ADDIS ABABA UNIVERSITY

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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Torsional Behavior of Multistory Plan Asymmetric RC Building Under Seismic Load

A Thesis in Structural Engineering

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ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
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The undersigned have examined the thesis entitled '**Torsional Behavior of Multistory Plan Asymmetric RC Building Under Seismic Load**' presented by **Solomon Liranso Jikamo**, a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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UNDERTAKING

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Solomon Liranso Jikamo

ABSTRACT

Nowadays, it is highly impossible to plan with regular shapes; due to architectural and functional requirements; thus, horizontal and/or vertical irregularity may be developed. These irregularities are responsible for the structural collapse of buildings under the action of dynamic loads. Past and recent earthquakes events demonstrate that plan asymmetric buildings are very susceptible to earthquake-induced damage due to torsional effects in addition to translational effects. So, it is essential to investigate the seismic response of these structures in active seismic zones to reduce the potential seismic damages. The objectives of this thesis are to study the torsional behavior of reinforced concrete buildings with plan asymmetry under earthquake load through parameters reflect the torsional effects and recommending the practical solution in order to reduce the torsional effects. To carry out the study, both regular reference and L-shaped plan asymmetric thirteen story buildings, three-dimensional finite element models have been developed and analysis is performed by modal response spectrum method according to EBCS EN 1998 2015 in ETABS 2016. The results prove that buildings with severe plan asymmetries are more vulnerable than those with regular counterpart resulting from torsion behavior. Two ways in order to reduce the torsional irregularity effects resulted from plan asymmetry are recommended as the practical solutions; those are an equivalent shear wall and diagrid system. From the study, those systems have provided considerable reduction in responses demand; based on such results, those ways in similar plan asymmetric building are suggested.

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LIST OF SYMBOLS AND ABBREVIATIONS

A_x torsional amplification factor

a_g peak ground acceleration

a_{gR} reference peak ground acceleration

ABS absolute sum

γ_I importance factor

CM center of mass

CQC complete quadratic combination

CR center of rigidity

DCM ductility class medium

d_{avg} average of the story drifts at the two ends of the structure including the accidental torsion

d_{max} maximum story drift, computed including the accidental torsion, at either end of the structure

e_a accidental eccentricity

E_{cm} modulus of elasticity of concrete

EI flexural rigidity

e_{ox}, e_{oy} distance between the center of stiffness and the center of mass, measured along the x, y-direction

EQ Earthquake

G permanent load

K_f stiffness coefficient

K lateral stiffness

LLRS Lateral Load Resisting System

ls radius of gyration of the floor mass in plan

M_s magnitude of earthquake

δ accidental torsional amplification factor

O_a cutout or open areas of diaphragm

Q variable-live loads

r_X, r_Y torsional radius along the x, y-direction

S_{dst} effective diaphragm stiffness

RM regular model (reference model)

R_i re-entrant corner irregularity

Ψ_{Ei} variable-live loads reduction factor

SDOF Single Degree-Of-Freedom

θ_f diaphragm rotation

SRSS square root of the sum of the squares

S soil factor

γ unit weight of the concrete

T_B lower limit of the period of the constant spectral acceleration branch

T_C upper limit of the period of the constant spectral acceleration branch

T_D value defining the beginning of the constant displacement response range of the spectrum

CHAPTER 1 INTRODUCTION

1.1 Background

In these modern days, most of the structures are involved with architectural importance and it is highly impossible to plan with regular shapes [1]; thus, horizontal and/or vertical irregularity may be developed. These irregularities are responsible for the structural collapse of buildings under the action of dynamic loads. Assessment of the performance of building structures during past earthquakes suggests that, among two types of irregularities, plan asymmetric buildings are very susceptible to earthquake-induced damage due to torsional effects in addition to translations [2]. The investigations deserve to understand the torsional behavior of this plan asymmetric buildings to avoid or minimize such problems during the design of structures.

Research works on plan irregular building systems started in early 1980's with Tso and Sadek (1985) [3] by performing one story mass eccentric model due to their simplicity and their ability to clearly depict the effect of different seismic response parameters. But several researchers proved that single story models resulted in the inaccurate prediction of torsional response [4]. The multi-story building models give a realistic prediction of torsional response. The development of powerful software tools has made modeling and analysis of multi-story building models much simpler.

1.2 Statement of the Problem

The seismic torsional response has been a principal cause of structural failure in every major earthquake. The torsion-induced failures have been especially catastrophic for plan asymmetric multi-story buildings. The seismic codes try to take into account the torsion effect during modeling; however, it is difficult to assess all the parameters that affect the behavior of this kind of structures.

Due to the torsional response, the structural design of plan asymmetric buildings requires advanced seismic assessment and design guidelines. Consequently, it usually needs more iterative assessments and adjustments.

1.3 Objective

1.3.1 General Objective

The main objective of this thesis is to study the torsional behavior of reinforced concrete multistory buildings with plan asymmetry under earthquake load.

1.3.2 Specific Objectives

- Review of research works regarding plan irregularities
- Review of Code provisions for the torsional effects
- Determination of parameters that reflect the torsional effects: include eccentricity, torsional irregularity ratio, story drift, story displacement, base shear force, and torsional diaphragm rotation.
- Recommending some practical solutions for torsional irregularity effects.

1.4 The Scope of the Research

This thesis focuses on the torsional behavior of plan asymmetric multi-story reinforced concrete building in a high seismic zone in Ethiopia. To carry out the study torsional behavior of plan asymmetric reinforced concrete building, both symmetric and L- shaped plan asymmetric multi-story (moderate rise) buildings have been modeled and analysis is performed by modal response spectrum method according to EBCS EN 1998 2015 in ETABS 2016. The irregularities other than L-shaped plan asymmetry is not considered in this thesis and the study does not consider low or moderate seismic zone rather than high seismic zone in Ethiopia. Also, a non-linear seismic analysis is not included in this thesis work.

1.5 Report Organization

The thesis report is made up of five chapters. Chapter 1 introduces the background, problem statement, objectives, and scope of this research. Chapter 2 presents a literature review related to the research. This chapter reports the structural irregularities in the buildings, classification of structural irregularity in buildings, review of research works on plan irregular building system and review codes provisions for torsional effects.

Chapter 3 discusses structural modeling and analysis. In this chapter, the building models with different level of plan asymmetry have been developed at first. Secondly, seismic and vertical actions are discussed. Finally, different analysis methods available to obtain the seismic response have been reviewed and based on the review of analysis methods a suitable method has been adopted for analysis of plan asymmetric building models.

Chapter 4 reports analysis results and discussions. In this chapter analysis reports of different parameters are reported in tables and figures and brief discussions are performed. Also, practical solutions for plan asymmetric effects are presented.

Chapter 5 caps the report with conclusions and recommendation drawn from the study and a discussion of future research requirements.

CHAPTER 2 LITERATURE REVIEW

2.1 Structural irregularity in buildings

When a building is subjected to seismic excitation, the damage generally initiates at a location of the structural weak planes present in the building systems. These weaknesses trigger further structural deterioration which leads to the structural collapse. These weaknesses often occur due to the presence of the structural irregularities [5]. The structural irregularity is defined by the location of the resistant elements: walls, columns, joints with nonstructural elements, floor systems, wall openings, masses, etc., and geometric arrangement result in non-uniform distributions of mass, stiffness, strength or structural form. Eccentricity may occur due to the presence of structural irregularities. Ones, eccentricities are developed in the buildings which further generate torsion. Torsion generates greater damage in the buildings [6].

2.2 Classification of structural irregularity in buildings

The structural irregularity can be broadly classified as plan (horizontal) and vertical irregularities as shown in Figure 2-1.

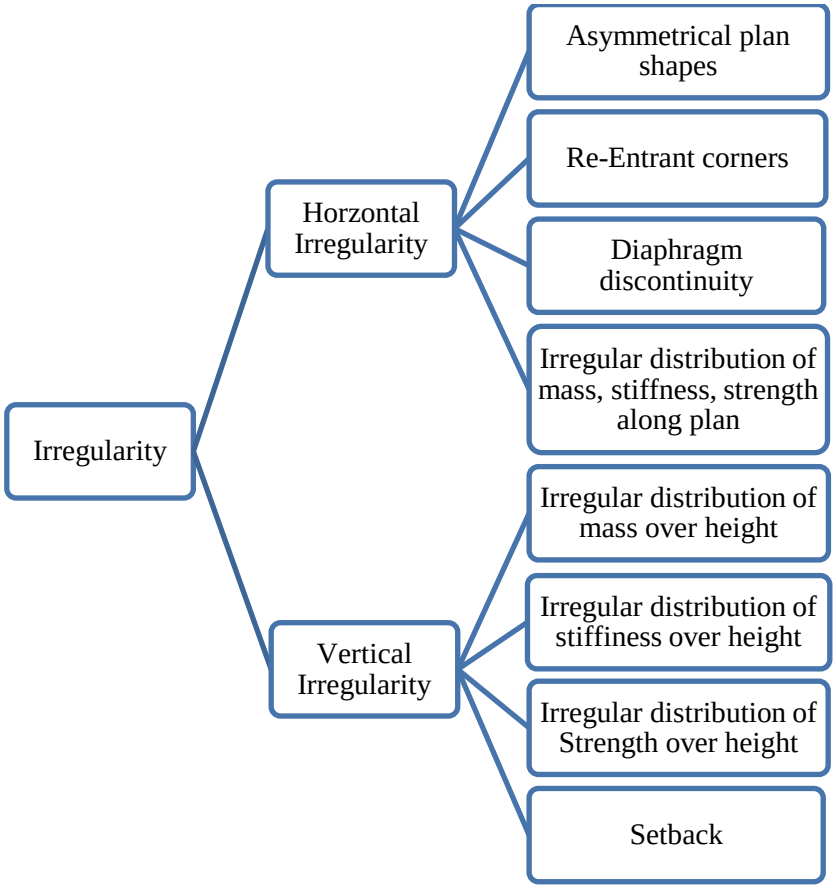


Figure 2-1: Classification of irregularity [3]

A structure can be classified as irregular if the structure exceeds the limits as prescribed by different seismic design codes. Among the irregularity limits prescribed by some codes; for both horizontal and vertical irregularities, based on the scope of the study only for horizontal irregularities have been discussed in Table 2-2.

2.3 Plan asymmetry of the buildings

Plan asymmetry may occur due to the irregular distribution of mass, stiffness, and strength along the plan [3]. Assessment of the performance of building structures during past earthquakes suggests that plan asymmetric buildings are very susceptible to earthquake-induced damage due to lateral torsional coupling, and the corners of these systems suffer heavy damage during earthquakes [2]. In past years a lot of research effort has been done to study the behavior of plan asymmetric buildings during seismic excitation as reviewed subsequently.

2.3.1 Lateral-torsional coupling

Pure torsional excitation in an earthquake may arise in a site across which there are significantly varying soils, but significant torsional excitations on buildings are unusual. However, coupled lateral-torsional excitation, arising from an eccentricity between centers of mass and stiffness, is common and is found to increase damage in earthquakes. For buildings with irregular floor plans having an uneven distribution of mass or stiffness, the center of mass (CM) and the center of rigidity (CR) often do not coincide with each other. During earthquakes, these buildings may vibrate in a lateral-torsional manner with significant swaying and twisting. In the seismic design of modern asymmetric multistory buildings, it is vitally important that the effects of lateral-torsional coupling on the response of such buildings are accurately evaluated and assessed [7, 8].

Symmetric-plan buildings, when subjected to ground motion, there would be no lateral-torsional coupling effects:

- Ground motion in the *X*-direction would cause only lateral motion in the *X*-direction.
- Ground motion in the *Y*-direction would cause only lateral motion in the *Y*-direction.
- The system would experience no torsional motion unless the base motion includes rotation about a vertical axis.

Asymmetric-plan building, when subjected to ground motion, say, the *X*-components of ground motion, would not be restricted to lateral motion only in the *X*-direction but simultaneously undergo lateral motion in two horizontal (*X* and *Y*) directions and torsion about the vertical (*Z*) axis that is coupled lateral-torsional motion [9].

2.3.2 Effects of the plan asymmetry on seismic response of the building

The behavior of a building during earthquakes depends critically on its overall shape, size, and geometry, in addition to how the earthquake forces are shaking the ground. In past and recent earthquakes, most of the damages are related to architectonic and structural configuration in plan and elevation and site ground effects (Raúl González Herrera 2008) [10]. Hence, at the planning stage itself, architects and structural engineers must work together to ensure that the unfavorable features are avoided and a good building configuration is chosen. In these modern days, most of the structures are involved with

architectural importance and it is highly impossible to plan with regular shapes (Momen M. M. Ahmed et al. 2016 [1]. These irregularities are responsible for the structural collapse of buildings under the action of dynamic loads. Hence, extensive research is required for achieving ultimate performance even with a poor configuration. Many researchers have studied the effects of the shape and plan configuration on seismic response of the building.

Raúl González Herrera and Consuelo Gómez Soberón, 2008 [10] showed in their study an analytical description of the damages caused by different plan irregularities, during seismic events of different magnitudes. The effects of different irregularity such as square, rectangular, section U, section L, and section T were studied with qualitative and quantitative analyses. A parametric study of the influence of different plan irregular systems in the elastic displacements responses were presented in their study. To do that elastic model or regular systems (square plan) and irregular models of rectangular, T, L and U plans were modeled in SAP2000 to determine the effect of the geometric form in the seismic behavior of structures with elastic analyses. The realized parametric studies allow them to identify the most important conditions of vulnerability in a qualitative and quantitative way. They concluded that constructions are more vulnerable when more irregular is.

Momen M. M. Ahmed et al. 2016 [1] studied irregularity effects on the seismic performance of multi-story buildings. Three-dimensional models were constructed by ETABS software for analysis and design of structural elements. Seismic analysis and design of reinforced concrete structures were performed based on the linear response. The objective of the study was to grasp the seismic behavior of the buildings with the irregular plan through the evaluation of the configuration irregularity of reentrant corners effects on seismic response demands. They concluded that that the floor shape plays a considerable role in the seismic behavior buildings.

Ravikumar C M et al. 2012 [11] studied the effect of irregular configurations on seismic vulnerability of RC buildings for two kinds of irregularities in the building models namely plan irregularity with geometric and diaphragm discontinuity and vertical irregularity with setback and sloping ground. In order to identify the most vulnerable building among the models considered, the analytical approaches were performed to identify the seismic demands in both linear and nonlinear way. From the studies, it was concluded that the seismic demand significantly varies with respect to the configurations.

Veena S Ravi1 and Sreedevi Lekshmi 2016 [12] studied the effect of shape and plan configuration on seismic response of the structure. The objective of the study was the seismic performance of different shape of structures located in the severe earthquake zone and minor earthquake zone. To carry out the study, seven models of G+11 story building with one regular plan and remaining irregular plan (C, E, H, L, T, PLUS shapes) had been taken. The plan area for each structure is same only there is different in geometry. The elevation is same for all models. The static and dynamic analysis (Response spectrum method) had been done on the computer with the help of STAAD-Pro software using the parameters for the design as per the IS-1893- 2002-Part-1. Performance in terms of base shear, time period, joint displacement was checked and concluded that the irregular shape buildings were severely affected; undergo more deformation during earthquake especially in high seismic zones.

Many researchers who had conducted their studies on the effects of the shape and plan configuration on seismic response of the building identified the vulnerability of irregular buildings qualitatively and /or quantitatively and they recommended as extensive research is required for this issue.

2.3.3 Torsional behavior of plan asymmetric buildings

The torsional behavior of plan asymmetric building is one of the most frequent causes of structural damage and failure during strong ground motions. Many researchers studied the torsional behavior of plan asymmetric building by means of single-story [e.g.: F. Crisafulli 2004 [13]; Ladinovic 2008 [14]] and multi-story building models. Most of the design criteria were formulated on the basis of results obtained in single story models. But researchers proved that single story models resulted in the inaccurate prediction of torsional response [e.g. Fajfar et al. 2002 [4]].

F. Crisafulli et al. 2004 [13] studied the torsional effects of the ductile structure using single story building. A parametric study was performed modeling the Structural systems namely torsionally restrained or unrestrained systems, depending on their capacity to restrain the torsional rotation under unidirectional seismic attack in the principal directions of the building. The different aspects of the problem were studied based on results obtained from static and dynamic analyses of inelastic systems. Based on the result of the study it was concluded that the torsional effects significantly affect the seismic response of buildings, producing an uneven distribution of the lateral displacements. To improve the

understanding of the problem; further investigation is required; for example, the behavior of multistory asymmetric buildings.

With the development of advanced software, the analysis of different types of complex multi-story structures can be easily performed. S.G. Maske and P.S. Pajgade 2013 [15], S.N. Suryawanshi et al. 2014 [16], Vipin Gupta and P.S. Pajgade 2015 [17] studied the torsional behavior of multistory plan asymmetric building by modeling and analyzing in ETABS and SAP2000 software. From the study, they found that torsion was the most critical factor leading to major damage or completes collapse of the building and recommended further investigation deserves for this issue.

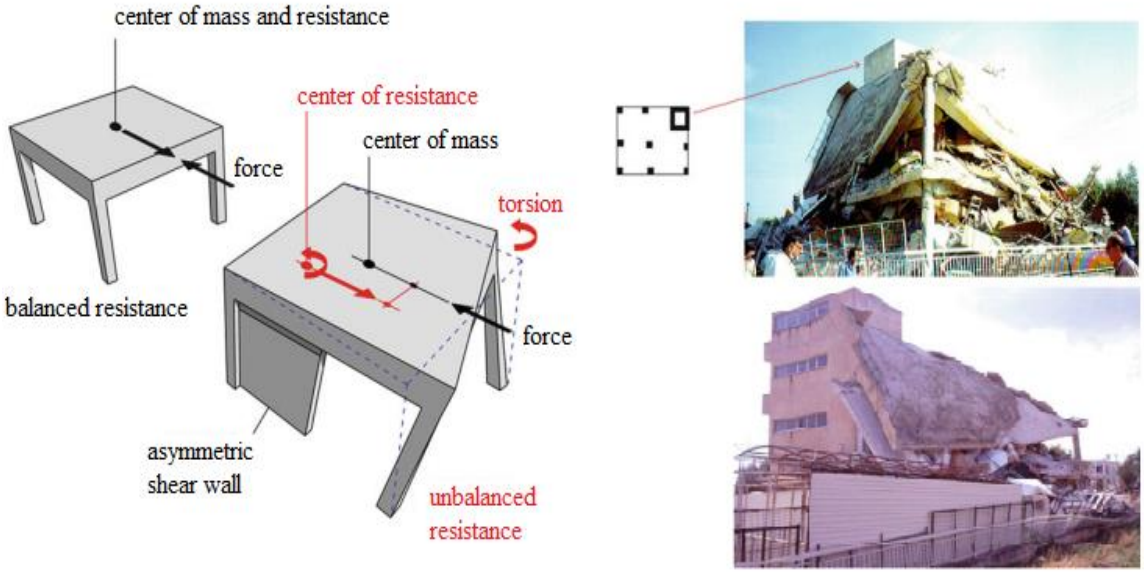


Figure 2-2: Strongly asymmetric stiffness distribution in the plan (Athens, 1999 EQ) [18].

2.3.4 Failure occurred during past earthquakes due to plan irregularity

Damage to irregular structures caused by asymmetry in the plan has been observed during many major and minor earthquakes during the past. The non-coincident centers of mass and stiffness in a structure generate plan asymmetry which causes torsional vibration resulting in severe damage to structural components in the more laterally flexible regions of the structure.

There are works of the literature showed the failure occurred during past earthquakes due to different irregularities. Raúl González Herrera et al. 2008 [10] had summarized the past earthquakes and causes of structures failures investigated by different researchers. Table

2-1 shows the summary of past earthquake. The torsion was the major cause of structures damaged.

Table 2-1: Summaries of past earthquakes damages related to plan irregularities [10]

Earthquake name	Date	M_w	Author		Earthquake name	Date	M_w	Author
El Asnam, Algeria	10/10/80	7.3	EERI, 1983		San Salvador, El Salvador	13/01/01	7.6	Alarcón, 2005
Viña del Mar, Chile	03/03/85	7.8	ICH, 1988		Bhuj, India	26/01/01	7.7	Humar et al, 2001
Michoacán; México	19/09/85	8.1	Popov, 1987 Y Tena, 2004		Tecomán, México	21/01/03	7.8	Alcocer et al, 2006
Spitak, Armenia	07/12/88	6.8	Tena, 2004		Lefkade, Greece	14/08/03	6.2	Karakostas et al, 2005
Luzon, Philippines	16/07/90	7.8	Hopkins, 1993		Bam, Iran	26/12/03	6.5	Tena, 2004
Erzincan, Turkey	13/03/92	6.7	Saatciogl u and Bruneu, 1993		Java, Indonesia	27/05/06	6.3	EERI, 2006
Northridge, USA	17/01/94	6.7	Tena, 2004		Pisco, Peru	15/08/07	8.0	Klinger, 2007
Kobe, Japan	17/01/95	6.9	Tena, 2004		Wenchuan, China	12/05/08	8.3	Xiao, 2008
Kocali, Turkey	17/08/99	7.4	Naeim et al, 2000					

From the review of works of literature, the irregular structures rendered poor seismic performance, and this is mainly due to ignorance of the irregularity aspect in formulating the seismic design methodologies by the seismic codes. Therefore, there is a need for a comprehensive evaluation of effects of different types of irregularity on the seismic response parameters to formulate improved design philosophy for these structures.

2.3.5 Evaluating regularity in plan and codes irregularity limits

Seismic codes set out quantified criteria for assessing structural regularity (Table 2-2).

1. Re-entrant corner irregularity

A ‘compact' shape, i.e. one in which the perimeter line is always convex. Definition of compact shaped and determination of Re-entrant area and perimeter line of the main structure is shown in Figure 2-3.

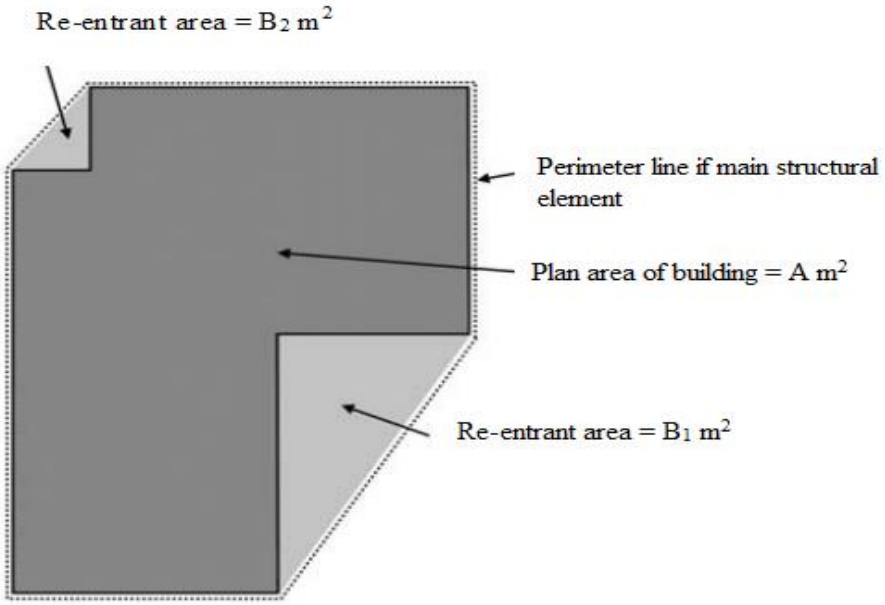


Figure 2-3: Definition of compact shape [7]

Re-entrant corner irregularity computed in Eq 2-1.

$$R_i = \frac{B_1}{A} \quad and \quad R_i = \frac{B_2}{A} \qquad \text{Eq 2-1}$$

2. Torsional Irregularity

According to ASCE 7-05 and UBC 97, torsional irregularity can be defined as the ratio of maximum drift to the average drift of the individual story. It is considered to exist when the maximum story drift, computed including the accidental torsion, at either end of the structure is more than 1.2 times the average of the story drifts at the two ends of the structure (Figure 2-4). The irregularity Limits prescribed in terms of d_{max} and d_{avg} by ASCE 7-05 and UBC 97 are shown in Table 2-2.

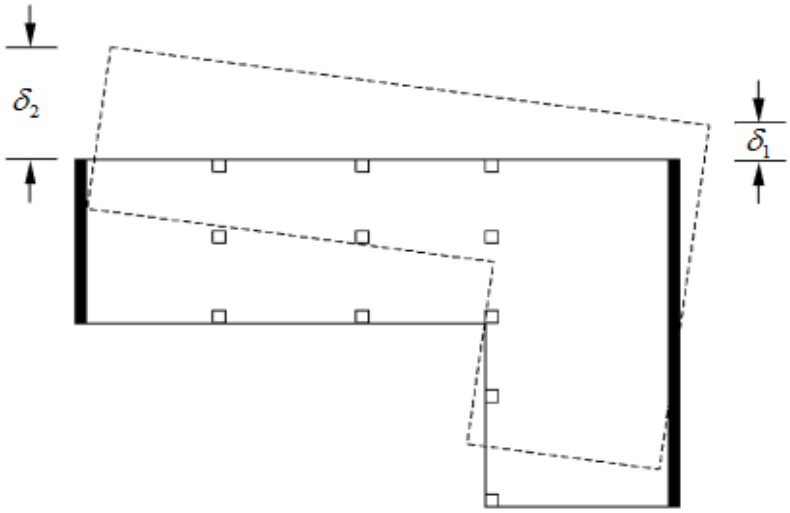


Figure 2-4: Torsional irregularity in ASCE 7-05 and UBC97 [19]

$$d_{avg} = \frac{\delta_1 + \delta_2}{2} \quad and \quad d_{\max} = Max\left(\delta_1, \delta_2\right) \qquad \text{Eq 2-2}$$

EBCS EN 1998-1(4.2.3.2) set out quantified criteria for assessing torsional irregularity in terms of torsional radius r_x (r_y) and eccentricity, e_{ox}, e_{oy} is the distance between the center of stiffness and the center of mass, measured along the x, y direction, which is normal to the direction of analysis considered and can be determined at level *i* from the following equations:

$$e_{0X,j} = \frac{R_{z,i}\left(F_{y,i}=1\right)}{R_{z,i}\left(M_i=1\right)} \quad and \quad e_{0y,j} = \frac{R_{z,i}\left(F_{x,i}=1\right)}{R_{z,i}\left(M_i=1\right)} \qquad \text{Eq 2-3}$$

The torsional radius r_x (r_y) is defined as the square root of the ratio of the torsional stiffness (K_M) to the lateral stiffness in one direction K_{FY} (K_{FX})**.**

$$r_{X,i} = \sqrt{\frac{K_{M,i}}{K_{FY,i}}} \quad and \quad r_{Y,i} = \sqrt{\frac{K_{M,i}}{K_{FX,i}}} \qquad \text{Eq 2-4}$$

Three static load cases are defined for each story level, and loads are represented by F_{TX}, $F_{TY,}$ and M_T, respectively. The forces and moment are applied in the center of stiffness in the case of the determination of the torsional radius and in the case of the determination of the structural eccentricity, forces and moment are applied in the center of mass. The torsional and lateral stiffness for both directions is calculated as follows.

$$K_{M,i} = \frac{1}{R_{Z,i}(M_{T,i}=1)}, \quad K_{FX,i} = \frac{1}{U_{X,i}(F_{TX,i}=1)}, \quad K_{FY,i} = \frac{1}{U_{Y,i}(F_{TY,i}=1)} \qquad \text{Eq 2-5}$$

where $R_{z,i}(M_{T,i} = 1)$ is the rotation of the story *i* about the vertical axis due to unit moment, $U_{X,i}(F_{TX,i} = 1)$ is the displacement at story level *i* in direction X due to unit force F_{TX} and $U_{Y,i}(F_{TY,i} = 1)$ is the displacement in direction Y due to unit force F_{TY} [20].

