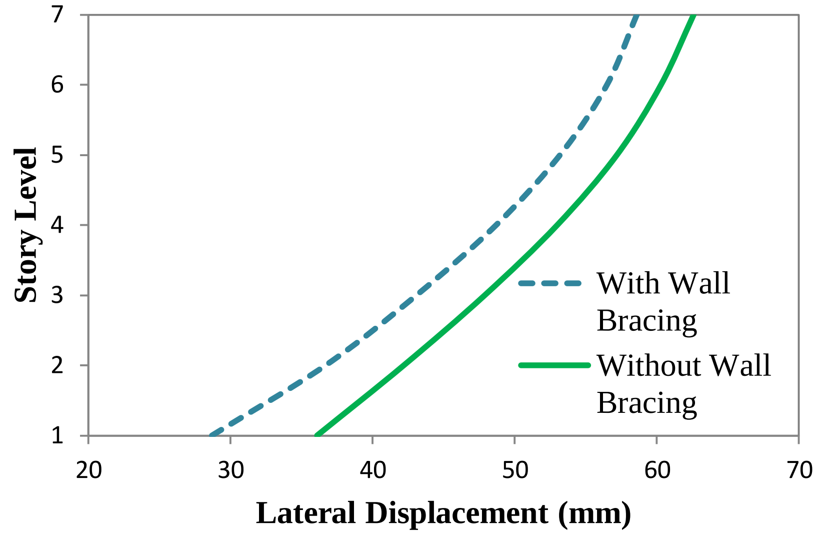

Figure 4.21: Comparison of story displacement with and without wall bracing system under failure pushover load in positive X direction

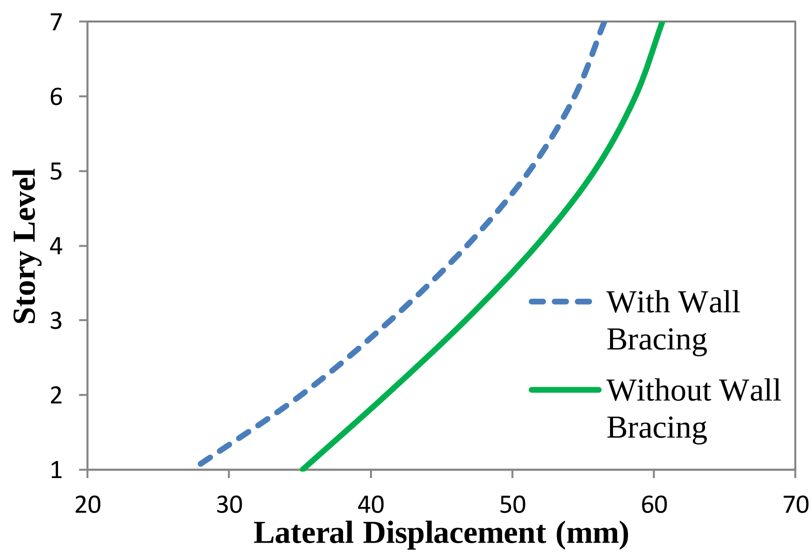

Figure 4.22: Comparison of story displacement with and without wall bracing system under failure pushover load in negative X direction

When pushover load applied on both positive Y direction and negative Y direction, lateral displacement of RC bracing framed model in both positive Y direction and negative Y direction has been slightly decreased by 2.94% to 16.29% as compared to normal RC hospital model without frame bracing system. Both Figure 4.23 and Figure 4.24 compare the lateral displacement of both models with wall bracing and without wall bracing.

