TORSIONAL RESPONSE OF ASYMMETRIC BUILDINGS UNDER EARTHQUAKE LOADS

Thesis

Submitted in partial fulfillment of the requirements for the degree of

DOCTOR OF PHILOSOPHY

by

ARCHANA J. SATHEESH

DEPARTMENT OF CIVIL ENGINEERING

NATIONAL INSTITUTE OF TECHNOLOGY KARNATAKA

SURATHKAL, MANGALORE -575 025

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DECLARATION

By the Ph.D Scholar

I hereby declare that the Research Thesis entitled **“Torsional Response of Asymmtric Buildings under Earthquake Loads”** which is being submitted to the National Institute of Technology Karnataka, Surathkal in partial fulfillment of the requirements for the award of the degree of **Doctor of Philosophy** in **Civil Engineering**, *is a bona fide report of the research work carried out by me*. The material contained in this Research Thesis has not been submitted to any University or Institution for the award of any degree.

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Place: NITK Surathkal

Date: 29-10-2019

CERTIFICATE

This is to certify that the Research Thesis entitled **"Torsional Response of Asymmetric Buildings under Earthquake Loads"** *submitted by* **Archana J. Satheesh** (Register Number: **148048CV14F13**) as the record of research work carried out by her, is accepted as the Research Thesis submission in partial fulfillment of the requirements for the award of the degree of **Doctor of Philosophy.**

Prof. B.R. Jayalekshmi	Prof. Katta Venkataramana
Research Guide	Research Guide
(Signature with date)	(Signature with date)

Chairman-DRPC

(Signature with date and seal)

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Archana J. Satheesh

ABSTRACT

Irregular buildings constitute a major fraction of the urban infrastructure due to various occupational and architectural demands.Most buildings are irregular to varying degrees due to asymmetry in plan, elevation, distribution of vertical members or mass distribution on the floors. Perfectly regular buildings are more of an idealized concept and in practice this condition is rarely satisfied. Under seismic loading, the presence of structural irregularity in buildings leads to large displacement amplifications and stress concentrations in the members which lead to their severe damage and ultimately, early collapse. The presence of irregularities in mass, stiffness, strength or geometry along the elevation of the building is categorized as vertical irregularity. Torsional irregularity or in-plan irregularity can be considered to exist if the building possesses non-concurrency in the lines of action of centers of mass and stiffness on a common vertical axis at each floor level. During earthquakes or any other lateral loads, the inertia force acts through the center of mass and resistive force through the center of stiffness or resistance. If an in-plan eccentricity is present, a time varying twisting moment is generated causing torsional vibration.

The proposed study investigates the effect of in-plan irregularity and vertical irregularity on the seismic response of buildings. Irregularities in mass and stiffness along the height of the buildings in combination with torsional irregularities along the plan of the buildings are evaluated. Transient analysis is carried out to analyse the seismic response of the shear wall buildings, mass irregular buildings and stiffness irregular buildings with in-plan eccentricity using LS-DYNA software. The responses of the irregular buildings and the effect of in-plan eccentricity in terms of variation in natural period, base shear, storey drifts, roof deflection, torsional resultant and roof rotations obtained from the analysis due to asymmetry have been studied in detail.

Based on the seismic responses of the irregular buildings, equations and irregularity coefficients are proposed to quantify and compare buildings with of vertical and torsional irregularity in combination. It is also attempted to suggest modification for the approximate natural period expression given in the IS 1893:2016 and ASCE 7-16 to incorporate the in-plan eccentricity and evaluate the natural period of irregular buildings. Also, the effects of frequency content of ground motions on the seismic responses of the irregular buildings are also evaluated. It is observed that the presence of in-plan eccentricity if present singly or in combination with any other irregularities, determines the overall seismic behavior of a building and tends to modify it's response.

Keywords: Torsion, Mass irregularity, Stiffness irregularity, Dynamic analysis

NOMENCLATURE

Static eccentricity	e_s
Dynamic eccentricity	e_d
Floor plan dimensions perpendicular to ground motion	b
Plan width	L
Maximum displacement at a particular floor level	δ_{max}
Average of the displacements at the extreme points of the same floor level	δ_{avg}
Minimum displacement at a particular floor level	δ_{min}
Torsional amplification factor	A_x
Torsion irregularity coefficient	η_t
Modified lateral stiffness of a storey	K_m
Initial lateral stiffness of a storey	K_o
Nominal height of storey	h_o
Modified storey height	h_m
Frequency ratio	Ω
Seismic weight of a building	W
Height of a building	H
In-plan eccentricity ratio	e_d/L
Proposed mass irregularity coefficient	α
Mass ratio	M_{ri}
Natural period of irregular building	T_i
Natural period of regular building	T_r
Base shear ratio of irregular building	B_i
Base shear ratio of regular building	B_r
Proposed stiffness irregularity coefficient	β
Stiffness modification ratio	S_{ri}

Fraction of the height over which the irregularity is considered	R_i
Predicted base shear ratio of irregular building	B_{ip}
Predicted natural period of irregular building	T_{ip}
Approximate natural period as per IS 1893:2016	$T_{a\ IS}$
Approximate natural period as per ASCE 7-16	$T_{a\ ASCE}$
Predicted natural period of regular building	T_{rp}
Proposed modification factor for natural period as per IS 1893:2016	γ_{IS}
Proposed modification factor for natural period as per ASCE 7-16	γ_{ASCE}
Predicted natural period of irregular building as per IS 1893:2016	$T_{ip\ IS}$
Predicted natural period of irregular building as per ASCE 7-16	$T_{ip\ ASCE}$

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CHAPTER 1

INTRODUCTION

A regular building generally happens to be an idealized concept since real buildings have numerous discrepancies or variations in mass, stiffness or strength distributions along the height or the planar directions. Multi-storey buildings with complicated geometry and structural systems are common due to various possibilities offered by advanced construction methods. Further buildings constructed are also inherently irregular in nature due to various constraints like the implementation of various architectural schemes, space constraints, functional demands so on and so forth. Various seismic damage surveys and analyses conducted on modes of failure of building structures during past severe earthquakes have concluded that asymmetric buildings are the most vulnerable building structures. The structural configurations of modern asymmetric buildings possess very complex lateral load paths. A building when subjected to lateral loads like wind or earthquakes undergoes damage which is generally initiated at the location of the structural weak planes in the building systems. These weaknesses originate due to the presence of any kind of structural irregularities in stiffness, strength or mass and cause further structural deterioration leading to the collapse of the building. For a building to be classified as symmetric it must possess, a coincident centre of mass and centre of stiffness lying on a common vertical axis, at each floor level. This criterion is rarely achieved in reality and most buildings are unsymmetrical to different extents along the plan, elevation and orientation of structural members or mass distribution on the floors.

Major seismic codes classify the structural irregularities into irregularities in plan and elevation, but quite often structural irregularity is present in buildings as a combination. The presence of irregularities in mass, stiffness, strength or geometry along the elevation of the building is categorized as vertical irregularity whereas, plan irregularities or horizontal irregularities refer to similar discrepancies in the plan of the building. This chapter briefly introduces the essence of the present study and attempts to highlight the classification of irregularities as per several codes. The details of the types of irregularities considered in the study are also explained.

1.1. Horizontal irregularities

Effects of seismic loads are the most important aspects to be considered while carrying out the design of structures. The performance of an irregular building under the influence of earthquakes is based on the action of different loads acting along the horizontal and vertical planes of the building. The presence of irregularities tends to change the loading path as well as the ductility demand of the building. The classification of horizontal irregularities in buildings as per most of the seismic codes is as given in Figure 1.1.

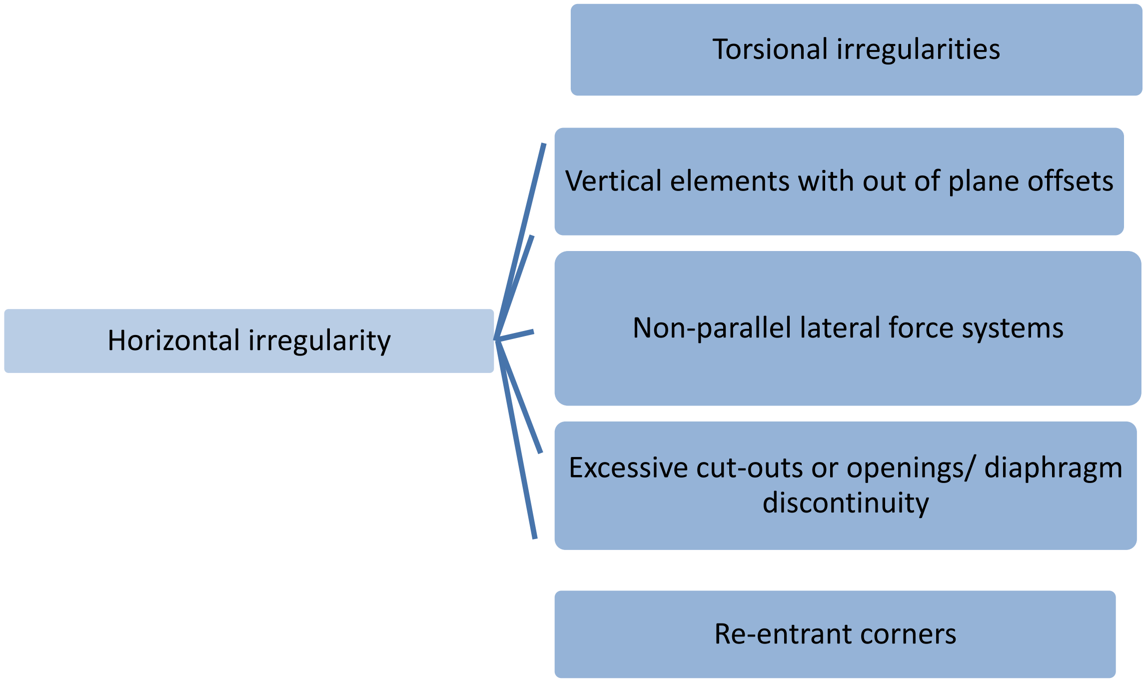


Figure 1.1 Classifications of horizontal irregularities

The most critical and significant among the horizontal irregularities is the torsional irregularity. Torsional responses in structures originate due to eccentricity in the mass and stiffness distributions, causing a torsional response coupled with translation response or due to accidental causes as a result of differences in coupling of the base of the building with that of the ground level or wave propagation effects. The lateral-torsional coupling as a result of eccentricity or difference between centre of mass (CM) and centre of stiffness (CS) in an irregular or asymmetric building generates torsional vibration even under the effect of pure translational ground motion. Under the action of seismic loading, inertia force developed in the structure acts through the centre of mass while the resistive force acts through the centre of stiffness. When there is non-concurrency in the lines of action of the inertia and resistive forces a twisting moment is induced which leads to torsional vibration in addition to the lateral vibration of the building. The dynamic response couples the torsion and translation in one or two orthogonal directions, asa result of which the lateral ductility capacity of the system reduces and such systems are termed as torsionally coupled systems.

As per IS 1893:2016, a well-proportioned building, does not experience torsion, if the stiffness distribution of the vertical elements which resists the lateral loads is balanced in plan according to the mass distribution and if the floor slabs are stiff in-plane. Torsional irregularity is said to exist when the irregularity coefficient, which is the ratio of the maximum displacement in the direction of the lateral force at one end of a floor to the minimum horizontal displacement at the far end of the same floor in the same direction, is more than 1.5 as shown in Figure 1.2. Also, the natural period of the building corresponding to the torsional mode of oscillation will be greater than that in the first two translational modes along each of the principal plan directions.

The static eccentricity (e_s) is given by the difference of positions of the center of mass and center of stiffness of the considered floor level. Under the application of dynamic loading, the effect of eccentricity in irregular buildings is higher as compared to the static load case for which, a dynamic amplification is considered as per IS 1893:2016 to calculate the design or dynamic eccentricity (e_d) at any floor level 'i' and floor plan dimensions b as in the Equation (1.1) given below:

$$e_{di}=\begin{Bmatrix} 1.5\, e_{si} + 0.05 b_i \\ e_{si} - 0.05 b_i \end{Bmatrix} \tag{1.1}$$

As per FEMA 450 and ASCE 7-16, torsional irregularity is considered to exist when the maximum storey drift, including accidental torsion, at one end of the structure transverse to an axis, is more than 1.2 times the average of the storey drifts at both ends of the structure. However, extreme torsional irregularity is considered to exist when the maximum storey drift, at one end of the structure transverse to an axis, is more than 1.4 times the average of the storey drifts at both ends of the structure.

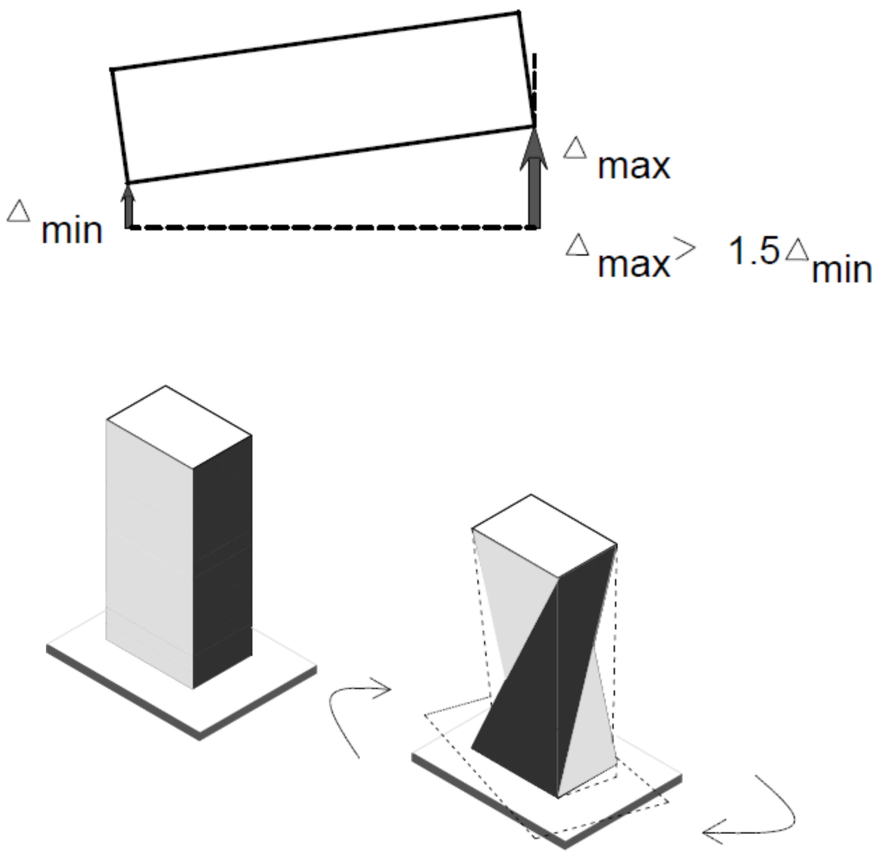


Figure 1.2 Torsional irregularity in buildings

When the plan irregular buildings are subjected to lateral loads such as wind or earthquake, torsional effects become significant. These buildings when subjected to dynamic loads, exhibit torsional behavior which is attributed to the significance of the higher modes of vibration. The irregularity in buildings due to the asymmetric placement of mass, stiffness and strength along the plan, causes the most severe damage since it leads to floor rotations or torsional response in addition to translations as observed from various earthquake damage histories. This makes their design for seismic loads substantially complicated as compared to the design of symmetric buildings whose response is purely translational. If the lateral resistance of the building to the earthquake forces is unbalanced torsionally, it can lead to large displacement amplification and high force concentration within the structural elements. This leads to severe damages to the structure thus affecting their load carrying capacity or in worst cases the total collapse of the structure under the influence of seismic loads. The resultant torsion in a building can be divided into inherent torsion that emanates from the dynamic properties of the system, and accidental torsion due to discrepancies in the stiffness and mass distributions as a result of rotational seismic excitation. Torsional effects occur even in symmetric buildings, known as accidental torsion, which may be generated by the rotational components of the ground motion during an earthquake. Accidental eccentricity is incorporated into design to compensate for actual distributions of both dead and live load during earthquakes, distributions of both stiffness and strength in the building, and torsional components of the ground motion, although it seems to be a significant criterion for buildings with large plan dimensions. In most of the building codes, the design eccentricity is estimated as :

$$e_d = \alpha e_s + \beta b \;; \quad e_d = \alpha e_s - \beta b \tag{1.2}$$

Where e_s is static stiffness eccentricity or the difference between the CM and CS, b is plan dimension of the building perpendicular to the direction of ground motion, and α, β are the specified coefficients. For each structural element, e_d is the design eccentricity. First term, αe_s in Equation 1.2 accounts for the coupled lateral torsional response of the building due to lack of symmetry in plan and second term, ± βb, stands for the accidental eccentricity introduced to account for the mass, stiffness, and strength discrepancies at the onset of an earthquake, torsional vibrations generated by the ground motion and other unprecedented sources. For design purposes, the accidental torsion is estimated as a fraction of the building dimension perpendicular to the direction of seismic action. In most of the codes, this fraction is taken as 0.05 to 0.10. As per 1893:2016, to evaluate the design eccentricity, an accidental eccentricity of 5% of the plan dimension of the storey perpendicular to the direction of applied ground motion is considered. The torsional amplification factor (A_x) shall not be less than 1 and should not exceed 3. As per the provisions of ASCE 7-16 and FEMA 450 the accidental torsion is estimated as.

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \tag{1.3}$$

Where, δ_{max} is the maximum displacement at a particular floor level and δ_{avg} is the average of the displacements at the extreme points of the same floor level of the building.

1.2. Vertical irregularity

Buildings which have significant physical discontinuities in vertical configuration or their lateral force resisting systems are termed as vertically irregular structure. A building is said to be vertically irregular if it possesses any sort of discontinuity in stiffness, strength or mass along the elevation or height of the building. The most significant among the vertical irregularities studied are the mass and stiffness irregularity. Several researchers have addressed the effects vertical discontinuity of mass, stiffness, strength or load paths in buildings. The important building codes classify the vertically irregular structures into different subcategories. It is also suggested that time history analysis or response spectrum analysis should be carried out to evaluate the response and lateral force distribution. Another widely researched area in vertically irregular buildings is the setback buildings as well as the stepped building frames which has a change in mass and stiffness at each level of setback or step. The classification of vertical irregularities as per the majority of seismic codes is as given in Figure 1.3.

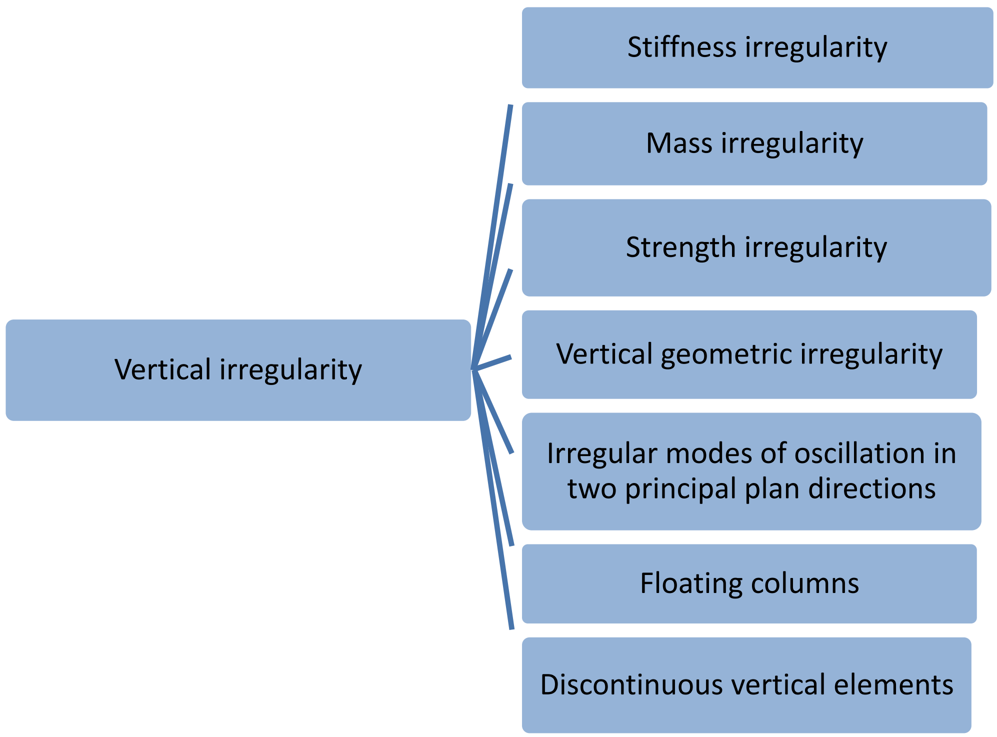


Figure 1.3 Classifications of vertical irregularities

1.2.1. Mass irregularity

Mass irregularities in buildings occur due to change in the mass distribution along the plan or the elevation of the building. Mass irregularity occurs due to the presence of a heavy mass on a floor or when one floor is much heavier than the others. Multi-storey high rise buildings usually have service floors or machine floors for the provision of storage of heavy masses as compared to regular floors. Heavy water tanks or swimming pools installed on an intermediate floor of a building can also lead to mass irregularity. With an increase in mass in one storey, the inertia force generated in that particular storey increases. If the percentage difference in mass or the change in mass as compared to the total mass of the building is minor, the effect of the mass irregularity is small on the mode shape in regular buildings. In case, the variation in the mass is considerable or pronounced, the difference in the seismic response is explicit when the buildings are subjected to strong ground shaking. Mass irregularities increase the ductility demands at the locations of irregularity and lead to unexpected effects in higher modes of vibrations.

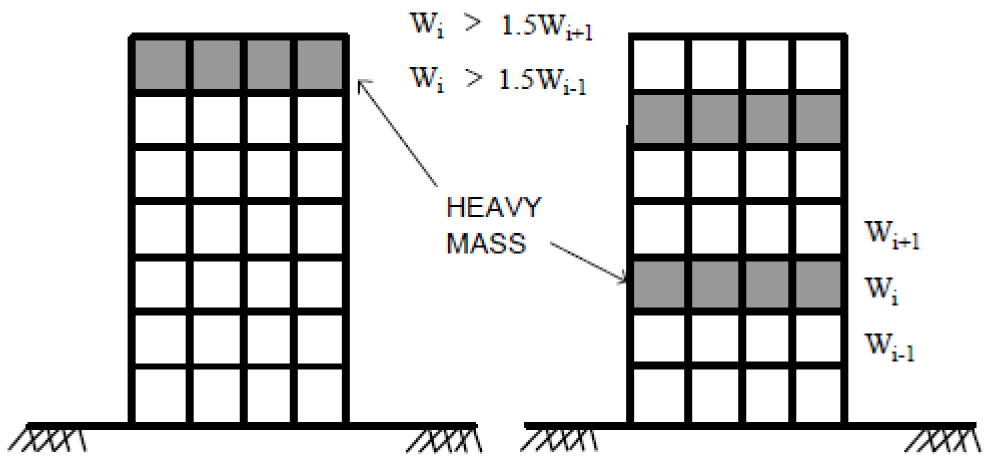


Figure 1.4 Mass irregularity in buildings

As per IS 1893:2016, ASCE 7-16 and FEMA 450, the criterion of vertical mass irregularity is considered to exist when mass of a storey is more than 1.5 times the mass of the storey below as shown in Figure 1.4.

1.2.2. Stiffness irregularity

A regular building is more of an idealization and practically all buildings are irregular in nature. Real buildings are designed to be irregular in nature to serve various purposes like basements for commercial purposes created by the elimination of central columns and reduction of the beams and columns size in the upper storeys, for functional requirements like storage of heavy machinery or equipment, etc. The difference in usage and purpose of a particular floor as compared to the consecutive floors results in irregular distributions of mass, stiffness and strength along the height of the building. Many other buildings are accidentally rendered irregular due to a variety of reasons like non-uniform material, methods, construction practices and strategies.

Soft storeys are the best examples of stiffness irregularity in buildings. Buildings with an open ground floor on the front side, tall ground storeys, etc. are examples of soft storeys and are commonly constructed in shopping malls, hotels and offices. If taller columns are provided in a storey and if they do not have the required strength and stiffness to resist the anticipated seismic forces, the buildings being stiffness irregular becomes vulnerable to seismic forces

In general, stiffness is the force required to cause unit displacement and is given by the slope of the force displacement curve. Strength is the maximum force that can be safely taken by the building. A soft storey in a building refers to the stiffness and weak storey refers to the strength distribution. Generally, the soft storey is also considered as weak storey. Soft storey condition emanates due to change in stiffness of adjacent storeys.

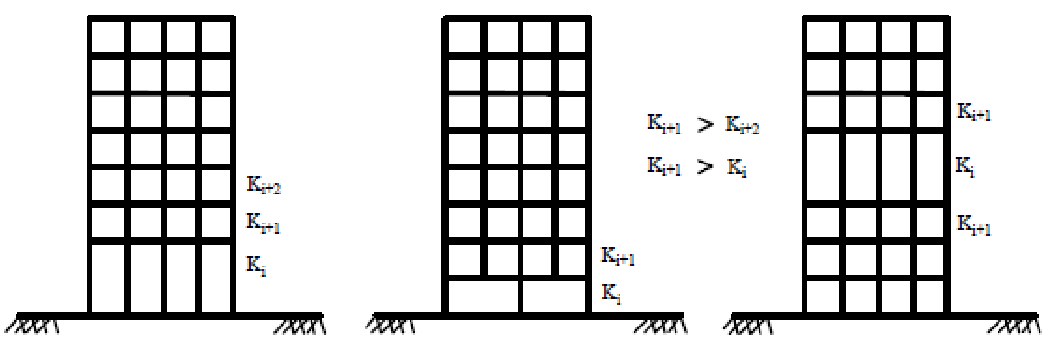


Figure 1.5 Stiffness irregularity in buildings

According to IS 1893:2016, stiffness irregularity or soft storey irregularity exists in a building when the lateral stiffness of any storey in it is less than that of the storey above it as shown in Figure. 1.5. As per ASCE 7-16 and FEMA 450, a soft storey is the storey whose lateral stiffness is less than 70 % as compared to the storey above or less than 80% of the average lateral stiffness of the three storeys above. Extreme soft storey is considered to exist when the lateral stiffness in any floor is less than 60 % of that in the storey above or less than 70% of the average lateral stiffness of the three storeys above.

1.3. Organisation of thesis

The dissertation has six chapters and the contents of each chapter has been organized as follows

Chapter 1 – **Introduction**

This chapter presents a brief overview of the types of irregularities prevalent in real buildings and the torsional coupling in buildings under the application of seismic loads.

Chapter 2 – **Literature Review**

A brief summary of literature focusing on research carried out related to irregular buildings. It is attempted to identify the improvements made in methodologies adopted for the analysis and study of irregular buildings through the years

Chapter 3 – **Objectives and Scope of the work**

Details of objectives framed for the study and the scope of the present study is elaborated.

Chapter 4 – **Methodology**

This chapter elaborates the methodology followed in the study towards the fulfillment of the objectives set forth. It gives the details of the idealization of the building models and the analysis carried out.

Chapter 5 – **Results and discussion**

The chapter discusses the observations and the inferences from the analysis carried out on the building models. The quantifications of various parameters are also discussed in this section.

Chapter 6 – **Conclusions**

This chapter summarizes the work that has been carried out. Also, the significant conclusions from the work are presented, highlighting the importance of consideration of integration of various irregularities in buildings which might lead to torsional coupling in buildings.

1.4. Summary

Seismic forces are occasional in nature, but when they crop up, they cause severe damages on buildings and property. Major seismic codes across the globe differentiate between irregularities in plan and elevation, but it must be comprehended that in practice, irregularity in the structure is the consequence of a combination of both types. It can be seen that the irregular structural alignments in elevation or plan are recognized as one of the major reasons for collapse through precedent seismic motions and numerous researchers have evaluated the effect of the irregularities on seismic response of the buildings. When present in combination, the irregularities lead to the most critical damages under the action of earthquake loading.

CHAPTER 2

LITERATURE REVIEW

Occupational or architectural demands or space constraints are some of the significant reasons behind the construction of irregular buildings. This irregularity or asymmetry in plan or elevation becomes a major cause for the damage of the buildings during earthquakes. The torsional coupling of the buildings due to mass or stiffness irregularity can detriment the seismic response of the structures leading to increased displacement demands. This chapter deals with an overview of the major research works carried out in this specific area to study the torsional response of asymmetrical buildings. The literature related to studies on different types of irregular buildings and their torsional behavior is available in abundance. Research works on the significance of torsion, in triggering the serious damages of the vast majority of earthquake affected buildings with different types of irregularities is highlighted in this chapter. The vast literature on irregular buildings reviewed in this work has been categorized into different sections based on the types of irregularities.

2.1. Dynamic analysis of buildings

The elastic analyses, as well as inelastic analyses of the single storey and multi-storey structures have been carried out by several researchers to study the effect of irregularities on building structures due to seismic loading. Equivalent static analysis, response spectrum analysis, non-linear static analysis or pushover analysis and non-linear dynamic analysis or time history analyses have been employed by the researchers to evaluate the seismic response of the asymmetric buildings under the influence of dynamic loading. Different building models were subjected to uni-directional as well as bi-directional ground motions in time history analysis to compare and study the influence of different parameters affecting the torsional response. In certain studies, envelopes of multiple ground motions were employed to study the effect of asymmetry in buildings. The structures are analyzed using different softwares like STAAD.Pro, ETABS, SAP2000, OpenSees, ABAQUS etc.

The non-linear static procedures were used as a solid tool to carry out nonlinear structural analysis since it required lesser computational effort and time as compared to the non-linear dynamic analyses. Fajfar and Fischinger (1988) suggested the N2 method which combined the pushover analysis for an MDOF system with the response spectrum of an equivalent SDOF system. The results of this method were seemingly accurate if the building vibrates in its first mode. However, the inadequacy of the N2 method in predicting the torsional response of in-plan asymmetric buildings was a major disadvantage. Further Fajfar et al. (2005) presented the extended N2 method, which was capable of estimating the torsional response of plan irregular buildings on applying various correction factors to the pushover results as per the original N2 method. Furthermore, Bhatt and Bento (2011a, 2011b) carried out extensive studies on the extended N2 in comparison with the original N2 method. The results obtained with the former were compared with that of the latter and also with the nonlinear dynamic analysis results evaluated based on semi-artificial ground motions. These analyses were performed for different seismic intensities to evaluate the torsional response of the building through different stages of structural inelasticity. The results obtained showed that the extended N2 method generally reproduces the real torsional behavior of the analyzed buildings in a very good fashion. However certain limitations were still evident with the static procedures as compared to the dynamic method like the de-amplification of torsional displacements, non-conservative roof displacements and so forth. Despite all the limitations, static methods are being used widely to assess the performance of irregular buildings. Mazza (2014) and Tarbali and Shakeri (2014) also employed pushover analysis to study irregular buildings. The latter proposed a single-run pushover procedure to evaluate the torsional response of in-plan asymmetric buildings subjected to unidirectional earthquake ground motions. Influences of the higher modes were incorporated into an invariant load pattern, calculated based on the height-wise distribution of the storey shear and torsional moment. The numerical investigations indicated that the proposed procedure was accurate in terms of the relative displacement of structures in comparison to that of time history analyses. Further, Poursha et al. (2014) carried out consecutive modal pushover analysis to study the seismic responses of tall buildings with two way unsymmetrical plan subjected to two-directional earthquake data considering the variations of the higher modes and the torsion. It was observed that the procedure could effectively estimate the displacements, storey drifts as well as the plastic hinge rotations. Further, Oyguca et al. (2018) investigated the degradation of irregular reinforced concrete structures subjected to the Tohoku ground motion employing pushover analysis. The structural characteristics of the developed irregular building models and their capacities were evaluated using the N2 and extended N2 procedures and the results indicated that irregularity effects increased the dispersed damage under these excitation sequences.