3. Diaphragm Discontinuity Irregularity

Diaphragm discontinuity irregularity as ASCE 7-05 and UBC97 is defined to exist where there are diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas(O_a) greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness (S_{dst}) of more than 50% from one story to the next (Table 2-2).

Table 2-2: Horizontal irregularity limits prescribed by EBCS EN 1998:2015, ASCE 7-05, UBC 97

Type of Irregularity	EBCS-EN 1998,2015	ASCE 7-05	UBC 97
Re-entrant Corners	$R_i \leq 5\%$	$R_i \leq 15\%$	$R_i \leq 15\%$
Torsional Irregularity	$e_{ox} < 0.3\, r_x$ $e_{oy} < 0.3 r_y$ r_x and $r_y > l_s$,	$d_{max} \leq 1.2 d_{avg}$	$d_{max} \leq 1.2\, d_{avg}$
Diaphragm Discontinuity	$r_x^2 > l_s^2 + e_{ox}^2$ $r_y^2 > l_s^2 + e_{oy}^2$	$O_a > 50\%$ $S_{dst} > 50\%$	$O_d > 50\%$ $S_{dst} > 50\%$

2.4 Code provisions for the torsional effects

Most of the current seismic design provisions require the consideration of torsional effects by adopting design eccentricities, which take into account both natural/inherent and accidental sources of torsion. Natural eccentricity is defined as the distance between the centers of mass and stiffness of a structure in the plan, while the accidental eccentricity generally accounts for factors such as a difference between the computed and actual values of the mass and stiffness, an unpredictable spatial distribution of load, and an effect of the rotational component of the ground motion, which is usually ignored in seismic design practice.

In EBCS EN 1998-1, 2015 and European code, EC8 2004-1 [21, 22], the accidental eccentricity is computed with the relationship

$$e_{ai} = \pm 0.05 L_i \qquad \text{Eq 2-6}$$

Where e_{ai} is the accidental eccentricity of story mass i from its nominal location, applied in the same direction at all floors; L_i is the floor-dimension perpendicular to the direction of the seismic action. If the lateral stiffness and mass are symmetrically distributed in plan and unless the accidental eccentricity is taken into account by a more exact method (modal analysis with response spectrum), the accidental torsional effects may be accounted by multiplying the loads' effects for individual resisting elements with the δ factor given by:

$$\delta = 1 + 0.6 \frac{x}{L_e} \qquad \text{Eq 2-7}$$

Where x is the distance of the element under consideration from the center of mass of the building in plan, measured perpendicularly to the direction of the seismic action considered; L_e is the distance between the two-outermost lateral load resisting elements, measured perpendicularly to the direction of the seismic action considered. The torsional effects can be considered by means of the torsional moments about the vertical axis according to EBCS EN 1998-1/4.3.3.3.3. They can be determined as a product of the horizontal forces in each horizontal direction and the corresponding accidental eccentricity.

In American Society of Civil Engineers/Structural Engineering Institute Standard 7, ASCE/SEI 7-05 [19], the design torsional moment at a given story is defined as the moment resulting from inherent eccentricities between the applied design lateral forces at the levels above the story and the lateral load resisting members in the story, plus an accidental torsion. The accidental torsional moment is determined by displacing the mass a distance equal to 5% of the plan dimension. Torsional irregularity is considered to exist when the maximum story drift, computed including the accidental torsion, at either end of the structure is more than 1.2 times the average of the story drifts at the two ends of the structure. In such a case, the torsional effects need to be reassessed by increasing the accidental torsion at each level with an amplification factor determined from the following formula:

$$A_x = \left(\frac{\delta_{\max}}{1.2 \delta_{avg}} \right)^2 \qquad \text{Eq 2-8}$$

Where, δ_{max} is the maximum displacement at level x and δ_{avg} is the average of the displacements at the extreme points of the structure at level x. The accidental torsional moment need not be amplified for structures of light-frame construction. The torsional amplification factor (A_x) is not required to exceed 3.0. The more severe loading for each element shall be considered for design.

2.5 Commonly recommended solutions to torsional irregularity effects

Currently, an additional Lateral Load Resisting System with the conventional moment-resisting frame is using for enhancing the performance of buildings in the seismic region. Some of the LLRS are Shear Wall, Diagrid, Bracing, hexagrid system, Tube Structures and etc. [23, 24]. In this thesis, the recommended practical solutions for plan asymmetric effects are presented subsequently.

2.5.1 Shear wall and Diagrid system

Non-uniform distributions of mass, stiffness, and strength caused by plan asymmetry results in an unbalanced drift demand for the lateral load-resisting members because of the more intense torsional response. This non-uniform distribution of stiffness and strength need to be balanced; such that the building has no or reduced eccentricity. This thesis tried to address such problems using some special lateral load resisting systems (shear wall and diagrid).

2.5.1.1 Shear walls

Use of structural walls or shear walls in RC buildings is one of the most commonly used strategies for earthquake mitigation. Shear walls provide large strength and stiffness to buildings in the direction of their orientation, which significantly reduces the lateral sway of the building and thereby reduces damage to the structure and its contents. Shear walls resist lateral forces through combined axial-flexure-shear action. Also, shear walls help reduce shear and moment demands on beams and columns in the frames of the building, when provided along with moment-resisting frame as lateral load resisting system. One way of avoiding torsional effect in buildings is increasing the torsional stiffness of building. The balancing of stiffness in each side can be done by introducing the shear walls. This is achieved by adding in-plane stiffness in the vertical plane in select bays along the flexible-side of the building; such that the building has no uneven stiffness distribution.

Advantages and disadvantages of shear wall

Advantages

- Shear walls are efficient, both in terms of construction cost and effectiveness in minimizing earthquake damage in structural and nonstructural elements.
- Shear walls are easy to construct because reinforcement detailing of walls is relatively straightforward and therefore easily implemented at the site.
- The shear walls with the moment-resisting frame (dual system) provide good redundancy, is suitable for medium-to-high rise buildings.

Disadvantages

- Shear walls present barriers, which may interfere with architectural and services required.
- Since shear walls carry large horizontal earthquake forces, the overturning effects on them are large. Thus, the design of their foundations requires special attention.
- Lateral load resistance in shear wall buildings is usually concentrated on a few walls rather than on the large number of columns [25, 26, 27].

2.5.1.2 Diagrid Systems

A Diagrid (Diagonal + Grid) is a structural system in a building formed with diagonally intersecting ribs of metal or concrete. The origin of diagrid structures is the Russian Vladimir Shukhov during the late 19th and early 20th century. Diagrid is suitable for constructing large buildings with both regular and irregular forms. Diagrid systems based on triangles, which inherently make the structure stiffer in all directions as compared to a structure using conventional rectangular grids (which can deform to parallelograms under lateral load). Both gravity and lateral loads from any direction are received and transmitted by the diagonal diagrid members throughout their triangulated configuration to the foundation. Compared with conventional framed structures without diagonals, diagrid structures are more effective in minimizing shear deformation because they carry shear by the axial action of the diagonal members, while conventional framed structures carry shear by the bending of the vertical columns. The diagrid system has higher inherent torsional rigidity than most other structural systems. Diagrids generally offer significant advantages over conventional rectangular grids where torsional loads are an issue.

Figure 2-5: Concrete diagrid building; Yellow Building in London, completed in 2008 [28]

Advantages and disadvantages of Diagrids

Advantages

- The diagrid structures are aesthetically dominant and expressive.
- The diagrid reduces the need for heavy shear walls within a structure in order to resist lateral loads.
- The diagrid systems are known for their redundancy, continuous and uninterrupted load paths.

Disadvantages

- The diagrid systems are that it is still not completely explored.
- The lack of availability of skilled workers having experience in diagrid construction [29, 28, 23, 24, 30].

CHAPTER 3 MODELING AND ANALYSIS OF SELECTED PLAN ASYMMETRIC BUILDINGS

3.1 Introduction

To carry out the study torsional behavior of plan asymmetric reinforced concrete building, both symmetric and L-shaped plan asymmetric buildings have been modeled in finite element software ETABS 2016. In this Chapter, the building models with different level of plan asymmetry have been developed at first. Secondly, seismic and vertical actions have been discussed. Finally, different analysis methods available to obtain the seismic response have been reviewed and based on the review of analysis methods a suitable method has been adopted for analysis of irregular building models.

3.2 Description of the building models

3.2.1 Definition of the base case:

The base case (reference model) is a building model without any irregularity in mass, stiffness and strength distribution and may be called as a regular building model. The torsional behavior of the plan asymmetric buildings has been studied based on that of the regular building model.

3.2.2 Definition of building models with plan asymmetry:

Three-dimensional model and finite element methods are introduced for the seismic analysis of the moment-resisting frame system. Where the seismic demands of horizontal irregularity L-shaped in plan buildings that built through the gradual reduction in plan area and then introducing slab hole and shear wall lift core are investigated and compared to that of the square regular building as the reference model (RM model).

These models are categorized into three types for studying each type individually.

I. Models without Staircase hole [Figure 3-2]
II. Models with staircase hole [Figure 3-3]
III. Model with lift core [Figure 3-4]

3.2.3 Details of the building models

The investigated buildings are the multi-story reinforced concrete structures. The buildings have 13 stories. The total height of the building above the ground level is 36 m and 3 m below the ground level. The heights of stories are equal to 3 m. The dimensions of the regular model (RM) floors are 24 m x 24 m, whereas the areas of other models' floors are smaller than the area of RM with the aim of developing plan asymmetry to the reference model (RM) for this study. The cross-sections of the construction elements (beams, columns, and walls) and the slab thickness are shown in Table 3-1. Concrete C25/30 is used. The corresponding modulus of elasticity amounts to E_{cm} = 31GPa (EBCS EN 1992-1-1:2013/ Table 3.1). Poisson's ratio was taken equal to ν = 0 (cracked concrete) according to EBCS EN 1992-1-1:2013/3.1.3. Steel S400 is used. The structures have been designed for ductility class DCM.

Table 3-1: Building model details.

Type of Structures	Multi-Story RC building	
Number of Stories	G+12	
Elements Dimension	Beam	300 x 400 mm^2
	Column	550 x 550 mm^2
	Slab Thickness	150 mm
	Shear wall Thickness	200 mm
	Story height	3 m
	Bay width	4m

3.2.4 Structural models

A three-dimensional (spatial) structural model is used in ETABS software. The plan view of each model is shown below (Figure 3-1 to Figure 3-4).

The basic characteristics of the models are as follows:

- Beams and columns are modeled as line elements.
- Slabs and walls are modeled as shell-thin.
- All elements are fully fixed in foundation
- Frames and walls are connected together by means of rigid diaphragms (in a horizontal plane) at each floor level.
- Masses and moments of inertia of each floor are lumped at the center of masses.

- The cracked elements are considered to take into account the effect of cracking (EN 1998 1/4.3.1(6)). The elastic flexural and shear stiffness properties are taken to be equal to one-half of the corresponding stiffness of the uncracked elements (EN 1998-1/4.3.1 (7)), i.e. the moment of inertia and shear area of the uncracked section were multiplied by factor 0.5. Also, the torsional stiffness of the elements has been reduced. The torsional stiffness of the cracked section is set equal to 10% of the torsional stiffness of the uncracked section.
- Infill is not considered in the model.
- The accidental torsional effects are taken into account according to EN 1998/ 4.3.2.

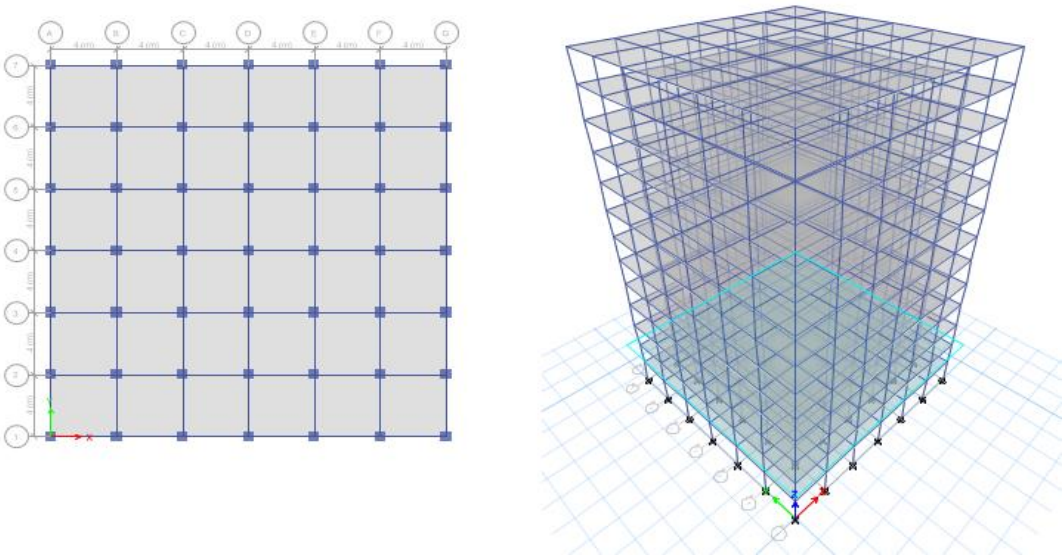
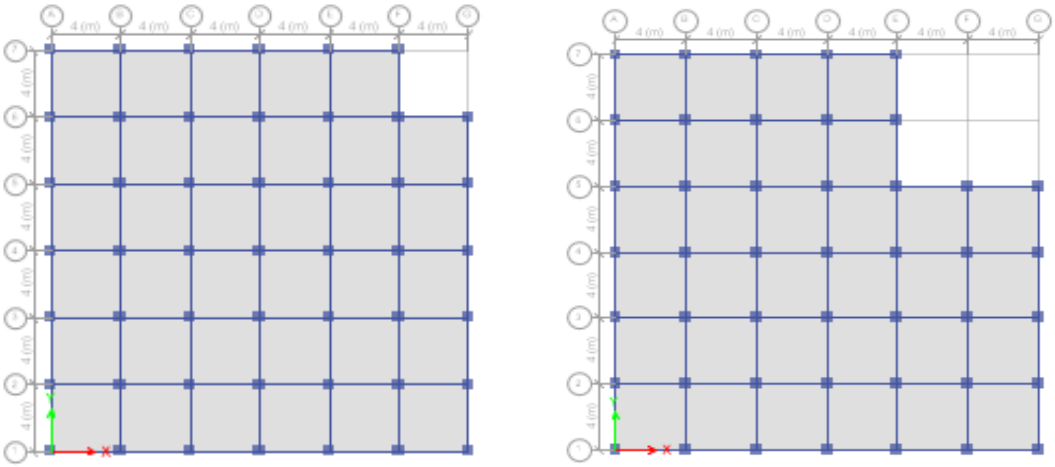
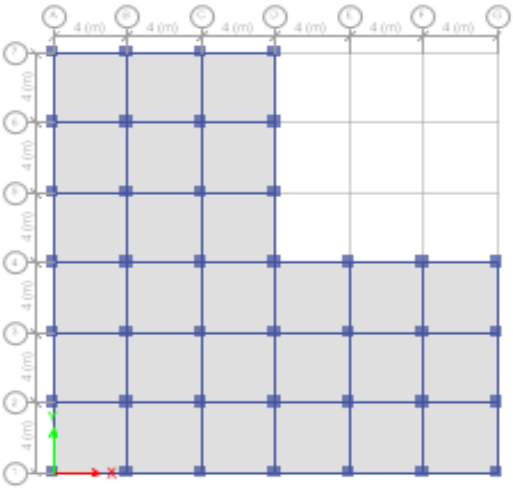


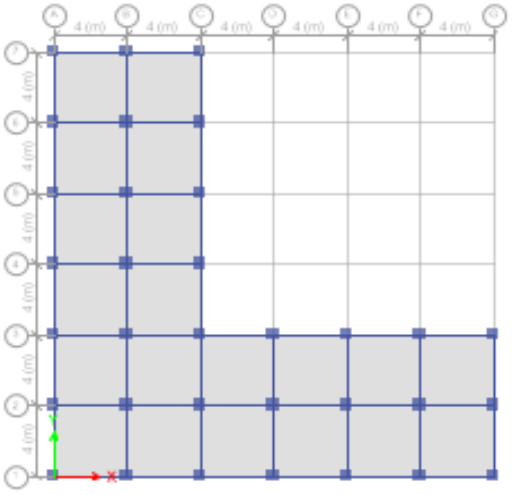
Figure 3-1: Regular building plan and 3-D model (RM)



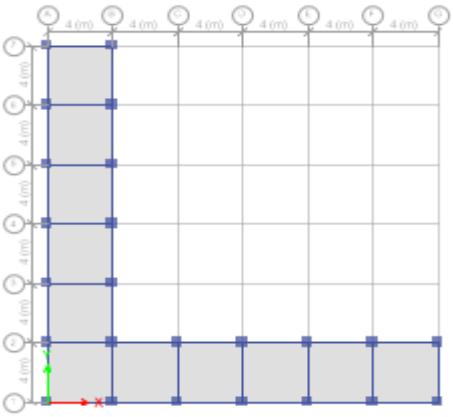
(a) Model 1-1 (b) Model 1-2



c). Model 1-3

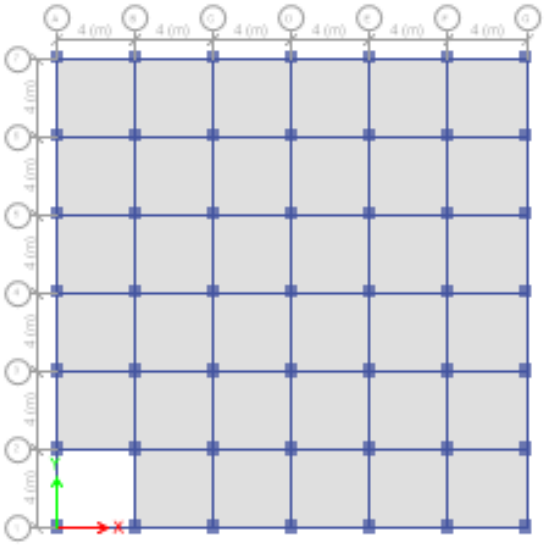


d). Model 1-4

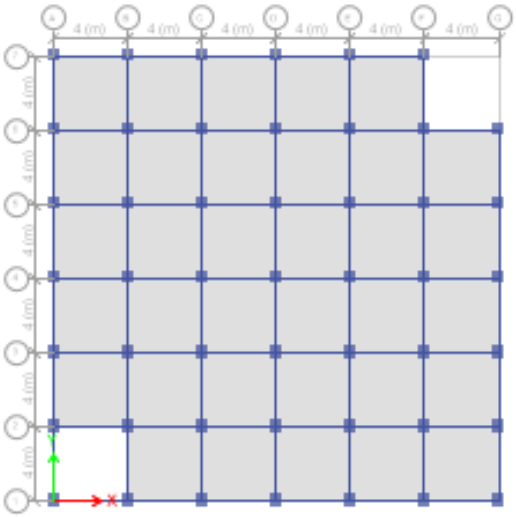


d). Model 1-5

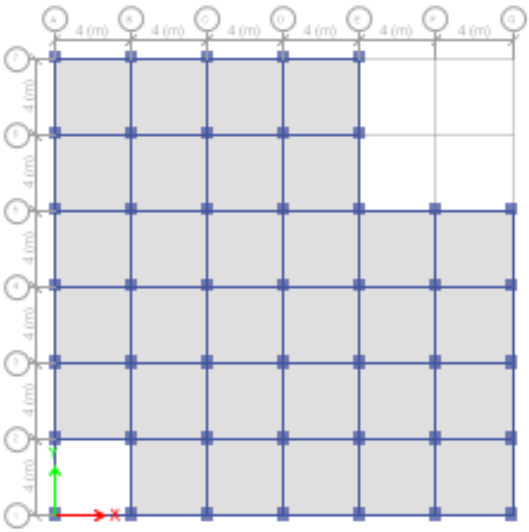
Figure 3-2: Type-I: Models without Staircase hole



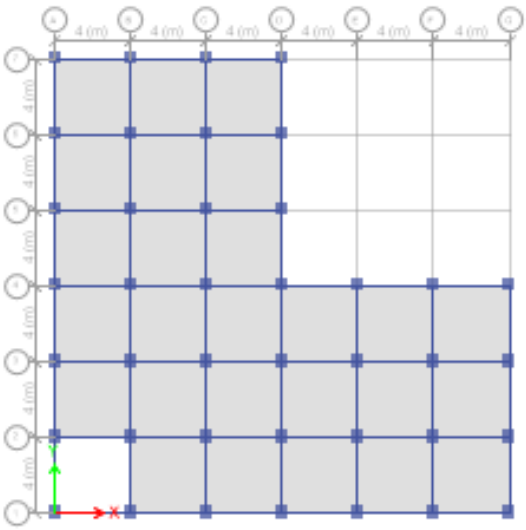
a). Model 2-1



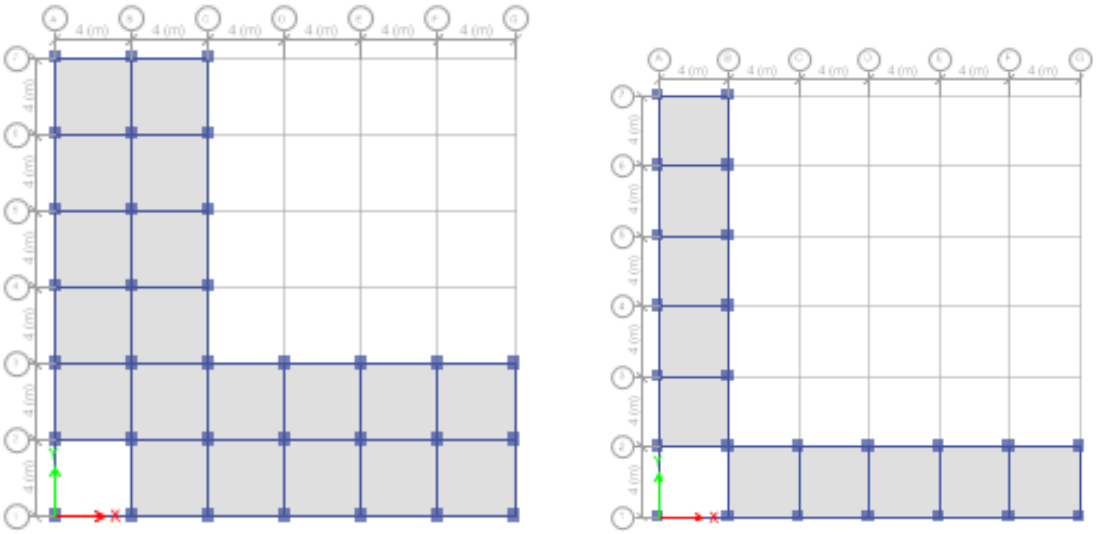
b). Model 2-2



c). Model 2-3

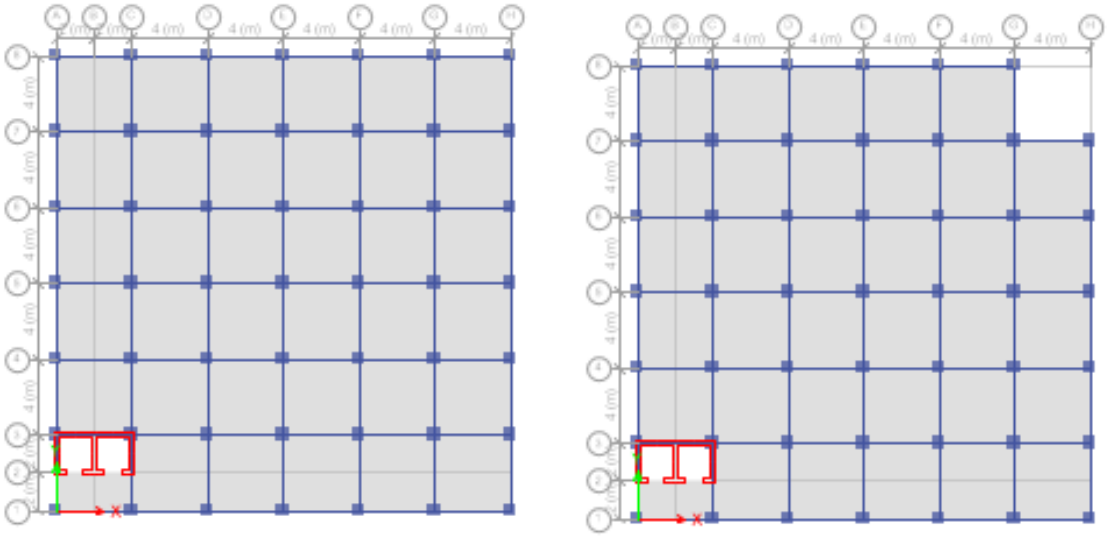


d). Model 2-4

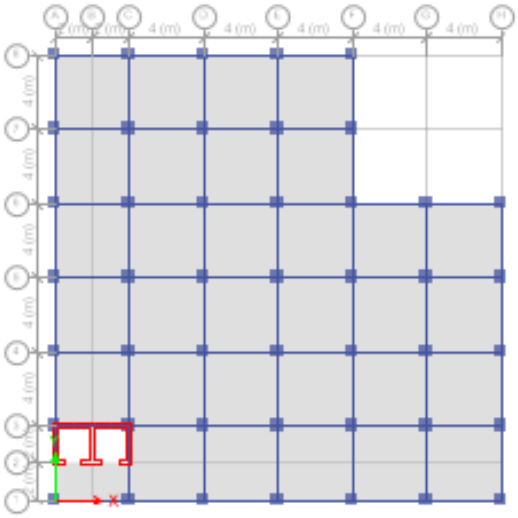


e). Model 2-5 f). Model 2-6

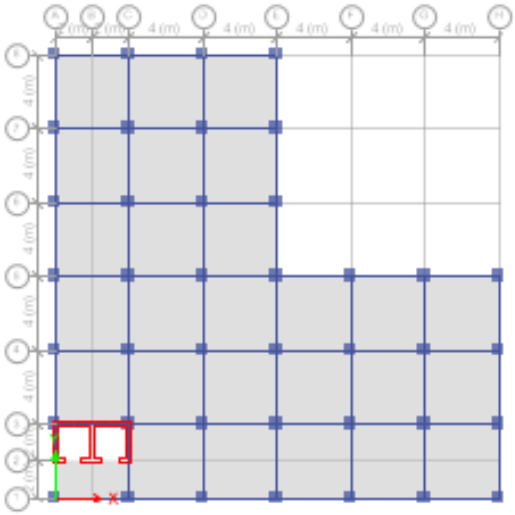
Figure 3-3: Type-II: Models with staircase hole



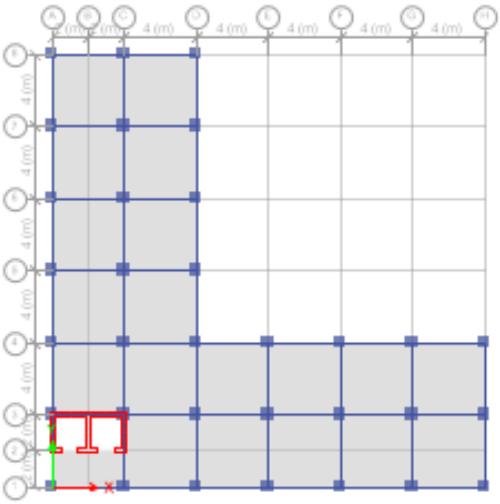
a). Model 3-1 b). Model 3-2



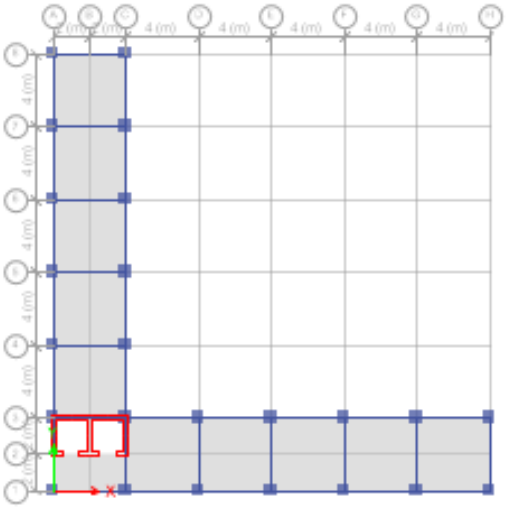
c). Model 3-3



d). Model 3-4



e). Model 3-5



f). Model 3-6

Figure 3-4: Type-III: Model with lift core

3.3 Actions

3.3.1 Seismic actions

The seismic action is represented by the elastic response spectrum, Type 1 ($M_s > 5.5$, EBCS EN 1998-1/3.2.2.2 (2)P) for soil B (EN 1998-1/Table 3.1). The reference peak ground acceleration in the highest seismic zone (zone 5) in Ethiopia amounts to a_{gR} = 0.2g. The values of the periods (T_B, T_C, T_D) and of the soil factor (S), which describe the shape of the elastic response spectrum, amount to T_B = 0.15s, T_C = 0.5 s, T_D = 2.0 s and S = 1.2 (EBCS EN 1998- 1/Table 3.2). The buildings are classified as importance class IV (EBCS

EN 1998-1/Table 4.3) and the corresponding importance factor amounts to $\gamma_I = 1.4$ (EBCS EN 1998-1/4.2.5 (5)P). Therefore, the peak ground acceleration is equal to the reference peak ground acceleration times importance factor $a_g = \gamma_I * a_{gR} = 0.28g$. Using the equation in EN 1998-1/3.2.2.2 the elastic response spectrum was defined for 5% damping. For the design of the building the design response spectrum is used (i.e. elastic response spectrum reduced by the behavior factor q). Determination of the behavior factor q, which depends on the type of the structural system, regularity in elevation and plan, and ductility class. It amounts to 3.9 for regular model, 3.45 for plan asymmetric models and 2 for torsionally flexible models (EBCS EN 1998-1/5.2.2.2).