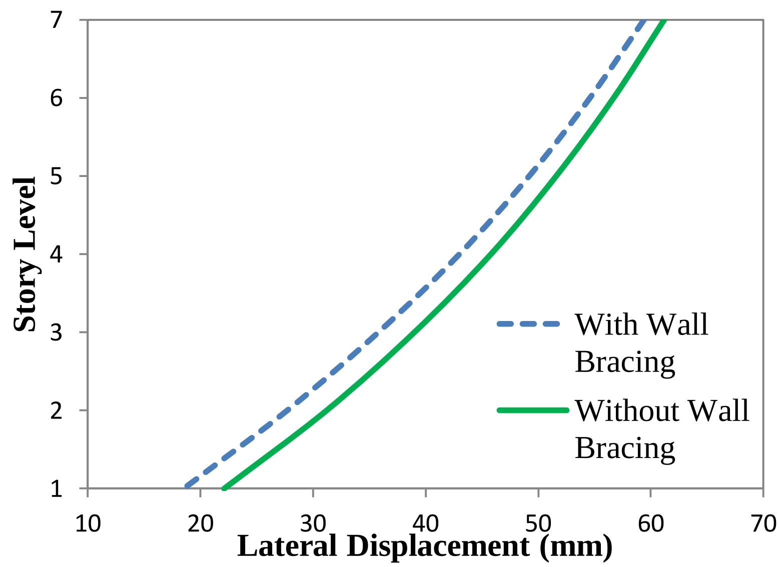

Figure 4.23: Comparison of story displacement with and without wall bracing system under failure pushover load in positive Y direction

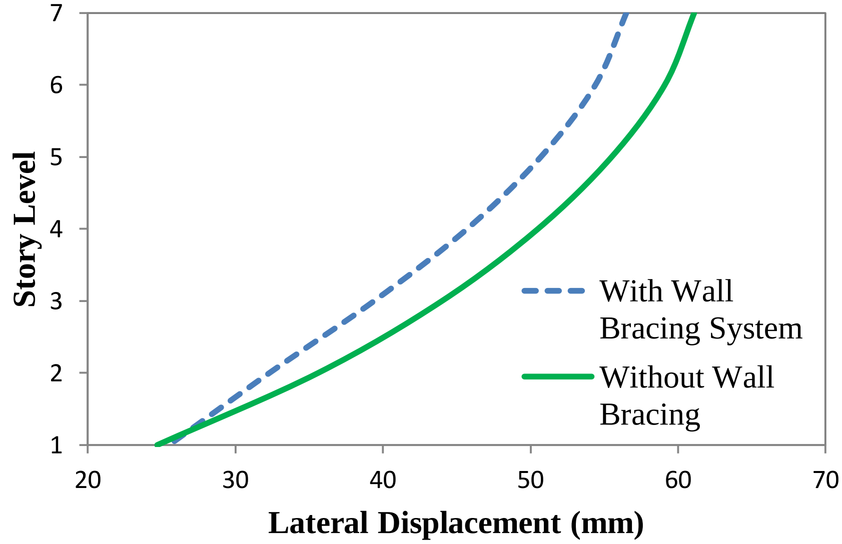

Figure 4.24: Comparison of story displacement with and without wall bracing system under failure pushover load in negative Y direction

A similar study found that braced frame system has the ability to increase the lateral load resistance of buildings by reducing the lateral displacement of high-rise buildings (Kulkarni, Kore and Tanawade, 2013). After the analysis of the structure with the comparison RC baring system, it can be concluded that the utilization of RC bracing system enables to improve the seismic performance of a structure by reducing the lateral displacement of structure during a seismic action. The total weight of structure do not alter significantly after the applying of RC wall bracing system (Mohammed and Nazrul, 2013).

4.5 Critical Review

The main achievement of this study is to obtain the crucial component of pushover analysis: capacity curve in 4 directions of the model to further determine the most vulnerable direction of the model. Results show that the most seismically vulnerable direction is negative X direction. In contrast, negative Y is the least seismically vulnerable direction.

Achievements of this study are the study of impacts of irregularities and braced frame system are obtained: The overall results of irregularities of buildings are adversely influence to buildings by weakening its seismic performance. However, irregularities are unavoidable during the design of real buildings or non-building structure such as discontinuity diaphragm and uneven mass distribution. Therefore, strengthening of seismic-resistant buildings is necessary in order to not collapse while resisting a seismic force. The overall results also show that the application of braced frame system effectively reduces the lateral displacement of buildings.

CHAPTER 5

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

Pushover analysis is a nonlinear static analysis used to evaluate seismic performance of buildings. The main objectives of this study have met which is to obtain the knowledge of the nonlinear behavior of high-rise buildings when subjected to seismic force.

5.1.1 Pushover Analysis

Pushover analysis is a nonlinear static analysis applied to simulate the seismic force acting laterally on the buildings: the key element of pushover analysis is the capacity curve (pushover curve), which enables the determination of failure mechanism of analyzed building.

5.1.2 Irregularity

The study regarding to the effects of irregularities of a buildings toward its seismic vulnerability has been conducted: Irregularities of a building creates weakness to itself. Irregularities is unavoidable in reality, therefore, understanding the influences of irregularity is a crucial part while designing a seismic-resistant buildings.