The aim of Moehle and Alarcon (1986) was to carry out an experimental study on two small scale models of reinforced concrete buildings with regular and irregular distributions of stiffness and strength in vertical plane subjected to ground motion with the help of shake table. The measured responses were compared with those computed by inelastic dynamic response time-history analysis, inelastic static analysis, elastic modal spectral analysis, and elastic static analysis. The maximum roof displacements obtained as per the experiments and by different inelastic dynamic and elastic analysis methods were compared. It was concluded that the prominent advantage of dynamic methods was that it captures accurately the maximum displacement response, as compared to the static methods. It was also inferred that the inelastic static and dynamic methods are reliable as compared to the elastic methods to interpret the discontinuities present in buildings. Goel and Chopra (1990) studied the effects of asymmetry on the seismic response of single storey elastic as well as inelastic systems for a wide range of system parameters like uncoupled lateral vibration period, the ratio of uncoupled frequency in the torsional mode to that of the lateral mode, stiffness eccentricity, and yield factor. Idealized one storey system with laterally loaded walls oriented in both the direction, with stiffness eccentricity along the x-direction subjected to a simple half-cycle input as well as El-Centro ground motion was considered for the study. It was shown that the response of inelastic systems is affected less by plan asymmetry compared to elastic systems. The effect of plan asymmetry on structural response was almost similar for the corresponding spectral regions of the simple input and the El-Centro excitation but differs considerably in the detailing. Furthermore, Hao and Duan (1995) investigated the effect of building asymmetry as well as multiple ground motions on an idealized single storey building subjected to earthquake time histories. When the translational mode is in phase with the multiple excitations, base shear reaches its maximum, and when they are out of phase base shear reaches the minimum, whereas, the torque reaches its maximum when the torsional mode and multiple excitations are out of phase, and minimum when in phase. It was concluded that in torsionally flexible structures, multiple excitations played a major role in producing torque.

Through several studies, dynamic analysis approach was found to be more realistic and valid as compared to the modal pushover analysis procedure and other static methods. Chintanapakdee and Chopra (2003) studied the seismic demands of vertically regular and irregular frames using modal pushover analysis and response history analysis and concluded that the latter was more accurate in estimating the seismic demands of irregular buildings with a strong or stiff lower half. It was deduced from several types of research that time history analysis provided an accurate evaluation of the structural response in comparison to the other prevalent methods. Hokmabadi et al. (2012) considered the inter storey drift as the main parameter in a 15-storey concrete moment resisting building using time-history analysis employing three earthquake ground motions, namely the Kobe (1995), Northridge (1994) and El-Centro (1940) earthquakes. It was also concluded that the absolute maximum drift upon time should be calculated over the maximum storey deflection as the latter may lead to un-conservative design. Similarly, Teruna (2017) studied the responses of mass and stiffness irregular buildings by non-linear static methods as well as time history analysis. It was observed that the accuracy of the former in the assessment of irregularities in the building is not a fully satisfactory solution since the irregularities of a building influence the dynamic responses of the building. Lagomarsino et al. (2018) employed nonlinear static analysis on existing irregular masonry buildings and compared the results with those obtained from nonlinear dynamic analyses, which was considered as the reference since it represents the actual seismic behavior.

In the dynamic analysis, the buildings were subjected to uni-directional as well as bi-directional excitations. Ghersi and Rossi (2000) studied the inelastic behavior of irregular asymmetric single storey buildings with resisting elements under the effect of bi-directional ground motion. It was observed that only minor variations were present in the responses of the structure due to the presence of the two horizontal components of the seismic action. Wilkinson and Thambiratnam (2001) modified the shear beam model and developed a three-dimensional procedure for the seismic analysis of asymmetric building systems considering the torsional coupling. Dutta (2001) attempted to study the inelastic range response of bi-directional eccentric systems with three lateral load resisting elements in each direction. Maximum displacement demand and maximum hysteresis energy dissipation demand of edge elements were considered as the responses. However, Shakib and Ghasemi (2007) determined the importance of consideration of near fault and far fault excitations on the earthquake response of the different types of stiffness irregular structures. Magliulo and Ramasco (2008) carried out non-linear static and non-linear time history analysis to study the effect of vertical strength discontinuities of RC frames by modifying the reinforcement details of beams or columns of the reference frame. Hong (2013) studied the effect of instantaneous eccentricity on the torsional response of one way and two-way symmetrical structures under the action of uni- and bi- directional seismic loading. This effect was incorporated in formulating the equations of motion under the effect of 123 bi-directional earthquake motions. Faggella et al. (2018) attempted to predict the seismic response of two way plan-asymmetric buildings. A nonlinear 3D frame model with eccentric in-fills was analyzed by linear and nonlinear response history analyses changing the earthquake incidence angle.

2.2. Torsional irregularities

The torsional effects, present in asymmetric buildings generally defined as the ratio between the floor edges displacements to the displacement of the center of mass. Due to architectural and functional requirements, multi-storey asymmetric-plan buildings have become quite common. Many researchers studied the seismic responses of single-storey asymmetrical buildings in order to understand the complicated seismic responses of multi-storey asymmetrical buildings. To reflect the variety of actual asymmetrical buildings, there were various prototypes of single storey asymmetrical buildings investigated in the aforementioned literature. Nevertheless, it seems difficult to project the inelastic properties of a multi-storey asymmetrical building in a single-storey asymmetrical building due to lack of a quantitative relationship between them. Hegal and Chopra (1989) studied the effect of lateral torsional coupling due to asymmetry in buildings for a wide range of parameters. The normalized response quantities were presented for flat and hyperbolic spectra against the ratio of uncoupled torsional to lateral frequency. The effects of coupling on the height wise distribution of the forces as well as the response spectra were also summarized. Further, Goel and Chopra (1990) studied the effects of plan asymmetry on the earthquake response of single storey elastic as well as inelastic systems for several response parameters and inferred that the response of inelastic systems is affected less by plan asymmetry compared to elastic systems. Guevara et al. (1992) showed that buildings with irregular plans were more vulnerable to earthquakes as compared to those with regular plans. The study was conducted to evaluate the effects of torsional coupling in buildings of irregularly shaped floor plans and it was proposed that if irregular rectangular buildings were divided into regular rectangular blocks by seismic joints and analyzed separately, it would be possible to obtain realistic models. Hutchison et al. (1993) extrapolated the dynamic torsional effects due to mass as well as stiffness eccentricities in single storey systems on to multi-storey buildings using probabilistic methods. It was observed that the effects of torsional coupling on a particular storey depend on its position along the building height. Dutta and Dutta and Das (2002) also studied the inelastic seismic response of code-designed asymmetric buildings idealized as asymmetric one storey systems.

Accidental torsion occurs due to random variations of the centre of mass or the centre of stiffness relative to the theoretical positions or due to ground motion rotational components. Chandler et al. (1994) evaluated the different interpretations of the inclusion of accidental eccentricity in the inelastic dynamic analysis of buildings. A single storey structure with three beam-column elements was considered for the study and the accidental eccentricity value of 0.05b for torsionally balanced systems was obtained as the most consistent case in terms of ductility demand. Further Chandler et al. (1995) also studied the inelastic torsional response provisions of various codes and their lateral period dependency on the same idealized structure as in the previous work. It was concluded that with appropriate inclusions of accidental torsion in the design of buildings as per the various code provisions there exists a strong period dependence on the deformation and ductility demand of the torsionally balanced as well as unbalanced buildings. De la Llera and Chopra (1994) and Lin et al. (2001) also developed procedures for evaluating accidental torsion from analysis of earthquake-induced motions of nominally symmetric-plan buildings. Paulay (1996) postulated a simple approach for the considerations of torsional effects in the seismic design of ductile building based on force displacement relationships.

In the recent past, realistic multi-storey models have been employed to analyse the torsional response in the inelastic range, also based on the evaluation of results of simplified one storey models as done by Stathopoulos and Anagnostopoulos (2003). These conclusions lead to further queries and an in-depth analysis of the various existing code provisions on torsion based on simplified, one-storey models. Stefano and Pintucchi

(2004) studied the torsional behaviour of mass and stiffness eccentric multi-storey buildings in light of the inelastic interaction effects between the axial and lateral forces on vertical load carrying elements of the building. It was concluded that the code provisions of Eurocode 8 along with the interaction effects resulted in maximum ductility values. Cosenza et al. (2000) studied the EC8 torsional provisions in comparison with other codes and also suggested improvisations for EC8. The static torsional provisions were observed to be satisfactory when the building has a large torsional stiffness but becomes deficient when applied to torsional flexible models. Zheng et al. (2004) evaluated the different criterions specified in different codes of China, USA and Europe by carrying out analyses on sample structures with torsion eccentricity about one as well as both the axes. The torsional effects were observed to have no dependency on the criterion specified in the codes and hence the torsion regulations were proposed to be modified. Jinjie et al. (2008) deduced the relationship between inter-storey torsion angle and torsion deformation ratio based on the provisions in tall building technical specifications in China for irregular buildings. A new methodology titled as torsion angle capacity spectrum method was put forth for the performance-based seismic evaluation of irregular plan frame structures using ETABS software which could identify the torsional displacement between the stories as well as among the structure members in a single storey.

The structural design and analysis of buildings with in-plan irregularities require an advanced and improved seismic evaluation and design principles to remedy the effects of torsion. Tabatabaei (2011) studied the free vibration characteristics of a single storey system and the effect of eccentricity in the torsional coupling of the irregular system. The coupling effect for a given value of eccentricity is the highest when the uncoupled torsional frequency and the translational frequency are equal. As their ratio increased, the effect of coupling was observed to decrease. Moon (2012) attempted to develop a consolidated seismic evaluation and design scheme to generate effective seismic designs for plan-irregular structures through temporal eccentricity variation. Validation was carried out for inelastic dynamic time history analyses and equivalent lateral force analyses with code-defined eccentricities. A tool to predict the torsional response and evaluation and design of irregular buildings was also developed. Broderick and McCrum (2012) also investigated the seismic response of plan irregular multi-storey steel braced framed structures employing the full-scale inelastic dynamic testing along with numerical modelling. The parametric study evaluated mainly the significant factors affecting the response of plan irregular structures, including static eccentricity and lateral-torsional frequency ratio. Lin et al. (2012) studied the ground basis of the various trends in asymmetric plan buildings. Torsional effects decreased when the plastic deformations increased and the torsional effects on displacements were smaller on the flexible side as compared to the stiffer side. It was observed that the seismic responses due to torsion on the stiffer side of a structure generally depend on the influences of several modes of vibration and ground motion in the transverse direction. Gokdemir et al. (2013) carried out studies on building models, with different storey numbers to study adversities caused by torsional irregularity under earthquake loads. Different earthquake code provisions on torsional irregularity were compared and it was concluded that proper separation distances between the big building sections and increased lateral rigidity on the weaker direction of the structures can reduce the torsional effect. Rajalakshmi et al. (2015) studied in detail the non-linear dynamic analysis on mass and stiffness irregular buildings separately and inferred that the irregular buildings are subjected to large displacements and localized damages as compared to regular buildings. Viti et al. (2016) studied the effects of variation of concrete strength as a reason for the generation of torsional effects in buildings. The strength variability which leads to stiffness and strength eccentricity at the first storey of the building, leading to a subsequent increase in the response of buildings were evaluated. Bensalaha et al. (2019) carried out an inelastic dynamic analysis on torsionally irregular buildings and inferred that the influence on the structural damage is significantly related to the input earthquakes characteristics. The probabilistic analysis was employed to evaluate the influence of seismic intensity on the building response in terms of the ultimate roof displacement and normalised dynamic eccentricity.

2.3. Shear wall buildings with irregularities

The building component, which resists the seismic forces, is considered as the lateral force-resisting system. The lateral load resisting system is incorporated in a building in the form of special moment resisting frames, shear walls and frame-shear wall dual systems. Shear walls are lateral load resisting systems in a building whose positions can considerably affect the dynamic response of the structure under the effect of earthquake loads. Moehle (1984) introduced vertical irregularity in buildings and carried out an inelastic dynamic analysis on buildings with discontinuous structural walls at various levels. Also, considering the inelastic behavior of structural elements provided a correct measurement of the distortions and shear distributions between frames and walls under strong seismic effects. Torsion which is directly proportional to the eccentricity ratio affected the strength and inter-storey drift at certain parts of the structure heavily considering the inelastic behavior. Bertero (1995) studied the strength demand and inter-storey drift at certain parts of the structure which differ for elastic and inelastic behavior due to the torsional mechanism of the structure. A simplified formula for the estimation of the reduction in strength due to inelastic torsion was proposed using which the inelastic torsion during preliminary seismic design could be controlled. Idealized one storey system with laterally loaded walls oriented in both the direction, with stiffness eccentricity along the x-direction or both orthogonal directions were considered by several researchers like Goel and Chopra (1990) and Hong (2013). Lee et al. (2000) evaluated the reliability of code formulae such as those of the current Korean Building Code (KBC), Uniform Building Code (UBC 1997), National Building Code of Canada (NBCC) 1995 and Building Law of Japan (BSLJ) 1994 for estimating the fundamental period of RC shear wall buildings. These code formulae were based on the fundamental natural periods of buildings under seismic action and the natural period was observed to vary with the amplitude of structural deflection or the strain level. Therefore the code formulas were found to be insufficient for estimating the seismic responses of the shear wall buildings. An improved formula was also proposed by regression analysis based on the measured period data. Gulay and Calim (2003) conducted a parametric study on 10 storeyed shear wall frame buildings with high torsional irregularities and different shear wall locations. It was concluded that additional torsional effects have to be considered for torsionally irregular buildings by increasing the value of design forces in structural elements to ensure the inherent safety of the structure. Similarily, Stefano and Pintucchi (2004) studied the torsional response of mass and stiffness eccentric multi-storey buildings and the torsional response of vertical load-carrying elements of shear wall buildings. Whereas, Demir et al. (2010) studied the effect of torsional irregularity factors on multi-storey shear wall buildings with varying shear wall locations, storey numbers, plan views and location of shear walls as per Turkish Earthquake Code (TEC) 2007. Colunga and Osornio (2005) studied the impact of the shear deformations of the wall systems on the torsional behavior of the building including the positions of the centre of rigidity, eccentricity and the distribution of the shear forces. It was observed that for the buildings with shear wall of different aspect ratios, the centres of torsion tend to shift from the vertical axis due to the effect of shear deformations on the rotational degrees of freedom of the shear walls. For short shear wall buildings, shear deformations were observed to be important and the computed static eccentricities increased from the upper storey to the lower but whereas in slender shear wall buildings, the effect of shear deformations is less significant and the computed static eccentricities increased from the lower to the upper storey. Lee and Ko (2007) analysed three building models with irregularities in the bottom two storeys, one symmetric, the second with an infill shear wall at the centre of the frame and the third with the infill shear wall at the exterior of the frame. The energy absorptions by the damage of the walls were observed to be similar regardless of its location. The damages were mainly due to overturning and secondly by shear deformation. Magliulo and Ramasco (2008) studied the effect of vertical strength discontinuities of RC frames due to the modification of the reinforcement details of beams or columns of the reference frame. On the other hand, Sigmund et al. (2008) stated that the elastic response of dual buildings is mainly governed by the wall response and the frame contribution could be neglected whereas in the inelastic range, the contribution by the frame increases. The base shear distribution, displacement and ductility demands of the walls and frames were recorded and it was observed that the wall contribution diminishes as the deformations increase. Heerema et al. (2015) studied the cyclic behavior of a reduced scale reinforced masonry asymmetric building with shear walls aligned in both the orthogonal directions based on experimental results. A two-storey reinforced masonry building was subjected to quasi-static loading and the contribution of the individual walls to the torsional response of the structure was also evaluated. Kocak et al. (2015) discussed the frame-wall irregularity on existing RC structures with shear walls subjected to the 1999 Kocaeli Earthquake in Turkey using nonlinear static analysis and nonlinear dynamic analysis. It is inferred that the irregularity checks should be intensified in the Turkish Seismic Code in synchronization with European standards.

Ozmen et al. (2014) investigated six groups of 3D building models with varying shear wall positions, storeys and the axes numbers. Torsional irregularity coefficients obtained in terms of maximum drift and average drift were observed to take a maximum for single storey structure. The irregularity coefficients were observed to take the maximum when the shear walls were placed close to the centre of mass whereas it was the reverse order in case of floor rotations. Hence it was concluded that floor rotations give an accurate representation of the torsional behavior in comparison with torsional irregularity coefficients as mentioned by several codes. Arabzadeh and Galal (2018) quantitatively assessed the non-linear responses of RC shear wall cores different torsional sensitivity factors based on the wide column method previously stated by Arabzadeh and Galal (2017). It was concluded that in totality, in the case of shear wall buildings, a combination of flexure, shear, and torsion determines the type of failure. The evaluation was done by comparing the seismic responses as per the response spectrum analysis as well as time history analysis. The range of torsional irregularity studied had no substantial effect on the bending moment envelope of the building but significantly increases the storey shear force demand during an earthquake.

2.4. Mass irregularities

Mass irregularities in buildings were studied in most of the research works and literature as a form of vertically irregular buildings with variations in mass through the height of the building. Many studies were based on the elastic seismic response of asymmetric buildings idealized as one storey systems which further paved the way for multi-storey inelastic building systems. These conclusions lead to further queries and in-depth analysis of the various existing code provisions on torsion based on simplified, one-storey models as reviewed in detail by Stefano and Pintucchi (2008). Valmundsson and Nau (1997) studied the response of vertically irregular multi-storey buildings of five, ten and fifteen storeys under earthquake loading. Mass, stiffness and strength were varied along the building height and it resulted in the increase in storey drifts and ductility demands. Time history analysis was carried out on structures by considering the floor mass ratios and strength and stiffness ratios ranging from 1.0 to 5.0. The irregularity limits laid down in the UBC were evaluated and the conditions under which equivalent lateral force procedure can be applied for an irregular building were also stated in the study. Das and Nau (2003) studied the various vertical irregularity effects on a large group of buildings of heights varying from 5 to 20 storeys. The seismic parameters computed by the equivalent lateral force method and time history analysis were compared for the symmetrical and asymmetric buildings. Ductility demand and storey drifts at the location of the combined irregularities showed an abrupt increase as per the limits as per UBC 1997. Bosco al. (2013) studied the static response of asymmetric buildings and suggested a procedure for the evaluation of the in-plan irregularity of singly or bi-eccentric building systems. Tremblay and Poncet (2005) and Ayidin (2007) examined the seismic response of mass irregular multi-storey buildings according to NBCC 2005 and the Turkish Seismic Code 2007 (TSC) respectively. This analytical study concluded that change in mass ratio affects the storey shear and that the time history procedure gives the accurate estimation of the seismic response of the multi-storey models in comparison with the ELF procedure. Karavallis et al. (2008) studied the seismic responses of steel moment-resisting frames with vertical mass irregularities and derived expressions to define the seismic response using regression analysis techniques. Sadasiva et al. (2008) studied the effect of the location of mass eccentricities on 9-storey frames designed as per New Zealand building code by carrying out inelastic time history analysis and concluded that the inter-storey drift recorded is the highest when the mass irregularity is present on the top storey of the building. Mansuri (2009) carried out nonlinear dynamic analyses on steel moment frame buildings subjected to Northridge and Loma Prieta earthquakes. Parameters such as the unsymmetrical distribution of mass or resistance along the plan, intensity and frequency content of ground motions, setbacks, accidental eccentricity and so on were discussed. Response parameters evaluated included lateral storey displacement, Inter-storey drift, plastic hinge rotation and torsional rotation of each floor. The torsional rotations of floors were considered as a significant parameter to determine the torsional response of the building. It is also reported by Anagnostopoulos et al. (2010) that if the element stiffness and strength of the real buildings, as well as their three lowest periods of vibration, is not comparable with that of the one-storey models, it may not give the accurate trend and behavior of the asymmetric buildings. These conclusions lead to further queries and in-depth analysis of the various existing code provisions on torsion based on simplified, single storey models. Rizwan and Singh (2012) classified buildings into mass symmetric systems with stiffness and strength irregularities and carried out dynamic time history analysis and stated that torsion caused a significant increase in the beam ductility demands of the frames. Thermou and Psaltakis (2017) proposed a design methodology to minimize the effect of structural eccentricities and torsional coupling by modifying the lateral response shape of the building in each direction and hence achieve an optimum distribution of inter-storey drift along the building height. Raheem, et al. (2017) studied the torsional behavior of L- shaped buildings from the results obtained for inter-storey drift, storey shear, overturning moment, torsional moment, roof displacement and torsional irregularity coefficient and stated that the irregular buildings are more vulnerable than those with a regular configuration resulting from stress concentration and coupled lateral-torsional behaviour.

Various researchers have attempted to quantify the mass irregularity in asymmetric buildings. Varadharajan et al. (2012b) discussed the applicability of proposed equations based on regression analysis for estimating the fundamental natural period, roof deflection and inter-storey drift of mass and stiffness irregular 2D as well as 3D frames. Bhosale et al. (2017) investigated the quantification of irregularity based on fundamental natural period and correlation between seismic risk and vertical irregularity coefficients. Few authors suggested an irregularity index to quantify the magnitude and location of the mass irregularity in the building frame along with modification for the expression for the natural period as per IS 1893:2002 (Varadharajan 2014; Varadharajan et al. 2015). Mwafy and Khalifa (2018) carried out an extensive study on the different types of vertical irregularities on real-life tall buildings designed as per international building codes subjected to far-field and near-source seismic excitations and concluded that the present design coefficients of irregular buildings are quite conventional and should be revised to yield effective limits and designs.

2.5. Stiffness irregularities

Stiffness irregularity is the most widely researched type of irregularity among the vertical variants of irregularity in buildings. Stiffness irregularity has the highest seismic demand especially in combination with strength variations and makes the buildings more vulnerable in comparison to the other irregularities of the same magnitude. Ruiz and Diederich (1989) studied the buildings damaged in the Mexico Michoacan earthquake (1985) due to the presence of weak lower storeys. The effects of lateral strength discontinuity, brittle infill walls in the upper floors and the variations in the lateral resistance of the upper storeys with as compared to the lower ones were observed to be the major reasons for the damages caused. Ali-Ali and Krawinkler (1998) studied in detail the responses of buildings with irregularities in mass, stiffness and strength individually and also in combination. It was concluded that the responses of the buildings were more influenced by the stiffness irregularities as compared to the mass irregularities of the same magnitude. The significance of non-linear analysis to design buildings and retrofit the existing ones was also highlighted. Dutta (2001) carried out a study on the inelastic torsional behaviour of reinforced concrete (RC) asymmetric buildings using idealized one-storey models based on the strength deterioration characteristics of RC members which magnified the displacement and ductility demand in structural elements increasing the eccentricity.

Further, in the recent past, studies on the seismic behavior of multi-storey irregular buildings have also escalated, mainly due to the efficiency of nonlinear dynamic programming and coding (Stefano and Pintucchi, 2008). Dimova and Alashki (2003) proved that even under small accidental eccentricities, the symmetric structures exhibited irregular behaviour and that the accidental torsional effects cannot be described properly by static application of torsional moments. Lađinovic and Folic (2008) stated a procedure to construct a base shear and torque (BST) surface with an arbitrary number of resisting elements in the direction of asymmetry and of ground motion since the inelastic seismic behaviour of plan asymmetric buildings was considered using the histories of base shear and torque. It was observed that the factors that determined the shape of the BST surface were the strength eccentricity, lateral and torsional capacity of the system and plan-wise distribution of strength. Michalis et al. (2006) carried out an incremental dynamic analysis on steel structures with vertical irregularities. With a nine-storey steel frame as the base, irregularities under four categories; stiffness, strength, combined stiffness and strength, and mass irregularities were considered. Non-linear time history analyses were performed for a suite of ground motion records scaled to several intensity levels and interpolation was carried out to calculate capacities for several limit-states, from elasticity to final global instability. Sadasiva et al. (2011) studied the stiffness irregular buildings with variation in inter-storey height causing stiffness reductions at various levels. The buildings with modified inter-storey heights were redesigned until a target inter-storey drift ratio was achieved at the critical storey. Time history analysis was employed for this purpose by subjecting the buildings to earthquake ground motions, and the maximum inter-storey drift ratio was evaluated to compare the responses of regular and irregular structures. Whereas, D'Ambrisi et al. (2013) studied the seismic performance of irregular 4-storey existing RC framed structures subjected to seismic loading using the computer code Seismo Structure and the obtained response domain was compared with the limits provided by FEMA 356. It was concluded that even lower values of the eccentricity lead to considerable variations in the seismic performance of the structure. Benavent-Climent et al. (2014) discussed the torsional response of a scaled RC framed structure subjected to several uniaxial shaking table tests and analysed the asymmetric behavior in terms of displacement, strain in reinforcing bars, energy dissipated at plastic hinges, and damage at section and frame levels. The results showed that within the elastic range, even the accidental eccentricity increases the lateral displacement demand in the frames by about 30% and this can cause significant damage to non-structural components.

Kumar and Gornale (2012) overviewed the performance of the torsionally balanced and unbalanced buildings subjected to pushover analysis. The buildings with the un-symmetrical distribution of stiffness in storeys were studied for the effect of eccentricity between centre of mass and centre of storey stiffness. Also, the effects of stiffness of in-fill walls on the performance of the building were assessed as per the procedure prescribed in ATC-40 and FEMA 273. Varadharajan et al. (2012a) summarized the research works carried out previously on different classifications of structural irregularities. Criteria and limits defined for these irregularities as per different codes of practice were discussed and it was observed that the limits of irregularities as prescribed by these codes were comparable. Many studies were carried out for developing torsional regularity coefficients based on various seismic codes and the regularity conditions and applications of the coefficients were researched thoroughly. Varadharajan et al. (2012b) proposed a single index to quantify mass, stiffness and strength irregularity in terms of both magnitude and location on the basis of the dynamic characteristics of the building. With the application of regression analysis and based on the proposed index, equations were proposed to estimate seismic response parameters such as fundamental natural period, maximum roof displacement and maximum inter-storey drift ratio, etc. for the irregular buildings. Ouazir et al. (2018) investigated the effects of the torsional coupling and soft storey effects on the seismic behavior of reinforced concrete frame buildings. Different building models, with a variation of structural stiffness in the plane of each storey and over the height of the structure, were generated and the effects of non-uniformly distributed masonry in-fills were also examined. It was observed that for the buildings considered, the maximum storey drift was concentrated on the first storey and further, the variation in displacement and storey drift of irregular building with respect to the regular one were more sensitive to the effect of coupling of stiffness ratio and stiffness eccentricity.

Most of the related research work was primarily on elastic models but were then substituted by inelastic models, since building response becomes inelastic under the action of severe earthquake loading. Further, these building models were also subjected to linear and nonlinear analysis which was further bifurcated into the static and dynamic analysis. Kara and Celep (2012) studied the structural irregularity due to the discontinuity of a column in a plane frame subjected to seismic loads by carrying out linear and non-linear static and dynamic analyses of the structural system. Similarly, La Brusco et al. (2016) carried out the seismic study of a real RC existing building by performing three different types of analysis namely pseudo-dynamic elastic, non-linear static analysis and non-linear dynamic analysis as per EC8. It was observed that the torsional response was sensitive to the type of analysis carried out. The linear analysis provided the largest top storey rotation. Along the direction of larger eccentricity, the two non-linear analyses gave similar results, while in the opposite direction the pushover analysis was observed to be more conservative than the dynamic analysis because the 5% eccentricity provided by EC8 largely covered the eccentricity. Hence the seismic performance in the ultimate limit state can be said to be highly dependent on the analysis type. Bakalis and Makarios (2018) stated that the torsional behaviour of the asymmetric single-storey building changes abruptly during nonlinear behaviour and hence if pushover analysis is carried out, dynamic eccentricities should be considered. For this, the magnitude of these dynamic eccentricities and the appropriate horizontal orientation of the lateral static floor force should also be known. Herrera et al. (2013) carried out studies on an original building and its redesigned version on which nonlinear static analysis and non-linear 3D dynamic analysis were applied. It was also observed that maximum torsional effects occur in the re-entrant corners of the irregular plan, which can be reduced in mid-rise buildings by using a rigid diaphragm. Dutta, et al. (2017) carried out case studies to evaluate seismic behavior of different configurations of irregular structures analysed by response spectral as well as non-linear time history analyses, the results of which indicated that the irregular structures may have lower base shear due to reduced seismic weight as compared to a similar reference building. Chances of higher stress concentration as well as higher ductility demand were observed in the members around the proximity of the irregularity which can further lead to early damage of these members and finally leading to the progressive collapse of the structure. The pushover analysis method even after the application of improvement techniques was found to be less accurate as compared to dynamic analysis.

Numerous studies have focused on set-back structures and most of them agree on the variation in drift demand of the tower portion of the set-back structures due to an abrupt change in stiffness at the level of the setbacks. Athanassiadou (2008) studied ten-storey 2D plane frames with two and four large setbacks in the upper floors respectively in comparison to a regular building designed as per EC8. The frames were subjected to inelastic static pushover analysis and inelastic dynamic time history analysis for the input ground motions. The effect of ductility class and criteria of the setback frames were evaluated and it was observed that the pushover analysis underestimates the responses of the upper floors of the irregular frames. Sarkar et al (2010) evaluated the seismic responses of stepped building frames, with vertical geometric irregularity and attempted to quantify irregularity in them, considering the dynamic characteristics. A modification for the code specified formula for estimating the fundamental period for regular frames was also proposed, to evaluate the natural period of the stepped building frame. Varadharajan (2013) also proposed an irregularity index for quantifying the setback irregularity along with a modified equation for estimating the fundamental period of vibration in the case of frames with setback irregularity. Georgoussis et al. (2015) carried out an approximate analysis of multi-storey setback buildings subjected to strong ground motions. Setback buildings with mass and stiffness eccentricity along with two structural walls provided through the full height of the tower section were considered for study. Ghosh and Rama (2017) studied the setback buildings resting on the plain ground as well as along the hill slope, with soft storey configuration. Equivalent static force, response spectrum and time history methods were carried out and three individual mitigation techniques were been adopted and the best among the three techniques were suggested.

Improper load applications lead to irregularities in buildings which further generates complex structural behavior. Earthquake loads give rise to extra shear, torsion, etc. on irregular buildings and therefore structural irregularities decrease their seismic performance significantly. Stathopoulos and Anagnostopoulos (2004) investigated the inelastic earthquake response of eccentric, 3 and 5 storey detailed structural buildings designed according to EC8 as well as a simplified single storey, shear beam type idealizations. The frames on the flexible side of the buildings exhibited higher ductility demands in comparison to frames on the stiff side and such substantial differences in such demands between the two sides suggested a need for reassessment of the pertinent code provisions. Bosco et al. (2013) described a new technique for the assessment of the static eccentricity and the uncoupled torsional to lateral frequency ratio from the structural response to arbitrary force distributions and torsional couples in real multi-storey buildings. This method was validated on some regular and irregular buildings with different in-plan irregularities and was concluded that the results of the method were rigorous in the former and depends on the height-wise distribution of the forces in the latter. Torsional effects can modify the seismic response of asymmetric buildings due to the influence of several complex and difficult parameters. A key input in seismic response assessment and a major reason for the existing controversies in the subject of asymmetry is the application of an appropriate set of ground motions (Chakroborty and Roy, 2016).