3.3.2 Vertical actions

In a seismic design situation, the vertical actions (permanent loads *"G"* and variable-live loads *"Q")* have to be taken into account. The permanent loads *"G"* are represented by the self-weight of the structure and additional permanent load which is assumed to amount to 2 KN/m^2. In the case of investigated buildings, the variable-live load in terms of uniformly distributed load amounts to 3 kN/m^2 (EBCS EN 1991/Table 6.2). The variable-live loads are, in a seismic design situation, reduced by a factor of Ψ_{Ei} (EBCS EN 1998/ 3.2.4). The unit weight of the concrete (γ = 25 kN/m^3).

$$\sum G_{k,j} + \sum \psi_{E,i} . Q_{k,i} \qquad \text{Eq 3-1}$$

The floor masses are determined according to EBCS EN 1998-1/3.4.2. Complete masses resulting from the permanent load are considered, whereas the masses from the variable-live load are reduced using the factor $\psi_{Ei} = \varphi \cdot \psi_{2i}$. Factor ψ_{2i} amounts to 0.3 in the case of our models (EBCS EN 1990/Table A.1.1) and factor φ is taken is equal to 1.0 (EN 1998-1/4.2.4). The combination coefficients $\psi_{E,i}$ take into account the likelihood of the loads $Q_{k,i}$ not being present over the entire structure during the earthquake. These coefficients may also account for a reduced participation of masses in the motion of the structure due to the non-rigid connection between them.

3.4 Combinations of the seismic action with other actions

The seismic design based on analysis method discussed in section 3.5 should be performed in order to optimize the cross sections of the elements of the building model. The

combination of the gravity loads and horizontal components of the seismic actions are accounted for as EBCS EN 1998, 2015/3.2.3 and 4.3.3.5 [21, 22].

3.5 Review of analysis methods

The seismic responses of the building systems show a large dependence on the type of analysis method adopted. Different analysis methods available to obtain the seismic response have been discussed and based on the review of analysis methods a suitable method has been adopted for analysis of irregular building models. The analysis methods can be divided into the linear method and nonlinear methods.

3.5.1 Linear approach:

In the linear approach, the force is assumed to be constant with time. This approach can be further sub-classified into linear static and linear dynamic approach as discussed in following subsections.

3.5.1.1 Equivalent static analysis

The Equivalent Lateral Load method is essentially a simplified static approach that involves the judicious application of an equivalent lateral load along the height of the building at the floor levels. In such a procedure, total design lateral force or design base shear along any principle direction is given in terms of design horizontal seismic coefficient seismic weight of the structure. Design horizontal seismic coefficient depends on the zone factor of the site, importance of the structure, response reduction factor of the lateral load resisting elements and the fundamental period of the structure. Once the base shear is evaluated for a certain structure its vertical distribution along the height of the structure is accomplished in a manner that resembles the first mode of a vertical cantilever.

The seismic design of buildings follows the dynamic nature of the load. But equivalent static analysis would become sufficient for simpler, regular in plan configuration and it will give more efficient results. The method of analysis is obviously limited by stringent constraints that include but are not limited to height, weight, and the relevant seismic zone category. For buildings that have irregular distributions of mass or stiffness, irregular geometries, or non-orthogonal lateral-force-resisting systems, the distribution of demands predicted by linear dynamic analysis will be more accurate than those predicted by the

equivalent static analysis. Either the response spectrum method or time history method may be used for evaluation of such structures [1, 31, 32, 21, 33].

3.5.1.2 Response spectrum method

In order to perform the seismic analysis and design of a structure to be built at a particular location, the actual time history record is required. However, it is not possible to have such records at each and every location. To overcome the above difficulties, earthquake response spectrum is the most popular tool in the seismic analysis of structures. The Response Spectrum is a method of estimation of maximum responses (acceleration, velocity, and displacement) of a family of SDOF systems subjected to a prescribed ground motion. The dynamic forces in this method are applied in the form of the code specified response spectrum to the structure. A response spectrum is defined as the maximum response (displacement, velocity or acceleration) of all possible SDOF oscillators, which can be described by their natural frequency and damping coefficient, natural period; it can be derived by analyzing a series of Single Degree-Of-Freedom (SDOF) systems.

The response spectrum analysis is applicable for all types of buildings, while the equivalent lateral force method of analysis has many restrictions on its use due to the fear that it would provide un-conservative results in certain conditions. The Response Spectrum procedure is obligatory for structures that are high in elevation with the vertical or horizontal irregularity of stiffness, mass or geometry. Therefore, it provides a greater insight into the structural response as compared to the linear static approach. Furthermore, the seismic response at each mode is combined individually to get the total seismic response using different modal combination rules (SRSS, ABS, and CQC). In this thesis, this analysis is carried out according to the code EBCS EN-1998, 2015. Here type of soil, seismic zone factor should be entered from EBCS EN-1998, 2015. The standard response spectra for the type of soil considered is applied to the building for the analysis in ETABS software [16, 1, 31].

3.5.1.3 Time history method

Dynamic analysis using the time history method shall calculate building response at discrete time steps using discretized recorded or synthetic time histories as base motion. Response parameters shall be calculated for each time history analysis. If three or more-time history analyses are performed, the maximum response of the parameter of interest shall be used for design. If seven or more consistent pairs of horizontal ground motion

records are used for time history analysis, use of the average of all responses of the parameter of interest shall be permitted for design. In this method, mode superposition method is applicable. In order to perform the seismic analysis and design of a structure to be built at a particular location, the actual time history record is required. However, it is not possible to have such records at each and every location [33, 9].

3.5.2 Nonlinear methods:

The nonlinear analysis methods perform better than linear analysis methods. These methods provide a realistic estimate of seismic demands of the structural components deforming in the inelastic range. Furthermore, these methods are very effective to obtain the rational estimates of structural components of strength and stiffness deterioration in the inelastic range. This approach has been further classified into nonlinear static and nonlinear dynamic analysis as discussed below:

3.5.2.1 Nonlinear static analysis

This method is also known as push-over analysis. The pushover analysis of a structure is a static non-linear analysis under permanent vertical loads and gradually increasing lateral loads. The purpose of pushover analysis is to evaluate the expected performance of structural systems by estimating the performance of a structural system by estimating its strength and deformation demands in design earthquakes by means of static inelastic analysis and comparing these demands to available capacities at the performance levels of interest.

The nonlinear static procedure or pushover analysis, as described in FEMA-356 is now used by the structural engineering profession as a standard tool for estimating seismic demands for buildings. It is generally a more reliable approach to characterizing the performance of a structure than are linear procedures. It allows the inelastic behavior of the structure. The base assumption of the pushover analysis is the invariable form of displacement through time, which is reasonably accurate for linear response of structures vibrating dominantly in the first mode. However, in a non-linear field, the displacement form, in reality, is changing according to the dynamic characteristics change of the structure due to stiffness degradation, i.e. the formation of the plastic hinges or higher mode effects. When the nonlinear static procedure is utilized on a structure that has significantly higher mode response, the linear dynamic procedure is also employed to

verify the adequacy of the design. Therefore, a more rational nonlinear approach needs to be adopted to get a realistic estimate of seismic demands [34, 33, 35].

3.5.2.2 Nonlinear time history analysis

A final alternative, which remains comparatively rare, is the use of full nonlinear dynamic analysis. The nonlinear dynamic analysis shall be permitted for all structures. If this procedure is selected for seismic analysis of the building, a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and elements of the building shall be subjected to earthquake shaking represented by ground motion time histories to obtain forces and displacements. A calculated response can be highly sensitive to characteristics of individual ground motions; therefore, the analysis should be carried out with more than one ground motion record. Because the numerical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake. Yet, due to its complexity and high standards, it goes beyond the frames of practical application and is appropriate only for the research and analysis of structures of special significance [7, 33, 35].

CHAPTER 4 RESULTS AND DISCUSSION

4.1 Introduction

The response spectrum analysis for the models have been carried out using ETABS 2016 software. The seismic details were incorporated in accordance with EBCS EN 1998-1, 2015. The torsional irregularity has been found in reference to EBCS EN 1998-1 and ASCE 7-05. The analysis was carried out in order to evaluate the torsional behavior of irregular structure on the basis of different parameters: vibration period and modes, story drift ratio, lateral displacements, torsional irregularity ratio according to submitted national codes, torsional diaphragm rotation, torsional moment and base shear.

4.2 Natural period of vibration and vibration modes

The fundamental natural periods and vibration modes have been considered to identify the dynamic properties of the building analyzed. The sixteen mode numbers verse the natural period and fundamental period of vibration of models are depicted in Figure 4-1.

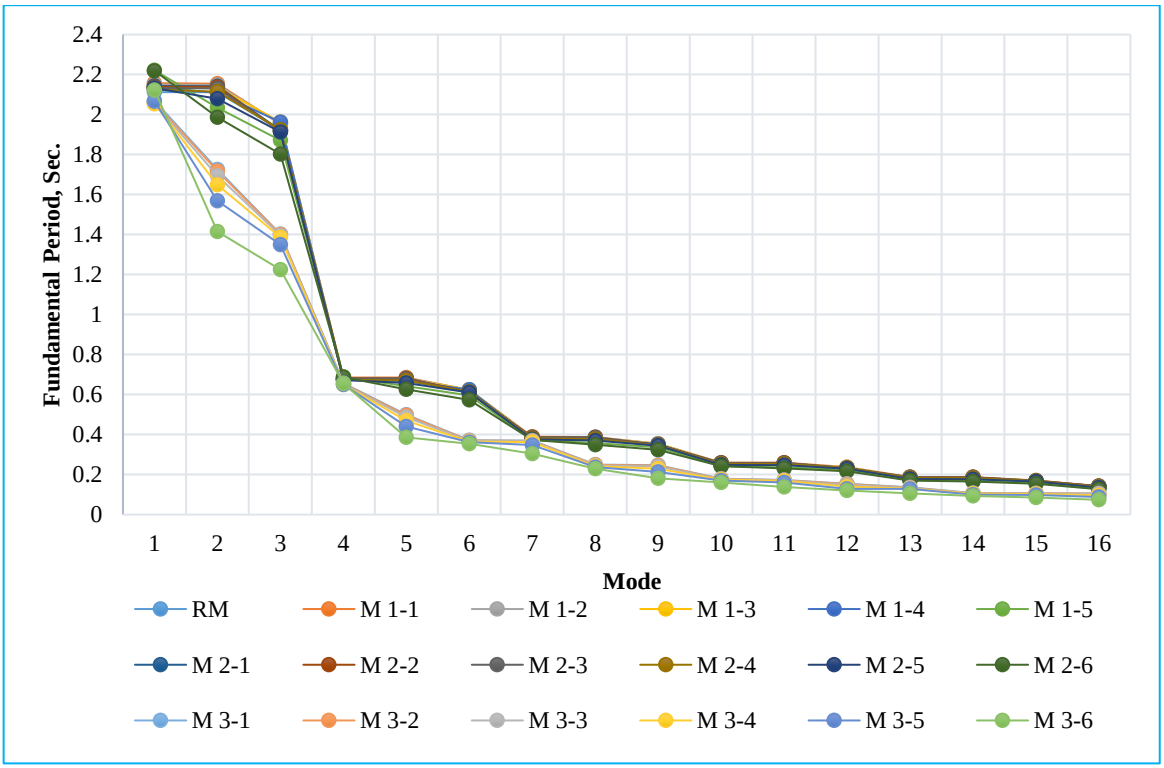
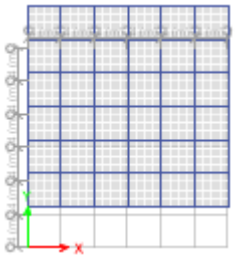
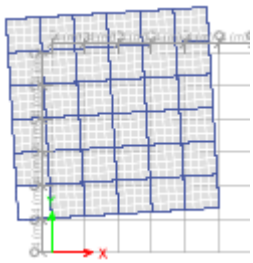
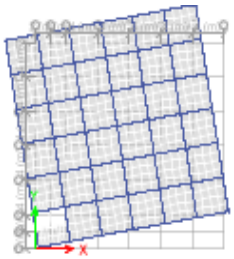
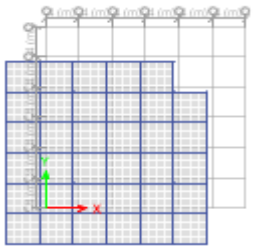
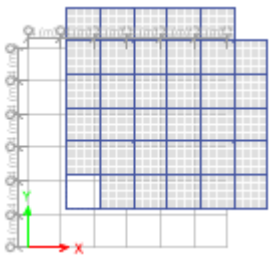
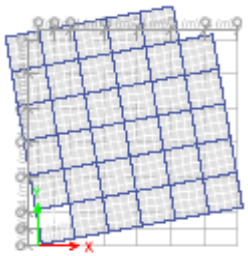


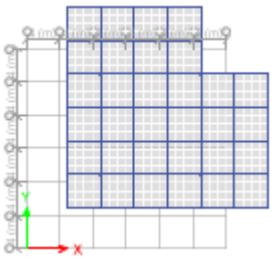
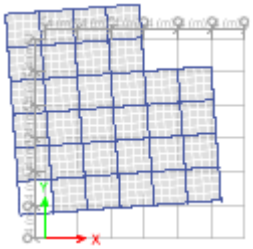
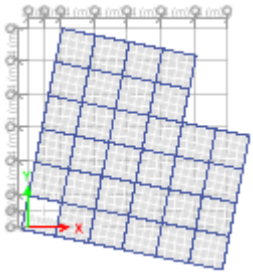
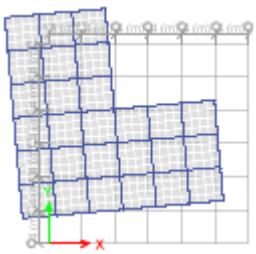
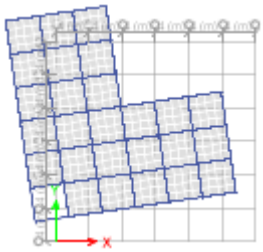
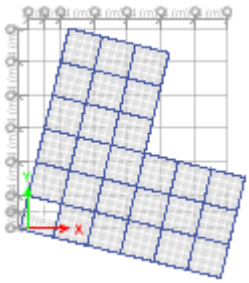
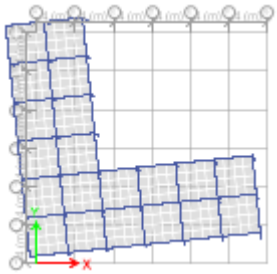
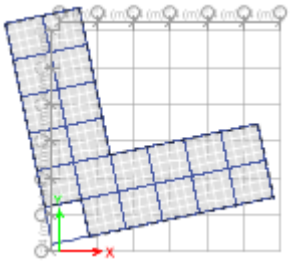
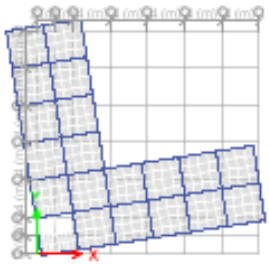
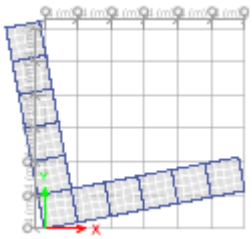
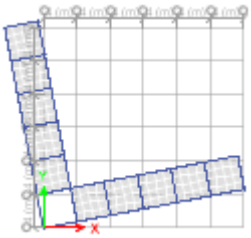
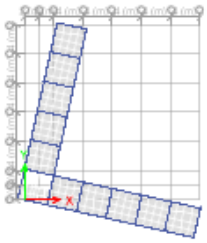
Figure 4-1: Mode numbers and natural period of vibration

Table 4-1: Fundamental period and modal participating mass ratio of models

Models		Fundamental period	Modal participating mass ratio		
			U_x	U_y	R_z
	RM	2.11	0.00	0.80	0.00
I	M 1-1	2.16	0.40	0.40	0.00
	M 1-2	2.15	0.40	0.40	0.00
	M 1-3	2.14	0.39	0.39	0.02
	M 1-4	2.14	0.33	0.33	0.14
	M 1-5	2.22	0.20	0.20	0.37
II	M 2-1	2.14	0.40	0.40	0.01
	M 2-2	2.14	0.40	0.40	0.00
	M 2-3	2.13	0.39	0.39	0.02
	M 2-4	2.13	0.37	0.38	0.05
	M 2-5	2.13	0.31	0.31	0.18
	M 2-6	2.22	0.20	0.20	0.38
III	M 3-1	2.07	0.17	0.37	0.25
	M 3-2	2.06	0.17	0.37	0.26
	M 3-3	2.06	0.16	0.36	0.28
	M 3-4	2.05	0.13	0.32	0.34
	M 3-5	2.07	0.10	0.26	0.43
	M 3-6	2.12	0.06	0.18	0.53

Table 4-2: First vibration mode shape of models

Models Type-I	Models Type-II	Models Type-III
 RM	 M 2-1	 M 3-1
 M 1-1	 M 2-2	 M 3-2

 M 1-2	 M 2-3	 M 3-3
 M 1-3	 M 2-4	 M 3-4
 M 1-4	 M 2-5	 M 3-5
 M 1-5	 M 2-6	 M 3-6

The overall response of a building is the sum of the responses of all of its modes. The response of all modes of vibration contributing significantly to the global response shall be taken into account. In the modal response spectrum analysis, all 16 modes of vibration were taken into account in all models. Note that for RM the first six modes, for Type-I and type-II model the first eight modes and for type-III model the first eleven modes would be sufficient to satisfy the requirements in EN 1998 1/4.3.3.3(3). This confirms that a number

of modes should be accounted, increase with irregularity increase. The first three modes of RM, M 1-1, M 1-2, M 2-1 and M 2-2 are pure translational along X and Y and pure torsional about Z respectively whereas modes of other models are the mixture of translational and rotational (torsional). It is noted that for regular and nearly regular models the first two modes are pure translational and third mode is pure torsional; with increasing of irregularity, it has become mixed translational and torsional.

The fundamental natural period is an inherent property of a building. It is an important parameter in earthquake-resistant design. The design horizontal base shear coefficient of a building is a function of its translational natural periods in the considered direction of design lateral force. Design codes give smoothened curves (design spectrum) to estimate design base shear coefficient as a function of the estimated fundamental translational natural period of a building. Models in Type-III which are stiffened with shear wall lift core, have got smaller fundamental natural period than their corresponding models in Type-I and II. Models Type-II have got slightly smaller fundamental natural period than their corresponding models in Type-I (Table 4-1). Asymmetry of the building is the result of the non-uniform distribution of mass and stiffness. Natural period of buildings depends on the distribution of mass and stiffness along the building (in all directions). An increase in mass of a building increases its natural period; whereas, increase in stiffness reduces the natural period. Buildings with smaller translational natural period attract higher design seismic force coefficient.

4.3 Eccentricity

Eccentricity is one of the important criteria for the assessment of torsion. The eccentricity along X and Y directions of the three types of models are as shown in Figure 4-2 to Figure 4-4. As it is seen from the figures, for Type-I and II models' eccentricities are increasing over the story height in X and Y direction but, for models in Type-III, eccentricities are decreasing over the story height in both directions.

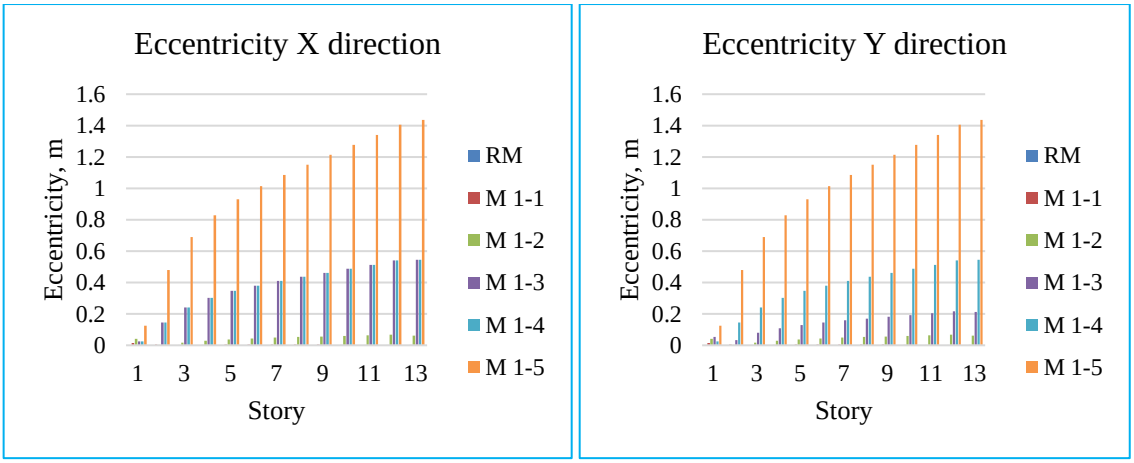


Figure 4-2: Eccentricity in X and Y direction for Type-I models

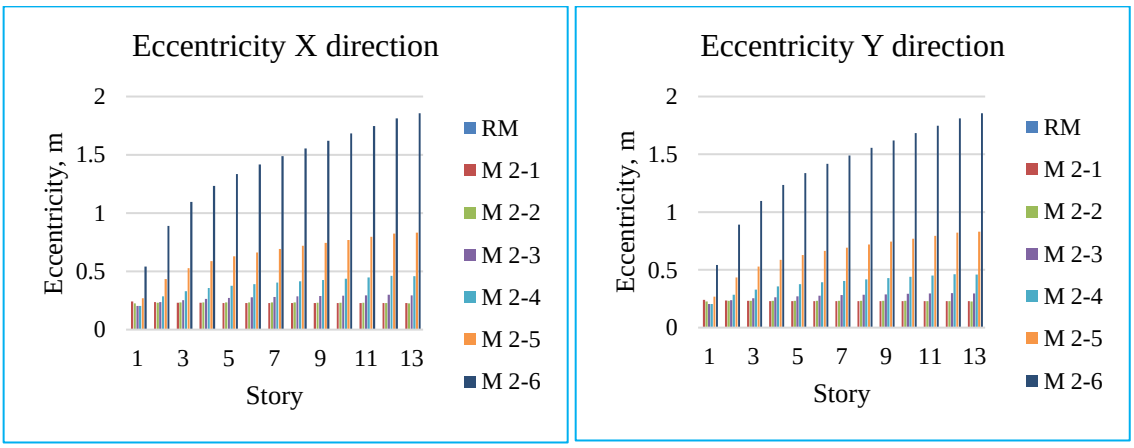


Figure 4-3: Eccentricity in X and Y direction for Type-II models

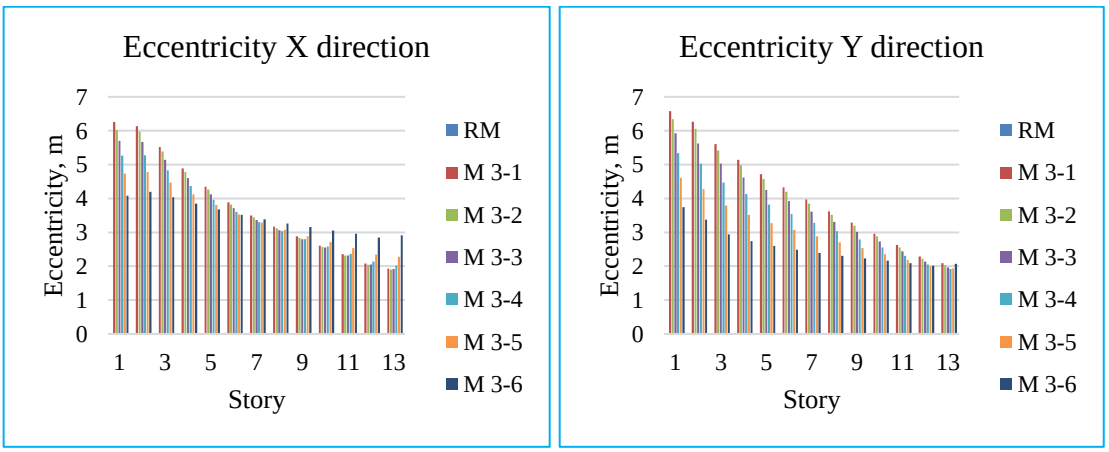


Figure 4-4: Eccentricity in X and Y direction for Type-III models

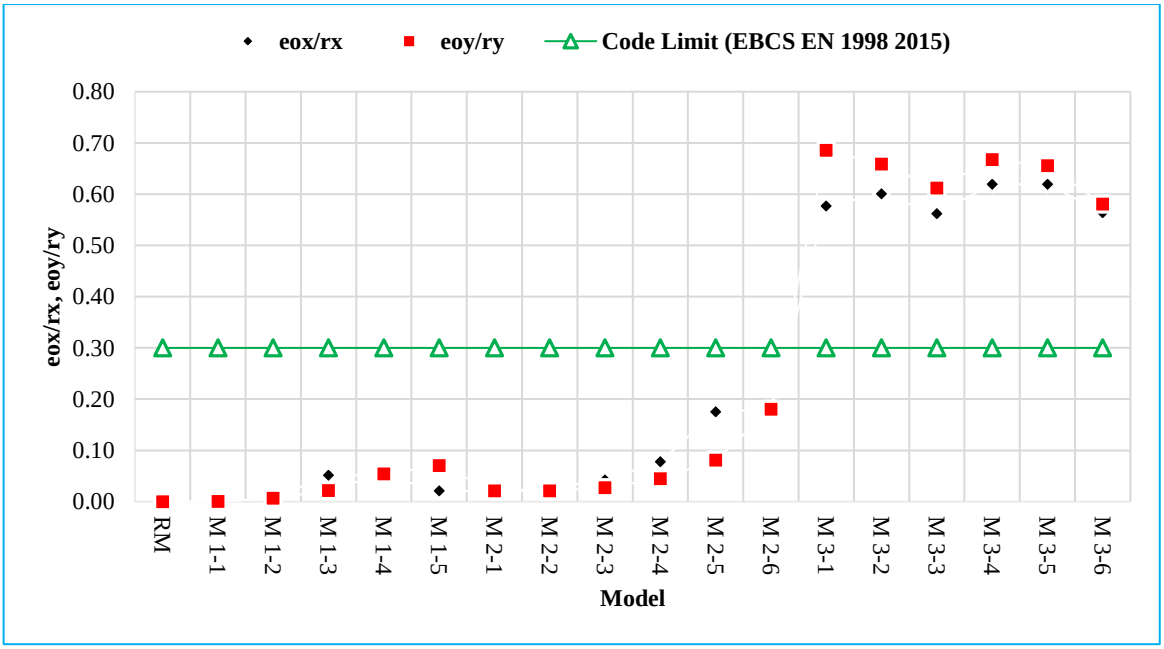
Torsional problems take place when the mass center and center of rigidity are not located at the same place. By increasing distance between the center of mass and center of rigidity, the building is forced to twist around the rigid structural section (rigid core) and subjected to great torsional moments and magnitude of the torsional moment is the function of eccentricity. Model M 1-5 have an eccentricity of 5.99% of plan dimension which is maximum of Type-I models. Model M 2-6 have the eccentricity of 7.74% which is maximum of Type-II models and the model is found in torsional irregularity limit. All Type-III models have eccentricity between 17% and 27% and entered into torsional irregularity limit as seen in Figure 4-6. It is also noted that increasing of eccentricity increases the torsional irregularity which in turn causes severe damage to the structure under seismic load. The maximum eccentricity of all modes has been shown in Table 4-3.