5.1.3 Strengthening of Seismic-resistant Buildings

This study regarding to the strengthen part of a design of seismic-resistant building by braced frame system. The application of braced frame system enables the reduction of displacement under seismic force without significantly altering of the self-weight of the models which is an economic way to strengthen the buildings without the application of shear walls.

5.2 Recommendation

Following are the main recommendations which are based on current study:

Pushover analysis capable to determine the failure mechanisms of buildings, but this analysis is lack of dynamic consideration of seismic action. Since seismic action is a dynamic force, combination with other types of dynamic analysis should be utilized to well-evaluate the seismic performance of buildings especially high-rise buildings. The further improvement can be carried out by pushover analysis is to find the performance point of the buildings by the intersection of demand capacity curve. It is also recommended to apply shear wall or frame bracing system to strengthen the buildings to become more seismic-resistance upon a seismic action.

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APPENDICE A: Reinforcement requirement in accordance of EC 8

1. Dimensioning & Detailing of Primary Seismic Beams

A beam is a horizontal component of structure that mainly subjected to bending and transverse loading. Beam does not develop significant axial compression in the design seismic situation. The critical region length of beam is equal to 1.5 h_w.

- Longitudinal bar requirement

i. The minimum ratio of longitudinal reinforcement should be equal to:

$$\rho_{min} = \frac{A_{s,min}}{bd} = 0.5\frac{f_{ctm}}{f_{yk}}\ (tension\ zone)$$

ii. The maximum ratio of longitudinal reinforcement should be equal to:

$$\rho_{max} = \rho' + 0.0018\ f_{cd}\ /\ \mathcal{E}_{yd}\ \mu_{\phi} f_{yd}\ (critical\ region)$$

iii. Minimum area of rebar at the bottom of the critical region:

$$A_{s,min} = 0.5A_{s,top}$$

iv. Minimum area of rebar at the support bottom:

$$A_{s,min} = A_{s,bottom-span}/4$$

v. Maximum diameter of longitudinal beam bars crossing joints:

$$interior\ joints{:}\ d_{bL}/h_c\ \leq 7.5f_{ctm}(1+0.8v_d)/\ \gamma_{Rd}f_{yd}(1+k\rho/\rho_{max})$$

$$exterior\ joints{:}\ d_{bL}/h_c\ \leq 7.5f_{ctm}(1+0.8v_d)/\ \gamma_{Rd}f_{yd}$$

- Transverse bar requirement

i. outside critical region:

Spacing of transverse bar should be less than 0.75d.

$$\rho_w \geq 0.08\ f_{ck}^{0.5}\ /f_{yk}$$

ii. Critical region:

Spacing of transverse bar was selected the minimum value of {225mm, $24d_{bw}$, $h_w/4$, $8d_{bL}$}

Diameter of transverse bar: $d_{bw} \geq 6mm$.

- Shear Design

i $$V_{Ed}, seismic = \sum M_{Rb}/l_{cl} \; + V$$

ii $$V_{Rd,max}, seismic = 0.3\, b_w z f_{cd} sin2\theta(1 - f_{ck}/250)\,, 1 \leq cot\theta \leq 2.5$$

iii $$V_{Rd,s}, outside\ critical\ region = b_w z \rho_w f_{ywd} cot\theta, 1 \leq cot\theta \leq 2.5$$

iv $$V_{Rd,s}, critical\ region = b_w z \rho_w f_{ywd} cot\theta, 1 \leq cot\theta \leq 2.5$$

2. Dimensioning & Detailing of Primary Seismic Columns

A column is defined as a generally vertical component that subjected to gravity loads by axial compression. The axial compression developed by columns cannot be neglected in the design seismic situation. The critical region length of the column is determined among

- Longitudinal bar requirement

i. The minimum ratio of longitudinal reinforcement should be equal to: $\rho_{min} = 1\%$

ii. The maximum ratio of longitudinal reinforcement should be equal to: $\rho_{max} = 4\%$

iii. Diameter of longitudinal bar: $d_{bL} \geq 8mm$

iv. Bar per each size ≥ 3

v. Spacing between restrained bar ≤ 200mm

vi. Distance of compression bar (unrestrained bar) to nearest restrained bar ≤ 150mm