2.6. Summary

The literature study shows that the building models, type of analysis, coupling of irregularities, and several other factors which lead to eccentricities in a building are substantial parameters that are to be considered in the study of irregular buildings. Literature available on plan irregularities is more in number as compared to that on the vertical irregularities due to the severity of the former in inducing torsion in buildings. Researches on single storey irregular buildings are more in number than those on multi-storey irregular buildings. So regularity studies and indices based on single storey models cannot be put into application to quantify irregular multi-storey buildings. Also, dynamic analysis and especially time history methods were found to be more reliable and accurate in comparison to that of static pushover methods. Overall it was concluded by many researchers that the extent of irregularity depends on the type, magnitude and location of the irregularities. The most significant problems associated with irregular buildings are identified to be the presence of heavy masses at the upper floor levels, discontinuous or unsymmetrical placement of infills or shear walls, discontinuous stiffness or stiffness reductions at the lower storey levels and the combination of different types of irregularities. Various works have been carried out individually to identify the effect of vertical mass and stiffness irregularity on buildings and to evaluate their responses to earthquake loading. It can be inferred that the literature on irregular buildings are available in plenty but the combination of irregularity along the height of a building and in-plan eccentricity is rarely researched. The studies on the combination of vertical irregularity and in-plan eccentricity or torsional coupling due to the integration of various irregularities together are limited. The present study focuses on the combination of vertical and torsional irregularity and aims to evaluate the effect of in-plan eccentricity, varying locations of irregularities and aspect ratios of buildings on their seismic responses. Problematic configurations and critical cases in the case of vertical irregularity have been assessed in combination with torsional irregularities and indices have been developed for quantification of torsional responses in terms of combinations of mass and stiffness irregularities with in-plan eccentricity. The torsional behavior of the buildings is assessed with respect to the measured roof rotation and torsional resultants and further, irregularity coefficients are developed to quantify the considered integration of vertical stiffness irregularity and in-plan eccentricity in terms of the location of irregularity and in-plan eccentricity.

CHAPTER 3

OBJECTIVES AND SCOPE OF WORK

3.1. Objectives

The research objectives set for the present study are as given below:

- To study the effect of torsional irregularities due to asymmetric plan on the structural response of reinforced concrete shear wall buildings of different heights and various location of shear walls under the influence of earthquake loading.

- To study the seismic response of reinforced concrete buildings of different heights, aspect ratios and mass ratios with vertical irregularity due to mass eccentricity under the influence of earthquake loading.

- To study the influence of stiffness irregularities on the earthquake response of reinforced concrete buildings of various heights and plan dimensions.

3.2. Scope

The presence of any type of irregularity in a building modifies its seismic response. A vast volume of literature is available on the study of irregularities in buildings. But researches and studies on certain areas dealing with the response of buildings with a combination of different irregularities are found to be limited. In addition, the effect of in-plan eccentricity in the seismic response of buildings is not addressed so far. Buildings irregular in plan or elevation when subjected to earthquakes forces have pronounced responses and are vulnerable to early damage. In practice, buildings may be subjected to different types of irregularities singly or in combination. The present study introspects the seismic responses of building with a combination of horizontal and vertical irregularities with specific attention to in-plan eccentricity. The significance of various parameters like the aspect ratio of the buildings, change in eccentricity ratio, magnitude and location of the irregularities are evaluated in the study. Initially, a set of shear wall buildings with in-plan irregularity due to change in the location of shear walls are considered. Here the in-plan mass and stiffness vary with the change in the eccentricity ratio due to the re-location of shear walls in a single direction. Secondly, vertically mass irregular buildings with in-plan eccentricity due to change in location of the masses along the plan as well as along the height are studied. Thirdly, stiffness irregular buildings with modified stiffness along the height due to variation in inter-storey height as well as a change in in-plan stiffness due to change in column dimensions are studied.

The three sets of irregular buildings of different aspect ratios are subjected to El-Centro ground motion and transient analysis is carried out. The seismic responses of the buildings are evaluated in terms of fundamental natural period, base shear ratio, roof deflection, roof rotation, storey drift and torsional resultant. Separate coefficients for mass and stiffness irregularities have been proposed to quantify the combination of in-plan and vertical irregularity in buildings which can be utilized to properly re-plan an irregular building. It is also attempted to propose an alternate equation to predict the natural period of an irregular building as a better alternative to the code proposed approximate equations for prediction of the natural period. Further, the critical cases among the buildings are subjected to two more ground motions with frequency content different from that of El-Centro to apprehend the influence of the frequency contents of ground motion on the irregularity variations.

CHAPTER 4

METHODOLOGY

The study deals with irregularities in buildings or in specific, the torsional irregularity due to variations in mass, stiffness or mass and stiffness in combination is evaluated thoroughly. The methodology adopted to study the irregular buildings and to investigate the seismic and torsional responses of the buildings is given in detail in this chapter. This section of the thesis also discusses the structural idealization adopted for study on the set of shear wall buildings with irregularities, mass irregular and stiffness irregular buildings. The different types of irregularities considered are explained in brief and then the detailed idealization of the buildings belonging to each category of irregularity is discussed.

4.1. Irregularity in buildings

Assessment of the performance of buildings subjected to seismic forces suggests that in-plan irregularities are one of the important causes of damage during the occurrence of an earthquake. The lateral-torsional coupling due to eccentricity between the centre of mass (CM) and centre of rigidity (CR) in asymmetric building structures generates torsional vibration even under purely translational ground shaking. Under the influence of seismic loading, inertia force acts through the centre of mass while the resistive force acts through the centre of rigidity. Due to this non-concurrency of lines of action of the inertia force and the resistive forces, a twisting moment is generated which causes torsional vibration of the structure in addition to the lateral vibration. Torsional effects occur even in symmetrical buildings, known as accidental torsion, which may be primarily induced by the rotational components of the ground motion during an earthquake. According to IS 1893: 2016, buildings with maximum displacement at any floor of the building more than 1.5 times the minimum displacement at the other end of the same floor in the same direction are said to be torsionally irregular as given in Equation 4.1. Also, the fundamental natural period in the torsional mode of a torsionally irregular building will be greater than the natural period of the building in the first two translational modes of oscillation along each direction.

$$\eta_t = \frac{\delta_{\max}}{\delta_{\min}} \tag{4.1}$$

Where, η_t is the torsion irregularity coefficient, δ_{max} and δ_{min} are the maximum displacement at one end of a floor and the minimum displacement at the other end of the same floor in the same direction respectively. The buildings are said to be torsionally irregular if η_t is greater than 1.5. If η_t is greater than 2, the entire building configuration should be revised as per the IS code provisions in order to ensure that the natural period of torsional mode of oscillation becomes smaller than that of the first two translational modes of oscillation. As per FEMA 450, ASCE 7-16 and most of the international codes, torsional irregularity is considered to exist if the maximum storey drift including the accidental torsion factors at the one end of the structure is more than 1.2 times the average of the storey drifts at the two ends of the structure. Extreme torsional irregularity is present when the maximum storey drift, is more than 1.4 times the average of the storey drifts at the two ends of the structure. The torsional irregularity in a building is represented in terms of the eccentricity of the building which is the divergence between the lines of action through the centre of mass and centre of rigidity or resistance. The static eccentricities (e_{si}) of the configurations are obtained from the difference of positions of the center of mass and center of stiffness of the considered floor level. Under the application of dynamic loading, the effect of eccentricity in irregular buildings is higher as compared to the static load case for which, a dynamic amplification is considered as per IS 1893:2016 to calculate the design or dynamic eccentricity (e_{di}) at any floor level 'i' as in the Equation 4.2 given below:

$$e_{di} = \begin{Bmatrix} 1.5\, e_{si} + 0.05 b_i \\ e_{si} - 0.05 b_i \end{Bmatrix} \tag{4.2}$$

From an extensive literature review it is noted that the effect of the position of mass or stiffness of elements causing in-plan eccentricity is not explored. Hence this study focuses on firstly, shear wall buildings, secondly, mass irregular building and thirdly, stiffness irregular buildings each with torsional or in-plan irregularities. In the first part of the study, irregularities present in buildings as a combination of mass and stiffness irregularities were considered. For this, a set of shear wall buildings were considered in which irregularities were incorporated by varying the position of shear walls within the plan of the building. As a result of this, shifting of the centre of mass, as well as the centre of stiffness, along the plan takes place leading to in-plan eccentricity and consecutively the buildings become torsionally irregular. In the case of shear wall buildings, shear wall ratio is taken as the ratio of the shear wall area to the total floor area. Two different sets of buildings were considered for this, first with L-shaped shear walls provided at diagonally opposite corners designated as 2W (shear wall ratio of 0.16) and the second with L-shaped shear walls on all four corners in the building frames designated as 4W (shear wall ratio of 0.32). To study the torsional behavior of shear wall buildings, in-plan eccentricity was generated in 2W buildings and 4W buildings by changing the location of one or two shear walls along the building plan in a single direction. 18 different plan configurations (2W1-2W9 and 4W1-4W9) with in-plan eccentricity due to the arrangement of shear walls were generated and studied for their seismic responses

The second part of the study focuses on the torsional irregularity in vertically mass irregular buildings. As per IS 1893:2016, ASCE 7-16 and FEMA 450, the criterion of vertical mass irregularity is considered to exist when the mass of a storey is more than 1.5 times the mass of the storey below. It is attempted to study the response of vertically irregular frames with varying mass ratios along the height and to identify the effects of torsional coupling on them. Varying mass ratios 1.25 to 5 were considered and the additional masses were located along the building height at different locations at the bottom floor level, middle floor level and upper floor level. The placement of the additional masses was done in three different patterns IM1, IM2 and IM3 on a floor level in order to develop in-plan torsional irregularities. These building configurations with mass irregularities were studied for their seismic responses and the effects of in-plan eccentricity were scrutinized.

The final part of this study deals with stiffness irregularity and studies the effect of in-plan eccentricity in vertically stiffness irregular buildings. As per IS 1893:2016, stiffness irregularity or soft storey irregularity exists in a building when the stiffness of any storey is less than that of the storey above it. Inter-storey height was modified in the buildings to give rise to stiffness reductions K_1 to K_4 which were applied at the bottom, middle and top floor levels. Further, to study the effect of in-plan eccentricity, stiffness modifications were made in the plan of the buildings to generate eight different plan configurations IS1-IS8 with torsional irregularities. The building configurations with a combination of the stiffness reduction along the height as well as stiffness variations in the plan were studied and their seismic responses were evaluated.

4.2. Structural idealisation

The effects of in-plan eccentricity on the differently asymmetric buildings were considered in this work. The varying eccentricity ratio in shear wall buildings and vertically irregular buildings specifically mass and stiffness irregular buildings modifies the seismic and torsional behaviour of the buildings under the application of seismic loading. The seismic responses of the irregular buildings were studied in terms of variation in the fundamental natural period, seismic base shear, roof deflection, roof rotation and storey drifts.

The buildings considered for the study of irregularities were three-dimensional (3D) idealized frames of 5 storey, 10 storey and 15 storey buildings categorized into group A, group B and group C respectively. The storey height and length of each bay of all the building frames were chosen as 3m and 4m respectively. The thicknesses of floor slab and the raft slab were taken as 0.15m and 0.5m respectively. The beam dimensions of 0.3 x 0.4m and column dimensions of 0.4m x 0.4m, 0.5m x 0.5m and 0.6m x 0.6m were considered for the 5, 10 and 15 storey buildings respectively. The dimensions of building components were adopted based on the structural design as per Indian standard codes for design of reinforced concrete structures IS 456:2000 and IS 13920:2016. Concrete of M_{25} grade and steel of Fe 415 grade were considered as the material for the structural elements and live loads of 3.0 kN/m^2 and 1.5 kN/m^2 were provided on floor and roof respectively. The loading for the residential building was considered based on IS 875(Part1):1987. The buildings were idealized as 3D frames in finite element software LS-DYNA using resultant Hughes-Liu beam elements with six degrees of freedom at each node. Four-noded Hughes-Liu shell elements with bending and membrane capabilities and six degrees of freedom at each node were used for modeling the roof, floor and foundation slabs. Non-linear concrete material type MAT_CONCRETE_EC2 was used as the material for the Hughes-Liu elements for representing a smeared combination of concrete and reinforcing steel. This material model includes concrete cracking in tension and crushing in compression, and reinforcement yield, hardening and failure as per Eurocode 2. The input data required for the material model includes mass density, compressive strength, tensile stress of concrete, Young's modulus, ultimate stress, Poisson's ratio of reinforcement and the fraction of reinforcement along both the directions. Type 6 Mander model (Mander et al. 1988) has been used to represent the material non-linearity of the reinforced concrete sections Mesh size of 1m was used to discretize all the building components.

4.2.1. Shear wall buildings

In the first part of the work, shear wall buildings were studied for their torsional behavior. The locations and orientations of shear walls were changed in the plan of the buildings to give rise to eccentricity. This eccentricity is specifically due to change in mass as well as stiffness in the plan due to variation in the location of shear walls. Seismic torsional responses of 5, 10 and 15 storey reinforced concrete buildings with varying shear walls positions were evaluated. Three-dimensional (3D) idealized building frames with plan dimensions of 20m x 20m were considered in the study. Buildings of varying aspect ratios 0.75, 1.5 and 2.25 categorized into group A, group B and group C respectively with shear walls at different locations were modeled in finite element software LS-DYNA software. Shear walls of thickness 0.25m were provided throughout the height of the building frames at specific locations in the plan of the buildings to incorporate the effect of in-plan eccentricity.

Two different sets of buildings were considered here, the first with L-shaped shear walls provided at diagonally opposite corners designated as 2W with a shear wall ratio of 0.16 and the second with L-shaped shear walls on all four corners in the building frames with a shear wall ratio of 0.32 designated as 4W. It is assumed that centers of gravity of storeys are at the geometric centers of floor plans. The shear walls along the x direction (x walls) and shear walls along y direction (y walls) were relocated perpendicularly and successively to the consecutive bays so as to introduce asymmetry in the plan of the building along a single direction to generate eccentricity in the same direction. To study the torsional behavior of shear wall buildings, the static eccentricity in the x-direction (e_s) was generated in 2W buildings and 4W buildings by changing the location of one or two walls along the building plan in a single direction. 18 different plan configurations with in-plan eccentricity due to the arrangement of shear walls were generated. 2W1-2W9 and 4W1-4W9 are the plan eccentric configurations thus generated and 2W0 and 4W0 are the symmetric configurations with L-shaped shear walls in the diagonally opposite corners and all four corners respectively. The bare frame buildings in group A, group B and group C are represented by 5R, 10R and 15R respectively. A total of 63 different buildings belonging to the three groups were thus generated as shown in Figures 4.1 and 4.2 which were analyzed to evaluate the effect of torsional irregularity in the seismic behavior of the shear wall buildings.

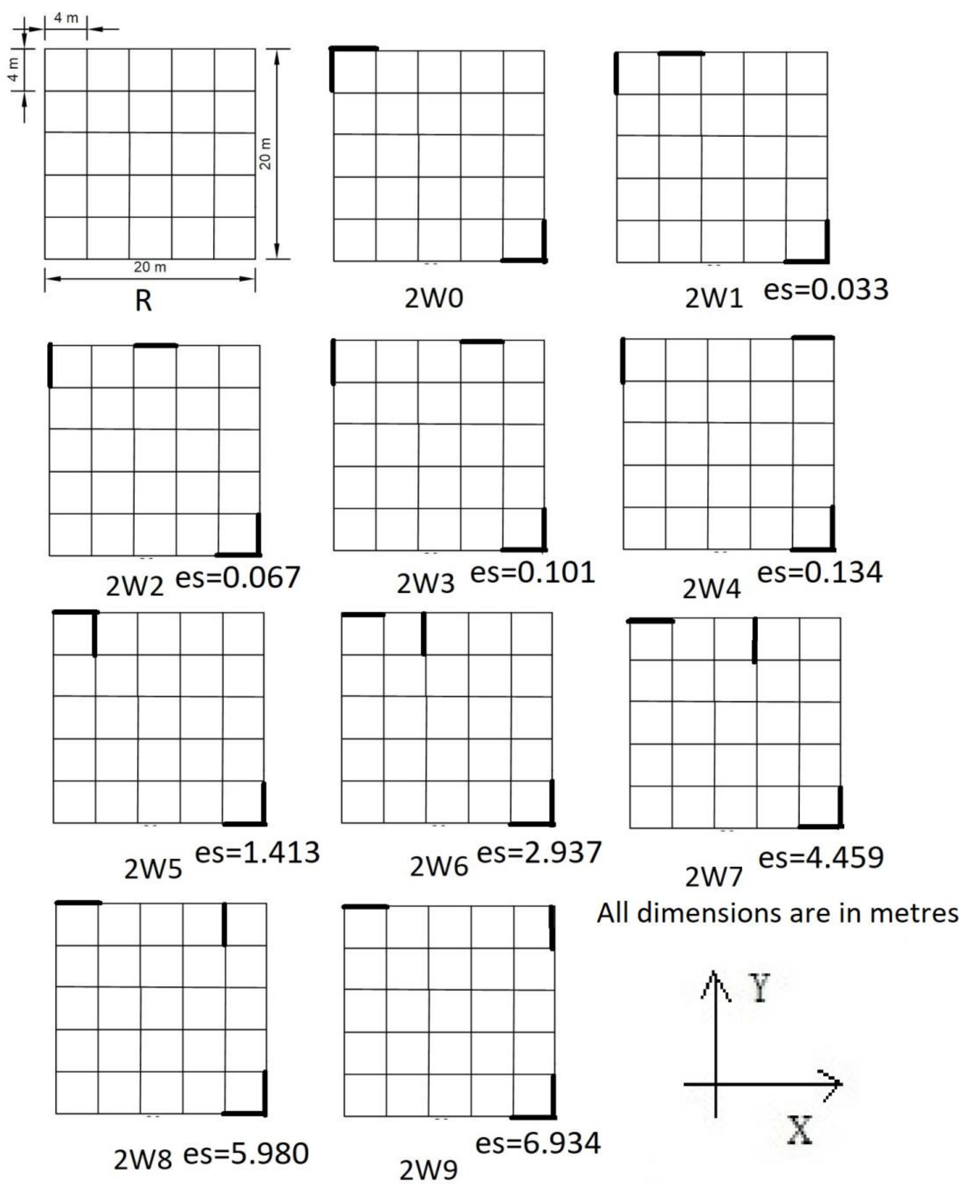


Figure 4.1 Plan layouts of 2W shear wall buildings

In 2W buildings with shear walls provided at the diagonal corners, the x wall or y wall were shifted along the x-axis to give irregular configurations 2W1 to 2W9 with x-directional static eccentricity (e_s). Considering 2W1 to 2W4, the x wall in the flexible side is re-located in the x-direction to the consecutive bays, in-plan eccentricity increases progressively and the e_s of 2W4 is 3.06 times as that of 2W1. From 2W5-2W9, it can be observed that the y wall is moved along the x-direction to the consecutive bays. In 2W9, e_s becomes 3.89 times as compared to that in 2W1.

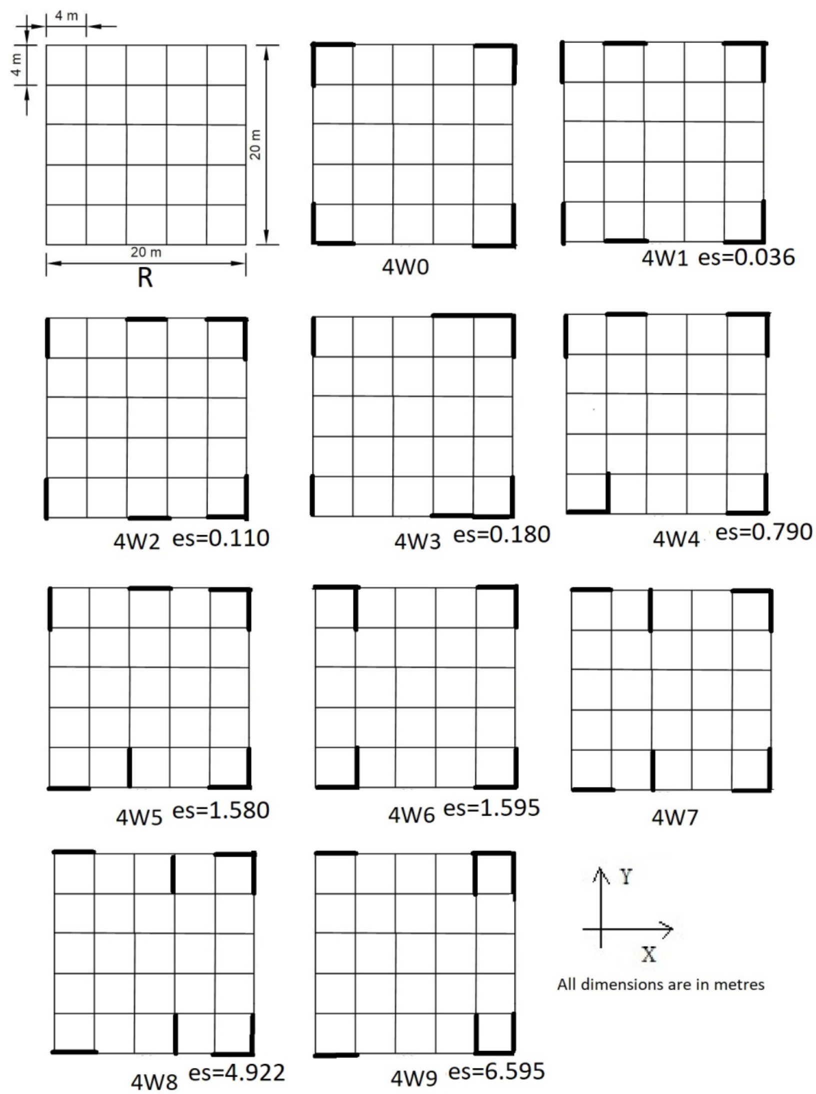


Figure 4.2 Plan layouts of 4W shear wall buildings

Whereas, in 4W buildings with shear walls at all four corners, x walls or y walls were shifted as a pair along the x axis to give irregular configurations 4W1 – 4W9 with x-directional static eccentricity. In the case of 4W1 to 4W3, a set of two parallel x walls are shifted in the x direction, in sequence and the eccentricity of 4W3 is 4 times as that of 4W1. Further, from 4W6 to 4W9, a set of two parallel y walls are re-located along the plan in the x-direction, wherein e_s in 4W9 increases by 3.13 times as that of 4W6. In 4W4 and 4W5, a set of x wall and y wall is positioned in the two consecutive bays in the x-direction, e_s increases by 33%. It can be observed that in general, when the walls aligned about an axis is shifted in the opposite direction the resultant eccentricity is higher in comparison to when it is moved in the direction parallel to its axis. The static and dynamic eccentricities of all the irregular shear wall configurations considered are as shown in Table 4.1.

Table 4.1 Dynamic eccentricities of the shear wall building configurations

Building configuration	e_s (m)	e_d (m)	e_d/L	Building configuration	e_s(m)	e_d(m)	e_d/L
2W0	0.000	1.000	0.050	4W0	0.000	1.000	0.050
2W1	0.033	1.050	0.052	4W1	0.036	1.054	0.053
2W2	0.067	1.101	0.055	4W2	0.110	1.165	0.058
2W3	0.101	1.152	0.058	4W3	0.180	1.270	0.064
2W4	0.134	1.201	0.060	4W4	0.790	2.185	0.109
2W5	1.416	3.124	0.156	4W5	1.580	3.370	0.169
2W6	2.937	5.406	0.270	4W6	1.595	3.393	0.170
2W7	4.459	7.689	0.384	4W7	3.263	5.895	0.295
2W8	5.980	9.970	0.499	4W8	4.922	8.383	0.419
2W9	6.934	11.401	0.570	4W9	6.595	10.893	0.545

Eighteen different plan configurations with in-plan eccentricity due to arrangement of shear walls were generated. 2W1 to 2W9 and 4W1 to 4W9 are the plan eccentric configurations and 2W0 and 4W0 are the symmetric configurations with L-shaped shear walls in the diagonally opposite corners and all four corners respectively. A total of 63 different buildings belonging to the three groups were thus generated and analyzed to evaluate the effect of torsional irregularity in the seismic behavior of shear wall buildings. Figure 4.3 shows FEM models of the regular shear wall buildings 2W0, 4W0 and irregular shear wall buildings, 2W5 and 4W7 of group B generated in LS-DYNA.

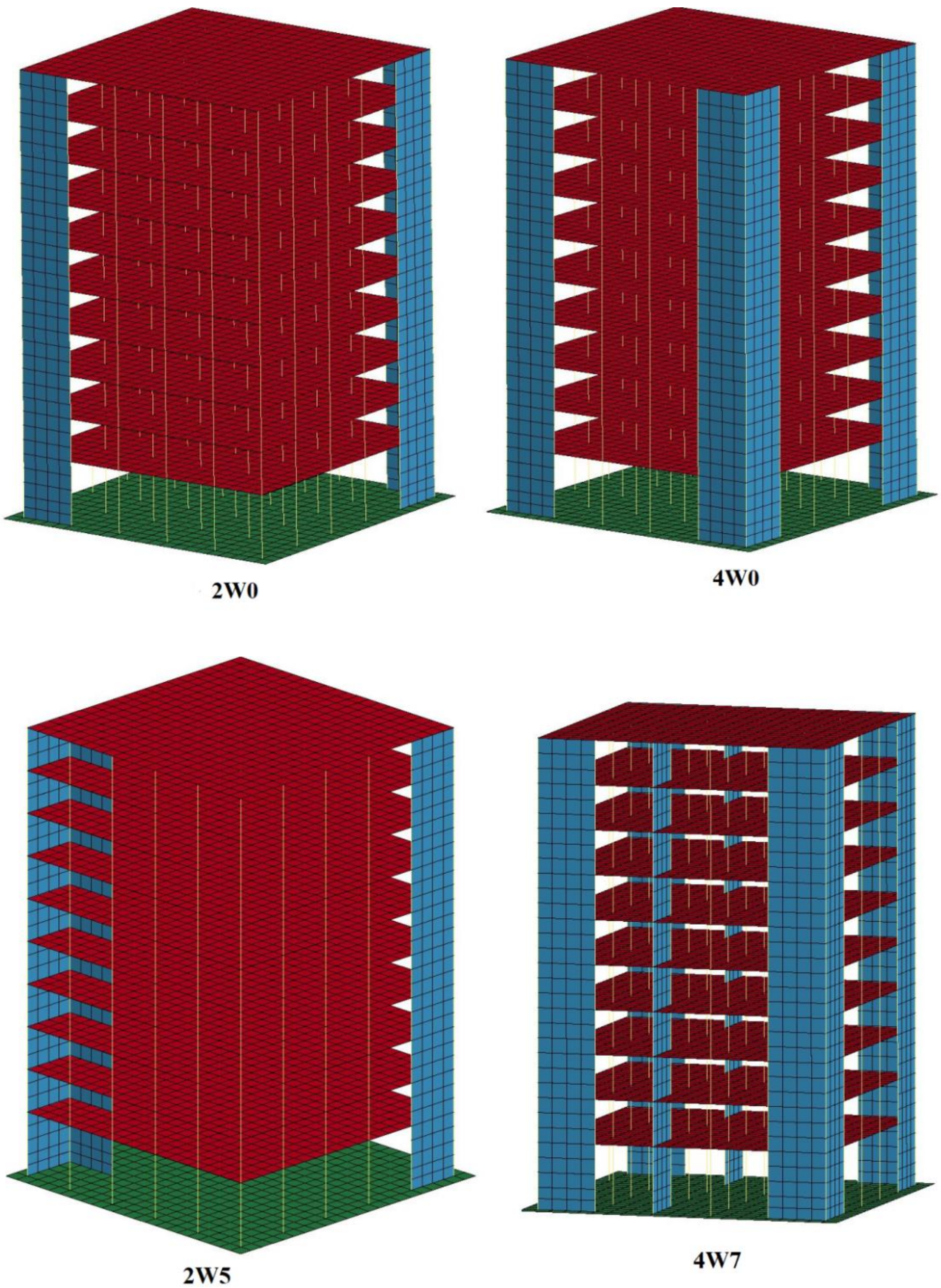


Figure 4.3 FEM models of the shear wall buildings in group B

4.2.2. Mass irregular buildings

Further to isolate the effect of in-plan eccentricity due to mass irregularity alone, mass irregular buildings with varying mass ratios, aspect ratios and location of masses along the height were idealized. Three-dimensional finite element models of building frames of 5, 10 and 15 storeys and plan dimensions 16 m x 16m (aspect ratio 0.937, 1.875 and 2.813) with 4 bays in each direction were considered. Mass ratios of 1.25, 1.5, 1.75, 2, 3, 4 and 5 were considered to be placed at the bottom, middle and top floor levels of the buildings. The mass ratio is defined as the ratio of the seismic weight of the floor considered to the seismic weight of the floor below. The highest mass ratio of 5 was considered such that even the equivalent distributed mass in three adjacent storeys together also causes vertical irregularity as per IS 1893:2016 code provisions. The mass density of concrete was taken as 2500 kg/m^3. The mass density of the slab was varied at different floor levels as well as at different locations in the plan, to represent different mass ratios along the plan without any variation in stiffness of the structure.

Since IS 1893:2016 limits the irregularity in mass of a particular storey to 1.5 times of the mass of storey below it, the discussion on the range of mass ratios considered in this study is bifurcated into two, the first with nominal mass ratios 1.25, 1.5, 1.75 and 2 and the second which comprises of buildings with higher mass ratios 2, 3, 4 and 5. Further, two major sets of buildings with uniformity and non-uniformity of distributed mass on a floor (without and with in-plan eccentricity) were studied. Figure 4.4 schematically represents the first set of buildings showing the location of mass irregularity in the elevation of the buildings by dark solid lines. Similarly, Figure 4.5 represents the second set of buildings showing the distribution of this mass irregularity with filled up areas in the plan of the corresponding storey. The 5, 10 and 15 storey building frames were categorized as group A, group B and group C and the regular buildings in each group were designated as 5R, 10R and 15R respectively. In the case of buildings with higher mass ratios M2 to M5, mass irregularities at the top, middle and bottom of the groups B and C building frames were distributed among two and three floors (2B and 3C) respectively keeping the total mass ratio constant with that of the single floor levels cases as shown in Figure 4.4. The groups designated as 2B and 3C refer to the buildings with higher mass ratios and have the same mass ratio as B and C itself but the same mass being distributed among two adjacent floors (2B) and three adjacent floors (3C) respectively, maintaining the same total seismic weight in each category. 'R' corresponds to the regular frame buildings without eccentricity; 'IM0' corresponds to the frame with vertical mass irregularity but without any in-plan mass eccentricity i.e. mass distributed uniformly throughout the entire area of the floor slab. In the designation of buildings, 'b', 'm', 't' corresponds to the bottom, middle and the top location floor levels along the height of the building.