Table 4-3: Maximum eccentricity of building models.

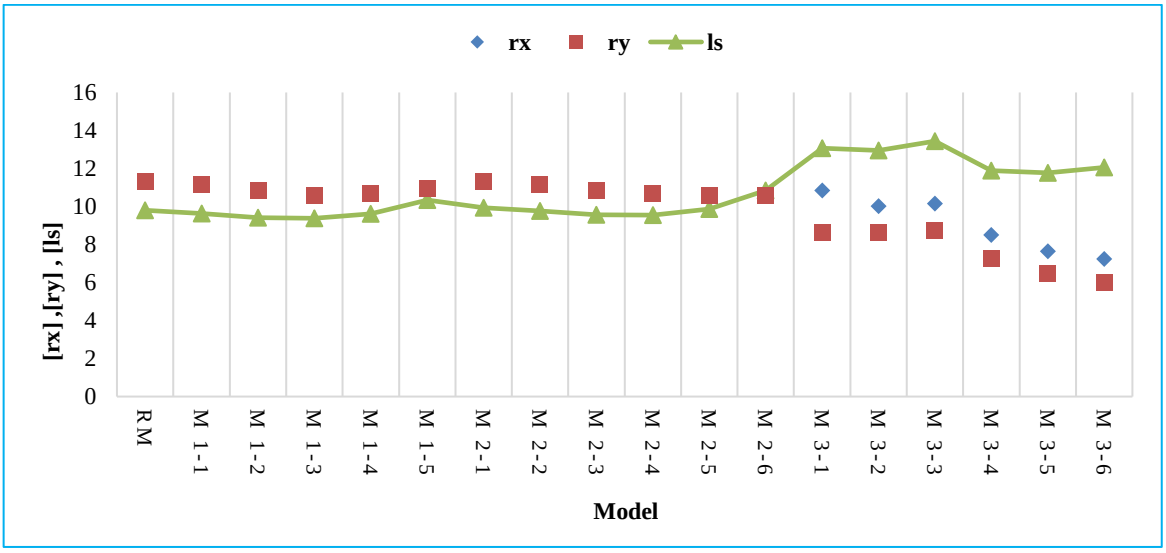
Type-I model							
Model	RM	M 1-1	M 1-2	M 1-3	M 1-4	M 1-5	
Eccentricity (%)	0.00	0.01	0.26	2.27	2.27	5.99	
Type-II model							
Model	RM	M 2-1	M 2-2	M 2-3	M 2-4	M 2-5	M 2-6
Eccentricity (%)	0.00	0.95	0.94	1.23	1.91	3.46	7.74
Type-III model							
Model	RM	M 3-1	M 3-2	M 3-3	M 3-4	M 3-5	M 3-6
Eccentricity (%)	0.00	26.05	25.10	23.77	21.95	19.73	17.00

4.4 Torsional irregularity

Torsional irregularity is one of the most important factors, which causes severe damage to the building structures. Torsional irregularity ratio is an important parameter which measures the extent of the torsional effect on the structure. The torsional irregularity values determined (as discussed in section 2.3.4) are shown below in figure for all building models and torsional irregularity is checked as per seismic codes.



(a)



(b)

Figure 4-5: (a), (b) Torsional irregularity check according to EBCS EN 1998 2015.

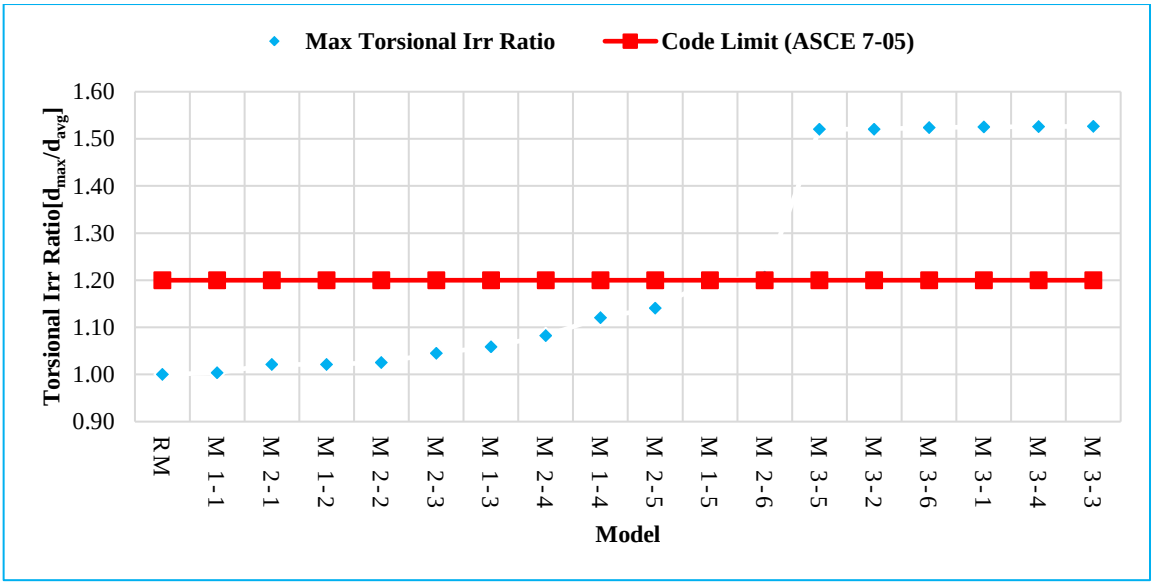


Figure 4-6: Maximum torsional irregularity ratio as per ASCE 7-05 and code limits for all models.

Table 4-4: Summary of torsional irregularity check according to EBCS EN 1998 and ASCE 7-05

Model		Torsional irregularity according to:	
		EBCS EN 1998-1, 2015	ASCE 7-05
	RM	No	No
I	M 1-1	No	No
	M 1-2	No	No
	M 1-3	No	No
	M 1-4	No	No
	M 1-5	No	Yes
II	M 2-1	No	No
	M 2-2	No	No
	M 2-3	No	No
	M 2-4	No	No
	M 2-5	No	No
	M 2-6	Yes	Yes
III	M 3-1	Yes	Yes
	M 3-2	Yes	Yes
	M 3-3	Yes	Yes
	M 3-4	Yes	Yes
	M 3-5	Yes	Yes
	M 3-6	Yes	Yes

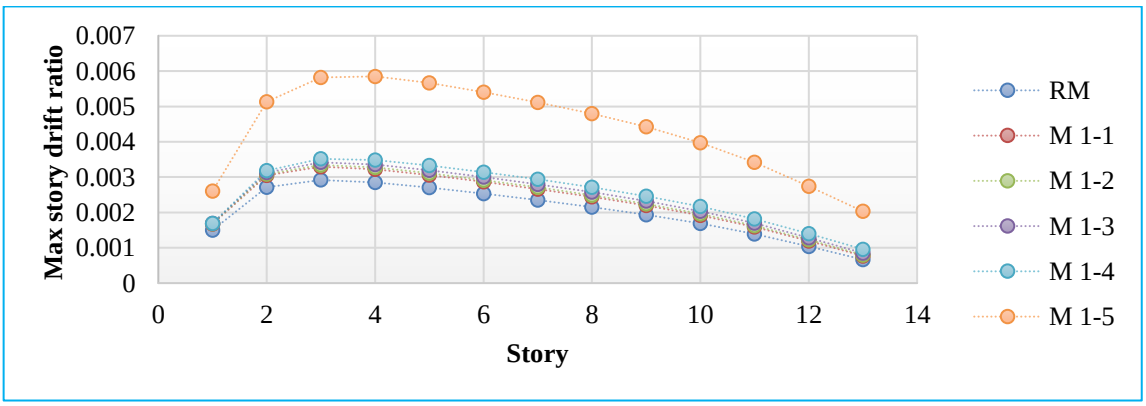
Figure 4-5 and Figure 4-6 show the maximum torsional irregularity ratios of each model and codes limit. Torsionally irregular models according to codes have been identified in Table 4-4. From the results, it is observed that torsional irregularity grows up with the increase of eccentricity. Model Type-III has got the maximum torsional irregularity and eccentricity, which implies irregularity introduced to regular model with the gradual reduction in plan area and shear wall (lift core) instead of stair hole (in Type-II model) promote to larger torsional effects.

4.5 Story drift and lateral displacement response

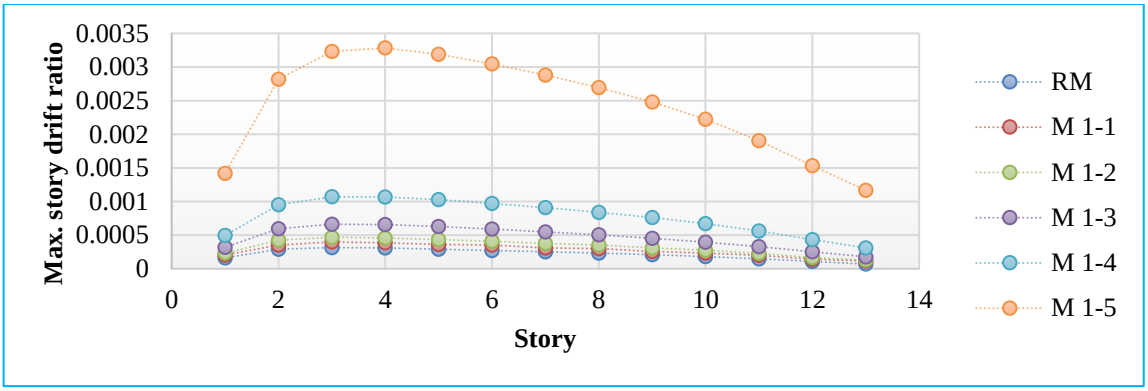
4.5.1 Story drift response

Story drift ratio is the maximum relative displacement of each floor divided by the height of the same floor is an important parameter that has been evaluated. Without proper consideration during the design process, large deflections and drifts can have adverse effects on structural elements, nonstructural elements, and adjacent structures [36]. The story drift ratio response demand is investigated for irregular building models and compared to the reference regular building model. The story drift ratios over the building's height for different models are introduced in Figure 4-7 to Figure 4-11.

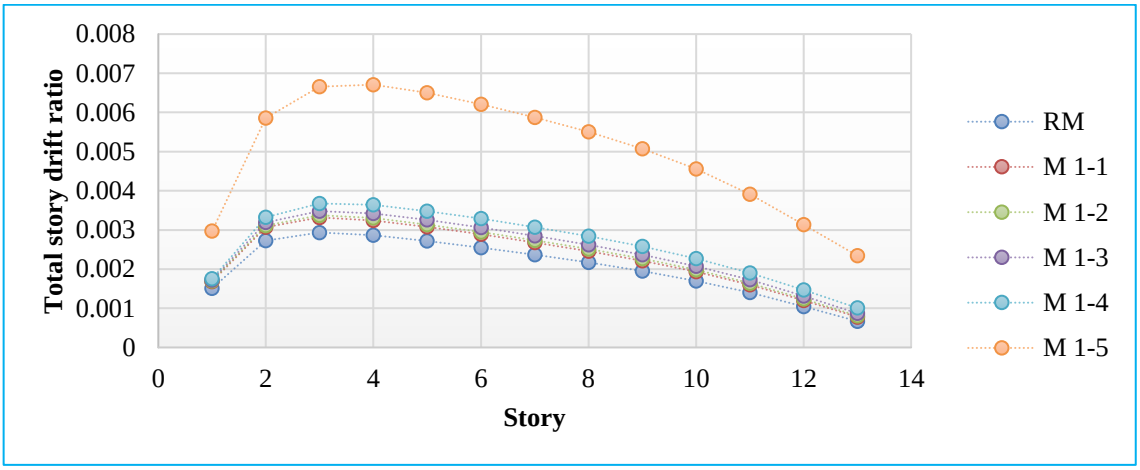
Type-I models



a) Maximum story drift ratio in EQ excitation direction



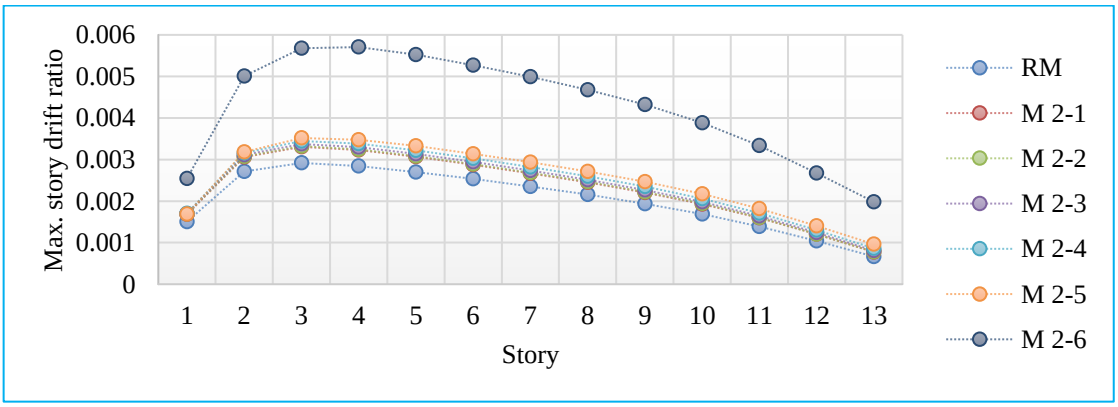
b) Maximum story drift perpendicular to EQ excitation direction



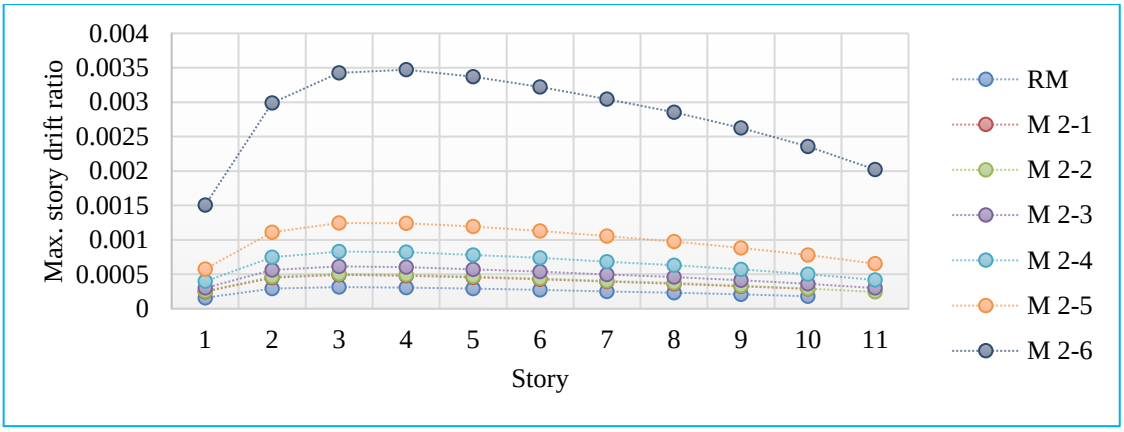
c) Total story drift ratio based on SRSS

Figure 4-7: Maximum story drift ratio for Type-I model.

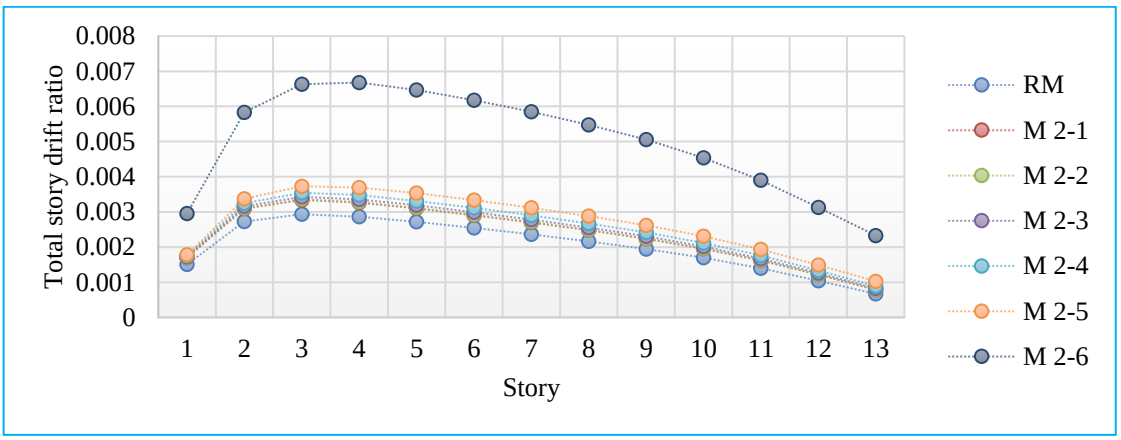
Type-II models



a) Maximum story drift ratio in EQ excitation direction



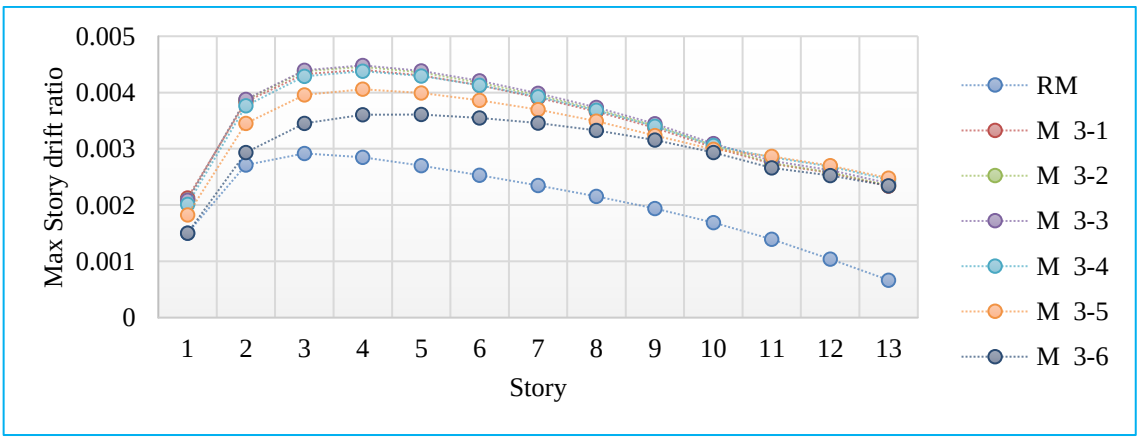
b) Maximum story drift ratio perpendicular to EQ excitation direction



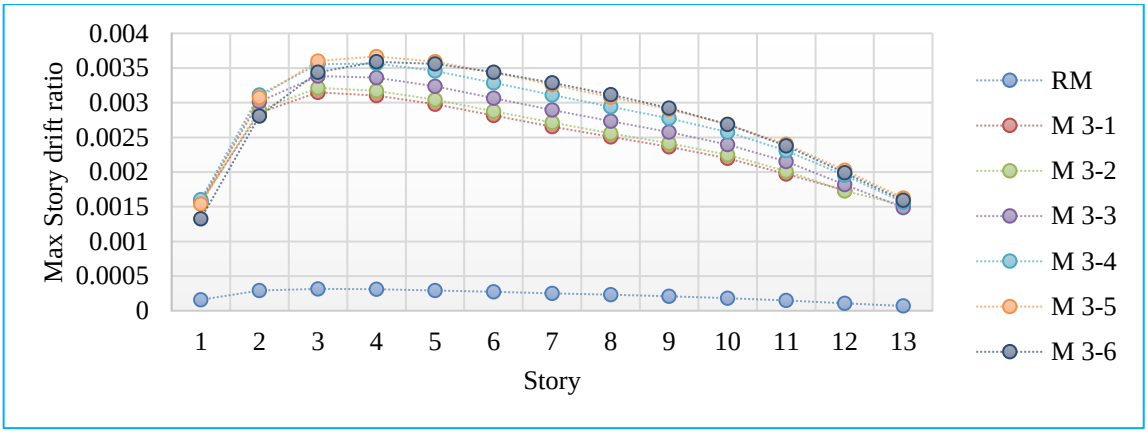
c) Total story drift ratio based SRSS

Figure 4-8: Maximum story drift ratio for Type-II models.

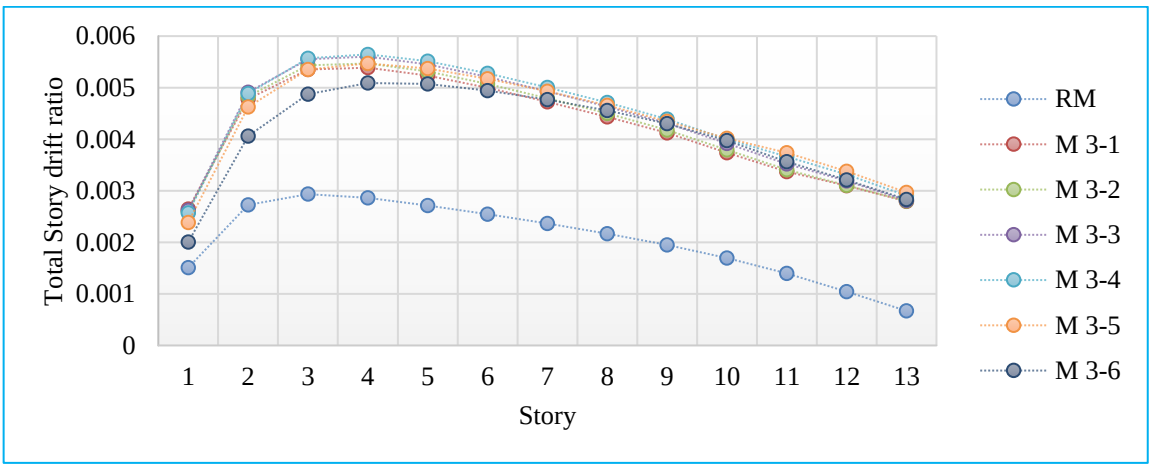
Type-III Models



a) Maximum story drift ratio in EQ excitation direction



b) Maximum story drift ratio perpendicular to EQ excitation direction



c) Total story drift ratio based on SRSS

Figure 4-9: Maximum story drift ratio for Type-III models.

Figure 4-7 to Figure 4-9 (a) show that story drift ratio distribution over the model's height that is created at earthquake direction. (b) show that story drift ratio distribution over model's height that is produced additionally in perpendicular to the earthquake direction. (c) shows the total story drift ratio distribution over model's height; that is calculated based on SRSS combination rule of story drift response in the direction of EQ excitation and perpendicular direction [in fig.(a) and (b)]. The distribution of total story drift ratio increases gradually over building's height and reaches its maximum value within 3rd and 5th story level then decreases at the higher levels in all models. The story drift response increase as the building plan irregularity increases from RM model to M 1-5, M 2-6 and M 3-6 of Type-I, II, and III respectively. Models M 1-5 of Type-I gets its maximum story drift response and reaches 0.00671 that is 131.03% over, compared to that of reference model RM (0.00293). Model M 2-6 of Type-II gets its maximum story drift response and

reaches 0.00668 that is 127.58% over, compared to that of reference model RM. Model M 3-4 of Type-III gets its maximum story drift response and reaches 0.00565 that is 92.38% over, compared to that of reference model RM.

4.5.1.1 Effect of lateral torsional vibration coupling in the Story drift ratio

Asymmetry in plan contributes significantly for translational-torsional coupling in the seismic response.