- Transverse bar requirement

i. Outside critical region

Diameter of transverse bar $d_{dw} \geq$ 6mm or $d_{dL}/4$

Spacing of transverse bar $s_w \leq 20d_{dL}$, min{h_c,b_c} or 400mm

s_w at lap splice $\leq$ $12d_{dL}$, 0.6 min{h_c,b_c} or 240mm

ii. Within the critical region

Diameter of transverse bar $d_{dw} \geq$ 6mm or $d_{dL}/4$

Spacing of transverse bar $s_w \leq 8d_{dL}$, $b_o/2$ or 175mm

iii. Capacity design – beam column joint

$$\sum M_{RC} \geq 1.3 \sum M_{Rb}$$

iv. Axial load ratio

$$v_d = N_{Ed}/A_c f_{cd} \leq 0.65$$

v. Shear design

$$V_{Ed}, seismic = \gamma_{Rd} M_{Rc,ends}/l_{cl}$$

$$V_{Rd,max}, seismic = 0.3\, b_w z f_{cd} sin2\theta(1 - f_{ck}/250)\,, 1 \leq cot\theta \leq 2.5$$

$$V_{Rd,s} = b_w z \rho_w f_{ywd} cot\theta + N_{Ed}(h - x)/l_{cl}, 1 \leq cot\theta \leq 2.5$$

3. Dimensioning & Detailing of Ductile Wall

- Dimension

i	Web thickness, b_{wo}	$\geq$ max{150mm, $h_{storey}/20$}
		$\geq$ max{l_w, $H_w/6$}
ii	Critical region length,h_{cr}	$\leq$ min{$2l_w$, h_{sotrey}} for n $\leq$ 6 storey
		$\leq$ min{$2l_w$,$2h_{sotrey}$} for n > 6 storey

- Boundary Elements

Critical region

i	Length of l_c from edge	$\geq$ max{$0.15l_w$, $1.5b_w$}
		Length over which ε_c>0.0035
ii	Thickness of b_w over l_c	$b_w \geq 0.20$m & $b_w \geq h_s/10$
		$l_c \geq$ max{$2b_w$,$0.2l_w$}
		and
		$b_w \geq 0.20$m & $b_w \geq h_s/15$
		$l_c \leq$ max{$2b_w$,$0.2l_w$}

- Vertical Reinforcement

i. Minimum ratio of vertical reinforcement: ρ_{min} = 0.005 over A_c= l_c b_w

ii. Maximum ratio of vertical reinforcement: ρ_{max} =0.04 over A_c

4. Confining Hoops

- Web

a. Vertical Reinforcement

i. Minimum ratio of vertical reinforcement: $\rho_{v,min} \geq 0.005$, ε_c>0.002

ii. Maximum ratio of vertical reinforcement: $\rho_{v,max}$ = 0.04

iii. Spacing of vertical bar, $s_v \leq$ min{ $3b_{wo}$,400mm}

b. Horizontal Reinforcement

i. Minimum ratio of horizontal reinforcement $\rho_{h,min}$ = max{$0.001A_c$, $0.25\rho_v$}

ii. Spacing of horizontal reinforcement, $s_h \leq$ 400mm

c. Axial Load Ratio

i. Normalized axial load $v_d \leq 0.4$

d. Design moments, M_{Ed}

i. If the $h_w/l_w \geq 2.0$, M_{Ed} for analysis cover only tension.

e. Shear Design

i. Design shear force: $V_{Ed}= 1.5\ V_{Ed,seismic}$

ii. Outside critical region

$$V_{Rd,max}, seismic = 0.3\ b_{wo}(0.8l_w)f_{cd}sin2\theta(1 - f_{ck}/250)\,, 1 \leq cot\theta \leq 2.5$$

$$V_{Rd,s} = b_{wo}(0.8l_w)\rho_h f_{ywd}cot\theta, 1 \leq cot\theta \leq 2.5$$

iii. Critical Region in Web

$$V_{Rd,max}, seismic = 0.3\ b_{wo}(0.8l_w)f_{cd}sin2\theta(1 - f_{ck}/250)\,, 1 \leq cot\theta \leq 2.5$$

$$V_{Rd,s} = b_{wo}(0.8l_w)\rho_h f_{ywd}cot\theta, 1 \leq cot\theta \leq 2.5$$

APPENDICE B: Strength classes of concrete (Mosley, Hulse and Bungey, 2012)