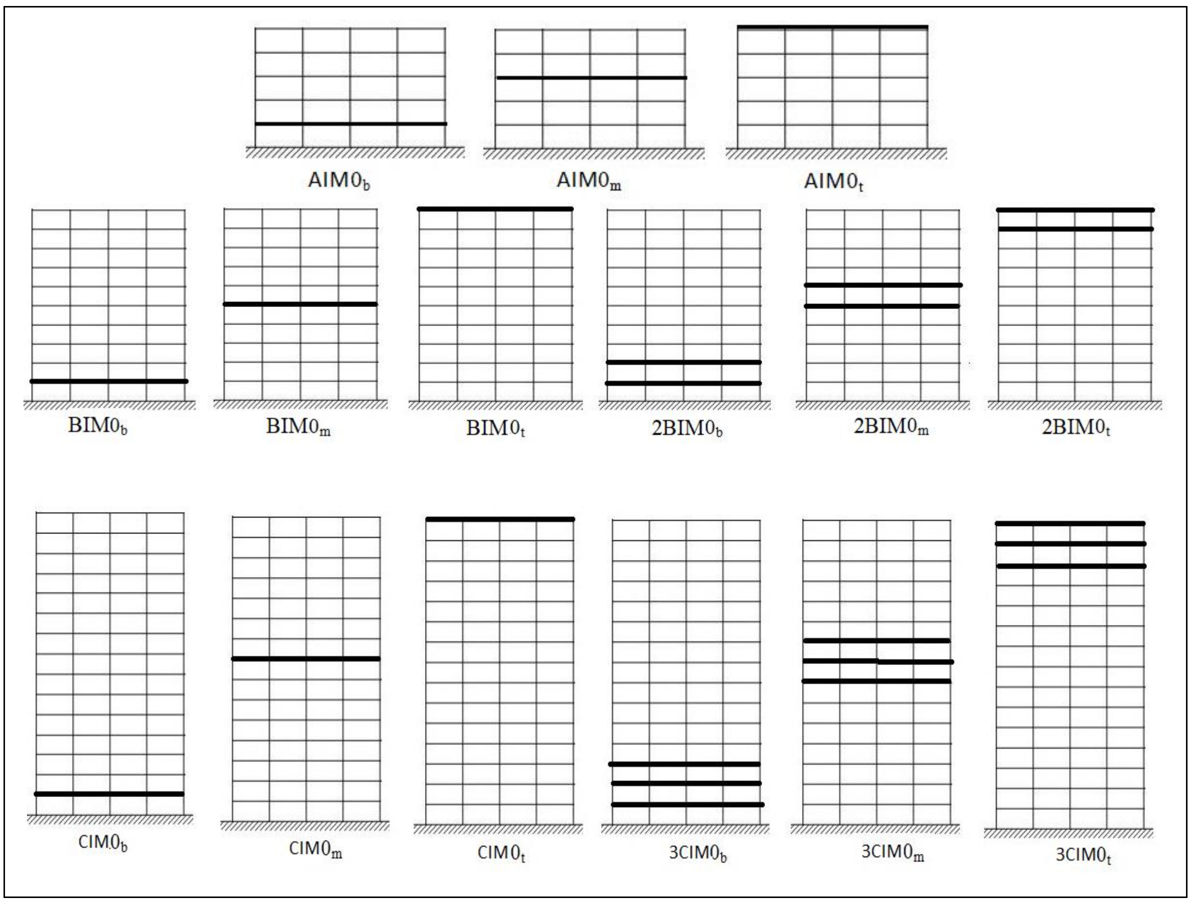


Figure 4.4 Elevation of group A, group B and group C buildings with mass irregularities at the bottom, middle and top floor levels

A second set of buildings in which the mass is concentrated at different locations in the plan for all the above groups of buildings were considered. Herein the irregularities were generated along the plan, in the initial set of buildings as shown in Figure 4.5 having vertical irregularities. Masses of varying mass ratios from 1.25 to 5 were provided in different patterns with varying eccentricity keeping the total seismic weight of the entire configurations belonging to a particular mass ratio as constant. The initial mass density of 2500 kg/m^3 in the floor slab was increased in portions along the floor slabs in three different patterns as shown in Figure 4.5 to generate in-plan eccentricity. The configurations IM1 to IM3 correspond to the three different patterns with increasing in-plan eccentricities 'AIM1$_b$' indicates a 5 storey building (Group A) with the mass irregularity provided at the bottom level having in-plan eccentricity pattern IM1. M1.25 to M5 represents the mass ratio of 1.25 to 5.

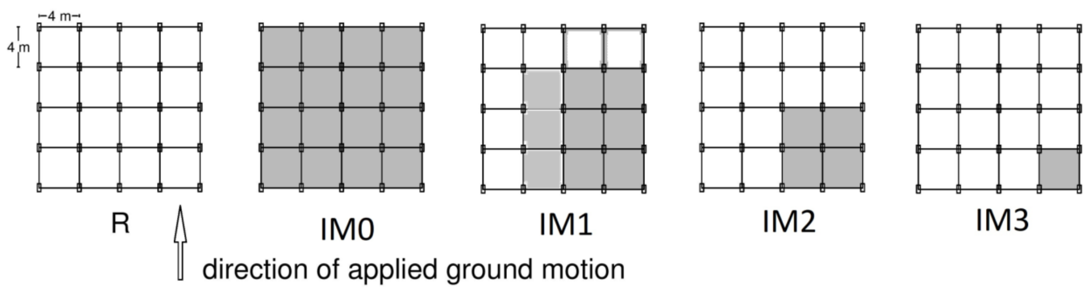


Figure 4.5 Plan layouts of the building frames depicting the placement of masses in a floor

The design eccentricities (e_d) of the buildings were calculated as per IS 1893:2016 and are represented in terms of the total plan width (L) for the buildings of mass ratios varying from M1.25 to M5 as Table 4.2.

Table 4.2 Dynamic eccentricities of the mass irregular building configurations

Building configuration	Dynamic eccentricity ratio (e_d/L)						
	M1.25	M1.5	M1.75	M2	M3	M4	M5
IM1	0.060	0.063	0.074	0.082	0.110	0.130	0.144
IM2	0.072	0.087	0.106	0.124	0.178	0.215	0.243
IM3	0.094	0.110	0.139	0.165	0.245	0.301	0.342

Figure 4.6 shows the FEM models of IM3 buildings with mass irregularities at the top floor level belonging to group B and Figure 4.7 shows the FEM models of IM3 buildings with mass irregularities at the top floor level belonging to group 2B.

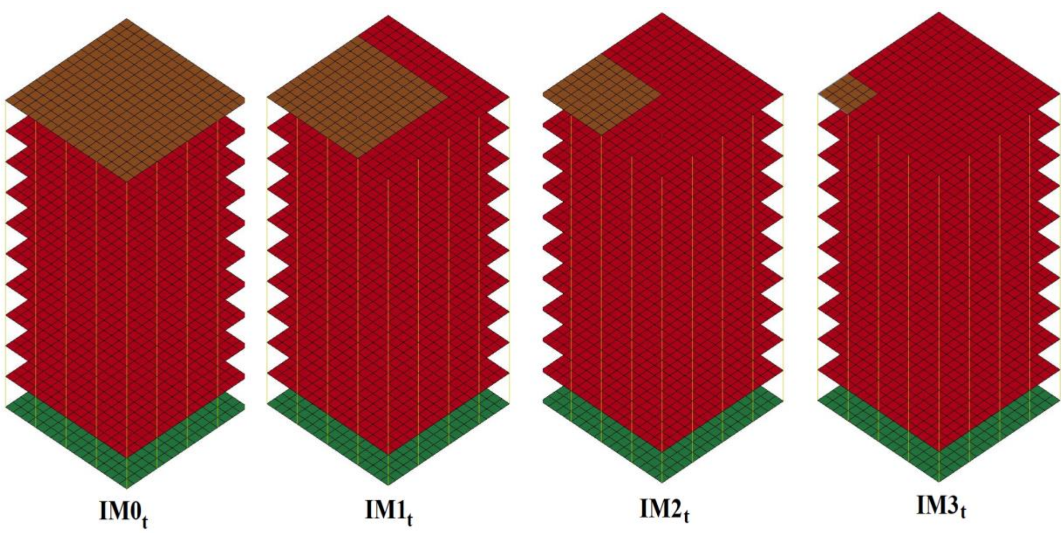


Figure 4.6 FEM models of the buildings in group B with mass irregularity at top floor

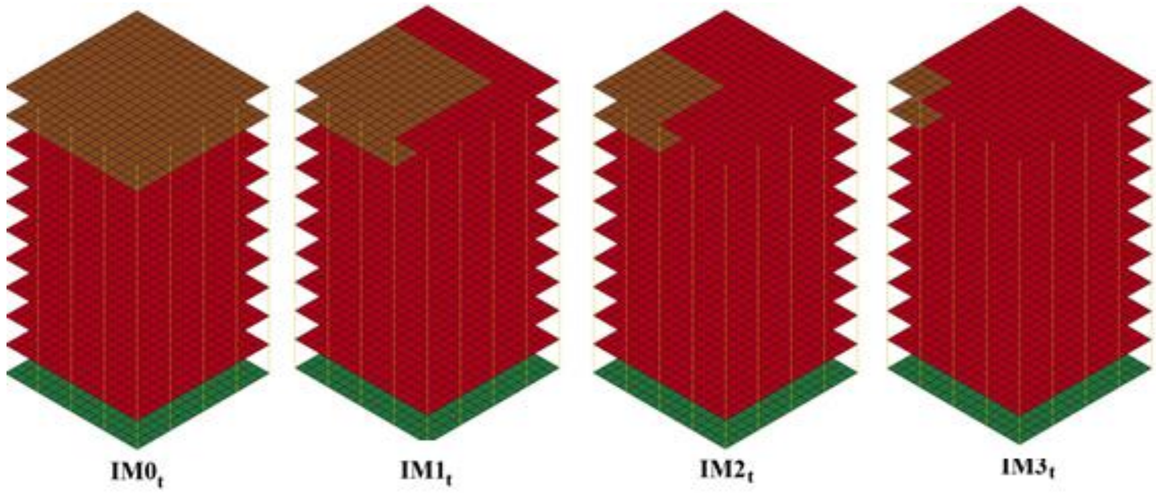


Figure 4.7 FEM models of the buildings in group 2B with mass irregularity at top floor

4.2.3. Stiffness irregular buildings

Seismic response variations in buildings due to stiffness irregularity is studied in the third part of the study wherein, stiffness irregularities were provided at the bottom, middle and top floor levels in the 5, 10 and 15 storey building frames designated as the AIS0, BIS0 and CIS0 respectively. The regular frame buildings with irregularities are designated as 5R, 10R and 15R similar to the other cases of irregularities. As per IS 1893:2016, the criterion of vertical stiffness irregularity or soft storey irregularity is considered to exist when the stiffness of a storey is less than that of the storey above it. As per FEMA 450, ASCE 7-16, a soft storey is one in which the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average stiffness of the three stories above. A change in inter-storey height along the height of the building results in a change in the storey stiffness. Relationships between the storey stiffness due to a modified inter-storey height, h_m can be obtained for various types of lateral force-resisting systems as in Sadashiva et al. (2011). Here, the modified lateral stiffness at a particular storey, K_m, is given by Equation 4.3 as the product of the stiffness modification factor corresponding to the lateral force-resisting system and the initial lateral stiffness at the chosen storey, K_o.

$$K_m = \left[\frac{h_o}{h_m}\right]^3 K_o \tag{4.3}$$

The nominal height of storey (h_o) was taken as 3m. Four inter-storey height variations were provided in the buildings as shown in Table 4.3 to give modified stiffness, K_1 to K_4. The storey heights at the bottom, middle and top floor levels were varied from 3m to 3.25m, 3.5m, 3.75m and 4m to generate modified stiffness K_1 to K_4.

Table 4.3 Modified storey stiffness

Modified stiffness	**Modified inter-storey height (h_m) (m)**	**Modified stiffness $(h_o/h_m)^3K_0$**
K_1	3.25	0.79
K_2	3.5	0.64
K_3	3.75	0.51
K_4	4	0.43

This forms the first set of buildings with three different aspect ratios 0.937, 1.875 and 2.813 with modified storey stiffness K_1 to K_4 located at the top, middle and bottom levels. These buildings without any in-plan eccentricity were designated as AIS0, BIS0 and CIS0 in group A, group B and group C buildings respectively. Further a subscript notation, 'b', 'm' or 't' corresponding to bottom, middle or top floor level was also given to indicate the location of irregularity. Therefore $CIS0_m$ corresponds to the 15 storey vertically irregular building with soft storey at the middle floor level. The elevation of the regular buildings 5R, 10R and 15R along with the plan regular configurations in group A, group B and group C are schematically represented in Figure 4.8. A second set of buildings was considered in which torsional irregularities were incorporated with the initial set having soft stories at different locations along the height. To incorporate in-plan stiffness eccentricity, column dimensions were varied in a particular fashion about a central Y axis, keeping the center of mass constant. Numerous cases with in-plan stiffness irregularities were generated in this manner, out of which, a group of 8 configurations designated as IS1- IS8 with dynamic eccentricities (e_d) in the range of 0.05 to 0.3 in terms of the total plan width (L) were considered in the study.

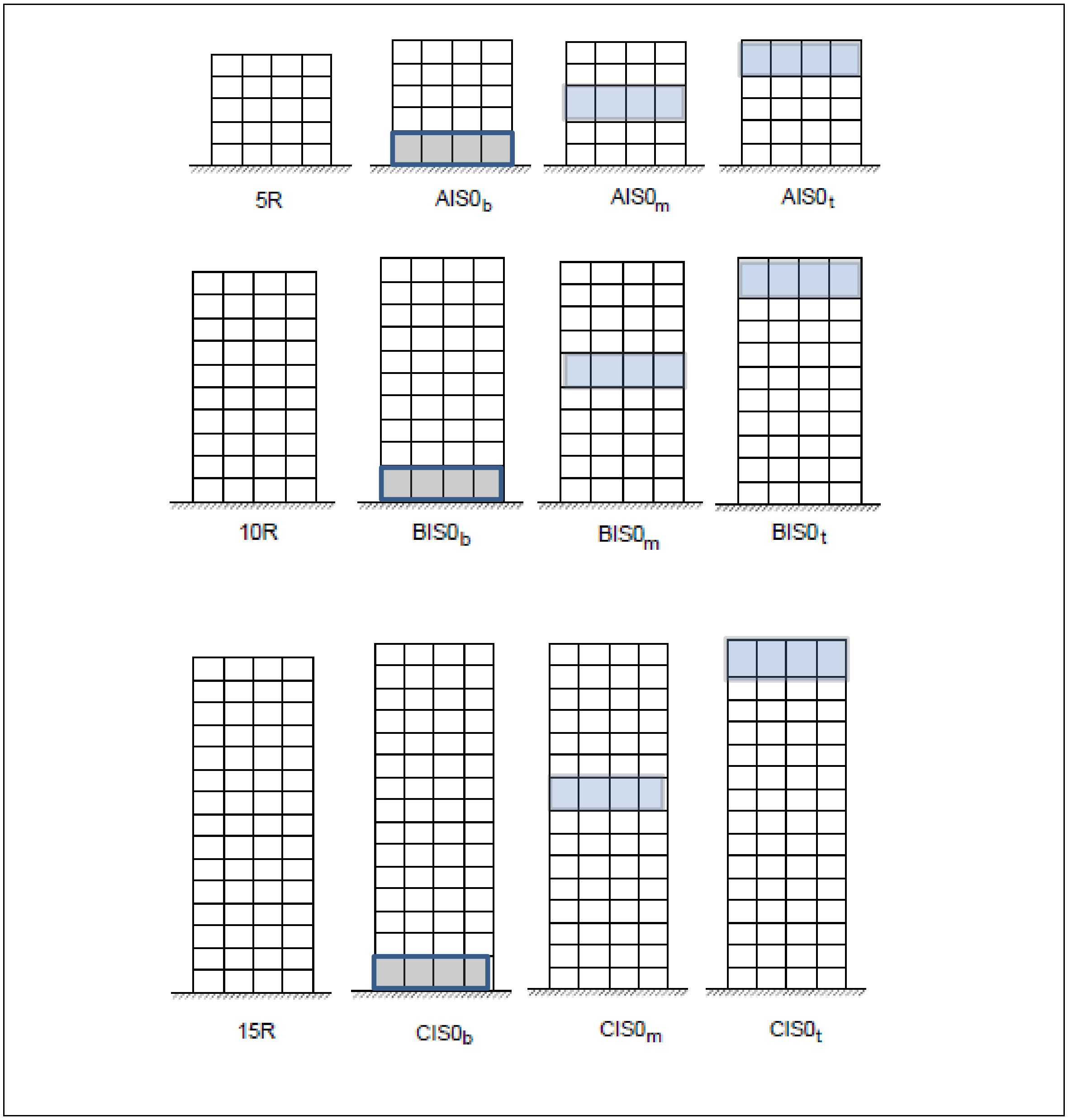
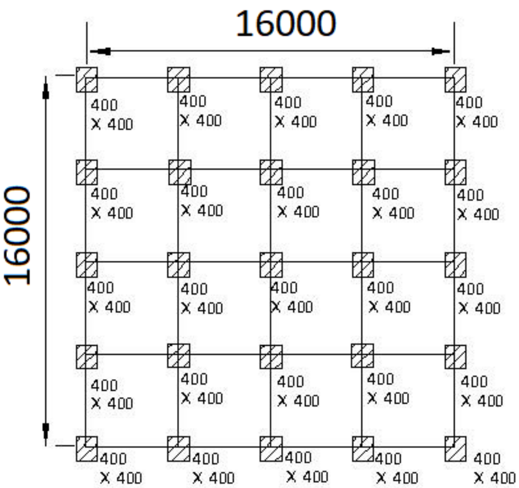
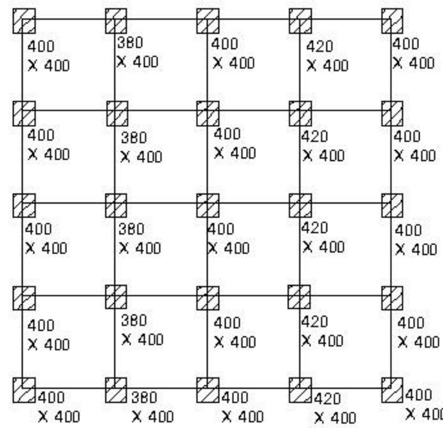


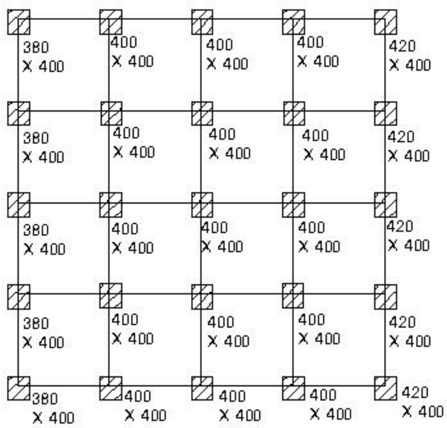
Figure 4.8 Elevation of plan regular buildings of group A, group B and group C



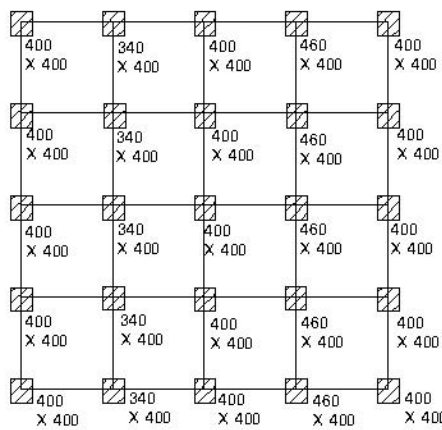
AIS0



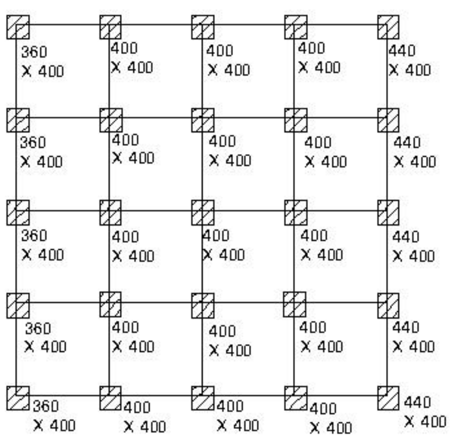
AIS1



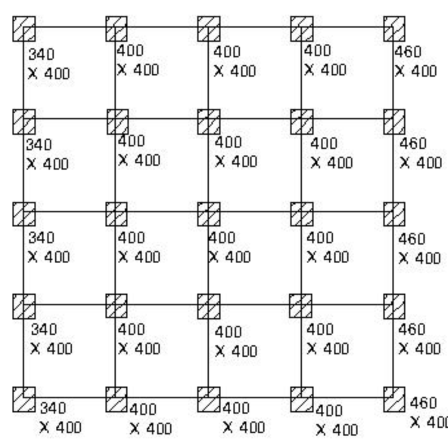
AIS2



AIS3



AIS4



AIS5

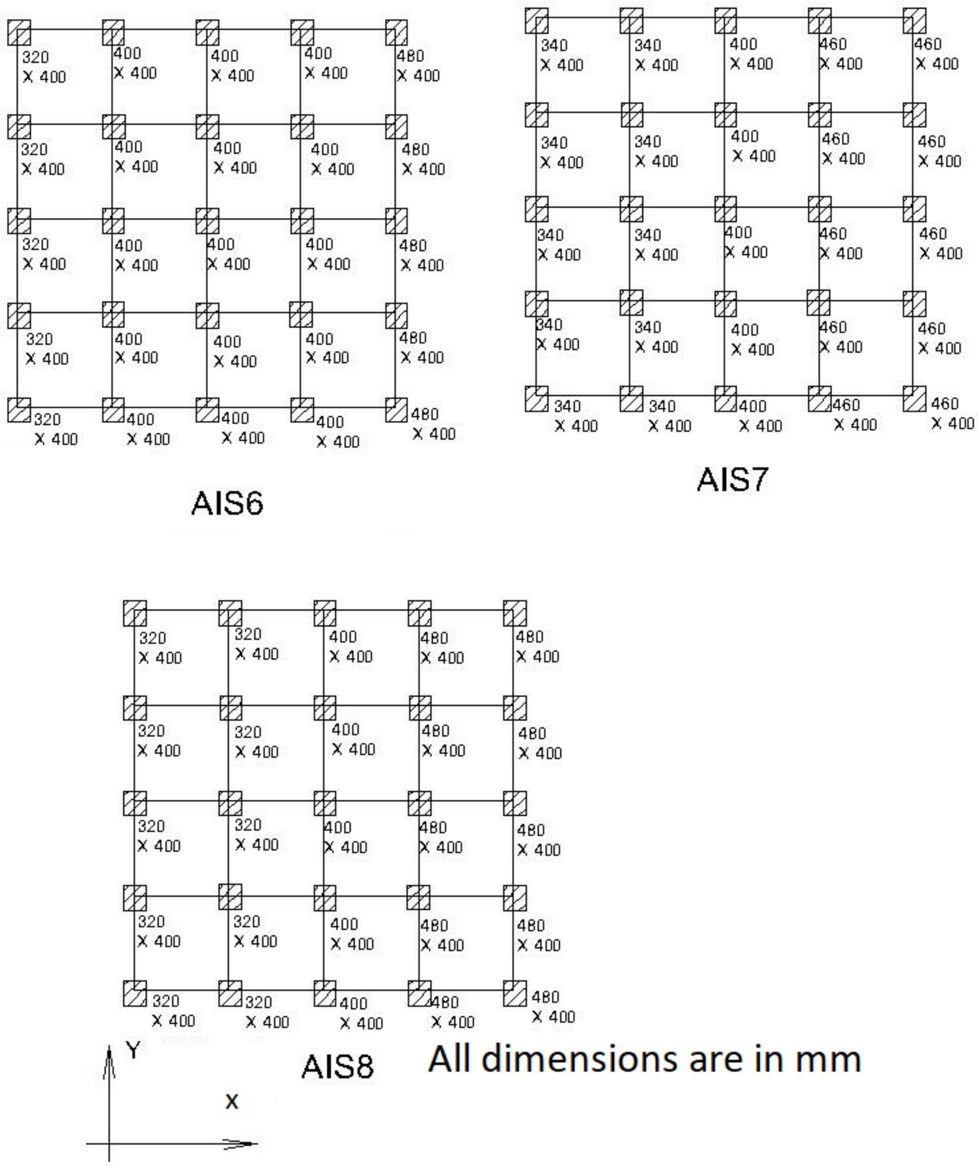


Figure 4.9 Plan layouts of the stiffness irregular buildings in group A

Figure 4.9 shows the in-plan stiffness irregular configurations AIS1 to AIS8 belonging to group A generated from the plan regular configuration IS0 with initial column size 400mm x 400mm. The dynamic eccentricities of the stiffness irregular buildings are given in Table 4.4. Three height variants namely, A, B and C groups with stiffness modifications K_0 to K_4 and eccentricity variants IS0 to IS8 were considered in totality.

The nomenclature of the building models indicated the height of the building, location and in-plan eccentricity of the irregularities, for example, AIS4$_t$ denoted the 5 storey building with in-plan eccentricity of 0.14 e_d/L as well as stiffness modification at the top floor level.

Table 4.4 Dynamic eccentricities of the stiffness irregular building configurations

Building configuration	e_s	$(e_s/L)\%$	e_d	e_d/L
IS0	0.00	0.00	0.80	0.05
IS1	0.24	1.50	1.16	0.07
IS2	0.48	2.99	1.52	0.09
IS3	0.71	4.44	1.87	0.12
IS4	0.95	5.95	2.23	0.14
IS5	1.41	8.83	2.92	0.18
IS6	1.86	11.63	3.59	0.22
IS7	2.07	12.91	3.90	0.24
IS8	2.67	16.68	4.80	0.30

Figure 4.10 shows the building models of the stiffness irregular buildings BIS8$_b$, BIS8$_m$ and BIS8$_t$ with modified inter-storey height at the bottom, middle and top floor levels respectively.

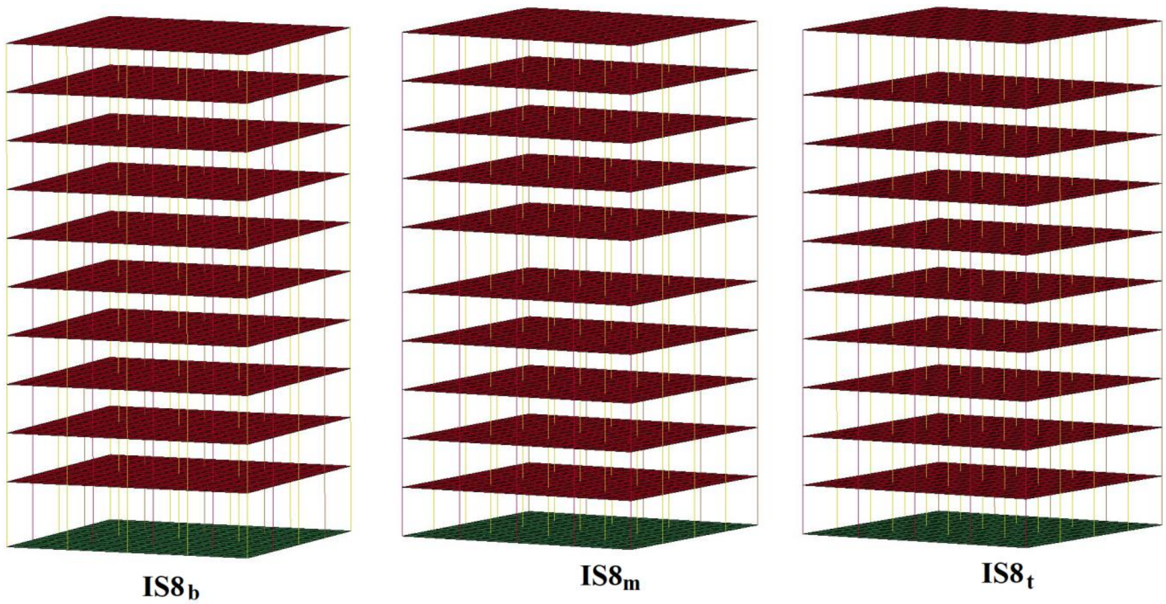


Figure 4.10 FEM models of the stiffness irregular buildings in group B

4.3. Eigenvalue analysis

The determination of the fundamental natural period is an integral part of the lateral load calculation of a building. Every building has it's sets of frequencies in which it starts vibrating if initiated by motion due to seismic or wind forces or due to any other building characteristics. The lowest associated frequency is termed as the natural frequency and the time period corresponding to this frequency is considered as the fundamental natural period. If the frequency contents of the earthquake is closer to the natural frequency of the building, more energy is introduced in the building and can lead to resonance. Buildings with shorter periods attract higher seismic forces whereas those with longer periods are susceptible to higher dynamic amplification. The most straightforward method to estimate the fundamental period is to employ the empirical formula for the determination of the approximate natural period as per the international codes. But in practice, the mode shapes of vibrations and the corresponding frequencies of a building are estimated by Eigenvalue analysis. In general, the discrepancies between the code estimated natural period and that obtained from the analysis is mainly due to the presence of non-structural claddings, infills or due to the stiffening effect of structural elements. In this study Eigenvalue analysis is carried out on the three-dimensional models generated using finite element software LS-DYNA in order to determine the natural period of vibration of the buildings and thus estimate the seismic load effects in the buildings.

4.4. Transient analysis

The transient analysis or the time history analysis of the buildings of different heights and varying location, magnitude and combinations of irregularities in the study were carried out using El-Centro (1940) earthquake data. The El-Centro (1940) earthquake (Imperial Valley earthquake) occurred on May 18 in the Imperial Valley of southeastern Southern California in the border region of the United States and Mexico. It had a moment magnitude of 6.9 and a maximum perceived intensity of X falling in the extreme category in the Mercalli intensity scale. It was located next to a fault rupture and is one of the strongest recorded earthquakes and caused widespread damage to structures and mainly irrigation systems. The El-Centro earthquake data is widely considered as the ground motion data in the seismic analysis since it has high amplitude frequency contents in the range of fundamental frequency of general RC frame buildings. In the present study, time history analysis was carried out on the group of regular and irregular multi-storey RC frame buildings of varying aspect ratios. Real bedrock ground motion corresponding to the longitudinal component of the Imperial Valley El-Centro (1940) earthquake with a PGA of 0.343g with a total duration of 60 sec was considered in the study to evaluate the seismic response parameters of the irregular buildings. The natural frequencies of all the buildings considered here lie in the range of 0.3Hz to 3.7Hz. The acceleration time history plot and Fourier spectrum plot of El-Centro earthquake data are given in Figures 4.11 and 4.12 respectively and it can be observed that this ground motion contains strong frequency contents in the range of natural frequency of the buildings considered in the study. The results of the time history analysis has been employed to assess the effect of the vertical irregularities with in-plan eccentricity on the seismic response of the frames in terms of fundamental natural period, base shear, roof deflection and roof rotation. A total of 63 shear wall frames with torsional irregularity, 375 mass irregular frames and 408 stiffness irregular frames were subjected to El-Centro ground motion to assess the effect of the irregularities on their seismic responses. Further, based on the results of time history analysis, irregularity indices are also developed to quantify the combinations of irregularities and equations are developed through regression analysis to predict the natural period and base shear of the irregular buildings.