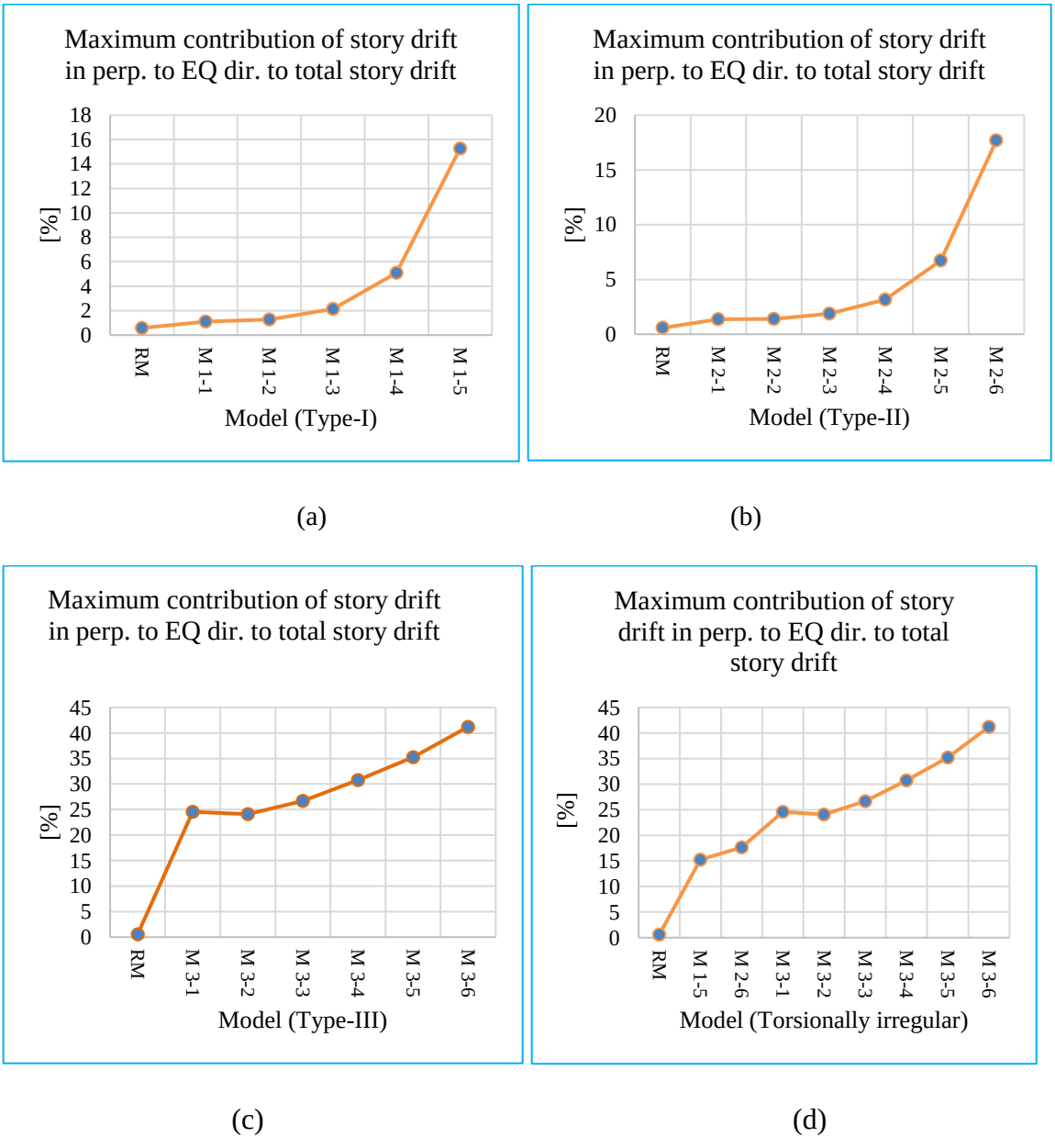
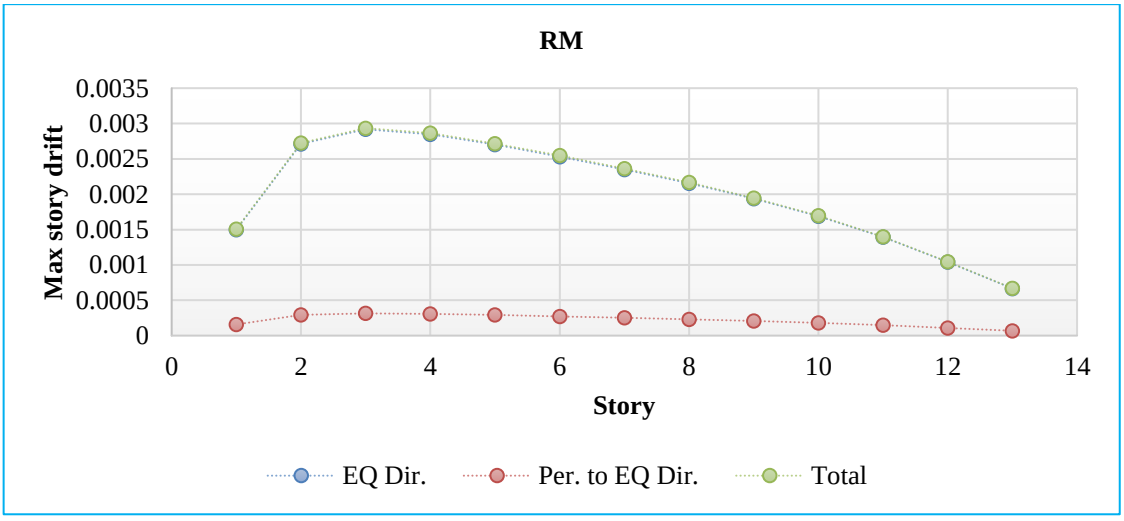
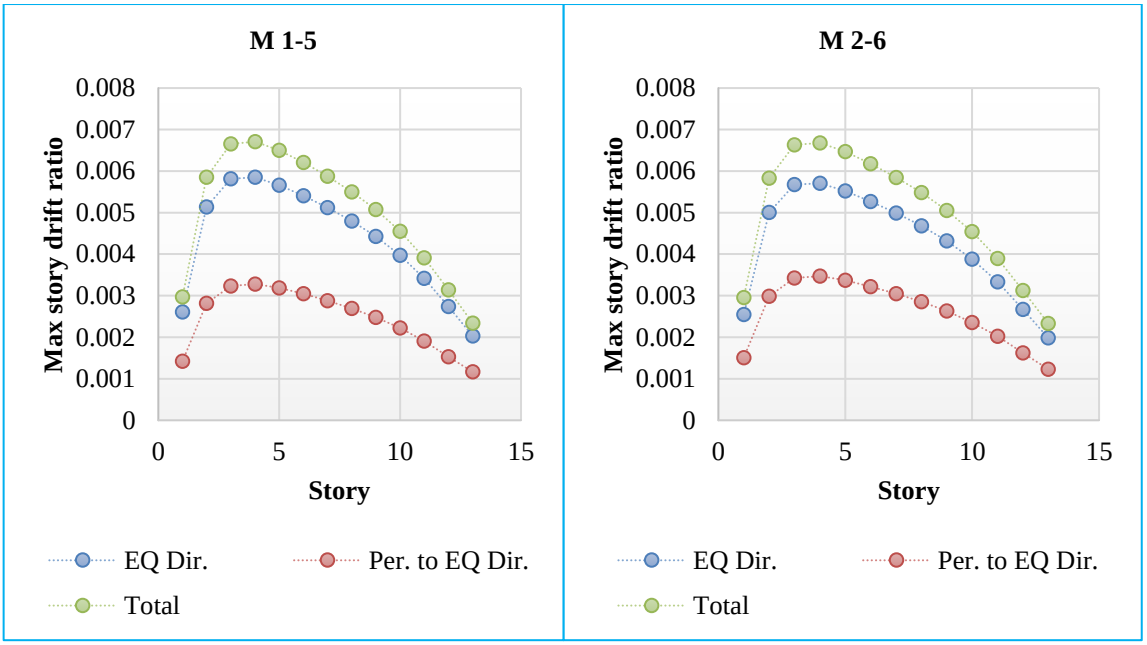


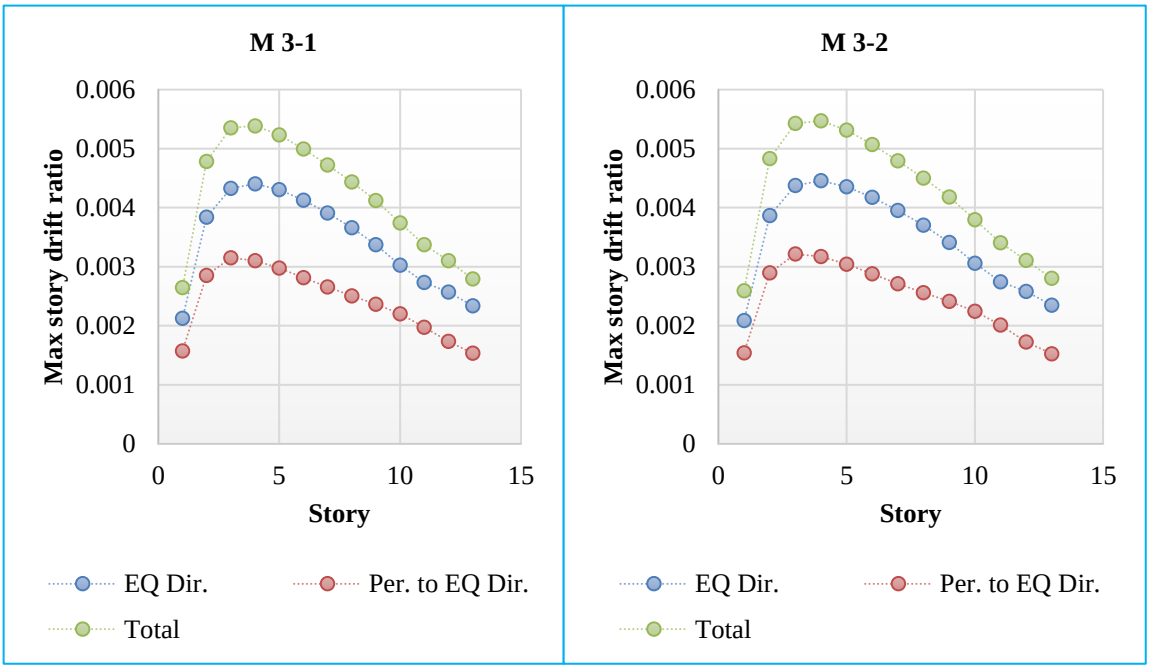
Figure 4-10: Contribution of story drift in perpendicular to EQ direction to total story drift response for a). Type-I, b). Type-II, c). Type-III and d) Torsionally irregular models.

Figure 4-10 (a, b, c, d) show the contribution of story drift additionally produced in perpendicular to EQ excitation direction to total story drift response due to lateral-torsional coupling effects. The contribution of story drift that is produced in the perpendicular direction to EQ excitation, increases as eccentricity increases. For regular and nearly regular models, its contribution is very small, however, for torsionally irregular models' significant contribution is observed (Figure 4-10(d)). Model M 1-5 of Type-I with its torsional irregularity ratio of 1.2 gets the maximum contribution of perpendicular to EQ direction that is 15.27% of EQ direction to total story drift. Model M 2-6 of Type-II with its torsional irregularity ratio of 1.21 gets its maximum contribution of perpendicular to EQ direction that is 17.68% of EQ direction to total story drift. In Type-III models, M 3-1, M 3-2, M 3-3, M 3-4, M 3-5, M 3-6 with their irregularity ratio of 1.524, 1.521, 1.527, 1.526, 1.520, and 1.525 get maximum contribution of 24.58%, 24.08%, 26.70%, 30.77%, 35.24%, and 41.22% of story drift in EQ direction to total story drift respectively. Type-III models are seen to exhibit abrupt changes from RM in story drifts in the direction perpendicular to EQ excitation, which is undesirable.




M 1-5
Max story drift ratio
0.008
0.007
0.006
0.005
0.004
0.003
0.002
0.001
0
0
5
10
15
Story
EQ Dir.
Per. to EQ Dir.
Total

M 2-6
Max story drift ratio
0.008
0.007
0.006
0.005
0.004
0.003
0.002
0.001
0
0
5
10
15
Story
EQ Dir.
Per. to EQ Dir.
Total


M 3-1
Max story drift ratio
0.006
0.005
0.004
0.003
0.002
0.001
0
0
5
10
15
Story
EQ Dir.
Per. to EQ Dir.
Total

M 3-2
Max story drift ratio
0.006
0.005
0.004
0.003
0.002
0.001
0
0
5
10
15
Story
EQ Dir.
Per. to EQ Dir.
Total

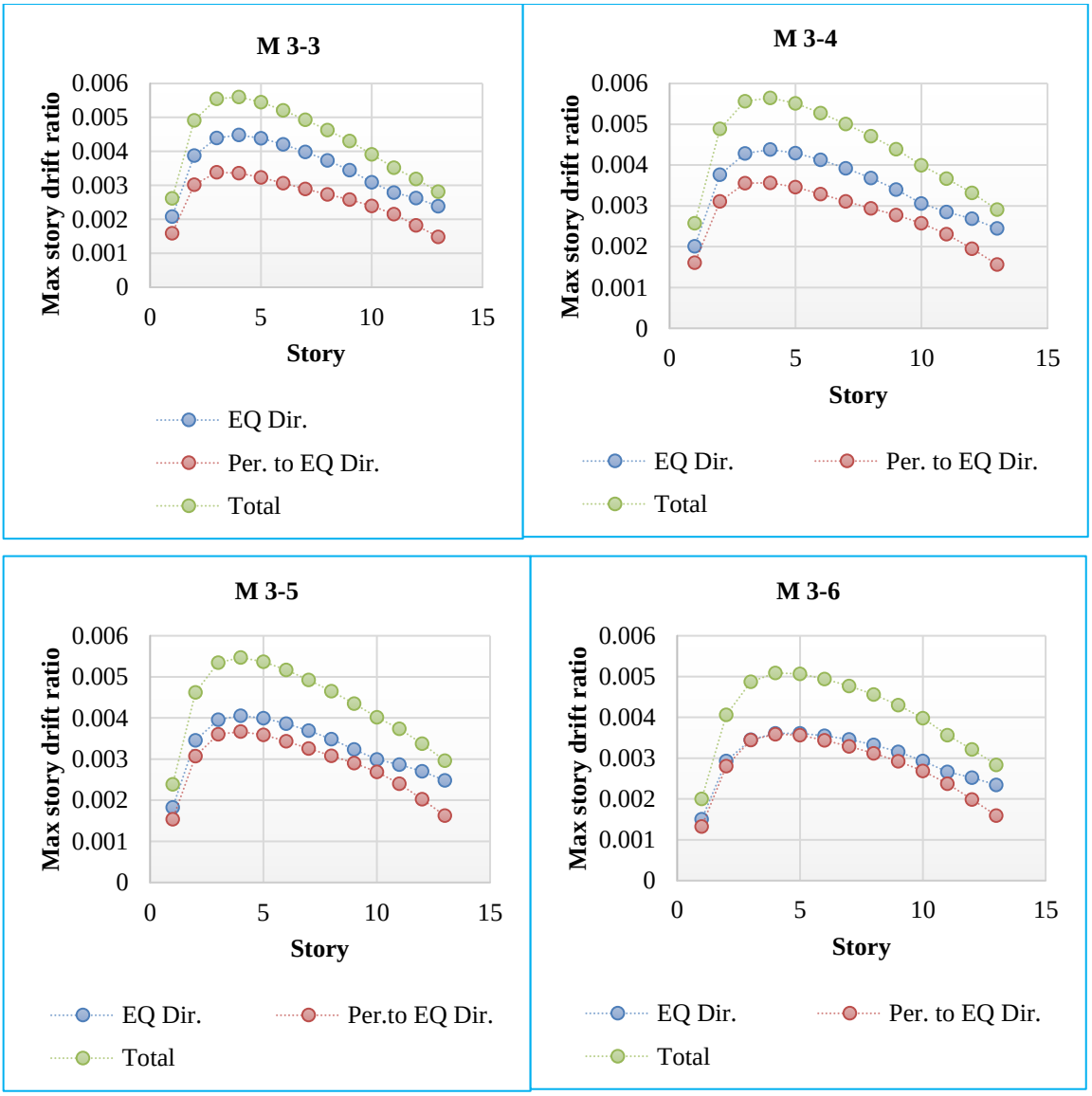


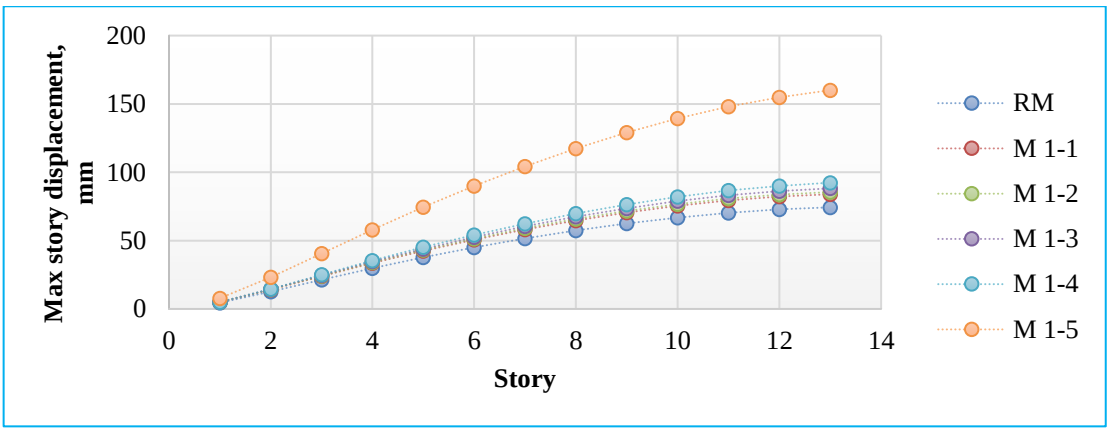
Figure 4-11: Effect of lateral torsional vibration coupling in the story drift ratio for the reference regular model and torsionally irregular models.

Figure 4-11 shows the effect of lateral torsional vibration coupling in the story drift ratio independently for the reference regular model and the torsionally irregular models. As it can be observed from the graph, for regular model RM, story drift in EQ excitation direction almost not changed, however, modification of story drift in EQ excitation direction significantly increased with plan irregularity increases.

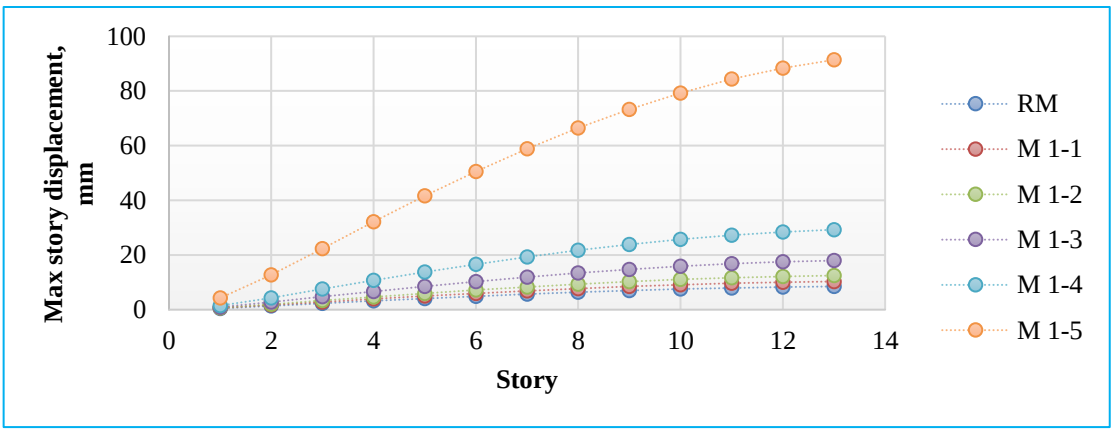
4.5.2 Lateral displacement response

The effect of plan irregularity on the lateral displacement demand is investigated based on analysis result hereafter. The story lateral displacements over the buildings' height for different models are introduced in Figure 4-12 to Figure 4-14.

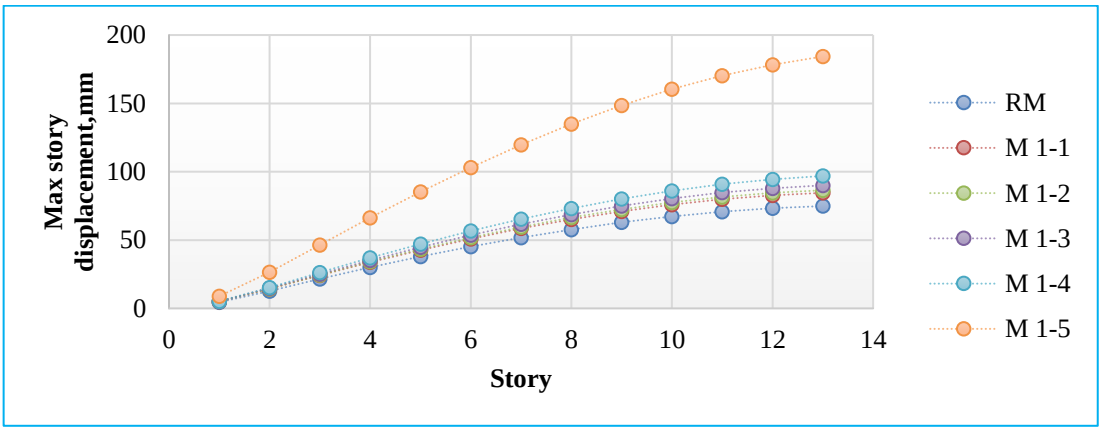
Type-I models



a) Maximum story displacement in EQ excitation direction



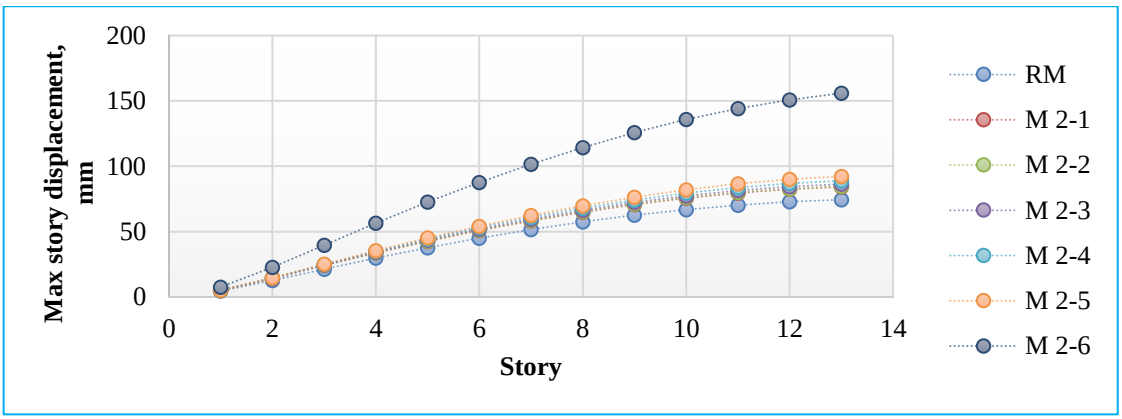
b) Maximum story displacement Perpendicular to EQ excitation direction



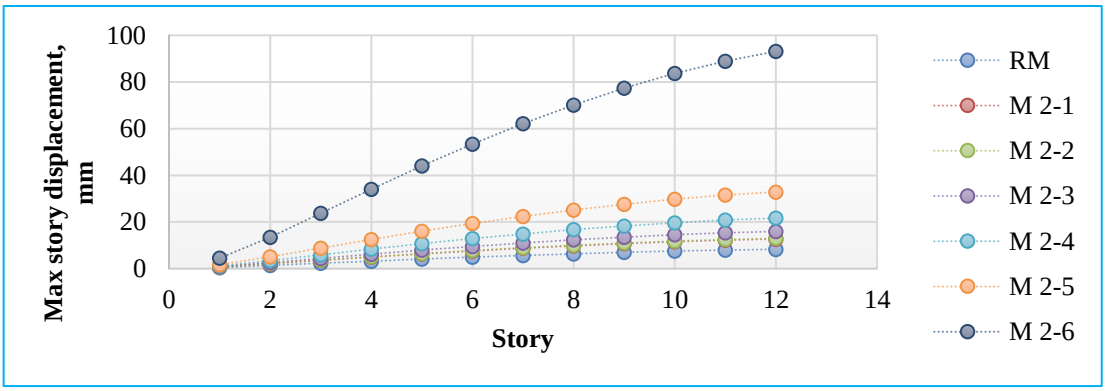
c) Total maximum story displacement based on SRSS

Figure 4-12: Maximum story displacement for Type-I models.

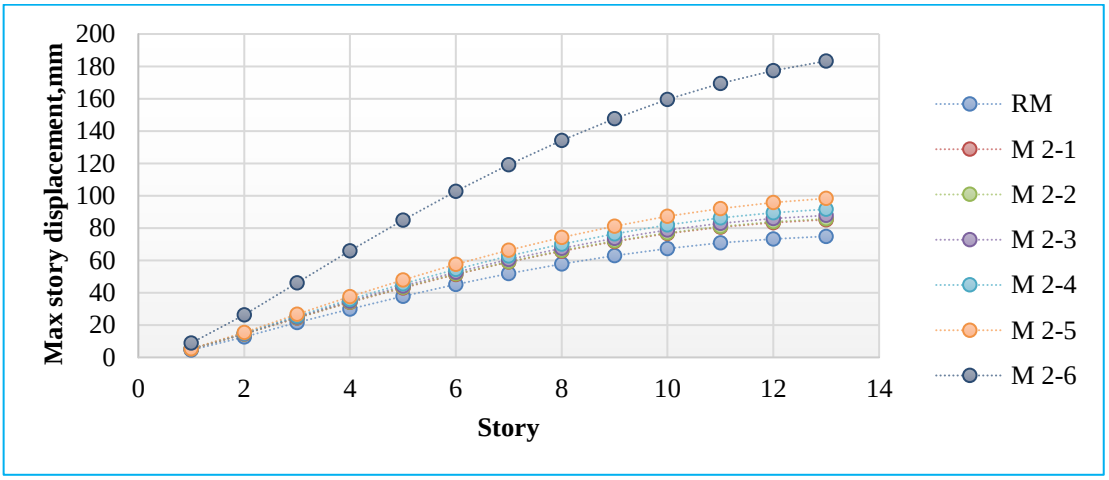
Type-II models



a) Maximum story displacement in EQ excitation direction



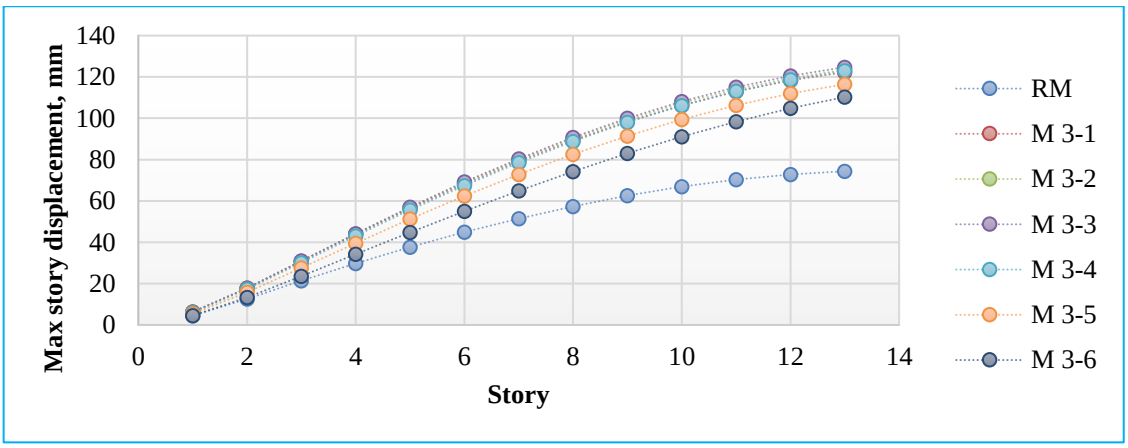
b) Maximum story displacement Perpendicular to EQ excitation direction



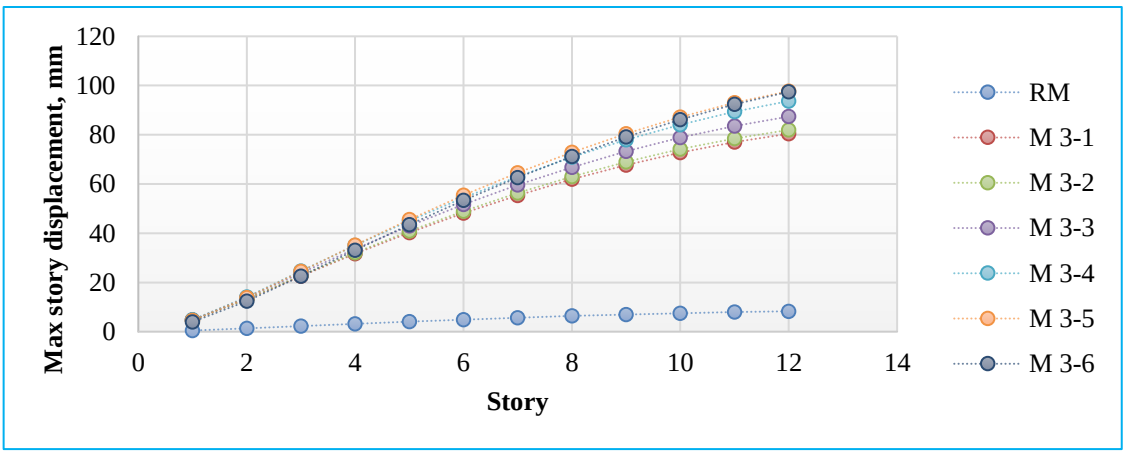
c) Total maximum story displacement based on SRSS

Figure 4-13: Maximum story displacement for Type-II models.