Table 1.2 Strength classes of concrete

Class	*f_{ck} (N/mm^2)*	*Normal lowest class for use as specified*
C16/20	16	Plain concrete
C20/25	20	Reinforced concrete
C25/30	25	
C28/35	28	Prestressed concrete/Reinforced concrete subject to chlorides
C30/37	30	Reinforced concrete in foundations
C32/40	32	
C35/45	35	
C40/50	40	
C45/55	45	
C50/60	50	
C55/67	55	
C60/75	60	
C70/85	70	
C80/95	80	
C90/105	90	

APPENDICE C: Partial safety factors at the ultimate limit state (Mosley, Hulse and Bungey, 2012)

Table 2.2 Partial safety factors at the ultimate limit state

Persistent or transient design situation	*Permanent actions (G_k)*		*Leading variable action ($Q_{k,1}$)*		*Accompanying variable actions ($Q_{k,i}$)*	
	Unfavourable	*Favourable*	*Unfavourable*	*Favourable*	*Unfavourable*	*Favourable*
(a) For checking the static equilibrium of a building structure	1.10	0.90	1.50	0	1.50	0
(b) For the design of structural members (excluding geotechnical actions)	1.35*	1.00	1.50	0	1.50	0
(c) As an alternative to (a) and (b) above to design for both situations with one set of calculations	1.35	1.15	1.50	0	1.50	0

Note: *Note that for a single variable action where permanent actions < 4.5 × variable action EC2 allows this figure to be reduced to 1.25. The figure of 1.35 has been used throughout this text.

APPENDICE D: Value of Ψ for different load combinations (Mosley, Hulse and Bungey, 2012)

Table 2.4 Values of Ψ for different load combinations

Action	*Combination* Ψ_0	*Frequent* Ψ_1	*Quasi-permanent* Ψ_2
Imposed load in buildings, category (see EN 1991-1-1)			
Category A: domestic, residential areas	0.7	0.5	0.3
Category B: office areas	0.7	0.5	0.3
Category C: congregation areas	0.7	0.7	0.6
Category D: shopping areas	0.7	0.7	0.6
Category E: storage areas	1.0	0.9	0.8
Category F: traffic area, vehicle weight < 30kN	0.7	0.7	0.6
Category G: traffic area, 30 kN < vehicle weight < 160 kN	0.7	0.5	0.3
Category H: roofs	0.7	0	0
Snow loads on buildings (see EN 1991-1-3)			
For sites located at altitude $H > 1000$ m above sea level	0.7	0.5	0.2
For sites located at altitude $H \leq 1000$ m above sea level	0.5	0.2	0
Wind loads on buildings (see EN 1991-1-4)	0.5	0.2	0

APPENDICE E: Level arm curve (Mosley, Hulse and Bungey, 2012)

$K = M/bd^2 f_{ck}$	0.05	0.06	0.07	0.08	0.09	0.10	0.11	0.12	0.13	0.14	0.15	0.16	0.167
$l_a = z/d$	0.954	0.945	0.934	0.924	0.913	0.902	0.891	0.880	0.868	0.856	0.843	0.830	0.820