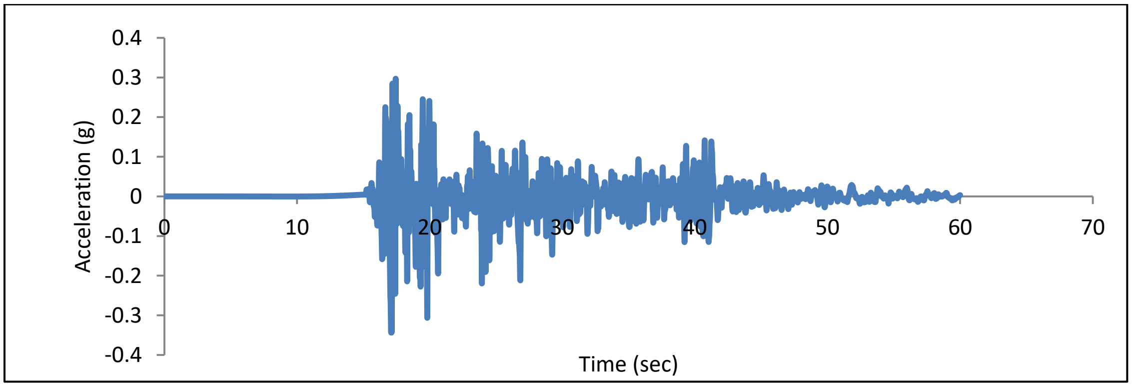


Figure 4.11 Acceleration time history plot of El-Centro ground motion

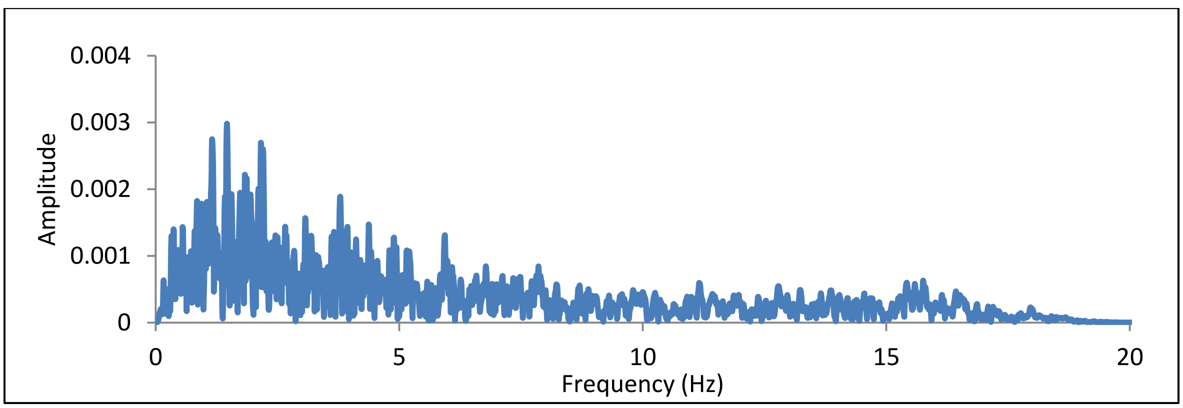


Figure 4.12 Fourier spectrum plot of El-Centro ground motion

Further two additional earthquake ground motions with frequency contents different from that of El-Centro was considered to analyse the critical cases of irregularities. This is to evaluate the responses of the buildings when subjected to earthquakes that do not exactly match with their natural frequency range and hence minimizing any possibilities of higher variations in responses due to resonance. Therefore, Kobe earthquake which has frequency contents of high amplitude below the range of natural frequency of the buildings and Koyna earthquake which has a high amplitude contents at a higher frequency range in comparison to the natural frequency of the buildings under study were also selected for further transient analysis.

The Great Hanshin earthquake or Kobe earthquake (1995) occurred on January 17, 1995, in the southern part of Japan. It measured 6.9 on the moment magnitude scale and had a maximum intensity of 7 on the JMA Seismic Intensity Scale. Among the major cities affected, Kobe was the closest to the epicenter and was hit by the strongest tremors. This was Japan's worst earthquake in the 20th century which claimed the lives of thousands of people. The Kobe earthquake data considered in the study has a PGA of 0.344g and duration of 40 sec. The time history of Kobe earthquake is shown in Figure 4.13 and the Fourier spectrum of Kobe earthquake is given in Figure 4.14. It can be observed that this ground motion has its peak and high amplitude contents in the range of 0.5Hz to 1Hz which is lower in comparison to that of frequency range of El-Centro earthquake.

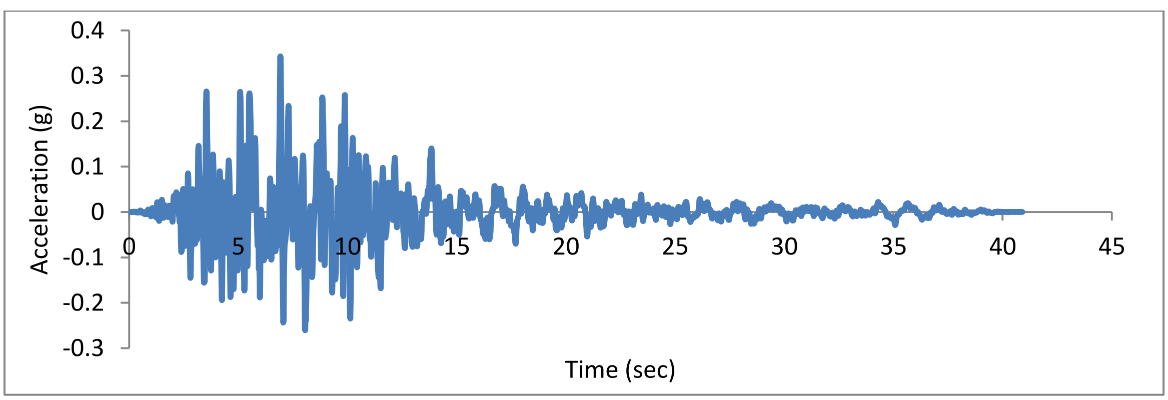


Figure 4.13 Acceleration time history plot of Kobe (1995) ground motion

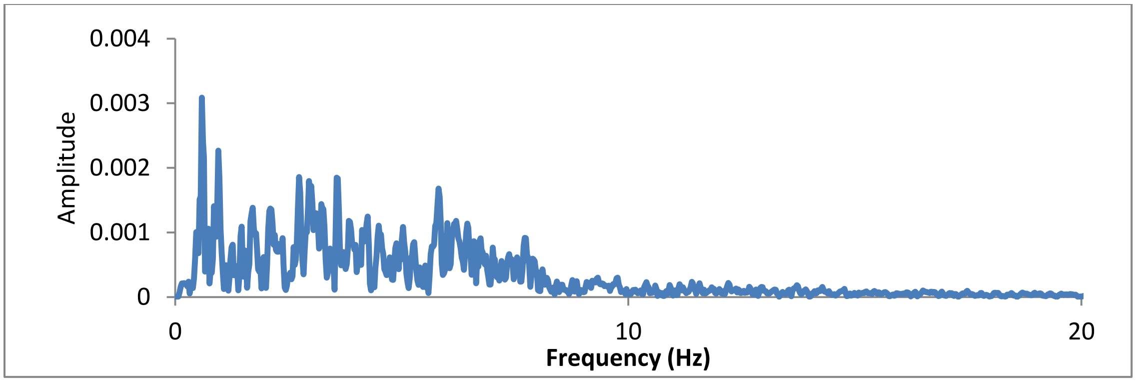


Figure 4.14 Fourier spectrum plot of Kobe (1995) ground motion

The Koyna earthquake (1967) occurred near the site of Koyna dam in Koynanagar town, Maharashtra, India on 11 December 1967 with a magnitude 6.6 and maximum Mercalli intensity of VIII. Ground motion corresponding to the longitudinal component of Koyna earthquake has a PGA of 0.48g with a total duration of 32sec. The acceleration time history and Fourier spectrum of the Koyna ground motion are given in Figures 4.15 and 4.16 respectively and it can be observed that this ground motion has its peak and high amplitude frequency contents in the range of 3Hz to 4.5Hz which is a higher range as compared to the frequency range of the considered buildings.

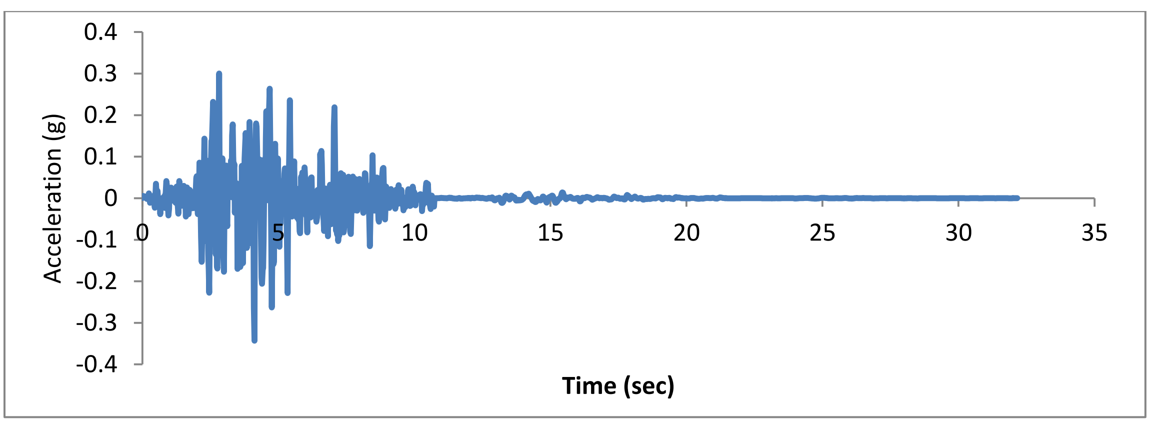


Figure 4.15 Acceleration time history plot of Koyna (1967) ground motion

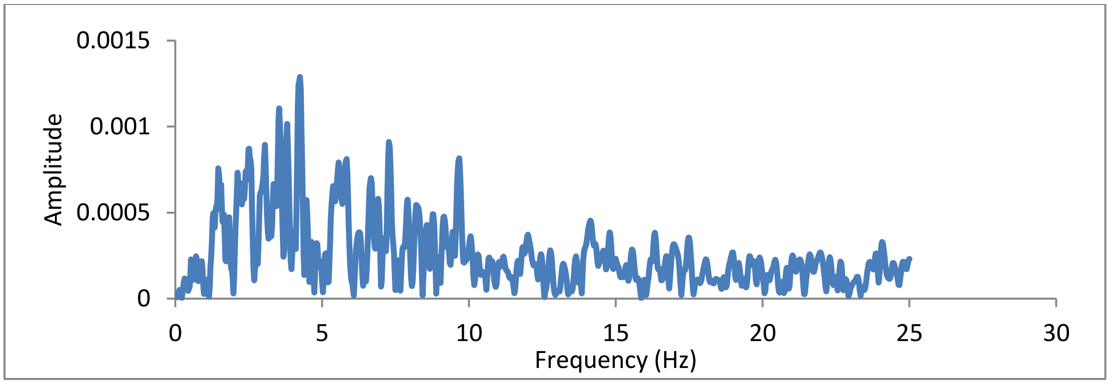


Figure 4.16 Fourier spectrum plot of Koyna (1967) ground motion

The effects of in-plan irregularities on the seismic responses of the considered irregular building configurations were evaluated with respect to El-Centro ground motion. Considering these responses, the critical cases of buildings with irregularities were identified and subjected to Kobe and Koyna ground motions, both scaled to 0.343g. Therefore the variations in responses of the critical irregular buildings subjected to the three ground motions were evaluated and studied.

4.5. Quantification of irregularity

Several researchers have attempted to quantify the regularity or irregularity of a building in different approaches and procedures. The regularity of a building can be quantified using regularity or irregularity coefficients or indices and few of the significant indices suggested by various researchers are listed here. Many irregularity studies on stepped building frames have been carried out based on which irregularity indices for such buildings have been developed. Karavasilis et al. (2008) had proposed storey-wise and bay-wise irregularity indices as follows:

$$\boldsymbol{\Phi_s} = \frac{1}{n_s - 1}\sum_{1}^{n_s-1}\frac{L_i}{L_{i+1}} \qquad \boldsymbol{\Phi_b} = \frac{1}{n_b - 1}\sum_{1}^{n_b-1}\frac{H_i}{H_{i+1}} \tag{4.4}$$

Where n_s is the number of storeys and n_b is the number of bays at the first storey of the frame. H_i and L_i are the height and width of the i^{th} storey respectively. However, this applies solely to stepped building frames and does not give a measure of the overall irregularity in the building. Sarkar et al. (2010) also proposed regularity index (η) which was based on the dynamic response of the stepped building frame and is given below:

$$\eta = \frac{\Gamma_1}{\Gamma_{1,\mathrm{ref}}} \tag{4.5}$$

Where Γ_1 is the first mode participation factor for the stepped frame and $\Gamma_{1,ref}$ is the first mode participation for the regular frame. The code-defined approaches quantify the irregularity limits in terms of magnitude only, and the effect of the location of irregularity was ignored. Varadharajan et al. (2012b) proposed the irregularity index whoch can be applied to mass as well as stiffness irregular buildings based on the results of sensitivity analysis as:

$$\boldsymbol{\beta_r} = \sum_{j}^{k} \frac{\boldsymbol{P_i}}{\boldsymbol{P_r}} \tag{4.6}$$

where, β_r, P_i and P_r are the combinations of participation factor from the j^{th} to the k^{th} mode for irregular and regular buildings. These irregularity indices mainly depend on the dynamic response, the properties of the building system and the type of irregularities under consideration. In general, the value of the irregularity index is less than unity for irregular building systems, as the participation factor of an irregular building is always less than that of a regular building.

The quantification of mass irregularity is necessary to define the variation of the response parameters with respect to the location as well as the in-plan eccentricity of the vertical irregularity. An index was proposed for quantification of mass irregularity based on location of mass irregularity along the height by Varadharajan et al. (2015) as:

$$\eta_m = \frac{b}{L}\frac{H_i}{H}\frac{M_i}{M} \tag{4.7}$$

Where M_i is the mass of the irregular floor, M is the total mass of the building, H_i is the height of the irregular floor from the base of the building, H is the total height of the building, b is the plan width in the direction and L is the plan width transverse to the direction of seismic excitation.

As per the literature study, thorough evaluation of the effects of in-plan eccentricity in vertically irregular buildings and the researches on the quantification of such irregularities in combination has not been carried out. Based on the results of transient analysis, the significant parameters are identified in this study through non-linear regression analysis and irregularity coefficients are proposed separately for mass irregular and stiffness irregular buildings based on their geometrical dimensions and in-plan eccentricity.

4.6. Prediction of natural period of irregular buildings

The fundamental natural period is an intrinsic property of a building. Every building has many natural frequencies, at which the building has minimum resistance to shaking induced by any load including lateral loads like earthquake and wind. Each of these natural frequencies and the associated deformation shape of a building constitute a natural mode of oscillation. The mode of oscillation with the lowest natural frequency and highest natural period is called the fundamental mode with the fundamental frequency and the associated natural period is termed as the fundamental natural period. The fundamental natural periods of the regular and irregular buildings in this study were evaluated by carrying out Eigenvalue analysis on the building frames modeled in LS-DYNA software.

The Indian as well as the international seismic codes suggests the equation to estimate the approximate natural period of vibration of buildings based on its height. Regression analysis was carried out on the irregular buildings considered in the study and relations were developed between the natural period of a torsionally irregular building and to that of a regular one based on the in-plan eccentricity. It is also attempted to propose equations to predict the accurate fundamental natural period of any building with irregularity based on the approximate equations for natural period as per IS 1893:2016 and ASCE 7-16.

4.7. Summary

The methodology followed to evaluate the effects of in-plan irregularities in buildings was detailed in the chapter. The idealization and the general assumptions considered and the analysis carried out to study the seismic behavior of the buildings in terms of natural period, base shear demand, rotation and displacement characteristics were elaborated. The chapter presented the proposition put forward to accomplish the objectives outlined in the study.

CHAPTER 5

RESULTS AND DISCUSSIONS

The irregular buildings considered in the study comprises of (i) the irregular shear wall buildings with a combination of mass and stiffness variation and dynamic in-plan eccentricity in terms of plan width L, in the range of 0.052L to 0.57L, (ii) the set of mass irregular buildings with mass ratios 1.25 to 5 in combination with dynamic in-plan eccentricity in the range of 0.06L to 0.34L and (iii) stiffness irregular buildings with a stiffness reduction of 0.79 to 0.43 in combination with dynamic in-plan eccentricity in the range of 0.07Lto 0.3L. Building models of three different aspect ratios incorporated with three types of irregularities were analysed and the transient analysis responses in 846 irregular building frames were studied. The variations in dynamic responses of the buildings due to the inclusion of different types of irregularities at different locations along the height were evaluated and are expressed in terms of absolute maximum values of the fundamental natural period, base shear, roof deflection and roof rotation. Also an attempt has been made to quantify the irregularities in the buildings in order to predict the responses of irregular buildings and thus propose amendments for proper planning of irregularly oriented buildings.

5.1. Shear wall buildings

Shear wall buildings categorized into two sets based on shear wall ratios were modeled and eccentricities were incorporated in the plan of the buildings by changing the positions of the shear walls along a single direction and the variation of the seismic response parameters were studied with respect to that of the bare frame building and the shear wall regular building.

5.1.1. Variation in fundamental natural period

The fundamental natural period being an intrinsic property of buildings was determined in this study by carrying out Eigenvalue analysis on the generated building models. It can be observed from Figure 5.1 that among all the three groups of buildings, there is a similar variation in the fundamental natural period among the two sets of buildings with different shear wall locations. It can be observed that in general, the natural period of the buildings increases with an increase in the height or the aspect ratio of the buildings. The addition of shear wall improves the overall stiffness of the buildings and when symmetrically arranged in plan hence tends to decreases the natural period as compared to the corresponding bare frame buildings R. Among all the groups of buildings, the highest natural period is observed in the R buildings.

The presence of in-plan eccentricity in buildings leads to torsional effects and tends to make the buildings flexible when subjected to dynamic loading. Therefore in the case of buildings with in-plan eccentricity, change in position of shear walls increases natural period in comparison to the 2W0 and 4W0 buildings. The changes in shear wall ratio and in-plan eccentricity are the parameters which affect the natural period among a single group of buildings of a particular height.

Considering 2W configuration buildings with a shear wall ratio of 0.16, the maximum reduction in natural period with respect to that of the corresponding bare frame building is observed in 2AW0 as 37%. As eccentricity increases, the natural period increases and the variation in the natural period between the 2W buildings and that of R building decreases. Considering the difference in the natural period between buildings with in-plan eccentricity and bare frame ones, 2AW9 has a variation of 17% with respect to 5R. The natural periods of 2BW9 and 2CW9 are very close to those of 10R and 15R respectively. Further, due to increasing eccentricity from 2W1 to 2W9, 2W9 has the highest variation in natural period with respect to 2W0. 2W9 with e_d/L of 0.57 has 29%, 33% and 39% variation with respect to 2W0 in group A, group B and group C buildings respectively.

Further, examining the variation of in-plan eccentricity in 4W configuration buildings, the highest variation of natural period in symmetric shear wall building with respect to regular buildings is seen in 4AW0 as 55%. Taking into account the variation of natural period due to change in eccentricity of the configurations, the shear wall buildings with in-plan eccentricity, increasing e_d/L leads to an increase in natural period of the buildings. The highest variation due to in-plan eccentricity is observed in 4AW9 with respect to 4AW0 as 60%. In the case of B and C group buildings, similar variation is obtained as 58% and 45% respectively. The variation of the natural period of 4W9 configuration building as compared to that of the bare frame buildings is lower and the maximum variation of 29% is observed in 4BW9.

Comparing the variation in the natural period due to in-plan eccentricity in 2W and 4W buildings, for the same value of e_d/L, 4W buildings are observed to have smaller variations due to increasing e_d/L as compared to that of the 2W buildings. This is due to the higher shear wall ratio in 4W which tends to stiffen the buildings though the increasing eccentricity increases the ductility of the structure.

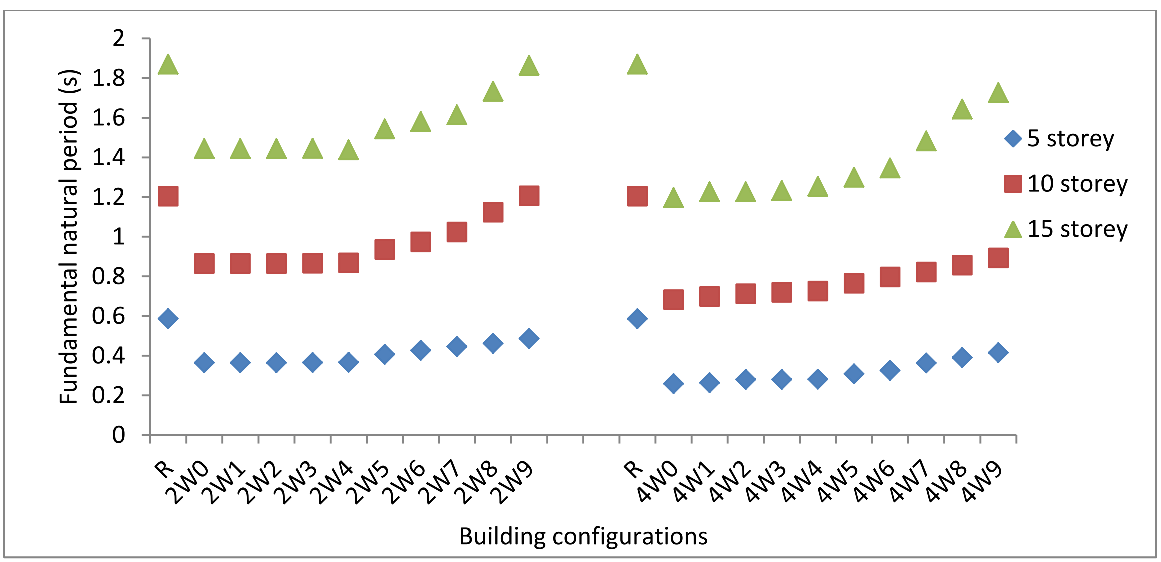


Figure 5.1 Variation in natural period of group A, group B and group C shear wall buildings

The frequency ratio, Ω which is the ratio of the frequencies of the building in the torsional mode to that in the translational modes decreases in all the three groups of buildings in the order of increasing e_d/L. 2W9 and 4W9 with the highest e_d/L among 2W and 4W buildings have the least Ω ratio among all the groups. Table 5.1 lists the frequency ratios of all the irregular buildings under 2W and 4W sets in all the three aspect ratio variants. For the configurations from 2W1-2W9, Ω is obtained in the range of 1.087 to 0.716 and for buildings 4W1-4W9, Ω is in the range of 1.036 to 0.775. Ω is less for the configurations with higher e_d/L and hence the buildings with the maximum torsional coupling are those with e_d/L of 0.545. Furthermore, the general pattern is that the Ω values of the buildings tend to decrease with an increase in the aspect ratio of the buildings and the variation of in-plan eccentricity has more influence on buildings with higher aspect ratios.

Table 5.1 Frequency ratios of the irregular shear wall building configurations

Building Configuration	Frequency ratio (Ω)			Building Configuration	Frequency ratio (Ω)		
	Group A	Group B	Group C		Group A	Group B	Group C
2W1	1.087	1.068	1.054	4W1	1.036	1.028	1.0185
2W2	0.995	0.986	0.981	4W2	0.984	0.981	0.975
2W3	0.979	0.975	0.969	4W3	0.976	0.973	0.971
2W4	0.962	0.954	0.952	4W4	0.939	0.935	0.931
2W5	0.921	0.918	0.911	4W5	0.903	0.893	0.890
2W6	0.846	0.831	0.822	4W6	0.875	0.872	0.867
2W7	0.812	0.803	0.795	4W7	0.852	0.847	0.841
2W8	0.787	0.785	0.783	4W8	0.824	0.818	0.809
2W9	0.735	0.724	0.716	4W9	0.798	0.781	0.775

5.1.2. Variation in seismic base shear ratio

Base shear, a very important parameter in the seismic analysis of buildings and is the maximum anticipated lateral force likely to occur at the base of a structure due to seismic ground motion. It directly implies the vulnerability of the building to an earthquake. Here the seismic base shear is normalized as a ratio of base shear with respect to the total seismic weight of the structure (W) as is termed as seismic base shear ratio. The seismic base shear ratio of the regular buildings is higher that the irregular configurations 2W1-2W5 and 4W1-4W5 in all the three aspect ratio variants as it can be observed from Figure 5.2. The provision of a shear wall considerably reduces the total lateral force acting on the structure under the application of ground motion. The seismic base shear ratio is observed to decrease with an increase in the aspect ratio and the 15 storey buildings are observed to have the lowest base shear ratio. The seismic base shear ratios of 5R, 10R and 15R are obtained as 0.1352W, 0.0981W and 0.0621W respectively. The in-plan regular shear wall frames 2W0 and 4W0 has a reduction in base shear ratio with respect to the bare frame R buildings. 2CW0 has the maximum decrease in the seismic base shear ratio of 52% with respect to the bare frame buildings. This is due to the stiffening effect on the structure due to the regular peripheral arrangement of the shear walls. Further due to increasing eccentricity, base shear also increases and 2CW9 has its base shear ratio higher by 17% with respect to R building. 2AW9 has the highest increase with respect to that of the regular bare frame as 31%. Considering the variation in maximum seismic base shear ratio due to the in-plan eccentricity of the frames, 2W9 frames with an e_d/L of 0.57 has the highest percentage increase of 43%, 52% and 89% in group A, group B and group C buildings respectively in comparison with 2W0 frames.

Similar variation is observed in 4W frames with shear wall ratio of 0.32 wherein the maximum decrease in seismic base shear ratio with respect to R building is observed in 4CW0 as 38% and the maximum increase is obtained in 4BW9 as 50.5% with respect to bare frame buildings. Further comparing the seismic base shears of buildings 4W1-4W9, with respect to 4W0, it can be seen that the maximum variation due to in-plan eccentricity in the frames is present in 4BW9 as 89%. In the case of both 2W and 4W buildings with the range of e_d/L from 0.1 to 0.5 causes significant and abrupt variations in seismic base shear among the entire set of irregular configurations. Figure 5.3Figure 5.4 shows the time history plots of base shear in 2W0, 2W9, 4W0 and 4W9 buildings in comparison with that of 15R building.

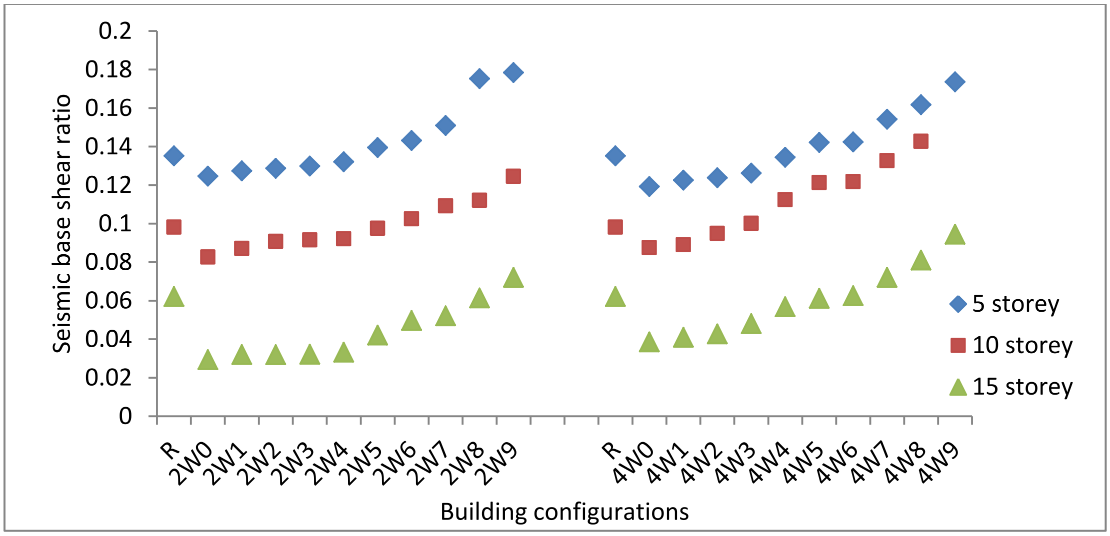


Figure 5.2 Variation in seismic base shear ratio of group A, group B and group C shear wall buildings

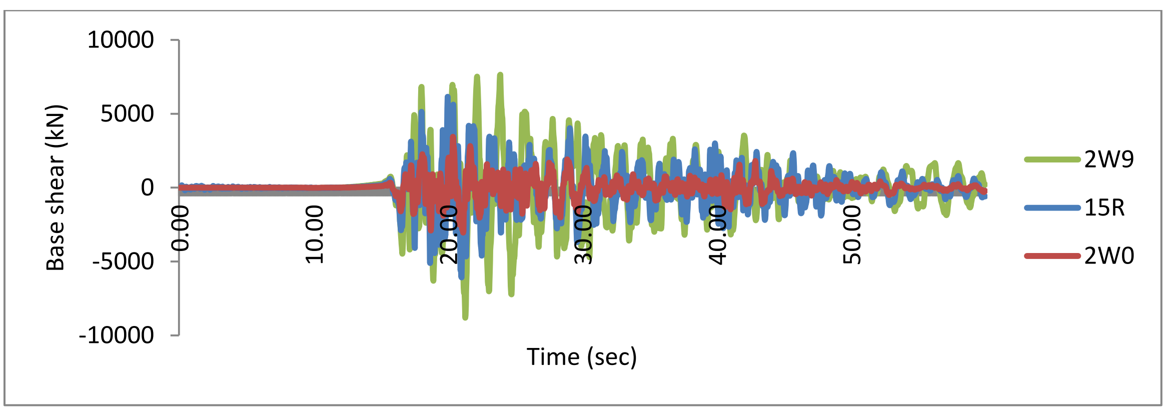


Figure 5.3 Time history plot of base shear of 2W9, 15R and 2W0 buildings

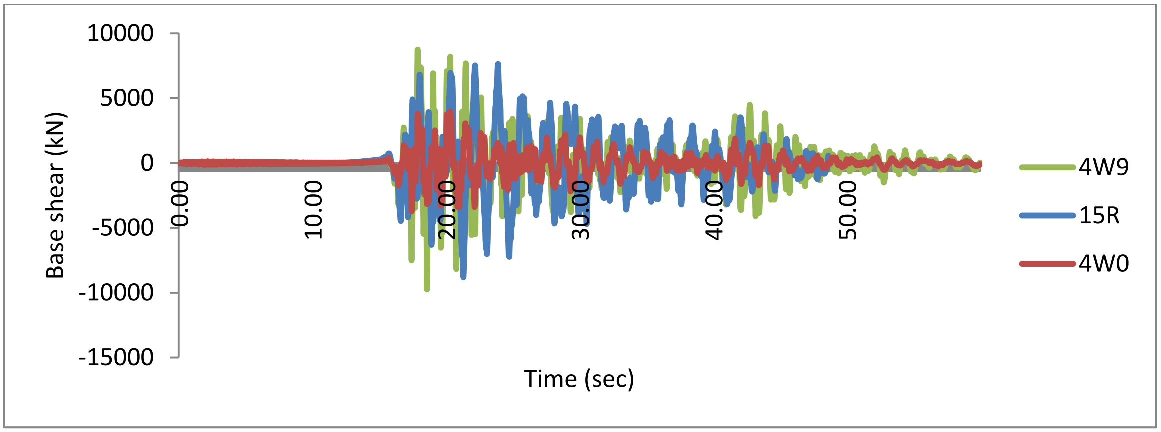


Figure 5.4 Time history plot of base shear of 4W9, 15R and 4W0 buildings

5.1.3. Variation in roof deflection ratio

The lateral deflection of the roof of a building with reference to its fixed base is referred to as roof deflection. The roof deflections of the buildings are expressed in terms of the total height of the buildings (H) as in Figure 5.5. Roof deflections of the buildings increases with an increase in the aspect ratio and the 15 storey buildings have the highest roof deflection among all the groups. Bare frame buildings have higher deflection ratios in comparison to the regular shear wall buildings due to the stiffening effect of the shear walls which tend to reduce the deflection in a building. The roof deflection ratios of the bare frame buildings are obtained in the range of 0.0042H to 0.0094H. The decrease in roof deflection due to the presence of shear walls with respect to the bare frame configuration in 2CW0 is 58% which is the highest among all the 2W configurations. Considering the configurations with in-plan eccentricity the roof deflection ratios increases proportionally with the increasing e_d/L. The variation in roof deflections due to the presence of eccentricity is nominal in the case of group A buildings or the variation in roof deflection due to increase in in-plan eccentricity becomes prominent in buildings of higher aspect ratios. Considering the variation of in-plan eccentric buildings with respect to bare frame ones, 2W1-2W4 have lower roof deflection and 2W5-2W9 have higher roof deflection in comparison to the bare frame buildings. Among 2W buildings, the highest variation in roof deflection with respect to regular 2CW0 building is observed in 2CW9 as 62%. Group B and group C buildings have more shifts in roof deflection ratios with respect to the increase in in-plan eccentricities.