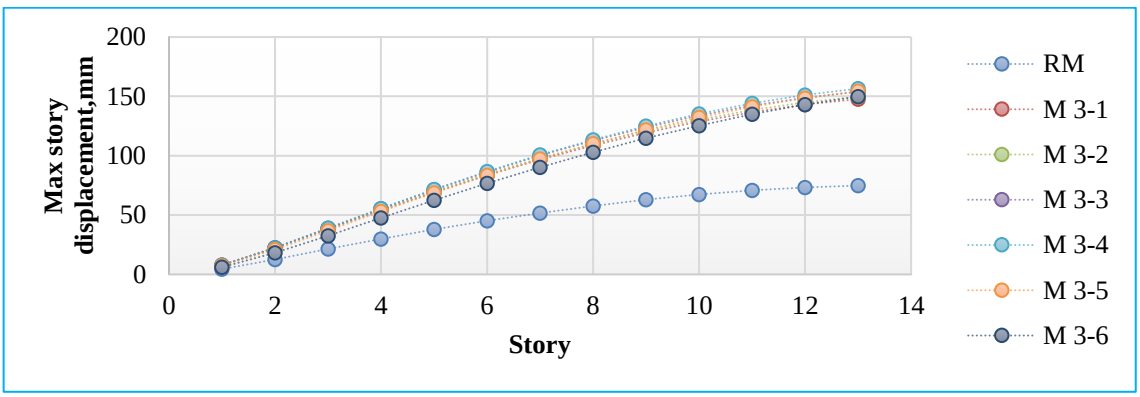
Type-III models



a) Maximum story displacement in EQ excitation direction



b) Maximum story displacement Perpendicular to EQ excitation direction



c) Total maximum story displacement based on SRSS

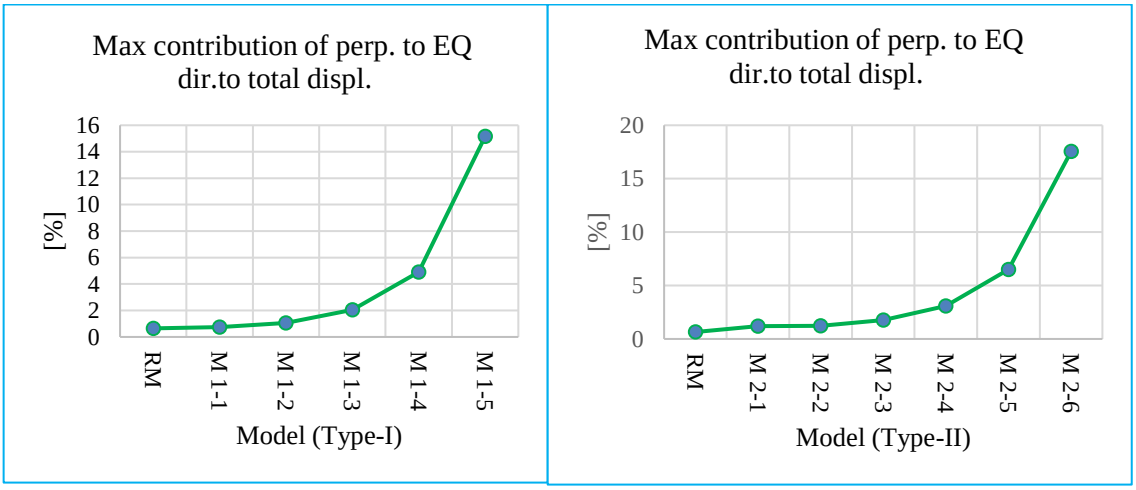
Figure 4-14: Maximum story displacement for Type-III models.

Figure 4-12 to Figure 4-14 (a) show that story displacement distribution over the model's height that is created at earthquake direction. (b) show that story displacement distribution over model's height that is produced additionally in perpendicular to the earthquake direction due to lateral-torsional coupling. (c) show the total story displacement distribution over model's height; that is calculated based on SRSS combination rule of story displacement response in the EQ excitation direction and perpendicular direction [in fig.(a) and (b)]. The distribution of total story displacement increases gradually over building's height in all models. The story displacement response increase as the buildings' plan irregularity increase from RM model to M 1-5, M 2-6 and M 3-4 of Type-I, II, and III respectively. Models M 1-5 of Type-I gets its maximum story displacement response and reaches 184.33mm that is 146.20% over, compared to that of reference model RM (74.87mm). Model M 2-6 of Type-II gets its maximum story displacement response and reaches 183.27mm that is 144.79% over, compared to that of reference model RM. Model M 3-4 of Type-III gets its maximum story displacement response and reaches 156.58mm that is 109.14% over, compared to that of reference model RM.

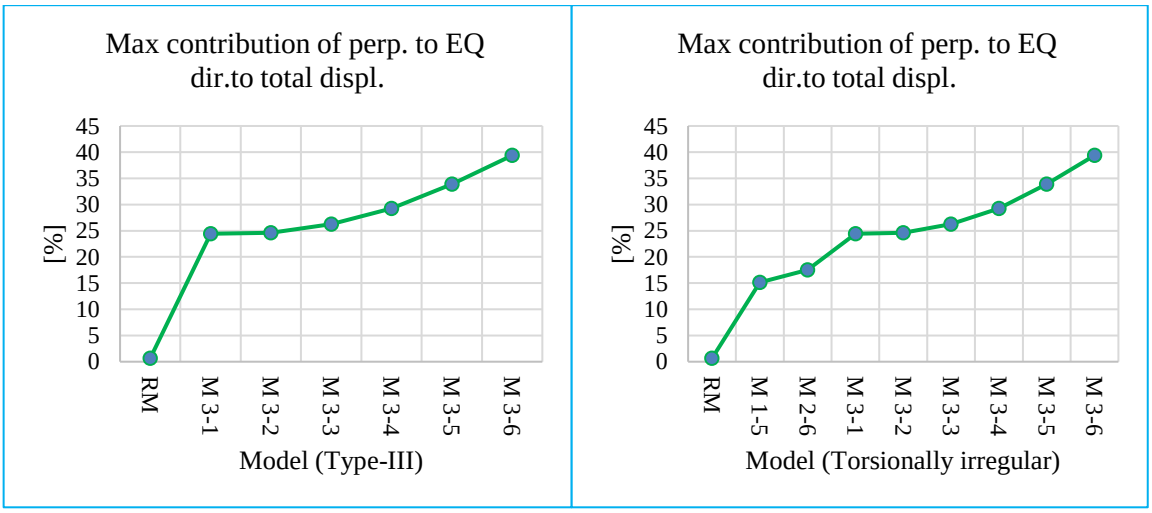
4.5.2.1 Effect of lateral torsional coupling in the Story displacement

The story displacement response demand in the perpendicular direction to earthquake load is significantly developed with the increase of lateral torsional vibration coupling. The contribution of story displacement that is produced in the perpendicular direction to EQ excitation, increases as eccentricity increases. For regular and nearly regular models, its contribution is very small, however, for the torsionally irregular models' significant contribution is observed. Figure 4-15 (a, b, c) illustrate the percentage of maximum contribution of story displacement additionally produced in perpendicular to EQ excitation direction to total story displacement response due to lateral-torsional effects. Model M 1-5 of Type-I with its torsional irregularity ratio of 1.2 gets the maximum contribution of perpendicular to EQ direction that is 15.15% of EQ direction to total story displacement. Model M 2-6 of Type-II with its torsional irregularity ratio of 1.21 gets its maximum contribution of perpendicular to EQ direction that is 17.54% of EQ direction to total story displacement. In Type-III models, M 3-1, M 3-2, M 3-3, M 3-4, M 3-5, M 3-6 with their irregularity ratio of 1.524, 1.521, 1.527, 1.526, 1.520, and 1.525 get maximum contribution of 24.44%, 24.63%, 26.28%, 29.25%, 33.88%, and 39.38% of story displacement in EQ direction to total story displacement respectively. Type-III models are seen to exhibit

abrupt changes from RM in story displacements in the direction perpendicular to EQ excitation, which is undesirable.



(a) (b)

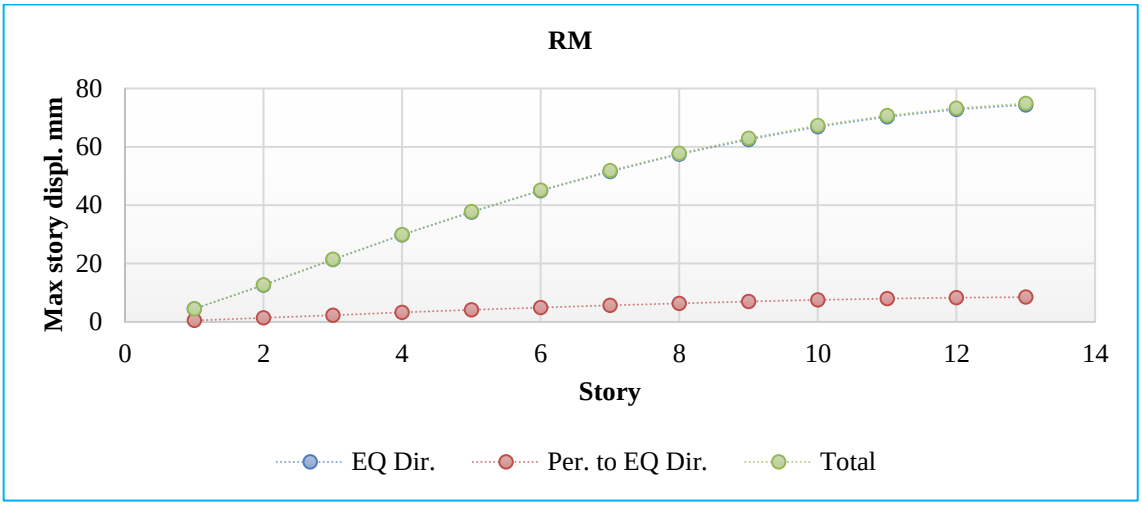


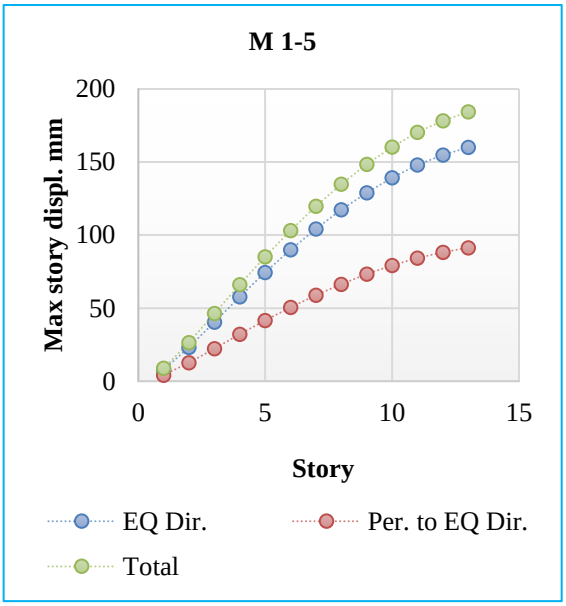
(c) (d)

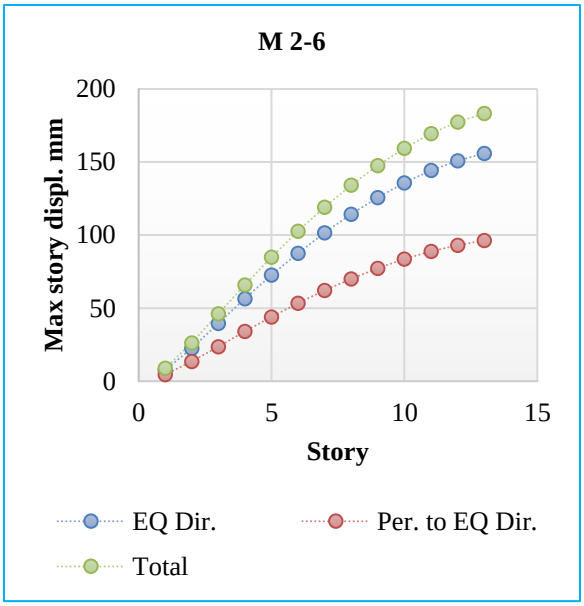
Figure 4-15: The percentage maximum contribution of story displacement perpendicular to EQ direction to total story displacement.

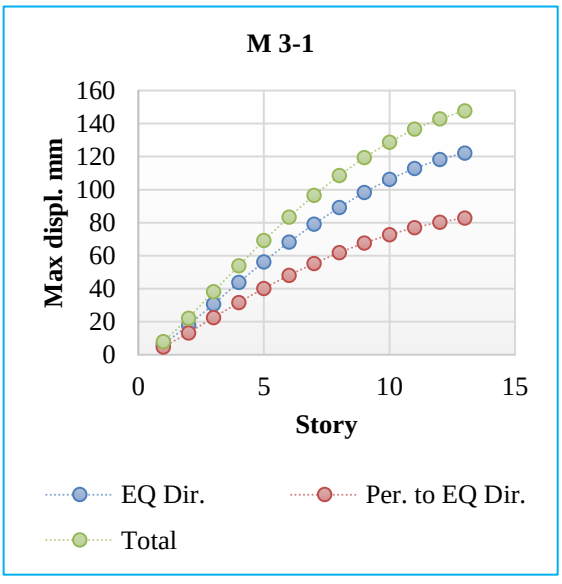
Figure 4-16 below shows the effect of lateral torsional vibration coupling in the story displacement independently for the reference regular model and torsionally irregular models. As it can be observed from the graph, for regular model RM, story displacement in EQ excitation direction almost not changed, however, modification of story displacement in EQ excitation direction significantly increased with plan irregularity

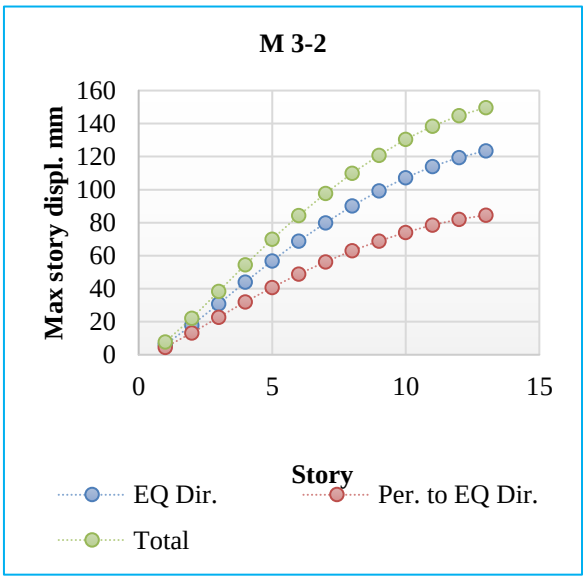
increases. Models that are torsionally irregular as per codes are seen to exhibit abrupt changes in story displacement, which is highly undesirable.











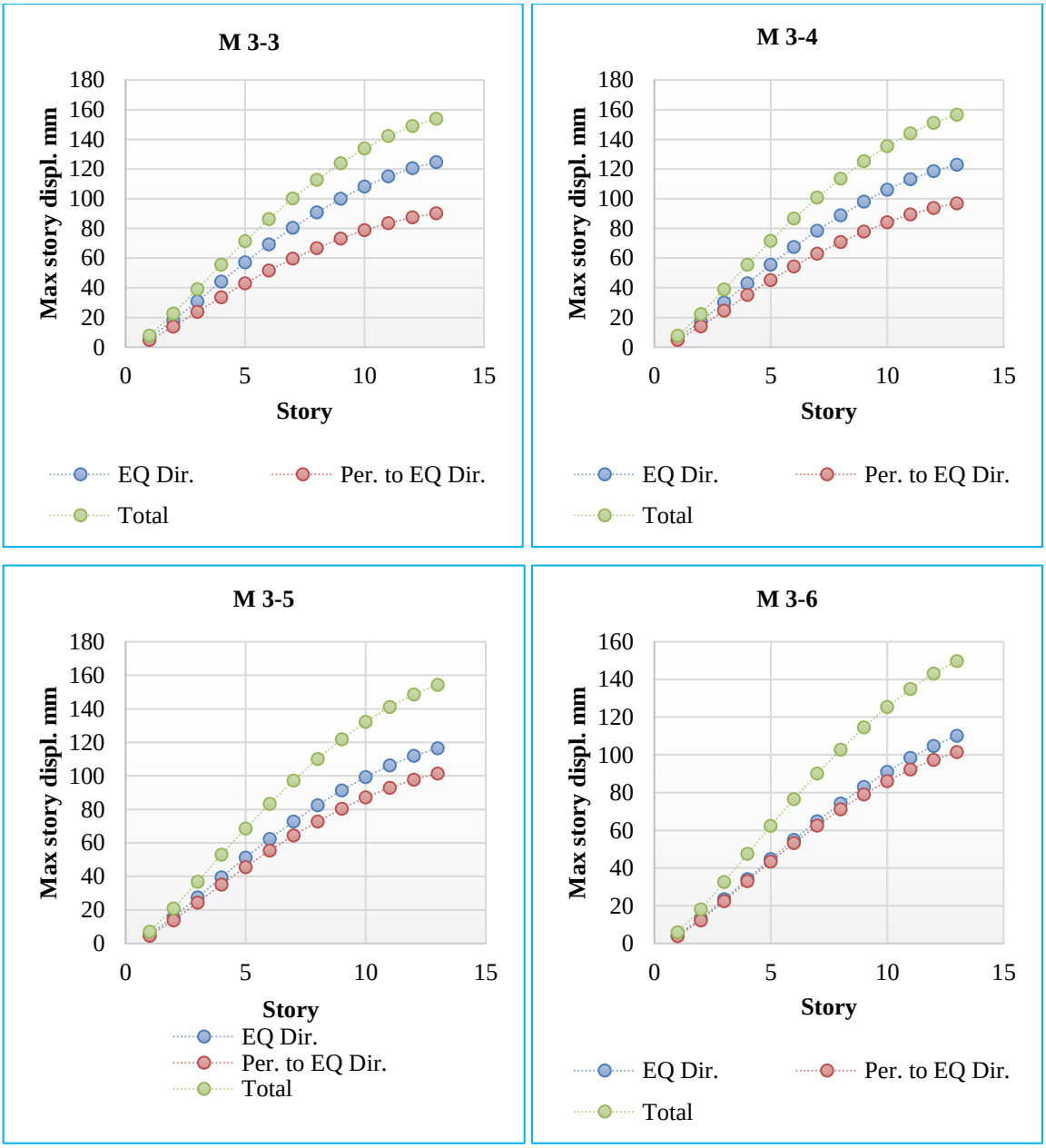


Figure 4-16: Effect of lateral torsional vibration coupling in the story displacement for the reference regular model and torsionally irregular models.

4.6 Torsional diaphragm rotation

Torsional rotation of floors considered as the main parameter of the torsional response of the building plus probability of local failure for outer element threatening the robustness of a structure that is highly dependent on the performance of the diaphragms [37, 38].

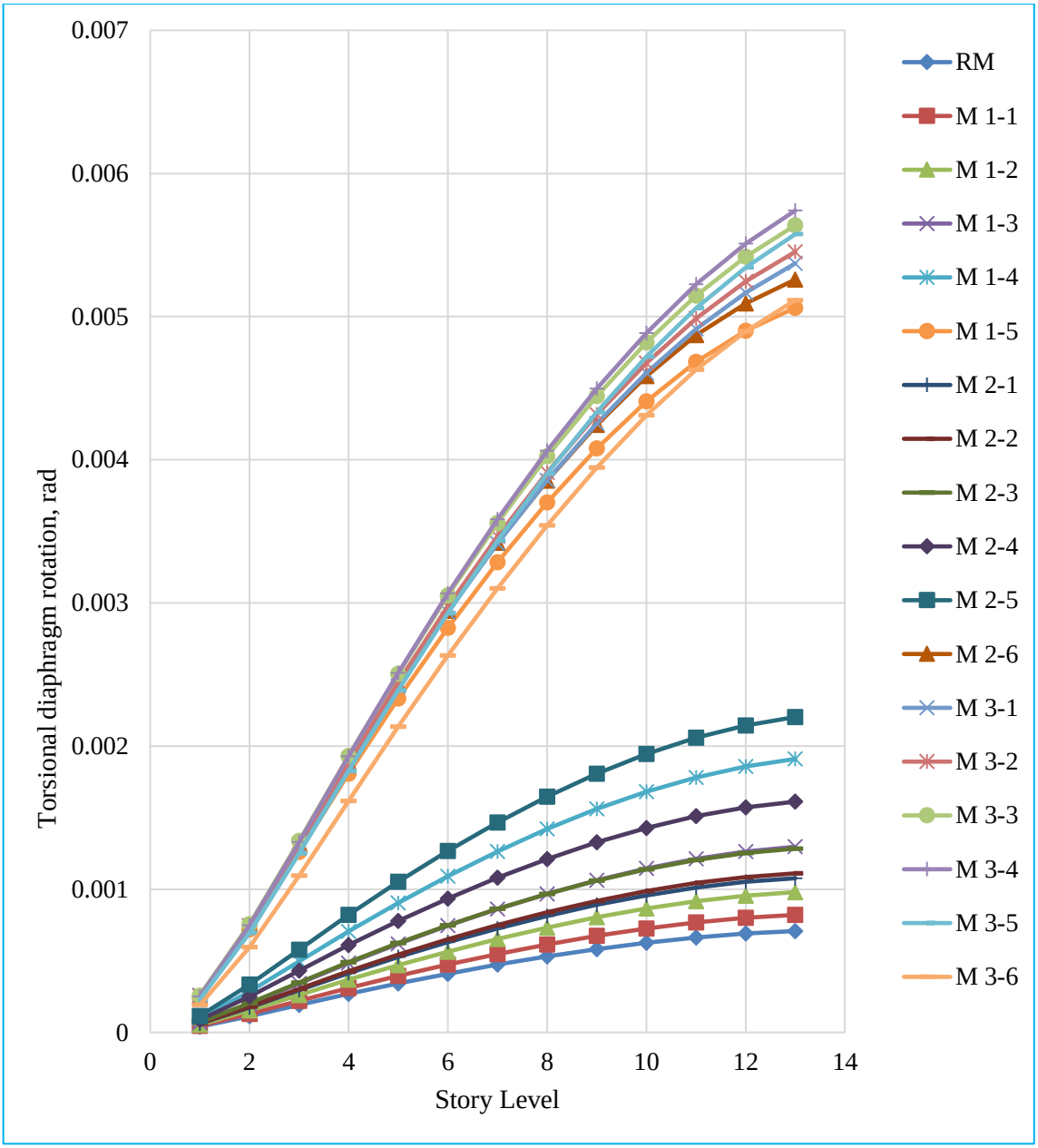


Figure 4-17: Torsional diaphragm rotation response.

Figure 4-17: shows the torsional diaphragm rotation for regular building model and all plan irregular building models over models' height. The torsional diaphragm rotation changes through the height of the structure reaching the maximum value at the top level of models. The increase of irregularity has a considerable effect on the torsional rotation; thus, the torsional diaphragm rotation increases with the increase of plan irregularity (Figure 2-4). It is also noted that torsional rotation increases with the eccentricity of the model. The maximum torsional rotation response demand for all building models; through which, maximum torsional diaphragm rotation for models RM, M 1-1, M 1-2, M 2-1, M 2-2, M 2-3, M 1-3, M 2-4, M 1-4, M 2-5, M 1-5, M 3-6, M 2-6, M 3-1, M 3-2, M 3-5, M

3-3 and M 3-4 are 0.00071(100%), 0.00082(116%), 0.00098(138%), 0.00130(152%), 0.00191(157%), 0.00506(181%), 0.00108(183%), 0.00111(228%), 0.00128(270%), 0.00161(311%), 0.00220(715%), 0.00526(722%), 0.00537(743%), 0.00545(759%), 0.00564(770%), 0.00574(788%), 0.00558 (796%)and 0.00512 (811%) rad respectively.

4.7 Base Shear Force

The plan asymmetry or lateral torsional coupling also has an effect on base shear. The discussion is carried out through normalized base shear. Normalized shear force presents shear force response demand at the base as the ratio to building's weight ($V_b/\Sigma W$) this parameter allows accurate comparison between buildings which accumulate different areas with different lumped masses. The weights of building models are computed taking into account the vertical actions (permanent loads and reduced variable-live loads) according to EBCS EN 1998/3.2.4 (see section 3.3.2).

Table 4-5: Base shear and total weights of building models.

Models	$V_{bx,}$ KN	$V_{by,}$ KN	V_{bT}, based on SRSS, KN	ΣW, KN
RM	3056.60	0.96	3056.60	75549.34
M 1-1	3226.84	72.87	3227.66	73602.10
M 1-2	2972.29	67.47	2973.05	67760.39
M 1-3	2545.48	58.18	2546.15	58024.20
M 1-4	1938.14	65.24	1939.23	44393.54
M 1-5	1926.39	292.83	1948.52	26868.40
M 2-1	3254.36	72.17	3255.16	74166.14
M 2-2	3169.56	70.66	3170.35	72218.90
M 2-3	2913.24	65.79	2913.98	66377.19
M 2-4	2482.74	63.25	2483.55	56641.00
M 2-5	1870.52	99.10	1873.15	43010.34
M 2-6	1821.67	368.79	1858.63	25485.20
M 3-1	5825.11	2506.80	6341.60	77263.74
M 3-2	5716.54	2438.87	6215.06	75316.50
M 3-3	5375.62	2260.18	5831.44	69474.79
M 3-4	4831.12	1919.34	5198.42	59738.60
M 3-5	4068.21	1384.42	4297.32	46107.94
M 3-6	2901.04	768.33	3001.06	28582.80

Table 4-5 shows that base shear force for each model in the earthquake loading direction (V_{bx}), perpendicular direction to earthquake loading (V_{by}) and total value (V_{bT}) calculated based on SRSS combination approach and total weight (ΣW) for each model.

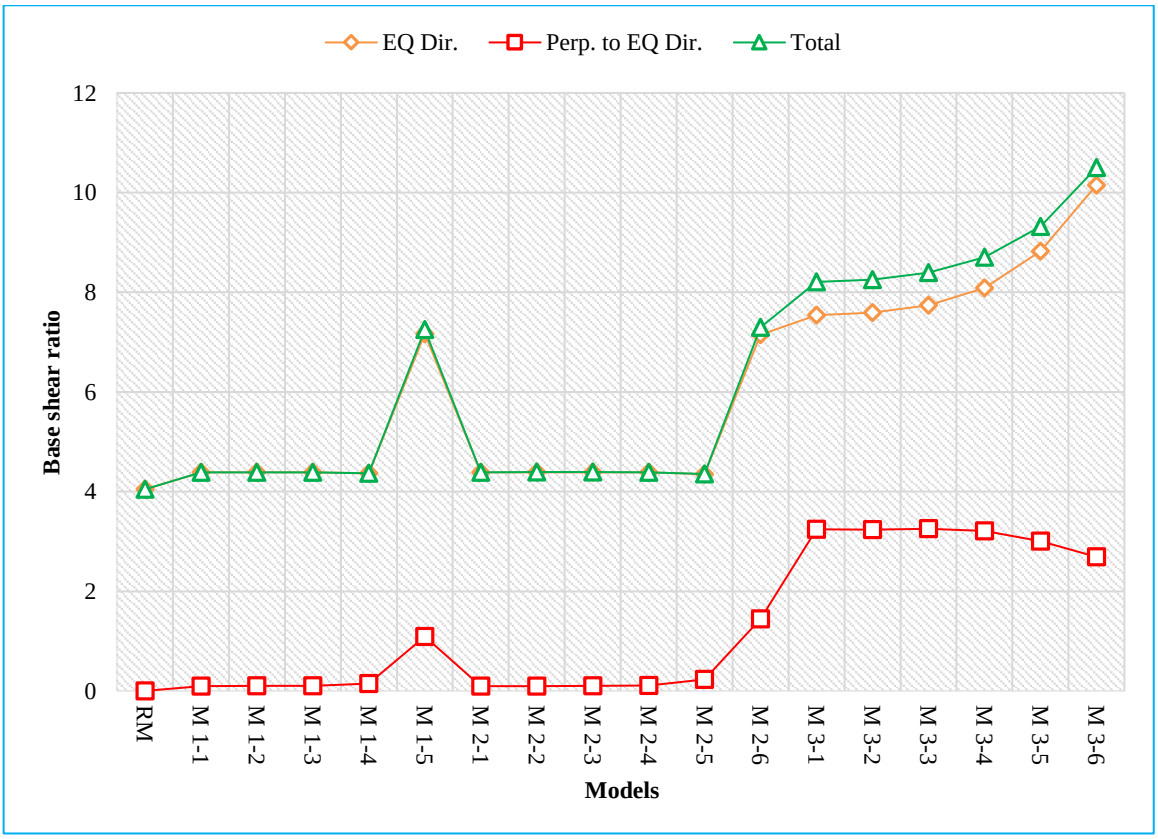


Figure 4-18: Normalized base shear for all building models.