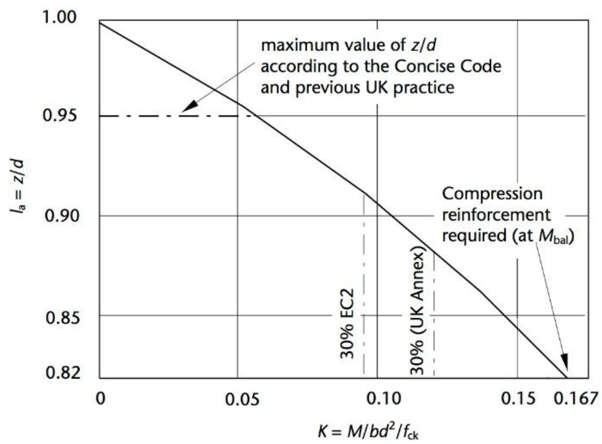
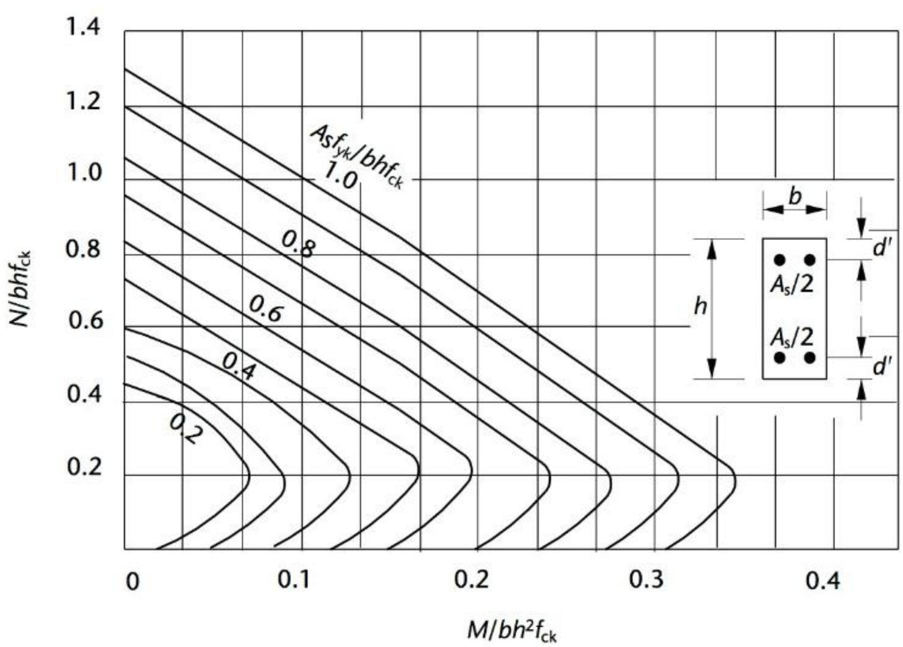


Figure 4.5
Lever-arm curve

The percentage values on the K axis mark the limits for singly reinforced sections with moment redistribution applied (see Section 4.7 and Table 4.2)

APPENDICE F: Typical column design chart (Mosley, Hulse and Bungey, 2012)



APPENDICE G: Nominal cover to reinforcement (Mosley, Hulse and Bungey, 2012)

Table 6.2 Nominal cover to reinforcement (50-year design life, Portland cement concrete with 20mm maximum aggregate size) [Based on BS 8500]

Exposure class	*Nominal Cover (mm)*								
XO	Not recommended for reinforced concrete								
XC1	25 →	→							
XC2	–	35	35 →	→	→				
XC3/4	–	45	40	35	35	35	30 →	→	→
XD1	–	–	45[1]	45	40[1]	40	35[1]	35	35
XD2	–	–	50[2]	50[1]	45[2]	45[1]	40[2]	40[1]	40
XD3	–	–	–	–	–	60[2]	55[2]	50[1]	50
XS1	–	–	–	–	50[2]	45[2]	45[1]	40[1]	40
XS2	–	–	50[2]	50[1]	45[2]	45[1]	40[2]	40[1]	40
XS3	–	–	–	–	–	–	60[2]	55[1]	55
Maximum free water/cement	0.70	0.65	0.60	0.55	0.55	0.50	0.45	0.35	0.35
Minimum cement (kg/m^3)	240	260	280	300	300	320	340	360	380
Lowest concrete	C20/25	C25/30	C28/35	C30/37	C32/40	C35/45	C40/50	C45/55	C50/60

Notes:

1. Cement content should be increased by 20 kg/m^3 above the values shown in the table.
2. Cement content should be increased by 40 kg/m^3 *AND* water–cement ratio reduced by 0.05 compared with the values shown in the table.

General Notes

These values may be reduced by 5 mm if an approved quality control system is specified.

Nominal cover should not be less than the bar diameter + 10 mm to ensure adequate bond performance.

APPENDICE H: Bending-moment coefficients for slabs spanning in two directions at right angles, simply supported on four sides (Mosley, Hulse and Bungey, 2012)

Table 8.4 Bending-moment coefficients for slabs spanning in two directions at right angles, simply supported on four sides

l_y/l_x	*1.0*	*1.1*	*1.2*	*1.3*	*1.4*	*1.5*	*1.75*	*2.0*
a_{sx}	0.062	0.074	0.084	0.093	0.099	0.104	0.113	0.118
a_{sy}	0.062	0.061	0.059	0.055	0.051	0.046	0.037	0.029

APPENDICE I: Column effective lengths (Mosley, Hulse and Bungey, 2012)