The variation of roof deflection due to in-plan eccentricity in the buildings belonging to the set of 4W buildings to that of 2W buildings is similar but the variation in the deflection responses are comparatively lower due to the higher shear wall ratio of the 4W buildings. 4AW0 has maximum reduction of 22% in the roof deflection ratio with respect to that of the regular buildings due to the stiffening effect provided by the shear walls with a ratio of 0.32. Considering the irregular buildings 4CW9 has the highest increase of 45% in roof deflection ratio with respect to the bare frame buildings. Evaluating the variation in roof deflection ratio in the plan irregular 4CW0 buildings due to in-plan eccentricity, the least variation is obtained in 4AW9 as 8.5% and the highest is observed in 4CW9 as 59% with respect to the corresponding 4W0 buildings respectively. Figures 5.6 and 5.7 show the time history plots of roof deflection in 2W0, 2W9, 4W0 and 4W9 in comparison with that of 15R.

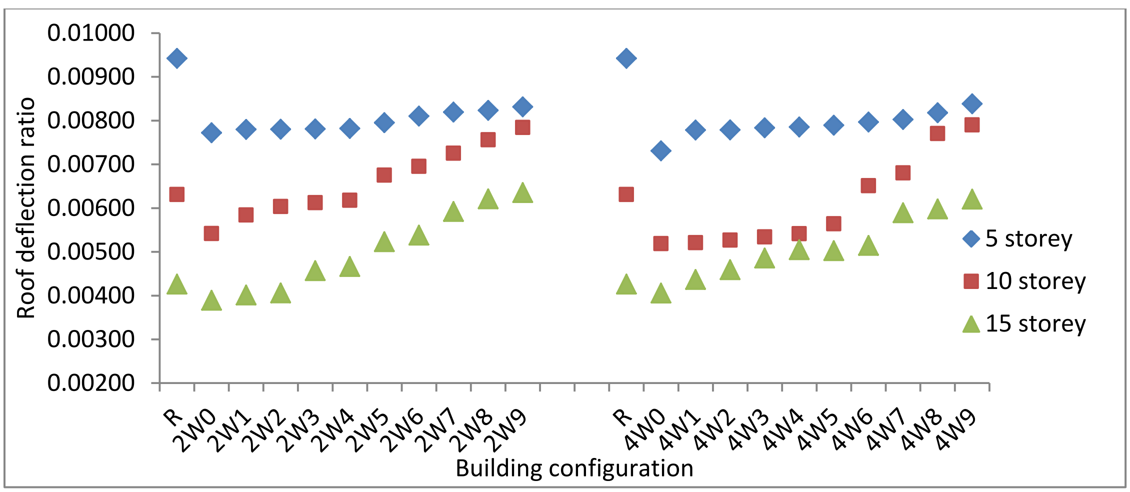


Figure 5.5 Variation in roof deflection ratio of group A, group B and group C shear wall buildings

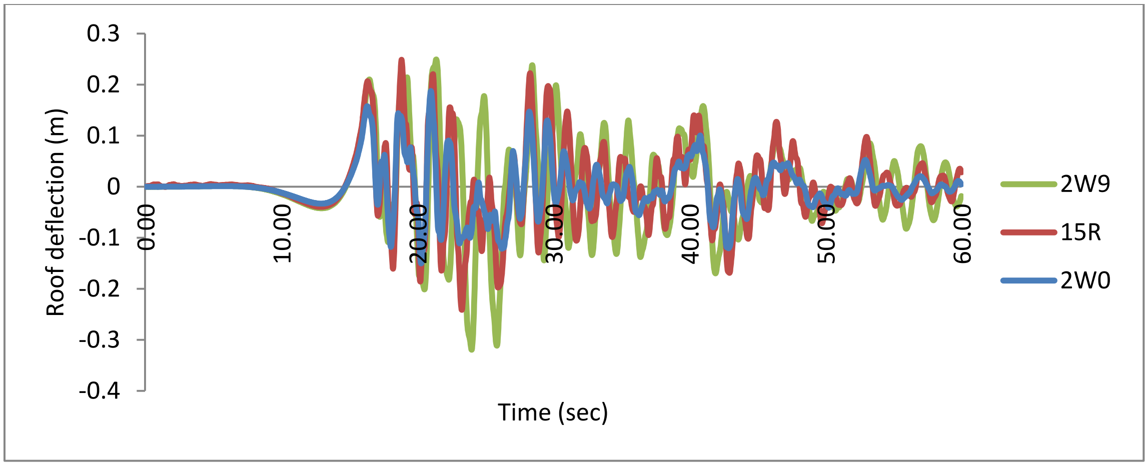


Figure 5.6 Time history of roof deflection in 2W0, 2W9 and 15R buildings

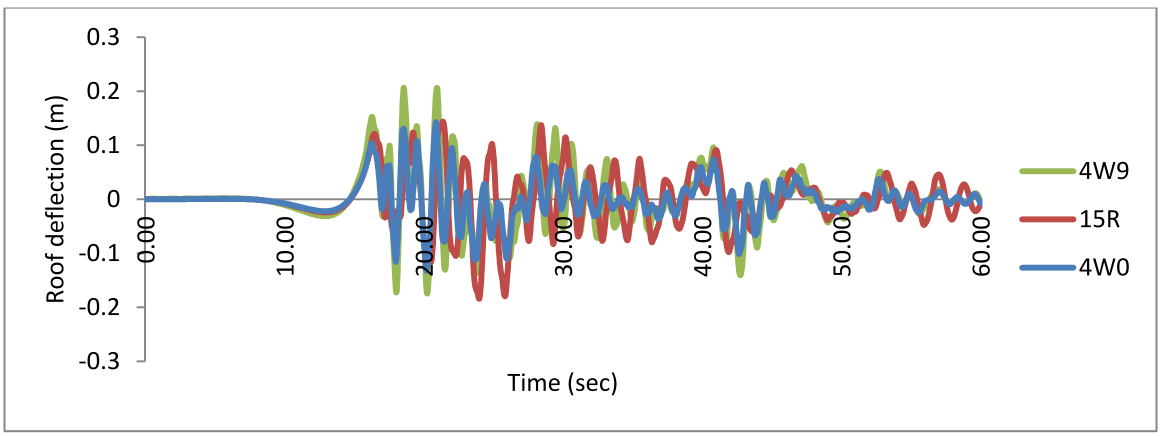


Figure 5.7 Time history of roof deflection in 4W0, 4W9 and 15R buildings

5.1.4. Variation in storey drift

Storey drift relates to the lateral deflections within the building or can be defined as the lateral displacement of one level of a multi-storey building relative to the level below. The greater the storey drift, the higher the likelihood of damage in the building. IS 1893:2016 specifies that storey drift should not be greater than 0.004 of the storey height under the action of design base shear. IBC 2015 sets the maximum drift for normal buildings at between 0.7% and 2.5% of storey height, while EC 8 specifies the range between 1% and 1.5%. The inter-storey drift or the difference in the displacements of two consecutive floor levels normalized by the inter-storey height of the 2W0, 2W5, 2W9, 4W0, 4W4 and 4W9 configuration belonging to all the three groups of buildings are shown in Figure 5.8. 2W5 and 4W5 configurations have an e_d/L of 0.156 and 0.169 and 2W9 and 4W9 have the highest e_d/L 0.570 and 0.545 among of 2W and 4W shear wall buildings considered in the study. It can be observed that the addition of shear wall in the buildings reduces the storey drift substantially.

The storey drift pattern of the shear wall buildings have major variation compared to in-plan regular shear wall building and this variation is substantially accentuated when the eccentricity increases. This variation is the highest in the case of group A buildings. As the aspect ratio of the building increases the variation in the drift pattern between the in-plan eccentric shear wall buildings and the bare frame building reduces. In the case of 2W buildings with a shear wall ratio of 0.32, the maximum storey drift of 2AW9 building is 1.02 times as that of 2AW0 and is lower by 80% with respect to 5R building. Similarly, in 10 storey buildings, 2BW9 has the maximum storey drift higher than 2BW0 by 60 % and is 40% less in comparison with 10R. In 15 storey buildings 2CW9 has the maximum storey drift higher than 2CW0 by 45% and is 85% less in comparison with 15R. Considering variation in 4W buildings, 4AW9 building has the maximum storey drift 1.2 times higher in comparison to that of 4AW0 and 78% lower as compared to 5R building. Similarly, in 10 storey buildings, 4BW9 has the maximum storey drift higher than 4BW0 by 1.2 times and is 54% less in comparison with 10R. In 15 storey buildings, 4CW9 has the maximum storey drift higher than 4CW0 by 43 % and 91% less in comparison with 15R. Another evident observation is that 2W buildings have higher storey drifts in comparison to that of 4W buildings.

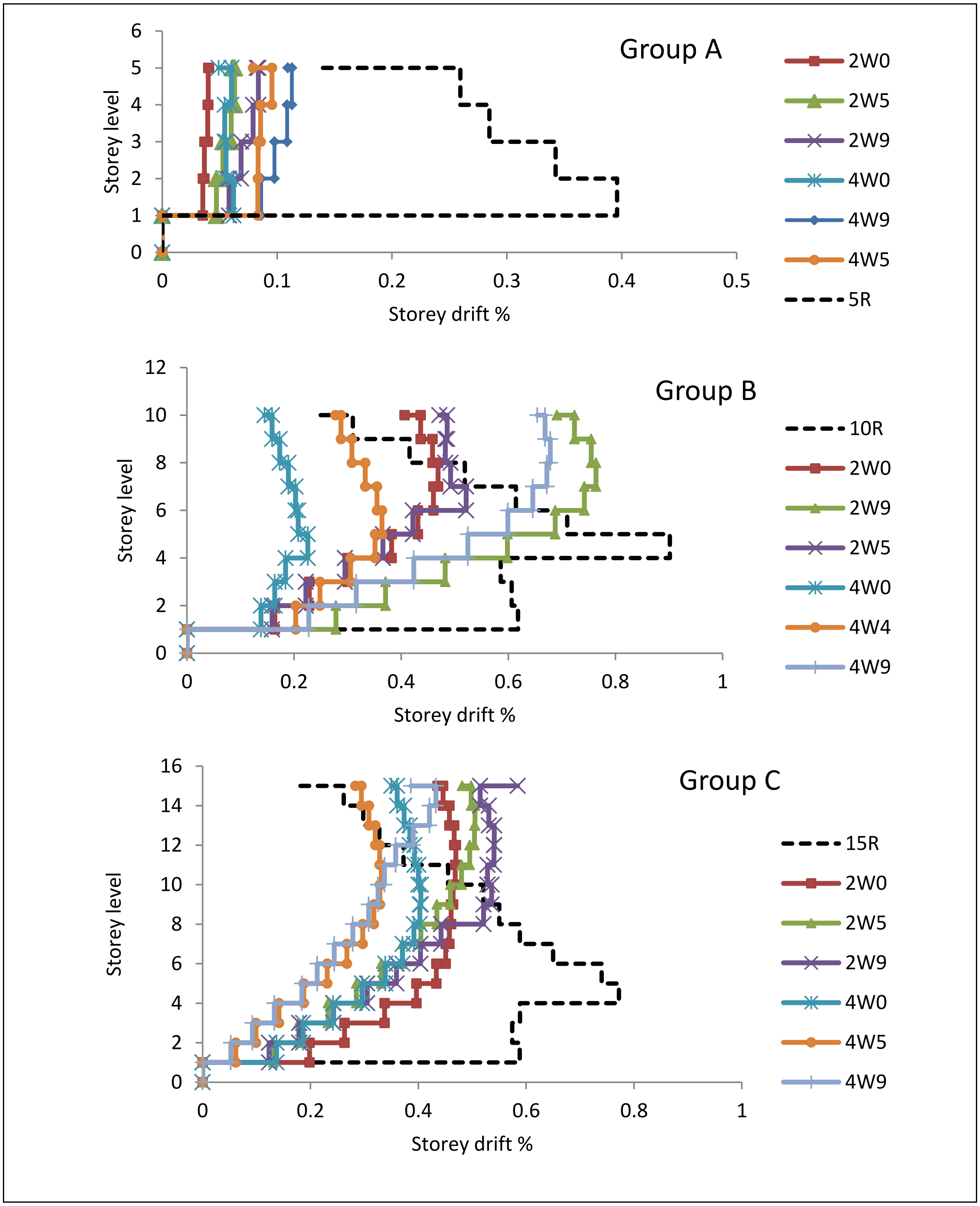


Figure 5.8 Variation in storey drifts of group A, group B and group C shear wall buildings

5.1.5. Variation in roof rotation

The roof rotations of the buildings were estimated by considering the maximum storey drifts of the extreme corners of the top floor of each model. The displacement time histories of the corner points with maximum drift are considered and the highest value of the difference of the drift values gave the maximum relative displacement of the corners. Roof rotation in radians is obtained by dividing the relative displacement by the width of the considered side of the building. Roof rotation is one of the significant parameters to be studied to evaluate the torsional effect in irregular buildings. The shear wall buildings with various unsymmetrical locations or arrangement of the shear walls have variations in roof rotations due to the change in the in-plan eccentricities of the buildings, as shown in Figure 5.9.

The roof rotations of the regular bare frame and symmetric shear wall buildings are near to negligible in comparison to that of the irregular configurations. However, in comparison, the shear wall buildings without eccentricity have lower roof rotation in comparison to that of the corresponding regular buildings due to the stiffening effect of shear walls provided at the corners of the building. Further, the variation of in-plan eccentricity has a high impact on the variation of roof rotation of the in-plan irregular buildings considered in the study. The maximum roof rotation among 2W and 4W buildings is observed as 0.917 radians in 2CW9 and 0.821 radians in 4CW9. Considering the set of 2W buildings with a shear wall ratio of 0.16, eccentricity in shear wall location remarkably increases the roof rotation and it becomes highest in 2CW9. 2CW9 has a variation of 96-98% with respect to that of regular buildings. Similarly, in the set of 4W buildings, 4CW9 has the highest variation of 96% with respect to 15R. The variations in the roof rotations of configurations up to 4W4 with an e_d/L of 0.109 are nominal and from 4W5 to 4W9evident increase in roof rotation can be observed. The roof rotations of irregular shear wall 15 storey buildings with a shear wall ratio of 0.16 have 0.14 -1.5 times in comparison to that of 5 storey buildings. In the case of 4W buildings with a shear wall ratio of 0.32, when the aspect ratio of the buildings increases by 2 times, the roof rotations increases by 70-170%.

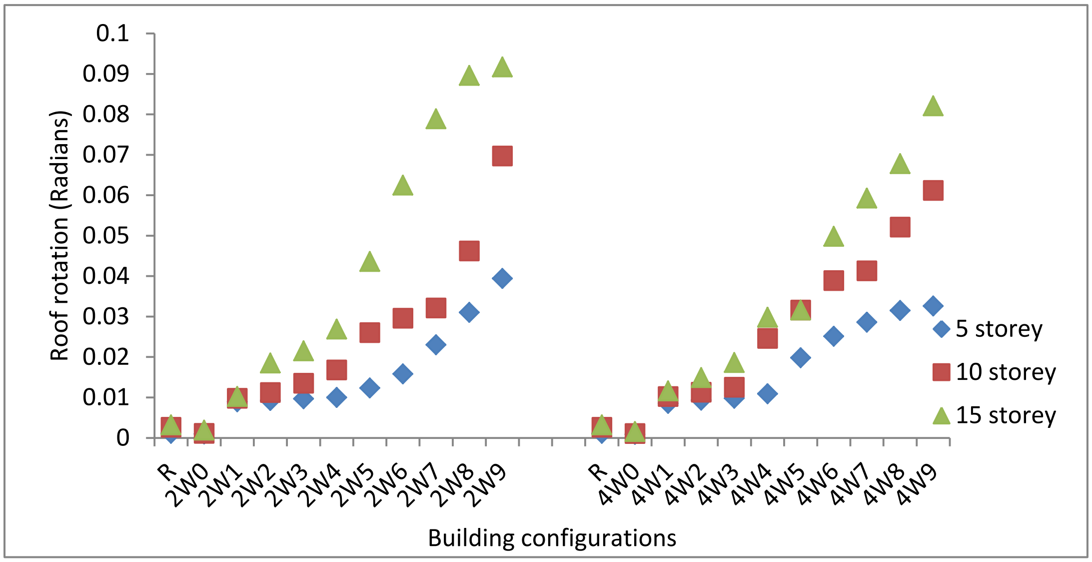


Figure 5.9 Variation in roof rotations of group A, group B and group C shear wall buildings

5.1.6. Torsional irregularity coefficient

According to IS 1893: 2016, buildings with maximum displacement at any floor of the building more than 1.5 times the minimum displacement at the other end of the same floor in the same direction is said to be torsionally irregular. Also, the fundamental natural period in the torsional mode of a torsionally irregular building is greater than the natural period of the building in the first two translational modes of oscillation along each direction.

The buildings are said to be torsionally irregular if the torsional irregularity coefficient, η_t as per IS 1893:2016 is greater than 1.5. If η_t is greater than 2, the entire building configuration should be revised. As per FEMA 450, ASCE 7-16 and most of the international codes, torsional irregularity is considered to exist if the maximum storey drift including the accidental torsion factors at the one end of a structure is more than 1.2

times the average of the storey drifts at the two ends of the structure. Extreme torsional irregularity is present when the maximum storey drift, is more than 1.4 times the average of the storey drifts at the two ends of the structure. Table 5.2 gives the torsional irregularity coefficients η_t of the irregular buildings 2W1-2W9 and 4W1-4W9 as per IS 1893:2016.

Table 5.2 Torsional irregularity coefficients of the irregular shear wall buildings

Torsional irregularity coefficient (η_t)							
Building configurations	**Group A**	**Group B**	**Group C**	**Building configurations**	**Group A**	**Group B**	**Group C**
2W1	1.475	1.496	1.506	4W1	1.452	1.492	1.503
2W2	1.517	1.523	1.531	4W2	1.511	1.524	1.529
2W3	1.524	1.536	1.542	4W3	1.532	1.552	1.563
2W4	1.556	1.548	1.562	4W4	1.628	1.693	1.706
2W5	1.723	1.765	1.785	4W5	1.705	1.763	1.789
2W6	1.842	1.837	1.855	4W6	1.824	1.836	1.924
2W7	1.934	1.964	1.975	4W7	1.934	1.985	1.996
2W8	2.015	2.021	2.048	4W8	2.063	2.098	2.037
2W9	2.185	2.196	2.215	4W9	2.118	2.125	2.196

It can be observed that the torsional irregularity coefficients increase with an increase in the height of the building. Also, the variation of the coefficients is almost in accordance with the variation of eccentricities of the building configurations. As per the international codes, extreme irregularity arises in configurations 2W8-2W9 as well as 4W8-4W9 with e_d/L greater than 0.4.

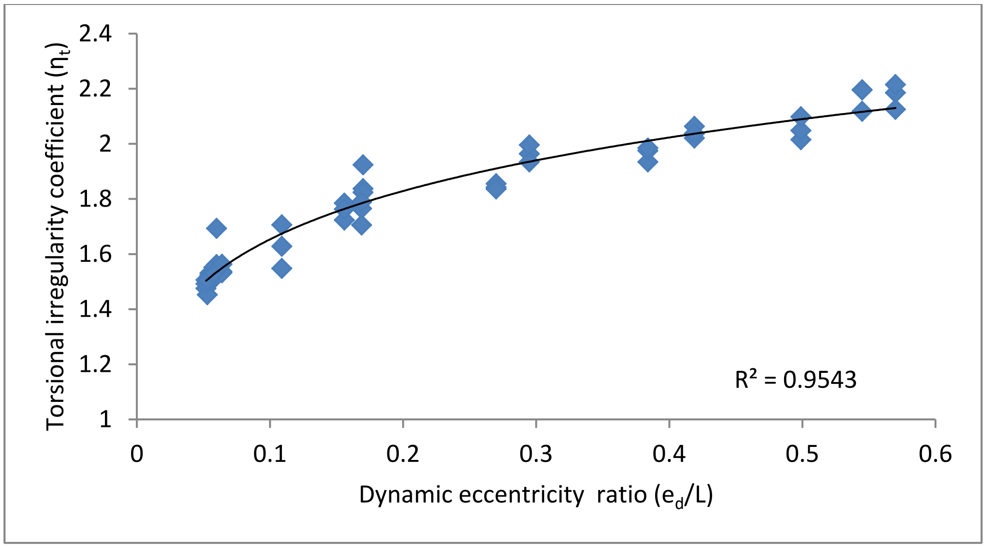


Figure 5.10 Variation of the torsion irregularity coefficient with the dynamic eccentricity of the building configurations

The dynamic eccentricities ratios, of the configurations to their plan width, have high compliance with the torsional irregularity coefficients as shown in Figure 5.10 and therefore the ratio e_d/L can be employed to represent the in-plan or torsional irregularities in the mass and stiffness irregular buildings. Further, the mass irregular and stiffness irregular buildings, in combination with torsional irregularities are arranged and evaluated as per their change in dynamic eccentricity ratio, in the consecutive sections.

5.1.7. Summary

From the study carried out on shear wall buildings with in-plan irregularity due to change in the symmetrical location of the shear walls, it can be inferred that the in-plan eccentricity plays a major role in determining the seismic behaviour of a building, despite the presence of lateral load resisting elements. Here, the in plan eccentricity incorporated was the highest in 2W9 as 0.57L and the highest variation of the same configuration with respect to the building with symmetrically configured shear walls (2W0) are 60% in natural period, 89% in base shear ratio, 62% in roof deflection ratio and 98% in roof rotations. It is also concluded here that dynamic eccentricity ratio is a good measure of the in-plan torsional irregularity in buildings

5.2. Mass irregular buildings

The variations in seismic and torsional behaviour of 375 building frames due to inclusion of mass irregularities along the height as well as in plan in terms of absolute maximum responses of fundamental natural period, base shear ratio, roof displacement and roof rotation are evaluated and their percentage difference with respect to the regular building frames are also computed. A mass irregularity coefficient which quantifies the effect of in-plan mass eccentricity in buildings has also been proposed which can be utilized to plan certain unavoidable mass eccentricities in buildings in a better way.

5.2.1. Variation in fundamental natural period

Fundamental natural period of vibration is determined by carrying out Eigenvalue analyses on building frames. The variation of natural period in the buildings with mass irregularities with the mass ratio M1.25 to M2 belonging to all the three height variants is given in Figure 5.11 and 5.12 represents the variation of fundamental natural period of the 5, 10 and 15 storey frames represented as A, B and C group buildings for higher range of mass ratios M2 to M5. The values are connected by solid lines for better representation of variation. The natural period of the regular buildings, 5R, 10R and 15R are in the range of 0.59s – 3.18s and that of the IM0 configurations are in the range of 0.57s to 2.27s.

Among the first set of buildings with mass irregularity distributed uniformly throughout the floor area, it can be observed that with the increase in the height of the location of irregular masses from the base of the buildings, the natural period increases. Therefore buildings with mass irregularities at the top floor level have the highest variation in natural period with respect to that of the regular frame building in comparison to buildings with irregularities located at other levels along the height of the building.

$CIM0_t$ with the mass ratio of 2 have the highest variation in natural period of 32% with respect to that of the regular frame due to the shift of the vertical mass centre upwards by 1.5m and with irregularities of the mass ratio of 5, the highest variation in the natural period becomes 44%. In the case of higher mass ratios, with irregularities distributed in more than a floor, 2BIM0 and 3CIM0 configurations have a higher natural period in comparison to BIM0 and CIM0 although the same mass is evenly distributed in two or three floors.

The fundamental natural periods obtained for torsionally irregular buildings belonging to the second set are higher compared to the plan regular frame buildings IS0 and this variation increases with increase in eccentricity. Buildings with mass irregularities at the bottom level do not have much variation in natural period with respect to that of the regular frames in all the three groups especially, with mass ratios up to 3. Also, the natural periods of buildings with irregularity $IM1_b$-$IM3_b$ are close to that of the plan regular frames IM0 in all the three groups indicating that masses placed at the lower floor do not cause much variation in the natural period due to in-plan eccentricity. When the masses of ratio 1.25 to 2 are present at the lower floor levels with mass eccentricity, there is nominal variation in natural period of 1.2% to 7.2% in group A, 1.3% to 7.7% in group B and 2.2% to 8.9 % in group C with respect to that of regular buildings. But when the masses are located at the top of the frame elevating the vertical mass centre, natural period increases and becomes highest in $IM3_t$ in all the mass ratio variants. The highest variation in the natural period between the placements of masses at the top floor level in comparison to that at the bottom floor is observed in group A buildings. Comparing the natural period of $AIM3_t$ and $AIM3_b$, both with mass ratio M2, a variation of 34.9% is observed. Considering mass ratio of 1.25 which is within the safe limits for mass eccentricity, the maximum variation in natural period of 28% with respect to regular frame buildings and 15% with respect to IS0 building is observed if the mass irregularities have an eccentricity of 0.09L when placed at the upper floor levels. Whereas in buildings of mass ratio M1.5 which is the limit for mass irregularity as per IS code recommendations, in-plan eccentricity at the upper levels causes a variation of 38% in the natural period with respect to regular frame buildings and 18% with respect to IM0 buildings. $AIM3_t$, $BIM3_t$ and $CIM3_t$ buildings with M2 placed at the upper levels have the highest increase of 25.2%, 35.9% and 39.2% in natural period among A, B and C group buildings with respect to 5R, 10R and 15R.

In the irregular frames with mass ratios from 1.25 to 3, the variation of the natural period with the change in e_d/L is also nominal. $AIM3_t$, $2BIM3_t$ and $3CIM3_t$ buildings with a mass ratio of 5 have the highest increase of 116.5%, 101.5% and 77% in natural period among A, B and C group buildings. The provision of irregularity at the top level of the buildings increases the natural period by 30.5% in comparison to IR0 due to maximum torsional coupling. In the case of buildings with higher mass ratios, it is observed that 2B and 3C buildings have higher natural periods in comparison to B and C buildings due to the change in mass distributed over more than a single storey leading to higher responses. Therefore the presence of heavy masses on any floor level of a building increases the natural period and it reaches the highest when placed at the top floor level elevating the vertical mass centre, natural period increases and becomes highest in $IM3_t$ with mass ratio 5. It can thus be interpreted that with the inclusion of heavy masses on any floors of the building, the natural period increases and the maximum variation is observed when the mass irregularities are present at the top floor levels. During lateral oscillation of a building under earthquake loads, the sum of the seismic masses at different floor levels become effective and the increase in seismic masses in a building increases the fundamental natural period. When a building has non-uniform distribution of masses along its height and masses vibrate at different heights from the base, simulating the changes in the effective stiffness and hence, the natural period varies. This increase is amplified when masses are placed with in-plan eccentricity, which further enhances the structure's flexibility.

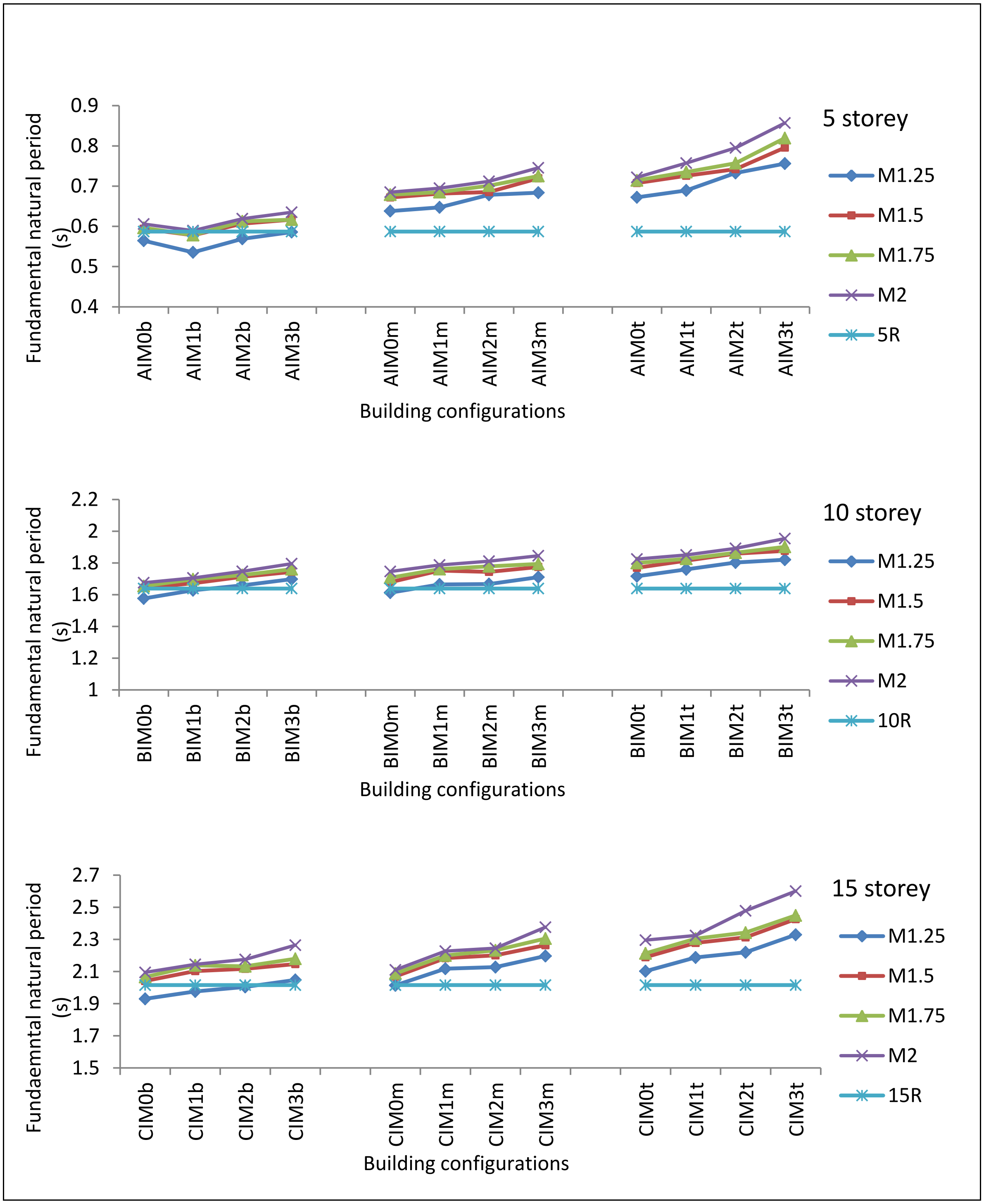


Figure 5.11 Variation in fundamental natural period of buildings with mass ratios M1.25-M2 in group A, group B and group C buildings.