Figure 4-18 shows that normalized base shear force for each model in the earthquake loading direction, perpendicular direction to earthquake loading and total value. The total base shear significantly increases with gradually increasing in eccentricity between the center of mass and the center of rigidity. The regular model displays no shear demand in perpendicular to EQ direction which contributes to total value whereas with the development of eccentricity due to plan asymmetry, the additional shear force developed in the perpendicular direction to earthquake direction could violate the safe design for resisting elements. The developed shear force in the perpendicular direction could reach 0.10, 0.10, 0.10, 0.15 and 1.09% of the building weight for M1-1 to M 1-5 of Type-I models; 0.10, 0.10, 0.10, 0.11, 0.23 and 1.45% of the building weight for M 2-1 to M 2-6 of Type-II models; 3.24, 3.24, 3.25, 3.21, 3.00 and 2.69% of the building weight for M 3-1 to M 3-6 of Type-III models respectively. For the torsionally irregular building models,

an abrupt change in values is observed. These effects result from lateral–torsional coupling action that may cause a disastrous effect for lateral load resisting element.

4.8 Practical solutions to torsional irregularity effects

4.8.1 Shear walls

In this thesis, one of the recommended practical solutions is stiffening the flexible-side by introducing shear wall so as to get balanced stiffness building model. This is achieved by adding equivalent in-plan stiffness of the stiffer-side to the flexible-side of the building. For the demonstration purpose, one of the torsionally irregular models, M 3-4 from Figure 3-4 is taken. The dimensions of the shear wall in each selected axis are determined by equating summation of lateral stiffness of columns lost from its regular counterpart or stiffer side of the corresponding axis and lateral stiffness of equivalent shear wall.

The lateral stiffness of column can be determined from Eq.4-1; considering the beams are rigid

$$K = \sum_{column} \frac{12EI_c}{h^3} \qquad \text{Eq. 4-1}$$

The lateral stiffness of shear wall can be determined from

$$K = \frac{12EI_w}{h^3} \qquad \text{Eq. 4-2}$$

The moment of inertia about axis of bending, I_c and I_w for column and shear wall respectively can be determines from

$$I_c = \frac{b_c h_c^3}{12} \text{ and } I_w = \frac{b_w h_w^3}{12} \qquad \text{Eq. 4-3}$$

Where b_c, b_w and h_c, h_w width and depth of column and shear wall respectively [9].

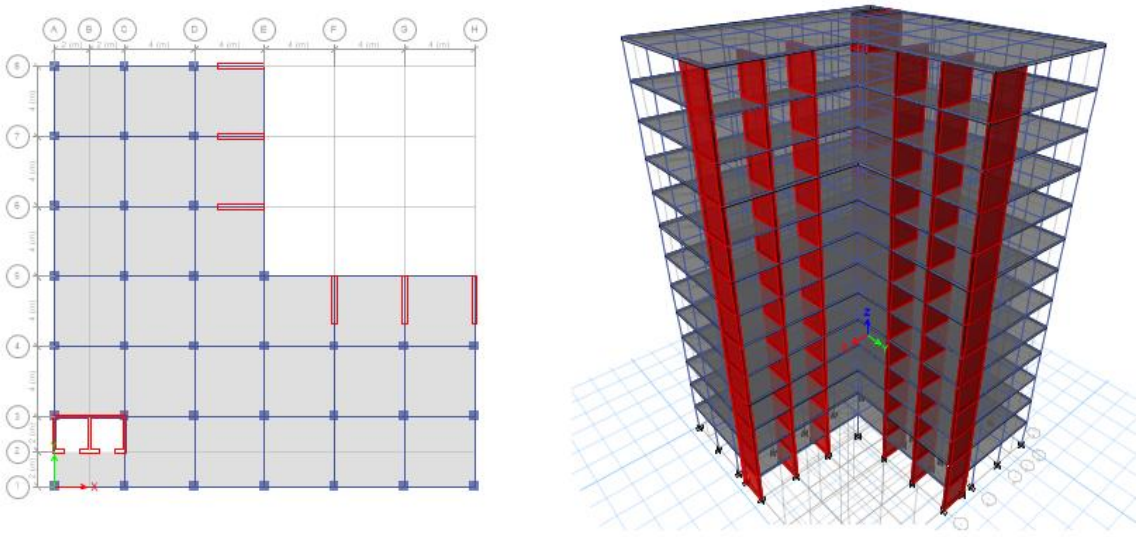


Figure 4-19: Model M 3-4 that is strengthened by equivalent shear walls.

4.8.2 Diagrid System

In this thesis, like an equivalent shear wall, the reinforced concrete diagrid system is provided on the weaker side so as to get balanced stiffness system.

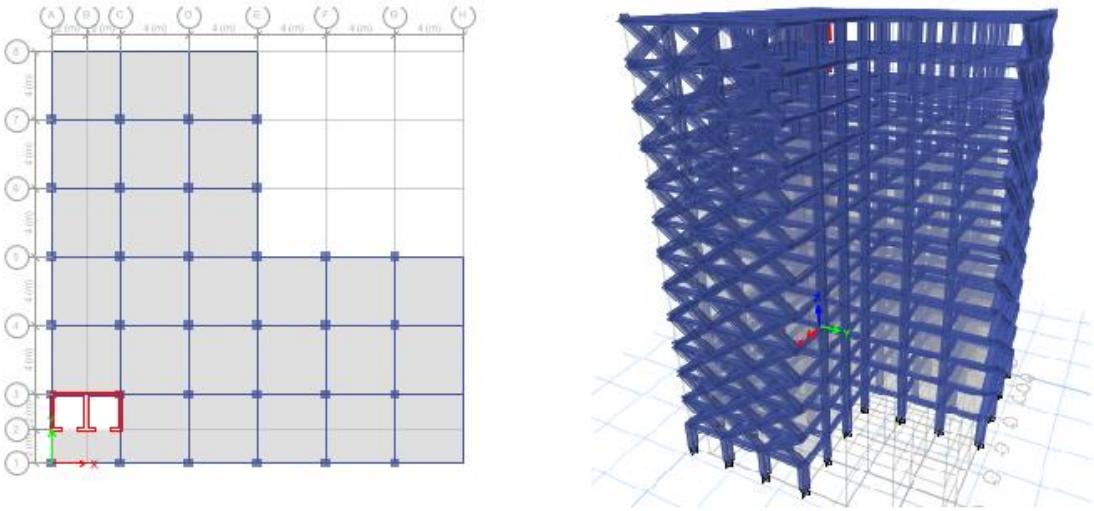
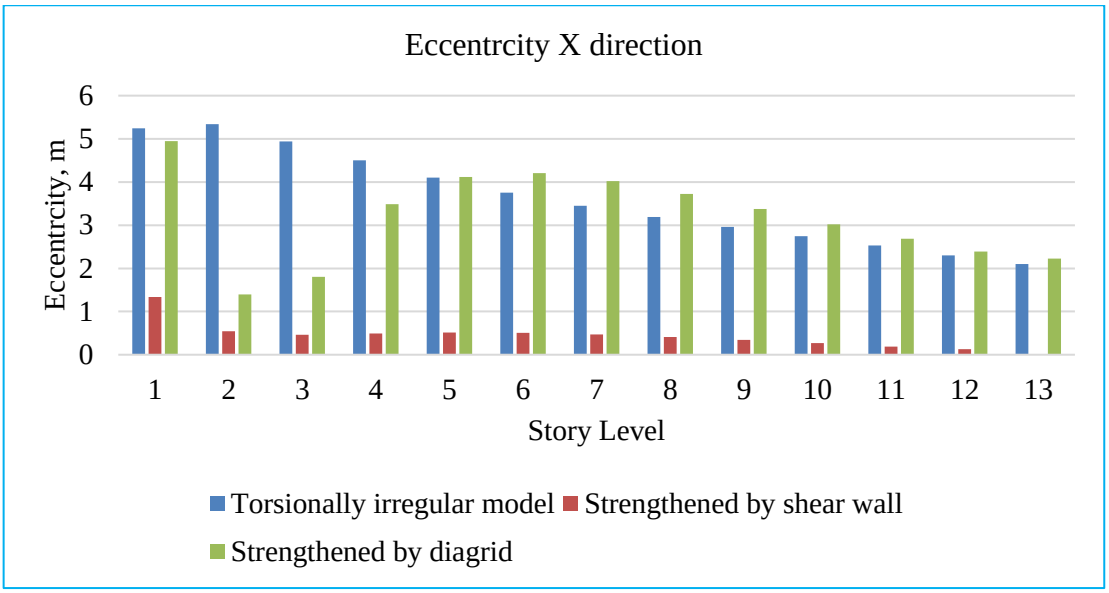


Figure 4-20: Model M 3-4 strengthened by diagrid system.

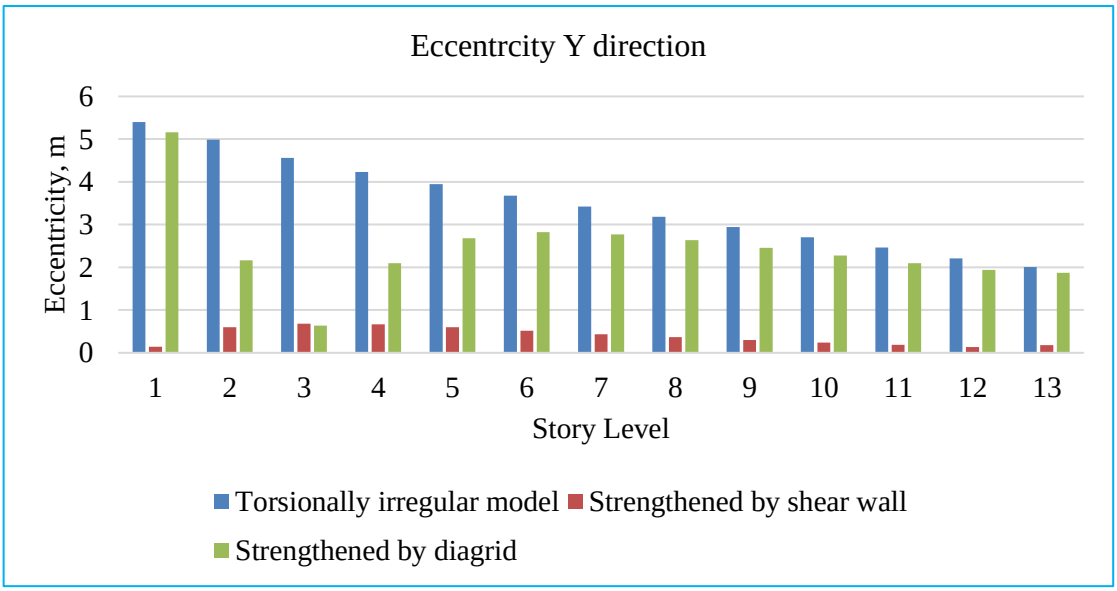
4.8.3 Summary of analysis results and discussions

Dynamic linear analysis using response spectrum method is performed for the model with shear wall and diagrid system. The performance of the structures is compared based on parameters such as eccentricity, Story displacement, Bending moment, shear force, axial force, and torsion.

4.8.3.1 Eccentricity



(a)



(b)

Figure 4-21: (a), (b): The eccentricity in X and Y direction respectively.

The eccentricities of the model with equivalent shear wall are reduced from 74% to 100% and 84% to 98% over the stories in X and Y direction respectively when compared with the eccentricities of torsionally irregular model M 3-4. Similarly, in the model with diagrid system, eccentricities are reduced from 5% to 74% and 4% to 87% in X and Y direction respectively. However, in the model with diagrid system, on some stories eccentricities

exceeded the eccentricities of the torsionally irregular model M 3-4 up to 17% in the X direction.

4.8.3.2 Lateral displacement

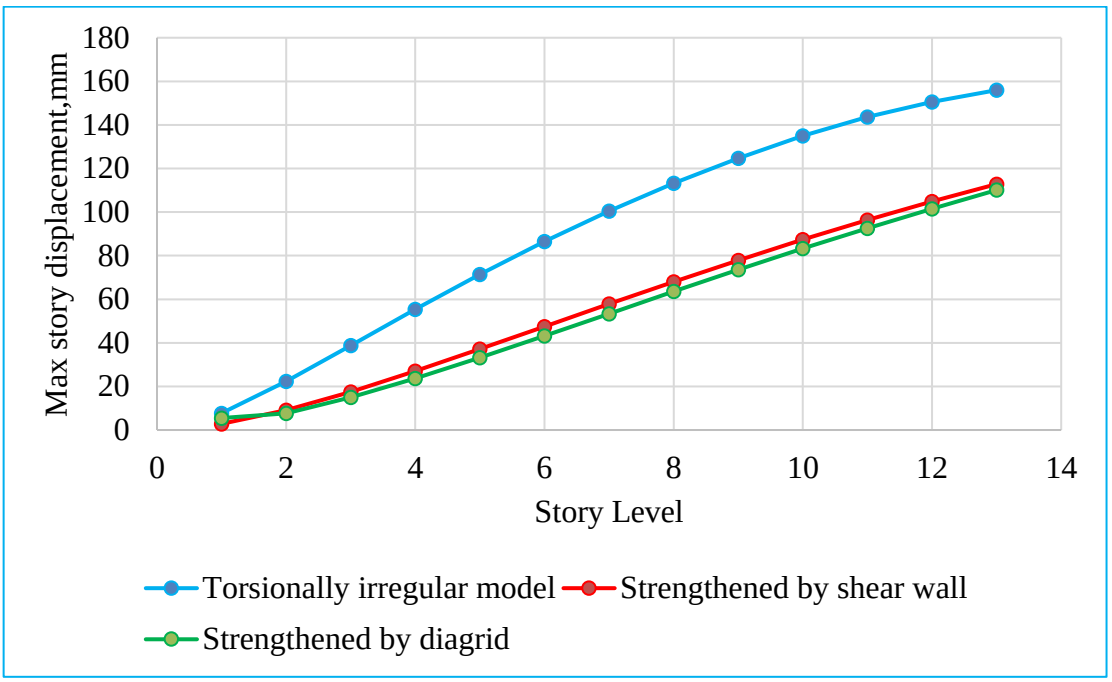


Figure 4-22: The maximum story displacement of irregular and strengthened models.

For the models with equivalent shear wall and diagrid system, significant lateral displacement reduction is observed when compared with model M 3-4 which is torsionally irregular. The reduction of maximum story displacement is from 27% to 64% and 28% to 67% over the stories for the model with equivalent shear wall and diagrid system respectively.

4.8.3.3 Torsional diaphragm rotation

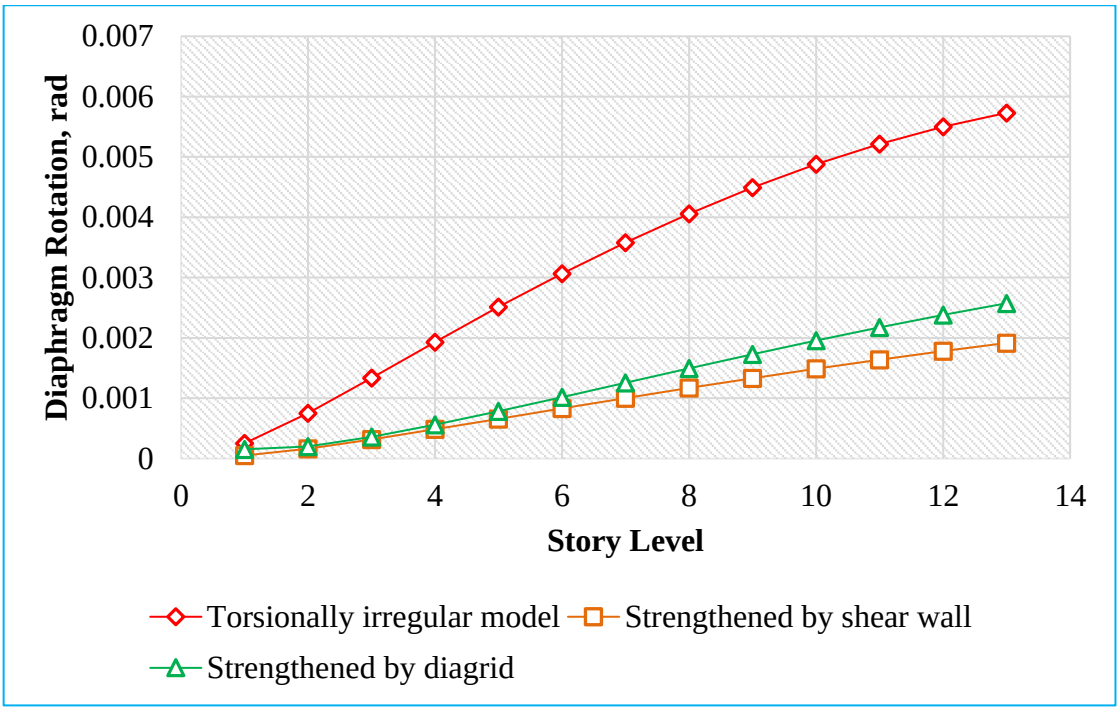


Figure 4-23: The torsional diaphragm rotation of irregular and strengthened models.

In terms of torsional diaphragm rotation which is considered as a significant parameter to evaluate the torsional effect, the appreciable reduction is observed that is from 66% to 80% and 38% to 74% over the stories for the model with equivalent shear wall and diagrid system respectively.

4.8.3.4 Effect of lateral torsional vibration coupling

Effect of lateral torsional vibration coupling in the story displacement is looked from the analysis results. The story displacement response demand in the perpendicular direction to earthquake load due to lateral torsional vibration coupling is significantly reduced for the models with equivalent shear wall and diagrid system.

Table 4-6: Percentage of story displacement in the perpendicular direction to story displacement in the direction of earthquake load

Story	M 3-4 Irregular Model	M 3-4 Strengthened by shear walls	M 3-4 Strengthened by diagrids
	%	%	%
1	79	26	66
2	81	26	52
3	82	26	51
4	81	25	49
5	81	25	48
6	80	25	47
7	80	24	46
8	79	24	45
9	79	24	44
10	79	24	43
11	79	24	43
12	79	24	42
13	78	24	42

4.8.3.5 Column and beam forces

For the selected columns and beams internal forces are extracted from the analysis for a given load combination and the comparisons are performed in terms of shear force, bending moment, axial force and torsion. Table 4-7 and Table 4-10 below show the shear force, bending moment, axial force and torsion in an exterior and interior column in each story for RM, M 3-4 and strengthened models with shear walls and diagrids. Also, Table 4-11 shows the shear force, bending moment and torsion in an interior beam in each story for M 3-4 and strengthened models with shear walls and diagrids. In the most of the stories, the reduction in terms of these forces is observed for the model with the equivalent shear wall and diagrid system comparing with M 3-4 which is torsionally irregular. In the beam shear force reduction, up to 43% and 75% observed for the model with the equivalent shear wall and diagrid system respectively. Similarly, bending moment reduction up to 72% and 93% observed for the model with the equivalent shear wall and diagrid system respectively. In the columns significant reduction in torsion ranges from 58% to 88% and 17% to 71% is observed over the stories for the model with the equivalent shear wall and diagrid system respectively.

Table 4-7: Shear force, bending moment, axial force and torsion in an exterior column for RM, M 3-4 and strengthened models with shear walls and diagrids.

` Story	RM, Regular Model				M 3-4, Torsionally irregular Model			
	V_2 KN	M_3 KNm	P KN	T KNm	V_2 KN	M_3 KNm	P KN	T KNm
1	27.83	74.78	1294.69	0.35	16.33	64.3	3359.54	2.42
2	24.46	44.42	1192.69	0.63	33.1	72.87	3167.38	4.81
3	22.39	33.95	10.8	0.68	30.37	54.01	2887.58	4.45
4	21.49	30.89	966.54	0.67	30.71	50.21	2563.57	5.5
5	20.71	29.22	854.45	0.63	30.6	47.62	2228.51	5.33
6	19.98	27.99	744.5	0.59	29.78	44.8	1899.68	5.09
7	19.23	26.72	636.95	0.55	28.6	42.21	1585.98	4.81
8	18.39	25.24	532.07	0.5	27.28	39.89	1288.98	4.81
9	17.46	23.56	430.28	0.44	26	37.48	1011.78	4.53
10	16.32	21.37	332.21	0.39	24.62	28.99	755	3.91
11	15.12	18.79	238.85	0.32	22.65	23.36	523.14	3.42
12	12.37	15.08	151.63	0.24	19.64	20.9	318.22	2.91
13	13.84	18.21	71.43	0.15	20.75	33.89	140.96	2.28

Table 4-8: The table continued from Table 4- 7

Story	M 3-4, Strengthened by Shear wall				M 3-4, Strengthened by diagrid			
	V2 KN	M3 KNm	P KN	T KNm	V2 KN	M3 KNm	P KN	T KNm
1	8.69	30.31	3282.5	0.29	37.37	72.45	2983.71	1.16
2	16.26	33.66	3108.22	0.65	34.36	79.45	2844.83	1.39
3	16.26	29.11	2850.73	0.89	23.6	49.2	2656.39	1.84
4	17.58	29.96	2549.39	1	25.23	45.64	2416.96	2.11
5	18.07	29.79	2233.55	1.05	25.38	43.38	2151.51	2.26
6	18.08	29.12	1918.8	1.06	25.22	41.44	1873.77	2.34
7	17.8	28.16	1613.27	1.04	24.87	40.1	1594.22	2.36
8	17.32	26.95	1320.88	1.04	24.35	38.49	1319.39	2.35
9	16.71	25.51	1044	1.04	23.75	36.63	1054	2.35
10	15.88	23.72	785.31	1.02	22.98	34.06	802.02	2.25
11	15.2	21.78	548.25	1.02	21.81	30.72	567.16	2.15
12	12.78	18.78	336.93	0.98	19.87	26.51	353.19	2.03
13	17.62	20.34	150.32	0.95	23.51	25.93	158.83	1.88

Table 4-9: Shear force, bending moment, axial force and torsion in an interior column for RM, M 3-4 and strengthened models with shear walls and diagrids.

Story	RM, Regular Model				M 3-4, Torsionally irr Model			
	V2 KN	M3 KNm	P KN	T KNm	V2 KN	M3 KNm	P KN	T KNm
1	24.68	60.14	1947.89	0.35	40.76	99.91	1980.08	2.42
2	25.98	44.22	1798.89	0.63	36.51	67.31	1829.84	4.8
3	25.03	37.67	1648.96	0.68	40.98	69.19	1678.82	5.45
4	23.8	34.46	1499.33	0.67	43.23	70.18	1527.24	5.5
5	22.43	32.03	1349.61	0.63	44.18	70.43	1375.21	5.33
6	21.03	29.79	1199.82	0.59	44.43	70.43	1222.86	5.09
7	19.57	27.44	1049.99	0.55	44.2	70.17	1070.27	4.81
8	18	24.84	900.12	0.5	43.84	69.92	917.54	4.53
9	16.25	21.94	750.23	0.45	42.79	68.61	764.73	4.24
10	14.28	18.57	600.34	0.39	41	66.18	611.91	3.91
11	11.9	16.46	450.43	0.32	39.01	62.86	459.09	3.47
12	8.83	13.43	300.49	0.24	34.87	59.15	306.3	2.91
13	5.52	9.87	150.62	0.15	42.6	53.62	153.68	2.28

Table 4-10: The table continued from Table 4-9.

Story	M 3-4, Strengthened by Shear wall				M 3-4, Strengthened by diagrid			
	V_2 KN	M_3 KNm	P KN	T KNm	V_2 KN	M_3 KNm	P KN	T KNm
1	9.25	25.74	1976.79	9.29	36.91	76.22	1975.8	1.16
2	15.61	30.48	1825.95	0.65	28.85	56.29	1825.75	1.39
3	18.81	32.25	1674.52	0.89	34.31	59.57	1675.03	1.84
4	21.4	34.7	1522.69	1	38.1	62.13	1523.93	2.11
5	23.18	36.38	1370.6	1.05	41.05	64.86	1372.42	2.26
6	24.28	37.29	1218.36	1.06	42.66	65.98	1220.58	2.33
7	24.87	37.63	1066.05	1.04	43.65	66.62	1068.49	2.36
8	25.05	37.43	913.74	1.04	43.98	66.35	916.2	2.35
9	24.87	36.72	761.48	1.04	43.83	65.35	763.78	2.31
10	24.78	35.45	609.32	1.04	42.99	65.37	611.024	2.24
11	23.65	33.89	457.28	1.02	41.89	63.3	458.66	2.15
12	20.92	30.78	305.29	0.99	37.63	60.49	306.03	2.03
13	28.92	43.4	153.81	0.95	49.11	55.26	153.52	1.88

Table 4-11: Shear force, bending moment and torsion in an interior beam for M 3-4 and strengthened models with shear walls and diagrids

Story	M 3-4, Torsionally irregular Model			M 3-4, Strengthened by Shear wall			M 3-4, Strengthened by diagrid		
	V_2 KN	M_3 KNm	T, KNm	V_2 KN	M_3 KNm	T KNm	V_2 KN	M_3 KNm	T KNm
1	60.59	35.58	3.66	34.54	13.4	2.81	43.69	19.72	2.99
2	69.46	48.69	5.08	39.07	14	4.35	37.44	17.4	3.5
3	72.32	53	5.7	41.9	16	5.32	41.42	19.96	4.24
4	72.59	53.6	5.99	43.62	17.44	5.81	43.95	22.52	4.79
5	71.76	52.62	6.1	44.6	18.33	6	45.78	25.54	5.12
6	70.51	50.98	6.03	45.05	18.84	6.01	47	28.04	5.3
7	68.94	48.87	5.85	45.14	19.07	5.92	47.72	29.6	5.35
8	66.95	46.14	5.59	44.94	18.85	5.73	48.04	30.58	5.34
9	64.41	42.58	5.24	44.46	18.45	5.46	48.02	30.94	5.28
10	61.25	38.13	4.82	43.74	17.76	5.12	47.76	30.91	5.16
11	57.33	32.57	4.32	42.66	17.49	4.72	47.13	30.12	4.96
12	53.35	26.88	3.94	42.09	17.46	4.48	47.34	31.12	4.97
13	45.04	19.24	2.78	38.11	14.51	3.26	42.21	22.13	3.72

CHAPTER 5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Observations and Conclusions

In this study, the parametric investigation has been performed on three groups of structures (18 models) with L-shaped plan asymmetry. The analysis was carried out in order to evaluate the torsional behavior of irregular structure on the basis of different parameters: vibration period and mode, story drift ratio, lateral displacements, torsional irregularity ratio according to submitted national codes, torsional diaphragm rotation, torsional moment and base shear. Also, the analysis was performed for the model with equivalent shear wall and diagrid system those were recommended as practical solutions for plan asymmetry effects. Based on the investigation the following observations and conclusions are derived:

1) The most plan irregular models, Type-III which was stiffened with shear wall lift core at a corner, had the smaller fundamental natural period than their corresponding models in Type-I and II. Models Type-II which is more irregular than Type-I had a slightly smaller fundamental natural period than their corresponding models in Type-I. It can be concluded that buildings with smaller translational natural period attract higher design seismic force coefficient. From the modal property, it can also be concluded that for regular and nearly regular models the first two modes can be pure translational and third mode pure torsional; with increasing of irregularity it will become mixed translational and torsional which is undesirable behavior.
2) Model Type-III had the maximum tendency for torsional effects with the higher value of eccentricity. Model Type-II had the second higher value of eccentricity.
3) The highest torsional irregularity was found in model Type-III which has shear wall lift core at one corner, which implied irregularity introduced to regular model with the gradual reduction in plan area and shear wall lift core instead of stair hole (in Type-II model) promoted to larger torsional effects.
4) The story drift and lateral displacement responses increased as the building plan asymmetry increases from RM model to M 1-5, M 2-6 and M 3-6 of Type-I, II, and III respectively. However, Type-III models were seen to exhibit abrupt changes from RM in story drifts and lateral displacement in the direction perpendicular to

EQ excitation due to lateral torsional coupling; thus, it can be concluded that with the increase in plan irregularity, the lateral torsional coupling effect increases which generates greater damage to the building.