Table 9.1 Column effective lengths

$\frac{1}{4}\frac{(I/l_{column})}{(I/l_{beam})} = k$	*0 (fixed end)*	*0.0625*	*0.125*	*0.25*	*0.50*	*1.0*	*1.5*	*2.0*
l_0 – braced (equation 9.2) $\{\times l\}$	0.5	0.56	0.61	0.68	0.76	0.84	0.88	0.91
l_0 – unbraced (equation 9.3(a) and	1.0	1.14	1.27	1.50	1.87	2.45	2.92	3.32
9.3(b)). Use greater value $\{\times l\}$	1.0	1.12	1.13	1.44	1.78	2.25	2.56	2.78

APPENDICE J: Sectional areas of groups of bar (mm^2) (Mosley, Hulse and Bungey, 2012)

Bar areas and perimeters

Table A.1 Sectional areas of groups of bars (mm^2)

Bar size (mm)	*Number of bars*									
	1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*	*10*
6	28.3	56.6	84.9	113	142	170	198	226	255	283
8	50.3	101	151	201	252	302	352	402	453	503
10	78.5	157	236	314	393	471	550	628	707	785
12	113	226	339	452	566	679	792	905	1020	1130
16	201	402	603	804	1010	1210	1410	1610	1810	2010
20	314	628	943	1260	1570	1890	2200	2510	2830	3140
25	491	982	1470	1960	2450	2950	3440	3930	4420	4910
32	804	1610	2410	3220	4020	4830	5630	6430	7240	8040
40	1260	2510	3770	5030	6280	7540	8800	10100	11300	12600

APPENDICE K: Sectional areas per metre width for various bar spacing (mm^2) (Mosley, Hulse and Bungey, 2012)

Table A.3 Sectional areas per metre width for various bar spacings (mm^2)

Bar size (mm)	*Spacing of bars*								
	50	*75*	*100*	*125*	*150*	*175*	*200*	*250*	*300*
6	566	377	283	226	189	162	142	113	94
8	1010	671	503	402	335	287	252	201	168
10	1570	1050	785	628	523	449	393	314	262
12	2260	1510	1130	905	754	646	566	452	377
16	4020	2680	2010	1610	1340	1150	1010	804	670
20	6280	4190	3140	2510	2090	1800	1570	1260	1050
25	9820	6550	4910	3930	3270	2810	2450	1960	1640
32	16100	10700	8040	6430	5360	4600	4020	3220	2680
40	25100	16800	12600	10100	8380	7180	6280	5030	4190

APPENDICE L: Shear reinforcement (Mosley, Hulse and Bungey, 2012)

Shear reinforcement

Table A.4 A_{sw}/s for varying stirrup diameter and spacing

Stirrup diameter (mm)	*Stirrup spacing (mm)*										
	85	*90*	*100*	*125*	*150*	*175*	*200*	*225*	*250*	*275*	*300*
8	1.183	1.118	1.006	0.805	0.671	0.575	0.503	0.447	0.402	0.366	0.335
10	1.847	1.744	1.57	1.256	1.047	0.897	0.785	0.698	0.628	0.571	0.523
12	2.659	2.511	2.26	1.808	1.507	1.291	1.13	1.004	0.904	0.822	0.753
16	4.729	4.467	4.02	3.216	2.68	2.297	2.01	1.787	1.608	1.462	1.34

Note: A_{sw} is based on the cross-sectional area of two legs of the stirrup.

APPENDICE M: Maximum and minimum areas of reinforcement (Mosley, Hulse and Bungey, 2012)

Maximum and minimum areas of reinforcement

Table A.7 Maximum areas of reinforcement

(a) For a slab or beam, tension or compression reinforcement
$100A_s/A_c \leq 4$ per cent other than at laps
(b) For a column
$100A_s/A_c \leq 4$ per cent other than at laps and 8 per cent at laps
(c) For a wall, vertical reinforcement
$100A_s/A_c \leq 4$ per cent

APPENDICE N: Floor and roof load ((Mosley, Hulse and Bungey, 2012)

Floor and roof loads

	kN/m^2
Classrooms	3.0
Dance halls	5.0
Flats and houses	1.5
Garages, passenger cars	2.5
Gymnasiums	5.0
Hospital wards	2.0
Hotel bedrooms	2.0
Offices for general use	2.5
Flat roofs, with access	1.5
Flat roofs, no access	0.60