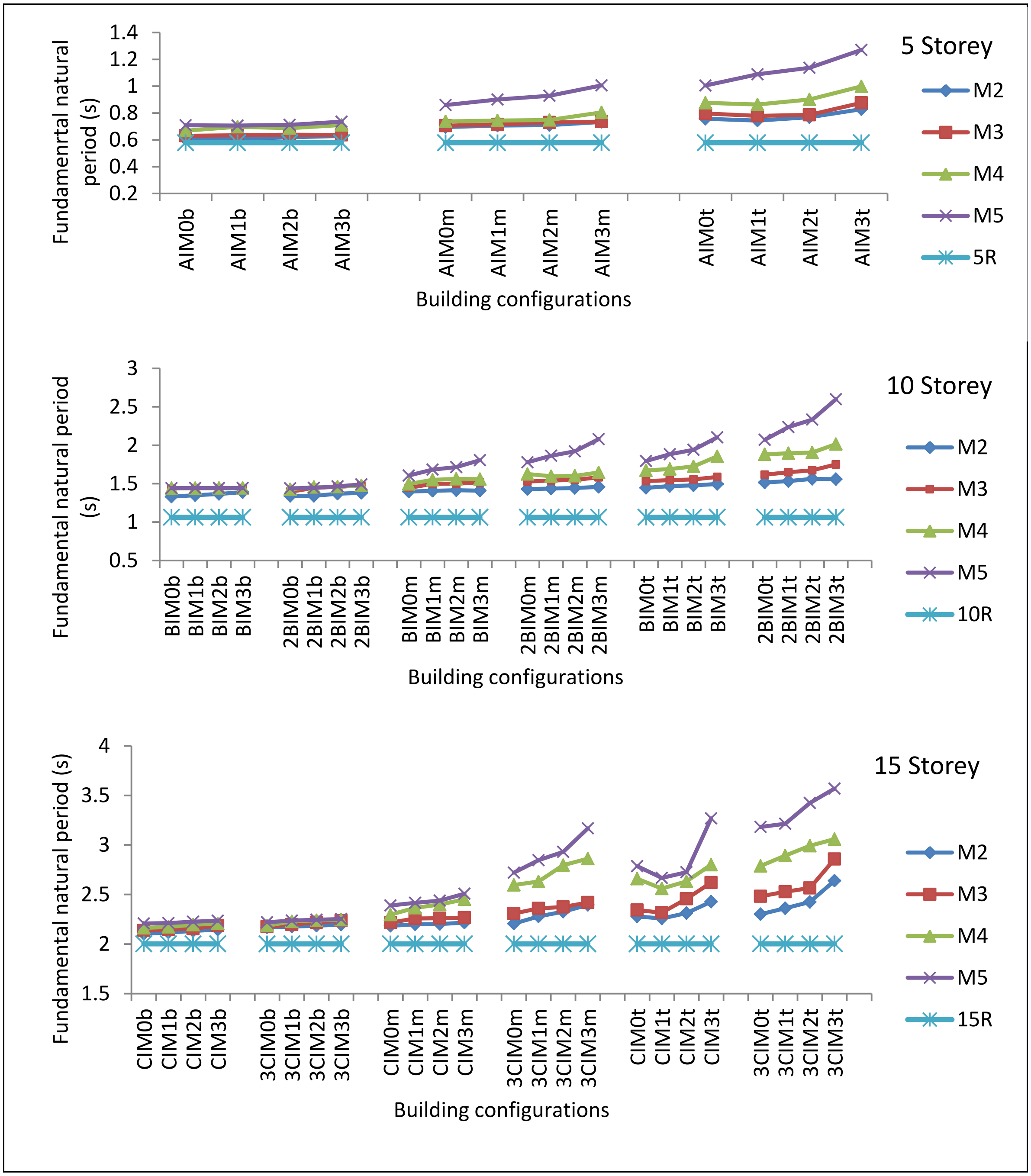


Figure 5.12 Variation in fundamental natural period of buildings with mass ratios M2-M5 in group A, group B and group C buildings

The frequency ratio, Ω decreases in all the three groups of buildings in the order of increasing e_d/L. IM3 with the highest e_d/L has the least Ω ratio among all the groups and the frequency ratios of buildings of IM3 configuration of the three groups of buildings are given in Table 5.3. Frequency ratio, Ω is less for higher eccentricity ratio of the configurations and hence the buildings with the maximum torsional coupling are $AIM3_t$, $2BIM3_t$ and $3CIM3_t$ with mass ratio 5 with frequency ratios of 0.905, 0.879 and 0.756 respectively. Furthermore, the general pattern is that the Ω values of the buildings tend to decrease with increase in the aspect ratio of the buildings, though there isn't any evident variation between the mass ratios especially in buildings with low eccentricity.

Table 5.3 Frequency ratios of mass irregular IM3 buildings

Building configuration	Frequency ratio (Ω)				
	M1.5	**M2**	**M3**	**M4**	**M5**
$AIM3_b$	0.980	0.985	0.984	0.983	0.981
$AIM3_m$	0.940	0.945	0.935	0.930	0.925
$AIM3_t$	0.913	0.918	0.915	0.910	0.905
$BIM3_b$	0.937	0.942	0.924	0.912	0.906
$2BIM3_b$	0.919	0.924	0.921	0.911	0.899
$BIM3_m$	0.912	0.917	0.908	0.899	0.892
$2BIM3_m$	0.903	0.908	0.900	0.890	0.885
$BIM3_t$	0.892	0.896	0.891	0.885	0.881
$2BIM3_t$	0.883	0.887	0.882	0.880	0.879
$CIM3_b$	0.951	0.956	0.927	0.904	0.895
$3CIM3_b$	0.893	0.897	0.886	0.875	0.863
$CIM3_m$	0.906	0.911	0.903	0.883	0.856
$3CIM3_m$	0.797	0.801	0.801	0.802	0.803
$CIM3_t$	0.887	0.877	0.852	0.831	0.822
$3CIM3_t$	0.865	0.869	0.842	0.796	0.756

5.2.2. Variation in seismic base shear ratio

Seismic base shear expressed in terms of total seismic weight (W) of buildings, with mass irregularity in the plan as well as elevation, which are subjected to El-Centro ground motion is as shown in Figures 5.14 and 5.15. Base shear is directly dependent on the total mass of the building and hence, with an increase in mass ratios, base shear increases. The base shear ratio decreases with an increase in the aspect ratio of the buildings and the ratios of 15R, 10R and 5R are obtained in the range of 0.051W to 0.115W. It can also be observed that base shear demand varies with the location of irregularity along the height of the buildings and the shift of irregularity location from the bottom storey to heights above the vertical mass centre of the building frame increases the base shear ratio. Considering the initial set of buildings without eccentricity, due to the shift of the masses to the upper levels of the buildings, the base shear ratio increases by a maximum of 29% in group C buildings. The base shear demand of irregular buildings is lower than that of the regular frame buildings in few cases with mass ratio 1.5 placed at the bottom level implying that smaller eccentricities do not cause much variation in base shear unless located along higher floor levels of the frame.

It is observed that the seismic base shear demand increases with increase in eccentricity when the irregularity is located at the top level of the building frame. Whereas, with irregularities positioned along the bottom half height of the frames, the increase in eccentricity leads to a reduction in the base shear ratio in comparison to the regular frames. The decrease in base shear with an increase in eccentricity when the irregularities are located at the bottom half of the frames may be attributed to the increase in the natural period. Further, when the mass irregularities are present in the upper floor levels with higher in-plan eccentricities, effects of increasing the base overturning moments becomes significant and hence base shear ratio increases. Therefore it can be observed that the building configuration with the highest dynamic eccentricity (IM3) has the highest base shear ratio when the additional mass is placed at the top level of the frame and has the least value when the additional mass is placed at the middle and bottom level of the frames. Hence eccentric masses located in the upper half of the frames amplify the effect of vertical irregularity remarkably, but if located in the lower half tend to stiffen the building frames. The lowest base shear is observed in CIM3$_b$ configuration with mass ratio 5 as 0.046 W and the highest base shear in AIM3$_t$ configuration with mass ratio 5 as 0.1965W. However, considering the buildings of mass ratio 1.25, in-plan eccentricity at the upper floor levels causes a maximum increase of base shear ratio by 25% with respect to regular frames and 11.5% with respect to IM0 frames. Similarly, buildings of mass ratio 1.5 which is the limit for mass irregularity as per several international codes of practice, due to in-plan eccentricity at the upper levels, the maximum increase of base shear ratio by 22% with respect to regular frames and 14% with respect to IM0 frames is observed. The highest variations of base shear of irregular frames with respect to the regular frames without mass irregularity are observed in AIM3$_t$, BIM3$_t$ and CIM3$_t$ building frames of the M2 variants as 44.3%, 52.2% and 68.5% respectively. Similarly, the highest variations of base shear demand of irregular frames with respect to the regular frames are observed in AIM3$_t$, 2BIM3$_t$ and 3CIM3$_t$ building frames of the M5 variants as 74.6%, 64.5% and 101.9% respectively. The maximum variation of base shear in buildings with in-plan eccentricity with respect to the plan regular ones is obtained in 3CIM3$_m$ as 34.6%. The time history plot base shear of IM3 and IM0 buildings in group C with mass ratio M2 in comparison with 15R building is given in Figure 5.13.

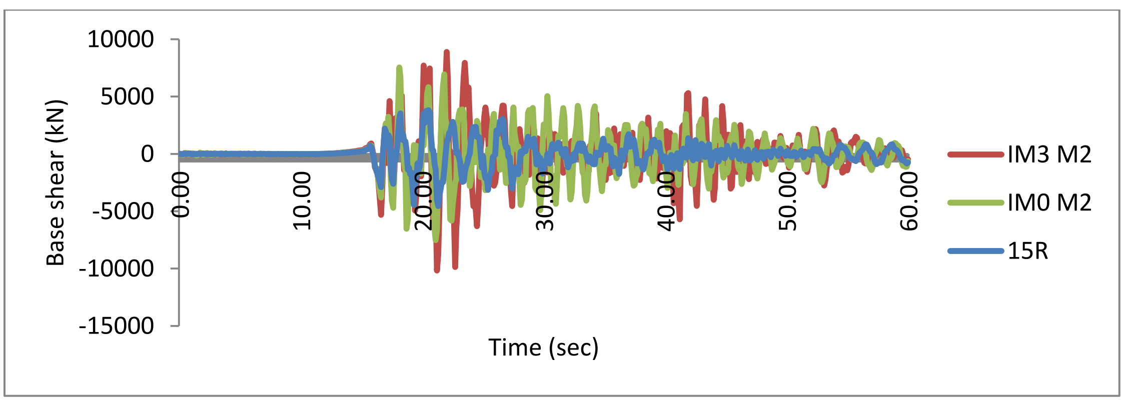


Figure 5.13 Time history of base shear of 15R and mass irregular buildings IM3 and IM0 with mass ratio M2

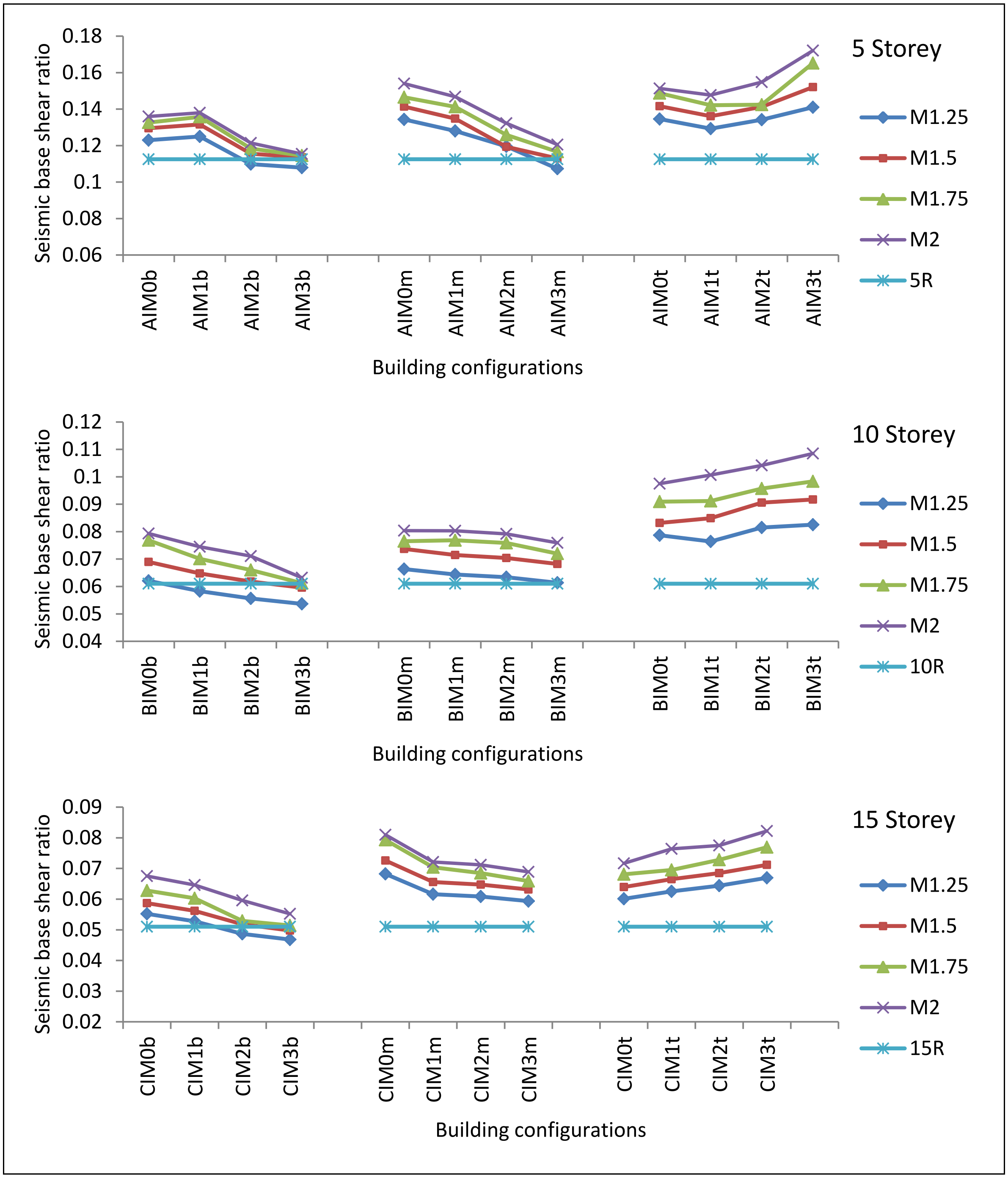


Figure 5.14 Variation in seismic base shear ratio of buildings with mass ratios M1.25-M2 in group A, group B and group C buildings

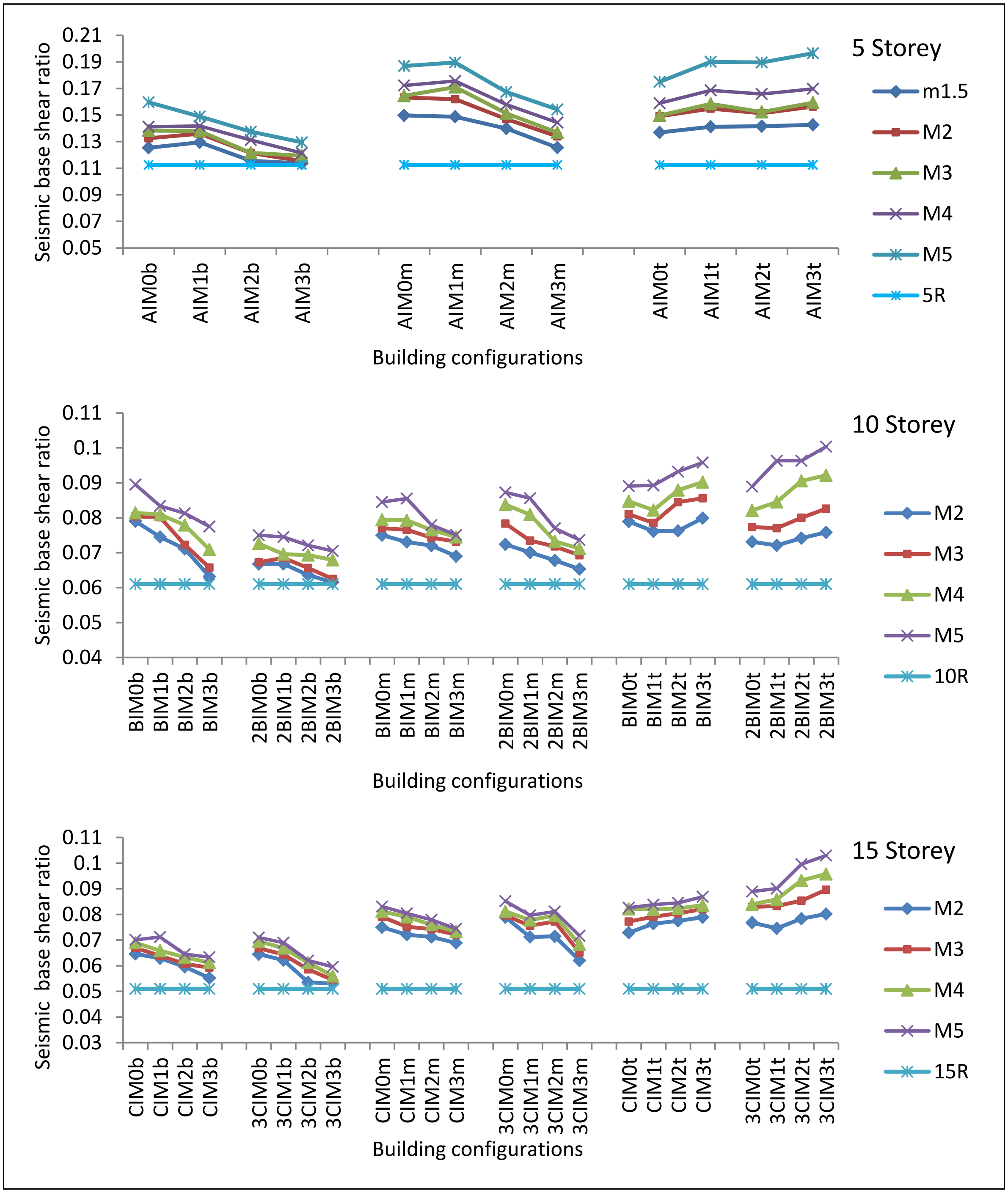


Figure 5.15 Variation in seismic base shear ratio of buildings with mass ratios M2-M5 in group A, group B and group C buildings

5.2.3. Variation in roof deflection

The maximum roof deflection of buildings with various mass ratios and irregularity locations under the application of El-Centro ground motion are represented in Figures 5.16 and 5.17. Roof deflection values are expressed in terms of the height of the buildings (H) as ‘roof deflection ratio.’ The roof deflection ratios of the regular buildings from 5 to 15 storey are in the range of 0.0052H to 0.0085H. It is observed that buildings without eccentricity i.e. AIM0, BIM0 and CIM0 in A, B and C groups respectively has lower roof deflection among the mass irregular buildings. The increase in roof deflection in mass irregular buildings is observed to be proportional to the increase in aspect ratio as well as to the location of irregularity. It can be observed that roof deflections of the frames are lower when the irregularity is located at the bottom of the frames in all the aspect ratio variants. The variation in maximum roof deflection between the regular and the vertically irregular buildings increases as the location of irregularity shifts upwards from the bottom to the top floor level. The variation of maximum roof deflection in buildings with mass ratio of 1.25 to 2 at the bottom floor levels with respect to the regular frame buildings is considerably low. When the masses of ratio M2 are located at the upper floor level of the buildings, the group B buildings show a maximum variation of 23% in roof deflection as compared to mass located at the bottom storey. In the case of masses of ratio 5, this variation in roof deflection due to the positioning of masses at the upper floor levels in comparison to that at the lower levels becomes 46%. The variation of maximum roof deflection in buildings with a mass ratio of 1.25 to 2 especially, at the lower floor levels, with respect to the regular ones, is considerably low. A nominal variation of 3.1% to 12% in group A, 1.5% to 10% in group B and 1.4% to 14% in group C are observed in irregular buildings with a mass ratio of 1.5 to 2 at the lower storey with respect to the regular buildings. This variation increases to 20%, 31% and 21% in group A, group B and group C buildings respectively when the same masses are located at the upper floor levels instead. In case of higher mass ratios 2 to 5, the maximum variation of

45% in group A, 58% in group B and 44% in group C is observed with respect to the regular buildings when the heavier masses are located at the top floor level

Considering the second set of buildings with in-plan eccentricity in combination with vertical mass irregularity, the variation of roof deflection ratio with respect to regular buildings has a maximum of 78% in group B buildings. The variation in roof deflection ratio with respect to the mass regular IM0 configuration is the highest, 29%, in the case of $BIM3_t$ with mass ratio of 5 located at the upper floor levels. The highest roof deflection of 0.0087H is observed in $CIM3_t$ configuration of 15 storey with a mass ratio of 5, which has a variation of 40% with respect to 15R and 25.8% with respect to $CIM0_t$. The roof deflection ratio of buildings with in-plan mass irregularity even in M1.25 located at upper levels increases by a maximum of 9% with respect to IM0 and 25% with respect to regular frame buildings. Among the 5 storey buildings, it can be observed that the highest roof deflection is obtained when the mass irregularities are present at the middle floor level of the frame. However, in buildings of higher aspect ratios, when irregularities are positioned at the upper levels, the buildings undergo maximum deflection under earthquake ground motion. It can also be observed that the 2B and 3C configuration have a higher deflection ratio, in comparison to the B and C buildings among all cases of irregularity locations. The roof deflection ratio of buildings with in-plan mass irregularity even in M1.5 located at upper levels increases by a maximum of 14% with respect to IM0 and 57% with respect to regular frame buildings. The roof deflection or displacement pattern comprises the global deformation demand of a building. Therefore the variation due to the change in position and magnitude of mass irregularities causes variation in the storey drift demands and hence roof deflection is the highest when the mass irregularities are present at the upper floor levels with highest in-plan eccentricity. Figure 5.18 shows the time history of of roof deflection in IM0 and IM3 buildings with mass ratio 2 in comparison with 15R building.

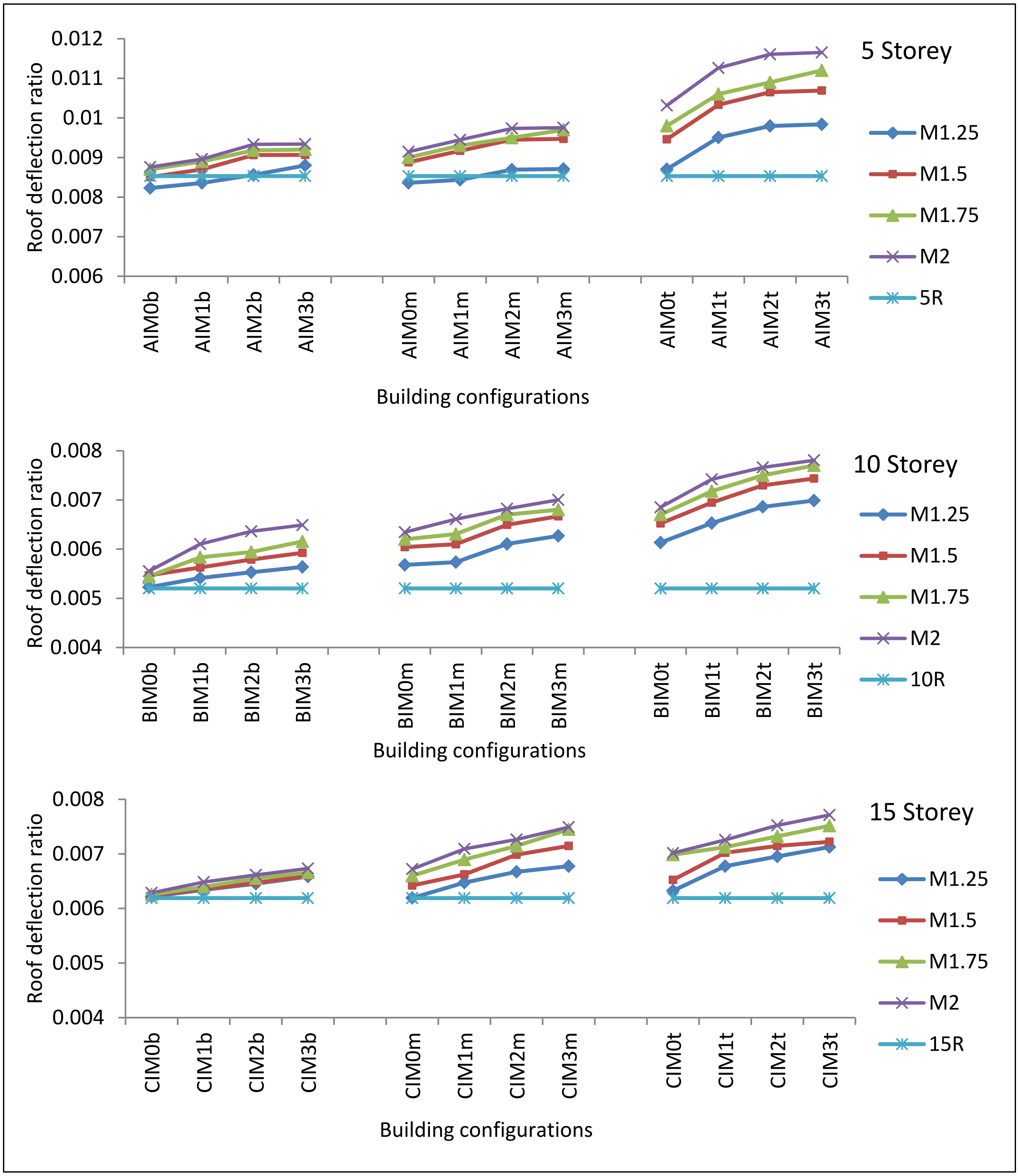


Figure 5.16 Variation in roof deflection ratio of buildings with mass ratios M1.25-M2 in group A, group B and group C buildings

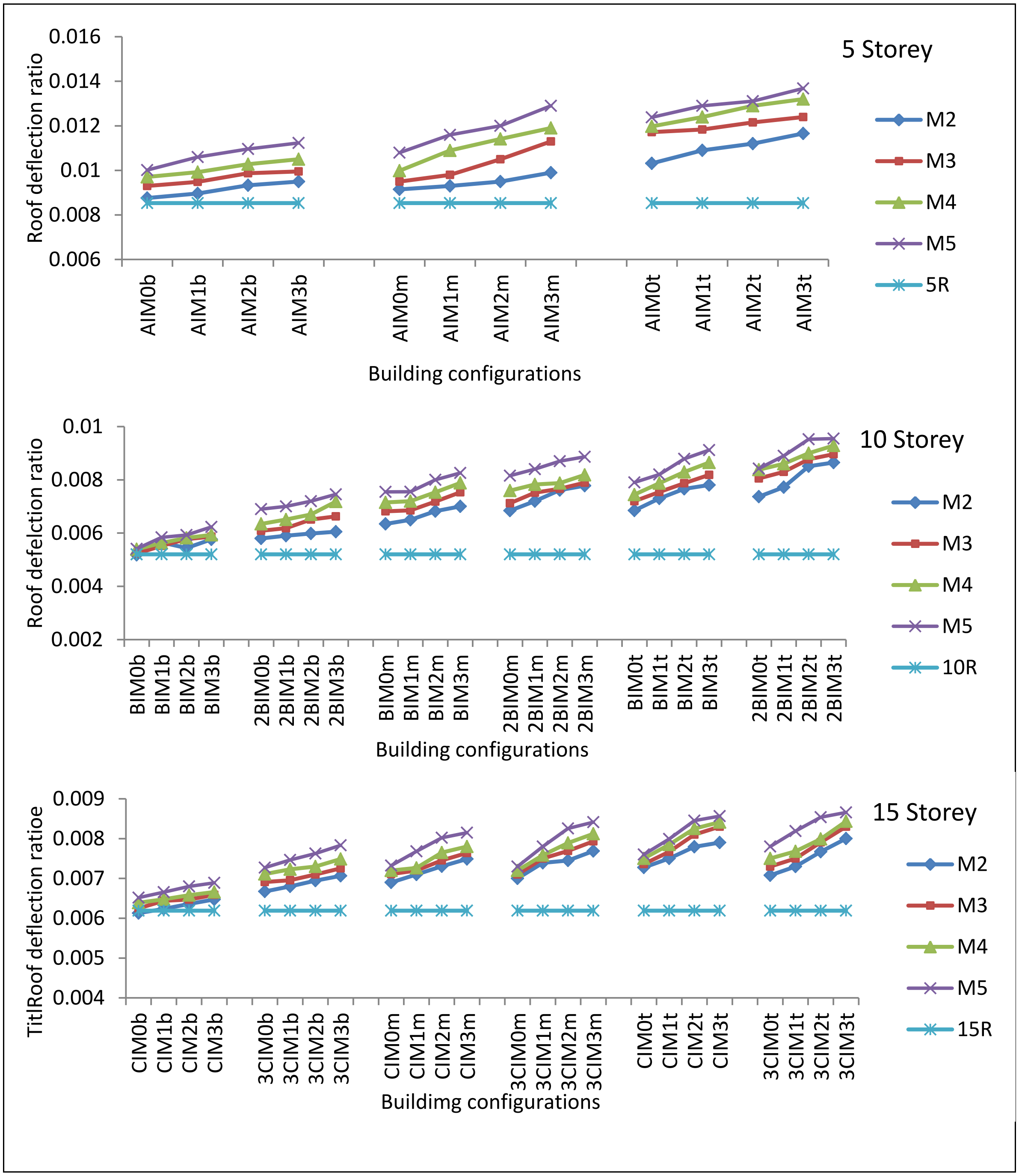


Figure 5.17 Variation in roof deflection ratio of buildings with mass ratios M2-M5 in group A, group B and group C buildings

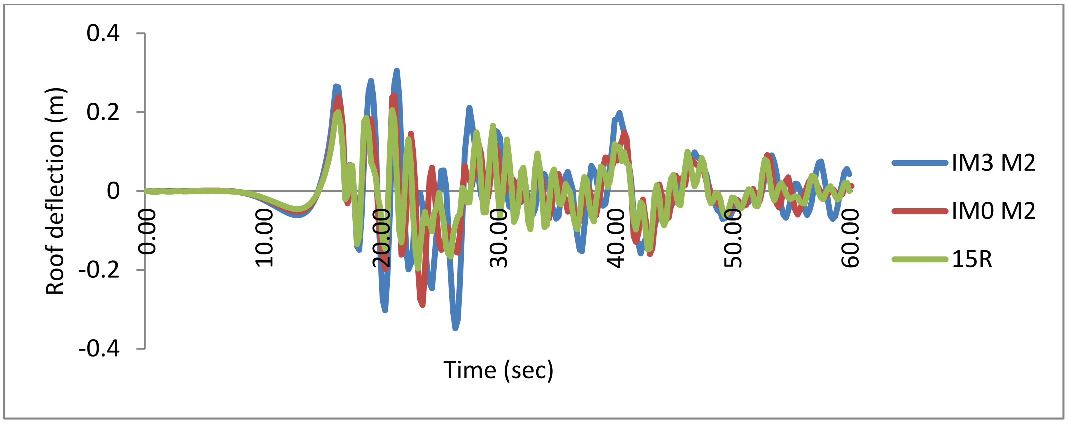


Figure 5.18 Time history of roof deflection of 15R and mass irregular buildings IM3 and IM0 with mass ratio M2

5.2.4. Variation in roof rotation

The roof rotation is estimated by considering the highest storey displacements of the extreme corners of the roof of each model. The displacement time histories of the corner points with maximum displacements were considered, and the highest value of the difference of the displacements gives the maximum relative displacement of the side of the building. Roof rotation in radians is obtained by dividing the relative displacement of the building under the effect of seismic loads by the plan width of the building. From the set of vertically mass irregular buildings without in-plan eccentricity, it can be observed that roof rotation increases with an increase in the aspect ratio and the 15 storey buildings have the highest roof rotation. It can be seen from Figures 5.19 and 5.20 that when the mass irregularity is present at the upper floor level of the building even without eccentricity, roof rotation increases in comparison to the presence of mass irregularities at the bottom floor levels. The roof rotation of 5R, 10R and 15R are obtained minimal, the plan being regular in mass and stiffness distributions. $IM0_b$ buildings with mass irregularities at the bottom floor level of the buildings also have nominal roof rotation in comparison with $IM0_m$ and $IM0_t$. Though the buildings with mass irregularities, IM0 is symmetrical in plan, with the shift in the location of these masses from the bottom to the higher levels, roof rotation is induced in the buildings. This is due to the increase in flexibility due to the shift of the vertical mass centre of the buildings upwards from the ground level. Considering buildings with smaller mass ratios, 1.25 to 2 the highest variation in roof rotation due to change in location of masses from the bottom to the higher levels is observed as 44% in group A, 53% in group B and 55.8% in group C buildings. In group C buildings with M2, the highest increase in roof rotation due to the shift in masses from the bottom level to the top level is 55.8%. In group 3C buildings with M5, the highest increase in roof rotation due to the shift in masses from the bottom to the top floor level is 67%.