5) It can be concluded as the increase of irregularity has a considerable effect on the torsional rotation; thus, the torsional diaphragm rotation increases with the increase of eccentricity due to plan irregularity.
6) The total base shear significantly increased with increasing in eccentricity between the center of mass and the center of rigidity. Regular model displayed no shear demand in perpendicular to EQ direction which contributes to total value whereas for the torsionally irregular building models, the abrupt change in values was observed. It can be concluded that with the development of lateral –torsional coupling effect due to plan asymmetry, the additional shear force developed in the perpendicular direction to earthquake direction could violate the safe design for resisting elements.
7) Two ways in order to reduce the torsional irregularity effects resulted from plan asymmetry were recommended as practical solutions; those were an equivalent shear wall and diagrid system. From the study, those systems had provided considerable reduction in responses demand such as an eccentricity, torsional irregularity ratio, story displacement, bending moment, shear force, axial force, and torsion.

5.2 Recommendations

The present study demonstrates that plan asymmetry has a significant effect on the response of buildings compared to the plan symmetric building. The dynamic response of the structure will be influenced significantly by the distribution of the lateral forces because of the torsion action which is the most critical factor leading to major damage or complete collapse of buildings. From the results of this study, the flowing recommendations are put forward:

1. Equivalent shear wall and diagrid system had provided considerable reduction in torsional irregularity effect. Based on such results, these ways in plan asymmetric buildings are suggested.
2. Although the seismic codes have provision for torsion, it is essential that irregular buildings should be carefully analyzed for torsion and the designer should introduce a structural system to make a torsionally balanced system or avoid these types of structures as much as possible.
3. The group of people involved in constructing the building facilities, including owner, architect, structural engineer, contractor and local authorities, contribute to the overall planning, selection of structural system, and to its configuration to avoid or minimize the torsion effect which is the most critical factor leading to major damage or complete collapse of buildings.
4. The structural engineer needs to have a thorough understanding of the torsional behavior of plan asymmetric structures.

5.3 Future Research Requirements

This study may not be sufficient for the complete understanding of the torsional behavior of plan asymmetric reinforced concrete multi-story building despite provides important information for torsion behavior. Therefore, further investigation should be done using non-linear pushover analysis or non-linear time history analysis considering different irregularities.

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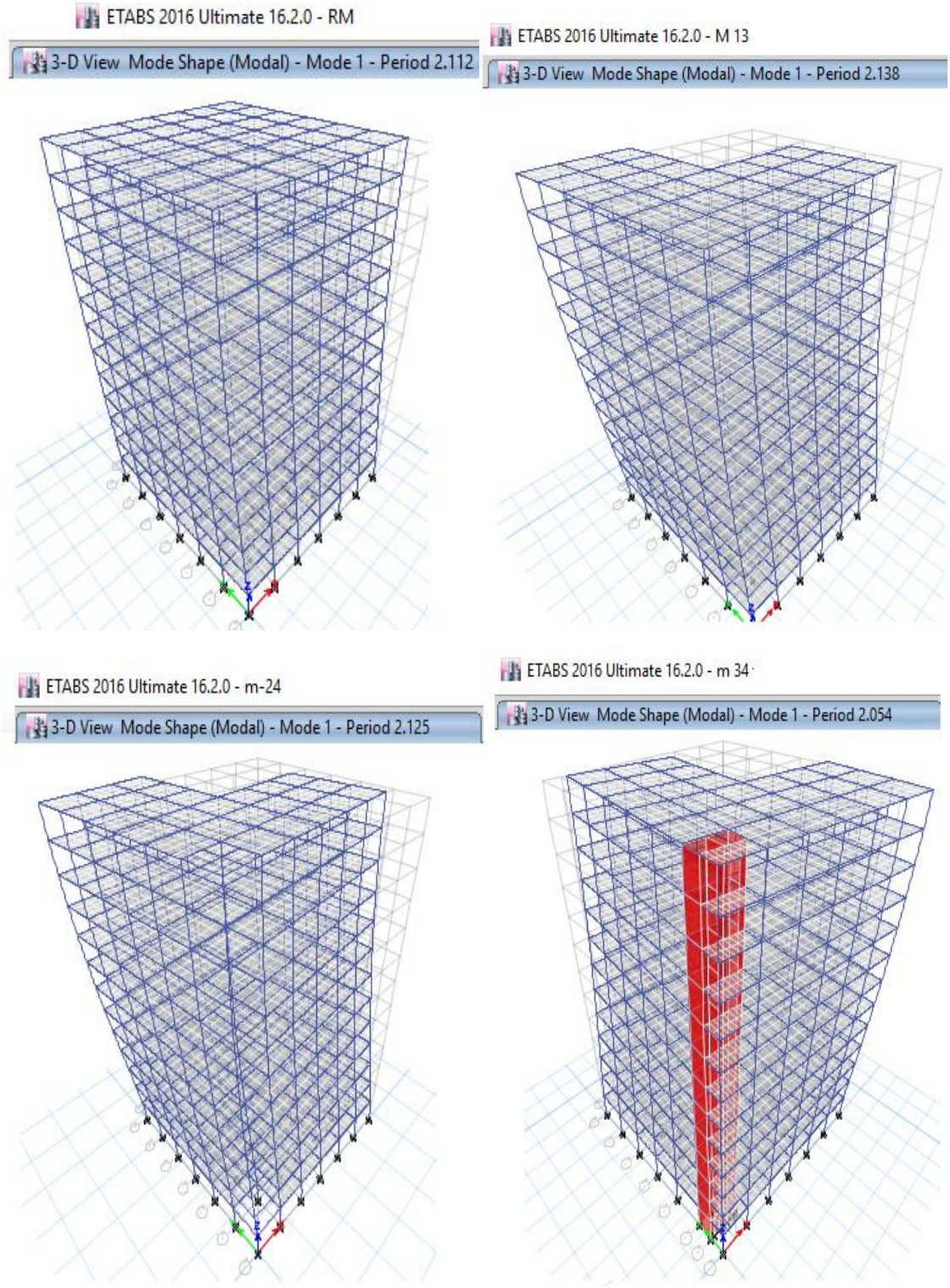
APPENDIX

APPENDIX A-FUNDAMENTAL MODE SHAPE OF REPRESENTATIVE MODELS



APPENDIX B- CALCULATION OF TORSIONAL RADIUS

Model	Ux (Fx=1000) mm	Uy (Fy=1000) mm	Rz (Mz=1000) rad	Kx= Fx/Ux KN/m	Ky= Fy/Uy KN/m	Km=Mz/Rz KNm/rad	rx= sqrt(Km/Ky) m	ry= sqrt(Km/Kx) m
RM	26.49	26.49	0.00	37748.67	37748.67	4854368.93	11.34	11.34
M 1-1	27.16	27.15	0.00	36820.21	36833.77	4587155.96	11.16	11.16
M 1-2	29.19	29.19	0.00	34259.48	34259.48	4016064.26	10.83	10.83
M 1-3	33.31	33.31	0.00	30022.82	30022.82	3344481.61	10.55	10.55
M 1-4	44.58	44.58	0.00	22433.09	22433.09	2564102.56	10.69	10.69
M 1-5	76.59	76.59	0.00	13057.39	13057.39	1562500.00	10.94	10.94

Model	Ux (Fx=1000) mm	Uy (Fy=1000) mm	Rz (Mz=1000) rad	Kx=Fx/Ux KN/m	Ky=Fy/Uy KN/m	Km=Mz/Rz KNm/rad	rx= sqrt(Km/Ky) m	ry= sqrt(Km/Kx) m
M 2-1	26.58	26.58	0.00	37616.61	37616.61	4830917.87	11.33	11.33
M 2-2	27.25	27.25	0.00	36699.94	36699.94	4566210.05	11.15	11.15
M 2-3	29.49	29.49	0.00	33910.95	33910.95	4000000.00	10.86	10.86
M 2-4	34.20	34.20	0.00	29244.04	29240.62	3322259.14	10.66	10.66
M 2-5	43.90	43.90	0.00	22780.60	22780.60	2551020.41	10.58	10.58
M 2-6	71.96	71.97	0.00	13897.38	13895.64	1557632.40	10.59	10.59

Model	Ux (Fx=1000) mm	Uy (Fy=1000) mm	Rz (Mz=1000) rad	Kx= Fx/Ux KN/m	Ky= Fy/Uy KN/m	Km= Mz/Rz KNm/rad	rx= sqrt(Km/Ky) m	ry= sqrt(Km/Kx) m
M 3-1	0.15	0.24	0.00	6711409.40	4255319.15	500000000.00	10.84	8.63
M 3-2	0.15	0.20	0.00	6711409.40	4975124.38	500000000.00	10.02	8.63
M 3-3	0.15	0.21	0.00	6578947.37	4854368.93	500000000.00	10.15	8.72
M 3-4	0.16	0.22	0.00	6329113.92	4608294.93	333333333.33	8.50	7.26
M 3-5	0.17	0.23	0.00	5988023.95	4273504.27	250000000.00	7.65	6.46
M 3-6	0.18	0.26	0.00	5524861.88	3816793.89	200000000.00	7.24	6.02

APPENDIX C- TORSIONAL IRREGULARITY CHECK

Torsional irregularity check as per EBCS EN 1998 2015.

Model		Direction X			Direction Y		
		r_x	l_s	e_{ox}, m	r_y	l_s	e_{oy}, m
	RM	11.34	9.80	0.00	11.34	9.80	0.00
I	M 1-1	11.16	9.63	0.00	11.16	9.63	0.00
	M 1-2	10.83	9.42	0.06	10.83	9.42	0.07
	M 1-3	10.55	9.38	0.55	10.55	9.38	0.23
	M 1-4	10.69	9.62	0.55	10.69	9.62	0.58
	M 1-5	10.94	10.35	0.23	10.94	10.35	0.77
II	M 2-1	11.33	9.93	0.23	11.33	9.93	0.24
	M 2-2	11.15	9.77	0.30	11.15	9.77	0.23
	M 2-3	10.86	9.56	0.46	10.86	9.56	0.30
	M 2-4	10.66	9.55	0.83	10.66	9.55	0.48
	M 2-5	10.58	9.87	1.86	10.58	9.87	0.86
	M 2-6	10.59	10.84	1.91	10.59	10.84	1.91
III	M 3-1	10.84	13.06	6.25	8.63	13.06	5.92
	M 3-2	10.02	12.94	6.02	8.63	12.94	5.69
	M 3-3	10.15	13.44	5.70	8.72	13.44	5.33
	M 3-4	8.50	11.90	5.27	7.26	11.90	4.84
	M 3-5	7.65	11.77	4.74	6.46	11.77	4.24
	M 3-6	7.24	12.06	4.08	6.02	12.06	3.49

Torsional irregularity ratio as per ASCE 7-05 for models

Model		d_1 mm	d_2 mm	d_{avg}	d_{max}	d_{max}/d_{avg}
	RM	76.94	76.94	76.94	76.94	1.00
I	M 1-1	87.14	86.55	86.85	87.14	1.00
	M 1-2	89.42	85.63	87.53	89.42	1.02
	M 1-3	93.76	83.38	88.57	93.76	1.06
	M 1-4	101.38	79.55	90.47	101.38	1.12
	M 1-5	187.79	125.63	156.71	187.79	1.20
II	M2-1	88.23	84.52	86.38	88.23	1.02
	M2-2	88.72	84.37	86.55	88.72	1.03
	M2-3	91.28	83.40	87.34	91.28	1.05
	M2-4	95.66	81.12	88.39	95.66	1.08
	M2-5	102.61	77.29	89.95	102.61	1.14
	M2-6	185.02	121.57	153.30	185.02	1.21
III	M 3-1	8.06	2.52	5.29	8.06	1.52
	M 3-2	7.93	2.50	5.22	7.93	1.52
	M 3-3	8.00	2.48	5.24	8.00	1.53
	M 3-4	7.86	2.44	5.15	7.86	1.53
	M 3-5	7.32	2.31	4.82	7.32	1.52

	M 3-6	6.17	1.92	4.05	6.17	1.53

APPENDIX D- MODAL PARTICIPATING MASS RATIOS

TABLE: Modal Participating Mass Ratios RM

Case	Mode	Period	UX	UY	UZ	Sum UX	Sum UY	Sum UZ	RX	RY	RZ	Sum RX	Sum RY	Sum RZ
		sec												
Modal	1	2.112	0.0001	0.8003	0	0.0001	0.8003	0	0.2037	1.80E-05	0	0.2037	1.80E-05	0
Modal	2	2.112	0.8003	0.0001	0	0.8003	0.8003	0	1.80E-05	0.2037	0	0.2037	0.2037	0
Modal	3	1.922	0	0	0	0.8003	0.8003	0	0	0	0.8012	0.2037	0.2037	0.8012
Modal	4	0.673	8.43E-06	0.0958	0	0.8004	0.8961	0	0.5246	4.62E-05	0	0.7283	0.2038	0.8012
Modal	5	0.673	0.0958	8.43E-06	0	0.8961	0.8961	0	4.62E-05	0.5246	0	0.7284	0.7284	0.8012
Modal	6	0.614	0	0	0	0.8961	0.8961	0	0	0	0.0945	0.7284	0.7284	0.8957
Modal	7	0.382	2.62E-06	0.037	0	0.8961	0.9331	0	0.0621	4.39E-06	0	0.7904	0.7284	0.8957
Modal	8	0.382	0.037	2.62E-06	0	0.9331	0.9331	0	4.39E-06	0.0621	0	0.7904	0.7904	0.8957
Modal	9	0.35	0	0	0	0.9331	0.9331	0	0	0	0.0371	0.7904	0.7904	0.9327
Modal	10	0.256	6.28E-06	0.0207	0	0.9331	0.9539	0	0.0775	2.35E-05	0	0.8679	0.7904	0.9327
Modal	11	0.256	0.0207	6.28E-06	0	0.9539	0.9539	0	2.35E-05	0.0775	0	0.868	0.868	0.9327
Modal	12	0.234	0	0	0	0.9539	0.9539	0	0	0	0.0208	0.868	0.868	0.9536
Modal	13	0.186	0.0002	0.0134	0	0.9541	0.9673	0	0.0316	0.0004	0	0.8996	0.8684	0.9536
Modal	14	0.186	0.0134	0.0002	0	0.9675	0.9675	0	0.0004	0.0316	0	0.9	0.9	0.9536
Modal	15	0.17	0	0	0	0.9675	0.9675	0	0	0	0.0137	0.9	0.9	0.9672
Modal	16	0.141	5.26E-07	0.0097	0	0.9675	0.9772	0	0.0325	1.76E-06	0	0.9325	0.9	0.9672
Modal	16	0.116	0	0.0103	0	0.9653	0.9756	0	0.0334	0	0	0.9286	0.8952	0.9651

TABLE: Modal Participating Mass Ratios M 1-5

Case	Mode	Period	UX	UY	UZ	Sum UX	Sum UY	Sum UZ	RX	RY	RZ	Sum RX	Sum RY	Sum RZ
		sec												
Modal	1	2.221	0.203	0.203	0	0.203	0.203	0	0.0595	0.0595	0.3731	0.0595	0.0595	0.3731
Modal	2	2.035	0.3955	0.3955	0	0.5984	0.5984	0	0.1093	0.1093	0	0.1687	0.1687	0.3731
Modal	3	1.871	0.1924	0.1924	0	0.7908	0.7908	0	0.05	0.05	0.413	0.2187	0.2187	0.7861
Modal	4	0.688	0.0286	0.0286	0	0.8194	0.8194	0	0.132	0.132	0.0502	0.3507	0.3507	0.8363
Modal	5	0.642	0.0503	0.0503	0	0.8696	0.8696	0	0.2515	0.2515	0	0.6022	0.6022	0.8363
Modal	6	0.596	0.0218	0.0218	0	0.8914	0.8914	0	0.1193	0.1193	0.0528	0.7215	0.7215	0.8891
Modal	7	0.373	0.0118	0.0118	0	0.9031	0.9031	0	0.0202	0.0202	0.0161	0.7417	0.7417	0.9051
Modal	8	0.357	0.0191	0.0191	0	0.9223	0.9223	0	0.0323	0.0323	0	0.774	0.774	0.9051
Modal	9	0.336	0.0074	0.0074	0	0.9297	0.9297	0	0.0121	0.0121	0.0229	0.7861	0.7861	0.928
Modal	10	0.243	0.0074	0.0074	0	0.9371	0.9371	0	0.0254	0.0254	0.0078	0.8115	0.8115	0.9358
Modal	11	0.236	0.0109	0.0109	0	0.9479	0.9479	0	0.0383	0.0383	0	0.8498	0.8498	0.9358
Modal	12	0.224	0.0034	0.0034	0	0.9514	0.9514	0	0.0129	0.0129	0.0145	0.8627	0.8627	0.9502
Modal	13	0.172	0.0056	0.0056	0	0.9569	0.9569	0	0.0132	0.0132	0.0037	0.8759	0.8759	0.9539

Modal	14	0.169	0.0071	0.0071	0	0.9641	0.9641	0	0.0169	0.0169	0	0.8928	0.8928	0.9539
Modal	15	0.161	0.0016	0.0016	0	0.9657	0.9657	0	0.0037	0.0037	0.0109	0.8965	0.8965	0.9648
Modal	16	0.128	0.0045	0.0045	0	0.9701	0.9701	0	0.0143	0.0143	0.0015	0.9108	0.9108	0.9663

TABLE: Modal Participating Mass Ratios M 2-6

Case	Mode	Period sec	UX	UY	UZ	Sum UX	Sum UY	Sum UZ	RX	RY	RZ	Sum RX	Sum RY	Sum RZ
Modal	1	2.219	0.1998	0.1998	0	0.1998	0.1998	0	0.0592	0.0592	0.3796	0.0592	0.0592	0.38
Modal	2	1.987	0.3954	0.3954	0	0.5952	0.5952	0	0.1098	0.1098	0	0.1689	0.1689	0.38
Modal	3	1.802	0.1953	0.1953	0	0.7905	0.7905	0	0.051	0.051	0.4068	0.2199	0.2199	0.7864
Modal	4	0.687	0.0278	0.0278	0	0.8183	0.8183	0	0.1276	0.1276	0.0518	0.3475	0.3475	0.8381
Modal	5	0.627	0.0503	0.0503	0	0.8686	0.8686	0	0.2511	0.2511	0	0.5985	0.5985	0.8381
Modal	6	0.574	0.0226	0.0226	0	0.8912	0.8912	0	0.123	0.123	0.0509	0.7215	0.7215	0.8891
Modal	7	0.373	0.011	0.011	0	0.9021	0.9021	0	0.0189	0.0189	0.0177	0.7404	0.7404	0.9068
Modal	8	0.349	0.0191	0.0191	0	0.9213	0.9213	0	0.0322	0.0322	0	0.7726	0.7726	0.9068
Modal	9	0.324	0.0082	0.0082	0	0.9295	0.9295	0	0.0134	0.0134	0.0212	0.786	0.786	0.928
Modal	10	0.243	0.0067	0.0067	0	0.9362	0.9362	0	0.0226	0.0226	0.0094	0.8086	0.8086	0.9374
Modal	11	0.231	0.0109	0.0109	0	0.947	0.947	0	0.0383	0.0383	0	0.8469	0.8469	0.9374
Modal	12	0.216	0.0042	0.0042	0	0.9513	0.9513	0	0.0157	0.0157	0.0128	0.8626	0.8626	0.9502
Modal	13	0.171	0.0047	0.0047	0	0.956	0.956	0	0.0113	0.0113	0.0054	0.8739	0.8739	0.9556
Modal	14	0.165	0.0071	0.0071	0	0.9631	0.9631	0	0.0169	0.0169	0	0.8908	0.8908	0.9556
Modal	15	0.155	0.0024	0.0024	0	0.9656	0.9656	0	0.0056	0.0056	0.0092	0.8964	0.8964	0.9648
Modal	16	0.127	0.0037	0.0037	0	0.9693	0.9693	0	0.0116	0.0116	0.0033	0.908	0.908	0.9681

TABLE: Modal Participating Mass Ratios M 3-4

Case	Mode	Period sec	UX	UY	UZ	Sum UX	Sum UY	Sum UZ	RX	RY	RZ	Sum RX	Sum RY	Sum RZ
Modal	1	2.054	0.1334	0.3242	0	0.1334	0.3242	0	0.0924	0.0359	0.3358	0.0924	0.0359	0.34
Modal	2	1.648	0.2885	0.372	0	0.4219	0.6962	0	0.1448	0.1106	0.0737	0.2372	0.1465	0.41
Modal	3	1.387	0.3036	0.0555	0	0.7255	0.7517	0	0.0235	0.1425	0.3446	0.2607	0.289	0.7541
Modal	4	0.65	0.018	0.0396	0	0.7435	0.7912	0	0.1849	0.0866	0.0408	0.4456	0.3757	0.7949
Modal	5	0.47	0.0432	0.0704	0	0.7867	0.8617	0	0.2168	0.1206	0.0148	0.6624	0.4963	0.8097
Modal	6	0.365	0.0335	0.022	0	0.8202	0.8837	0	0.0419	0.0681	0.0006	0.7043	0.5644	0.8103
Modal	7	0.363	0.0559	0.0009	0	0.8761	0.8846	0	0.0036	0.1243	0.0768	0.7079	0.6887	0.887
Modal	8	0.243	0.0057	0.0039	0	0.8818	0.8884	0	0.0165	0.0186	1.30E-02	0.7244	0.7073	0.9
Modal	9	0.23	0.0129	0.0358	0	0.8947	0.9243	0	0.068	0.0238	0.0043	0.7924	0.7311	0.9043
Modal	10	0.175	0.0105	0.0064	0	0.9052	0.9307	0	0.0144	0.0236	0.0012	0.8068	0.7547	0.9056
Modal	11	0.168	0.0233	0.0008	0	0.9285	0.9316	0	0.0012	0.0454	2.88E-02	0.808	0.8001	0.9343
Modal	12	0.14	8.40E-03	0.0183	0	0.9369	0.9499	0	0.0496	0.0238	0.0032	0.8575	0.824	0.9375
Modal	13	0.131	0.0018	0.0039	0	0.9387	0.9537	0	0.0123	0.0061	0.0046	0.8698	0.83	0.9422
Modal	14	0.104	0.0148	0.0039	0	0.9535	0.9576	0	0.0104	0.0388	5.00E-03	0.8802	0.8688	0.9472
Modal	15	0.103	7.00E-04	0.0004	0	0.9541	0.9581	0	0.0012	1.50E-03	0.0123	0.8814	0.8704	0.9595
Modal	16	0.097	0.0059	0.011	0	0.96	0.9691	0	0.0281	0.0152	1.20E-03	0.9095	0.8856	0.9607

APPENDIX E- DAMAGE LIMITATION LIMIT STATES

For buildings having non-structural elements fixed in a way not to interfere with structural deformations.

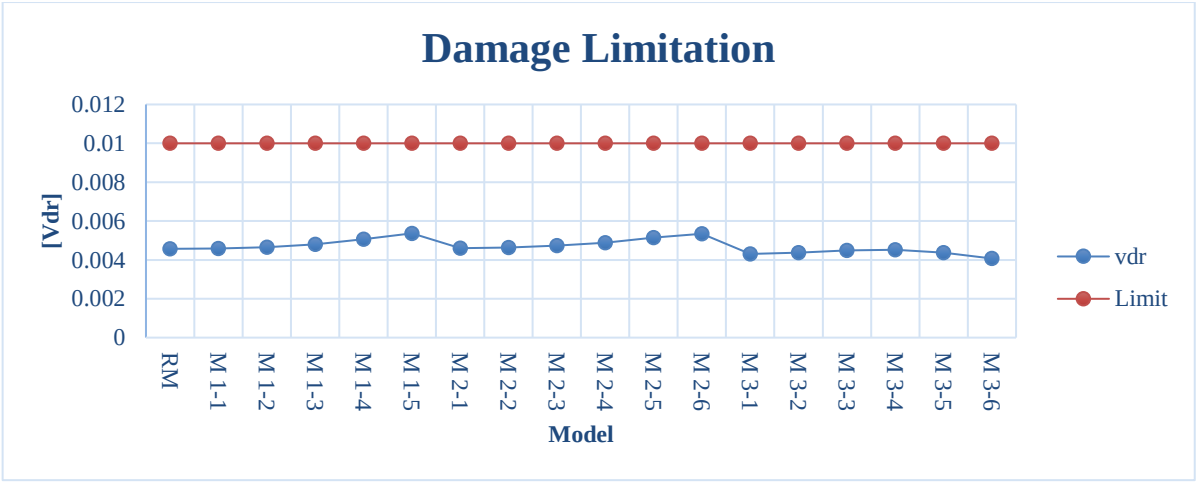
drv ≤ 0.010 h

where *h* is the story height *v* is the reduction factor to consider lower return period of EQs. The recommended values of v are 0.4 for importance classes III and IV and v = 0.5 for importance classes I and II.

*dr= de * qd* design inter-story drift

where q_d is behavior factor defined in section 3.3.

Type	Model	de	qd	dr	v	vdr	Limit	Remark
	RM	0.0029	3.90	0.0114	0.4	0.0046	0.01	OK
I	M 1-1	0.0033	3.45	0.0115	0.4	0.0046	0.01	OK
	M 1-2	0.0034	3.45	0.0116	0.4	0.0047	0.01	OK
	M 1-3	0.0035	3.45	0.0120	0.4	0.0048	0.01	OK
	M 1-4	0.0037	3.45	0.0127	0.4	0.0051	0.01	OK
	M 1-5	0.0067	2.00	0.0134	0.4	0.0054	0.01	OK
II	M 2-1	0.0033	3.45	0.0115	0.4	0.0046	0.01	OK
	M 2-2	0.0034	3.45	0.0116	0.4	0.0046	0.01	OK
	M 2-3	0.0034	3.45	0.0118	0.4	0.0047	0.01	OK
	M 2-4	0.0035	3.45	0.0122	0.4	0.0049	0.01	OK
	M 2-5	0.0037	3.45	0.0129	0.4	0.0051	0.01	OK
	M 2-6	0.0067	2.00	0.0134	0.4	0.0053	0.01	OK
III	M 3-1	0.0054	2.00	0.0108	0.4	0.0043	0.01	OK
	M 3-2	0.0055	2.00	0.0109	0.4	0.0044	0.01	OK
	M 3-3	0.0056	2.00	0.0112	0.4	0.0045	0.01	OK
	M 3-4	0.0056	2.00	0.0113	0.4	0.0045	0.01	OK
	M 3-5	0.0055	2.00	0.0109	0.4	0.0044	0.01	OK
	M 3-6	0.0051	2.00	0.0102	0.4	0.0041	0.01	OK


Damage Limitation
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0.01
0.008
0.006
0.004
0.002
0
[Vdr]
RM
M 1-1
M 1-2
M 1-3
M 1-4
M 1-5
M 2-1
M 2-2
M 2-3
M 2-4
M 2-5
M 2-6
M 3-1
M 3-2
M 3-3
M 3-4
M 3-5
M 3-6
Model
vdr
Limit