Considering the set of buildings with in-plan mass eccentricity, IM3 has the highest roof rotation, and IM0 without eccentricity has minimal roof rotation in all the groups of buildings with the different location of mass irregularities and aspect ratios. In the case of buildings with irregularities of mass ratios 1.25 to 2, the maximum rotation among all the cases is observed in $CIM3_t$ with mass ratio 2 as 0.0487 rad. The minimum rotation is observed as 0.0068 rad in $AIM0_b$ with M1.25. The maximum roof rotation among all the cases of mass irregularities in combination with in-plan eccentricity is observed in $3CIM3_t$ with mass ratio 5 as 0.0847 rad. The minimum roof rotation is observed in $AIM0_b$ with M1.5 as 0.0117 rad. $IM3_b$ of mass ratio 1.25 increases the roof rotation by 14% to 37.6% with respect to the regular frames which shows that eccentrically placed smaller masses at lower floor levels can also lead to considerable increase in roof rotation. When the masses of mass ratio 1.25 are placed at the upper levels, a maximum variation in roof rotation of 67% with respect to regular frame and 62.5% with respect to IM0 buildings is observed. Due to varying e_d/L, a maximum variation of 96.9% is observed in roof rotation in IM3 configuration with mass ratio 2 at the upper floor levels with respect to the plan regular IM0 buildings. $IM3_b$ of mass ratio 1.5 increases the roof rotation by 22% to 62%, implying that eccentrically placed smaller masses at lower floor levels can also lead to a significant increase of roof rotation. When the masses of mass ratio 1.5 are placed at the upper levels, a maximum variation in roof rotation of 88% with respect to regular frame and 85% with respect to IM0 buildings is observed. Considering buildings with higher mass ratios, 2B and 3C configuration have higher rotation in comparison to B and C configuration since the irregularities are provided as distributed in two and three floors respectively along the height. When irregularities are located at the upper levels with eccentricity, the storey drifts increase along with a considerable increase in twisting moments, which leads to increase in maximum roof rotation. Due to varying e_d/L, a maximum variation of 138.5% is observed in roof rotation in IM3 configuration with respect to the plan regular IR0 buildings.

5.2.5. Variation in torsional resultant

As a result of the torsional coupling in the buildings, it can be observed that the structure tries to pass on the consequences of global torsion in the most convenient manner. The torsion generated in the plan under the effect of lateral loads is passed on to columns initially as additional horizontal forces depending upon the distance from the center of rigidity. The columns bear these additional horizontal shear forces by frame action. In case, frame action is deficient or exhausted, then the system of scattered columns starts resisting the torsion in the plan through the local twisting of columns, as a redistributed behavior of the structure. Figures 5.22 and 5.23 depict the variations of maximum torsional moments in the corner columns of the mass irregular buildings. Similar to the variation of floor rotation in buildings, torsional moments of the mass irregular buildings also have remarkable variations with respect to the variation of in-plan eccentricity. The highest torsional moments of the corner columns are recorded and their variations with respect to the varying in-plan eccentricities of the configurations are studied. IM0 buildings without in-plan eccentricity have the least torsional moment among the irregular buildings in all the three groups of buildings considered. The mass irregularities with in-plan eccentricity at the top floor level of the buildings increase the torsional moments by 5 times as compared to the location of the mass irregularities at the bottom floor levels.

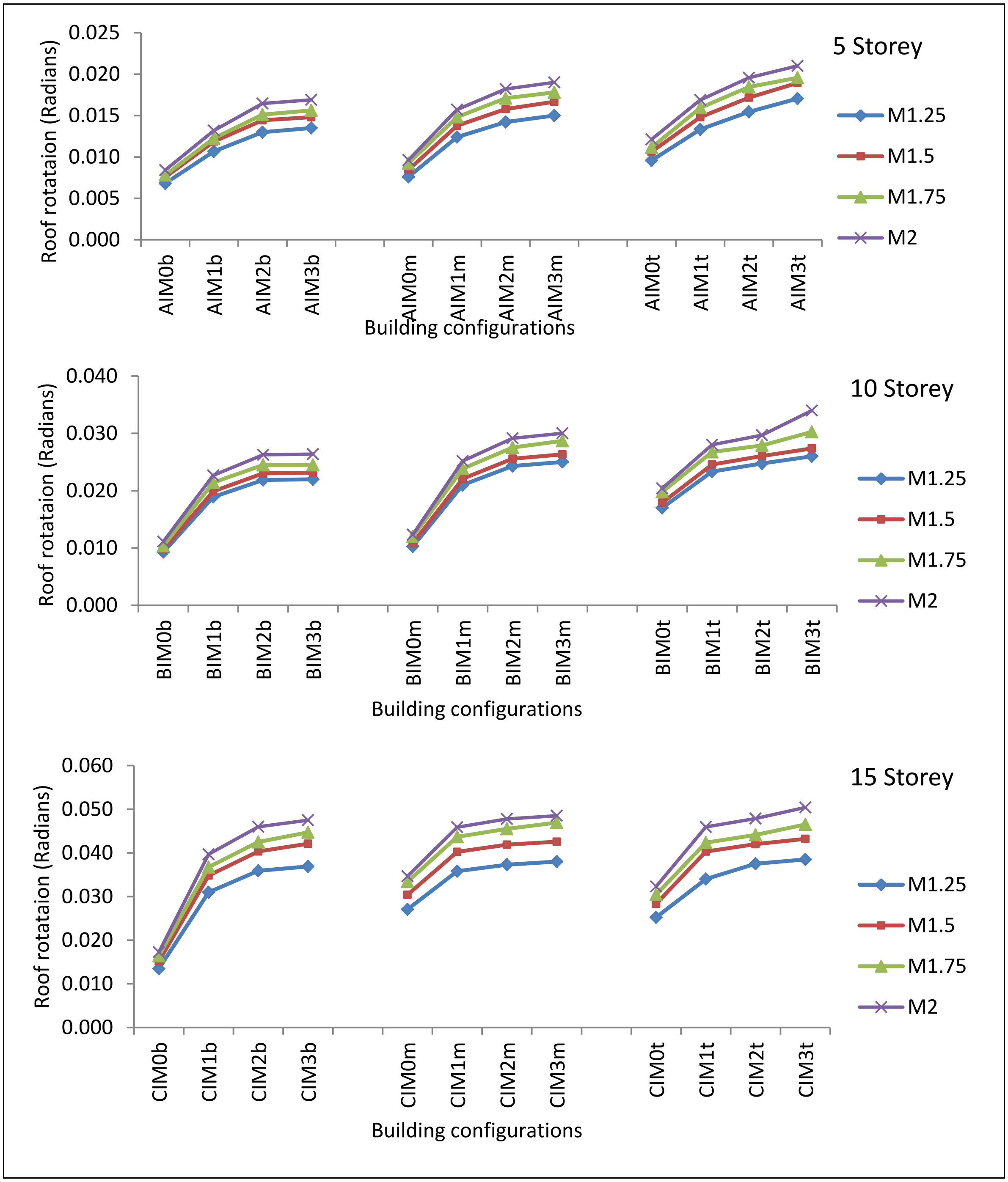


Figure 5.19 Variation in roof rotation of buildings with mass ratios M1.25-M2 in group A, group B and group C buildings

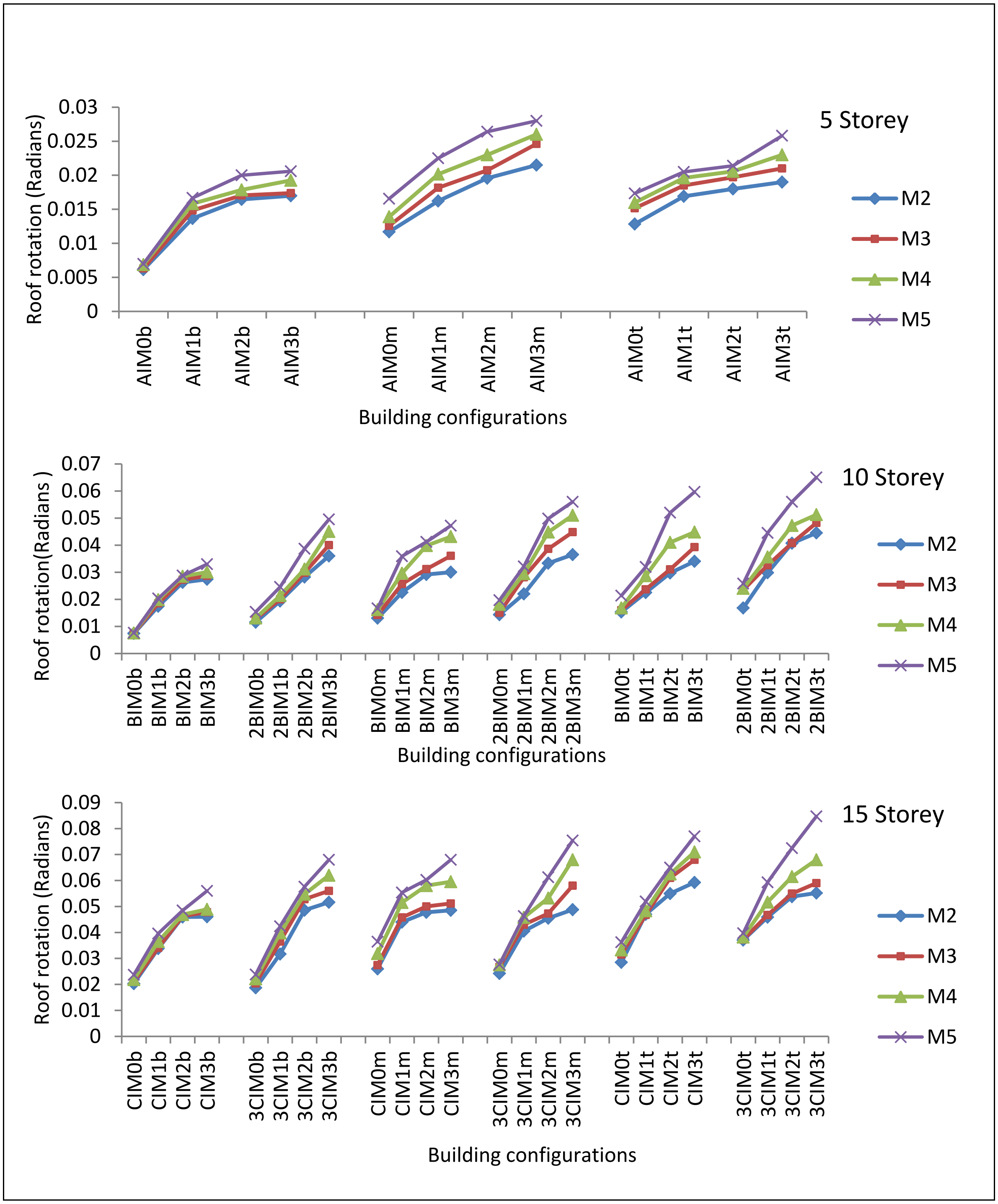


Figure 5.20 Variation in roof rotation of buildings with mass ratios M2-M5 in group A, group B and group C buildings

Comparing the variation of torsional moments in buildings with nominal mass ratios 1.25 to 2, IM3 buildings with mass ratio1.25 and e_d/L of 0.09 at the bottom floor level increases the torsional moment by almost 4 times with respect to that of the IM0 building. This is further amplified when the location of the irregularities shifts toward the upper floor levels. Considering the torsional moments of M1.25 and M2 buildings, maximum of 70% variation can be observed in the case of group B buildings. In the case of buildings with higher mass ratios, a similar variation of torsional moments between the in-plan eccentric M2 and M5 buildings is about 3.5times. Figure 5.21 gives the time history plot of torsional resultant in IM0 and IM3 buildings with mass ratio 2 in comparison with that of 15R building.

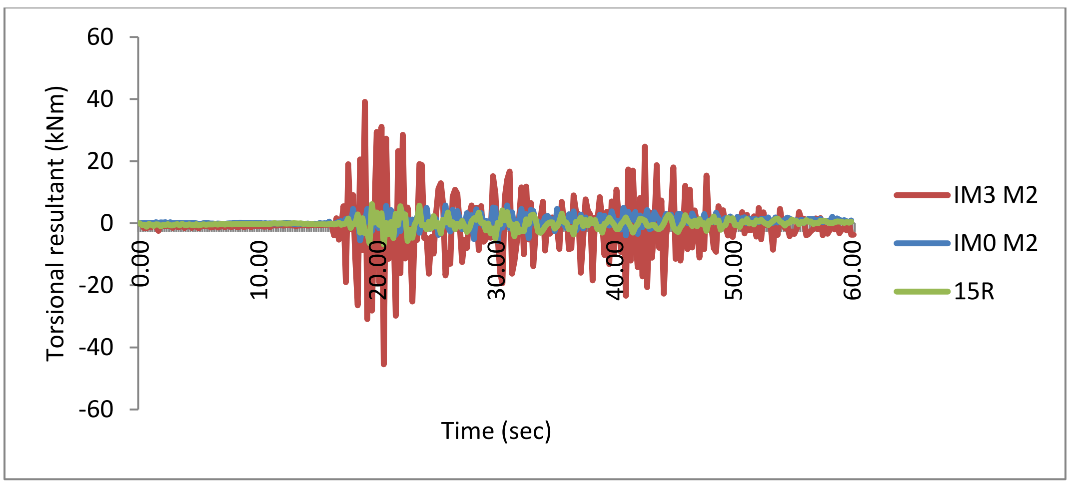


Figure 5.21 Time history of torsional resultant of 15R and mass irregular buildings IM3 and IM0 with mass ratio M2

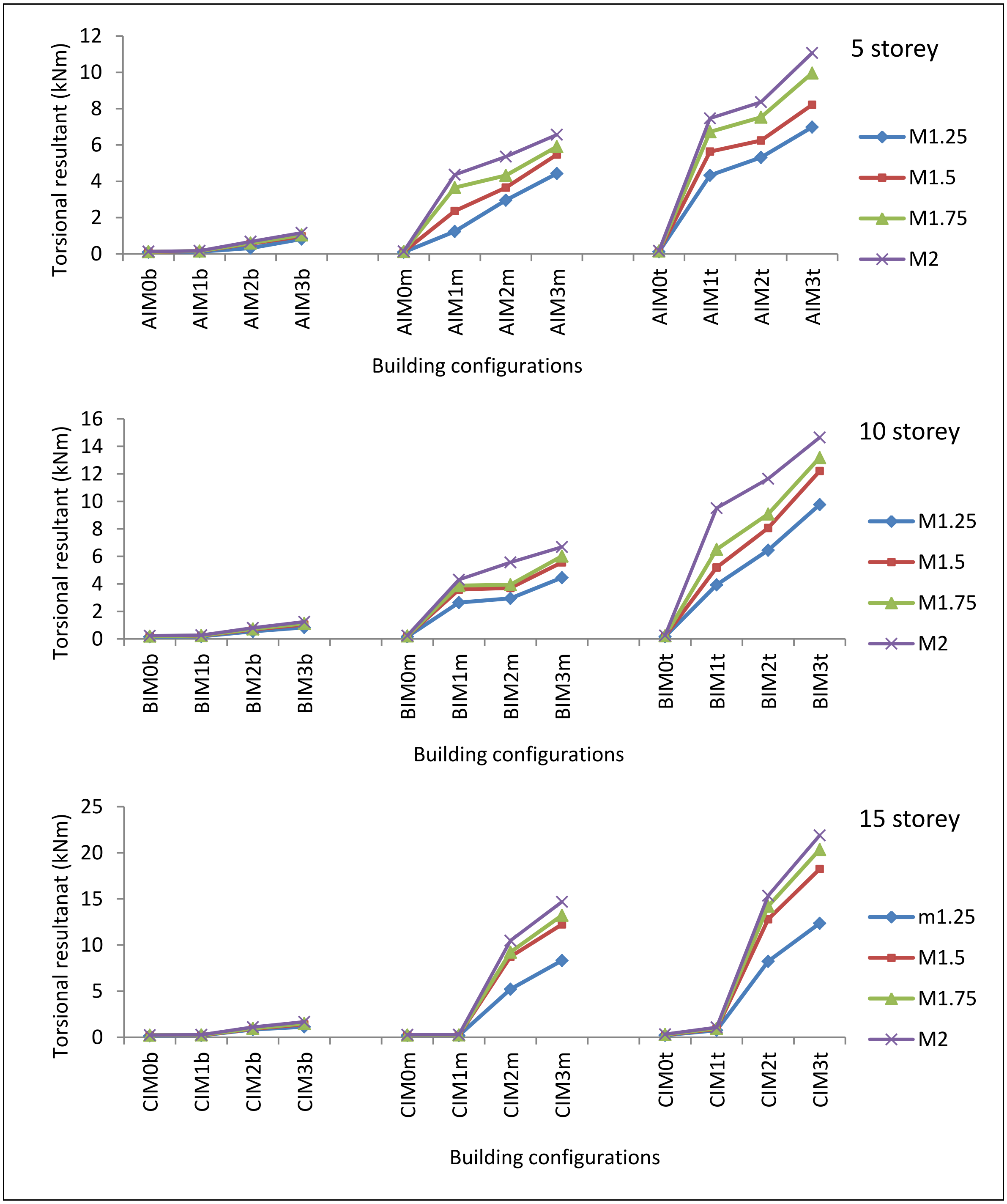


Figure 5.22 Variation in torsional resultant of buildings with mass ratios M1.25-M2 in group A, group B and group C buildings

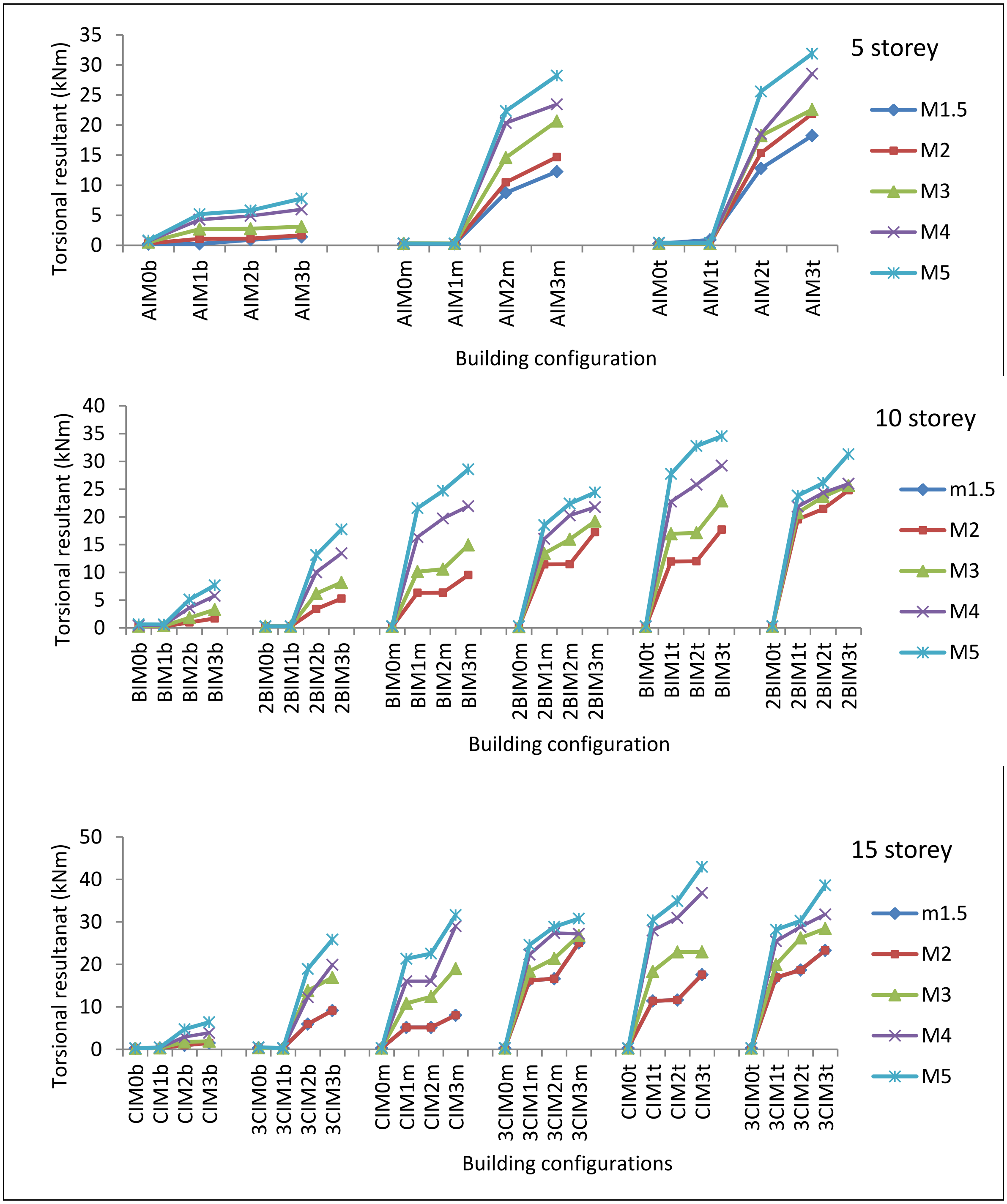


Figure 5.23 Variation in torsional resultant of buildings with mass ratios M2-M5 in group A, group B and group C buildings

5.2.6. Mass Irregularity Coefficient

A new irregularity coefficient ‘α’ has been proposed from the present study for the prediction of natural period and base shear in irregular buildings, which includes the effects of in-plan eccentricity for buildings having equal dimensions along the direction of seismic excitation and the transverse direction as:

$$\text{Mass irregularity coefficient, } \boldsymbol{\alpha} = \frac{\sum_{i=1}^{n} M_{ri} H_i \frac{e_{si}}{L^2}}{n} \tag{5.1}$$

Where M_{ri} is the mass ratio and e_{si} is the static eccentricity of the irregular floor considered and n denotes the number of floors with irregularity. This irregularity index ‘α’ is applicable for buildings with eccentric mass irregularity along ‘n’ number of floors located at any height from the base of the building. The irregularity coefficient varies from a minimum of 0.0016 to a maximum of 2.74 for all the building configurations considered in the study and increases with increase in eccentricity as well as a change in position of irregularity from bottom floor level to the top floor level. The value of α is lowest for buildings with irregularity present at the bottom of the frames and the variation of α due to eccentricity becomes prominent when the irregularity location shifts from the bottom to the upper floor levels of the frame. The mass irregularity coefficients get almost doubled when the location of mass irregularity is shifted from the bottom to the top level of the buildings. α also varies remarkably with the height of the buildings and the magnitude of the increase is the same for all the mass ratios considered. α is obtained in the range of 0.0016 to 0.61 in group A buildings, 0.002 to 1.83 in group B buildings, and 0.0028 to 2.74 in group C buildings. The coefficient α also increases correspondingly with increase in the in-plan eccentricity ratio. Comparing buildings with mass ratio 1.25 to mass ratio 5 when e_d/L increases by 2.6 times, α increases by 25 times.

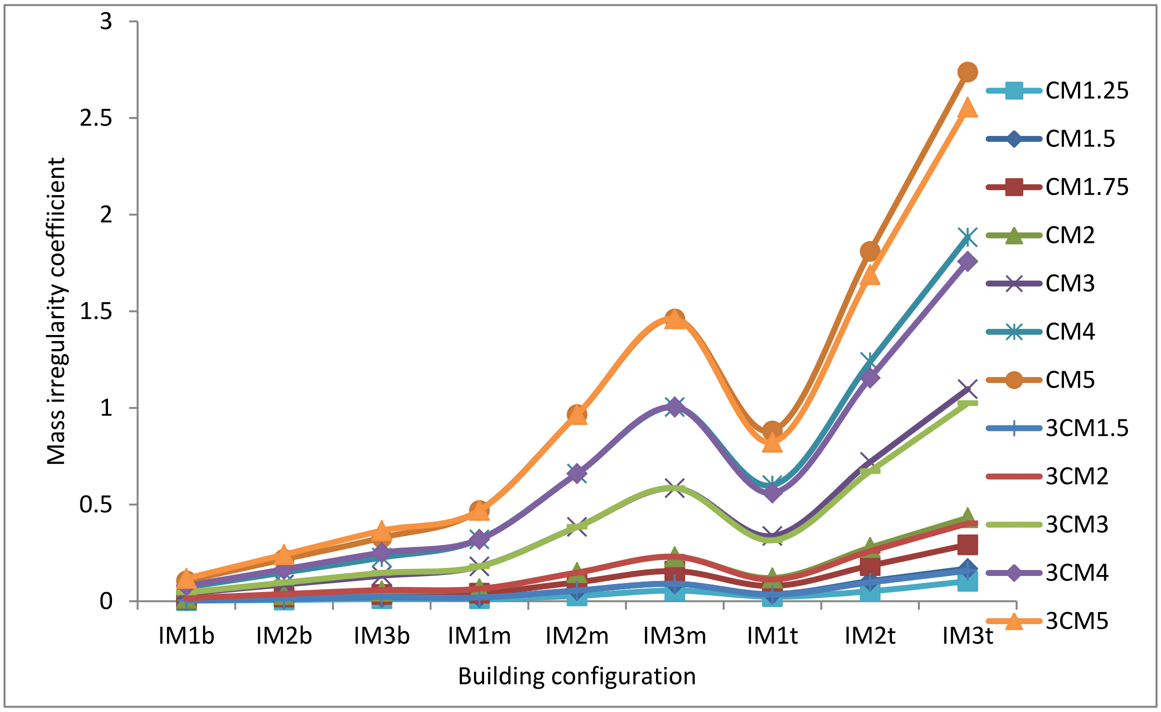


Figure 5.24 Mass irregularity coefficients for different irregular building configurations in group C

The pattern of variation of α with respect to the eccentricity of the buildings is identical in all the three aspect ratio variants. Figure 5.24 depicts the representative variation of mass irregularity coefficients of the building configurations in group C for various mass ratios of M1.25 to M5 which are placed in single (C) as well as distributed in three adjacent floor levels (3C). It can be observed that the higher mass ratios of 4 to 5 have very high α, which is almost 10 times of that obtained for the mass ratio variants of 1.25-2. Buildings with nominal mass ratios of 1.25 to 3 even with highest in-plan eccentricity have the irregularity indices within a range of 0.5 and this is generally the case of real buildings. Within this range of α from 0 to 0.5, with respect to regular frame buildings, natural period, base shear ratio, roof deflection ratio and roof rotation are higher by 41%, 57%, 66% and 122% respectively with respect to that of the regular frame buildings. Furthermore, when the same masses are distributed in two or three adjacent floors as in

2B and 3C, α is lower in comparison to that of B and C configuration buildings. Figures 5.25 and 5.26 show the variation of ratios of natural period (T_i/T_r) and base shear (B_i/B_r) of irregular buildings to that of the regular ones with respect to the proposed coefficient α where, T_i and B_i denote the natural period and base shear ratio of irregular building and T_r and B_r correspond to the natural period and base shear ratio of the regular building.

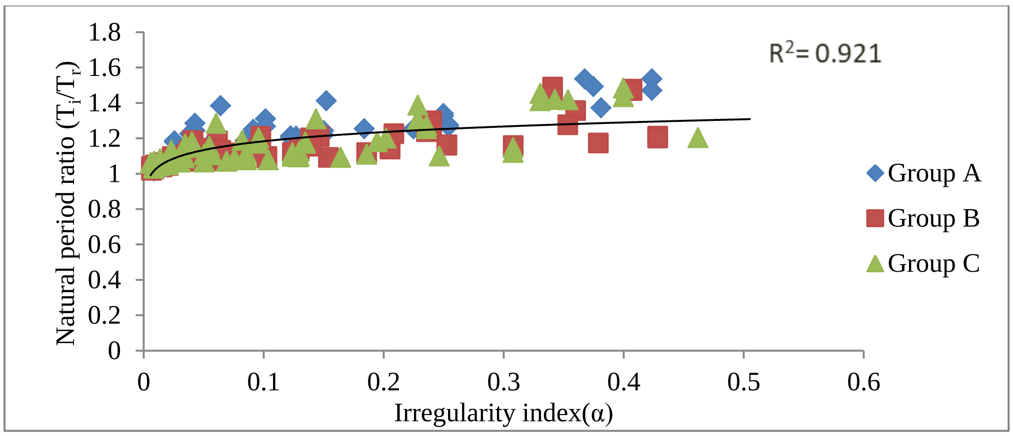


Figure 5.25 Variation of mass irregularity coefficient with natural period ratio

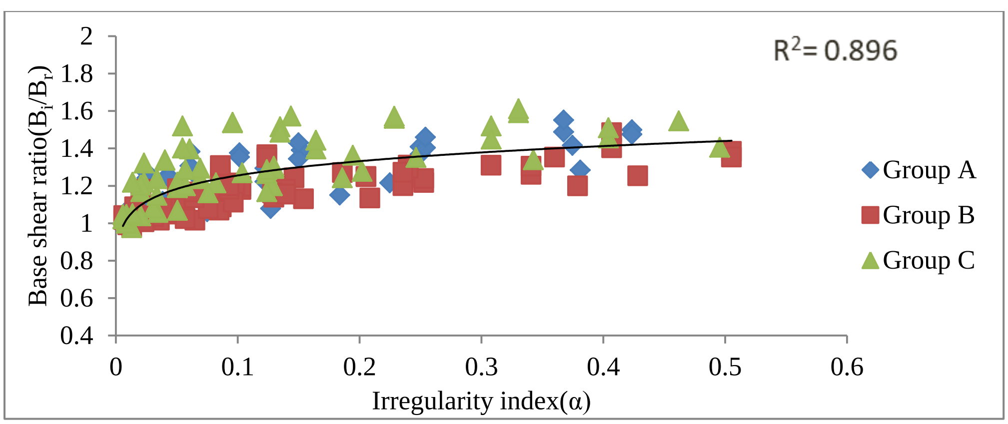


Figure 5.26 Variation of mass irregularity coefficient with base shear ratio

The variations of base shear ratio and natural period ratio with respect to the irregularity index for buildings of three different aspect ratios having α ≤ 0.5 give a well fit plot. In the case of group A buildings of M2 variant, when eccentric irregularity shifts from the bottom to the top level, α increases by 4 times and T_i/T_r ratio and B_i/B_r ratio increases by

32% and 25% respectively. Similarly, in the case of group B buildings of M1.5 variant, with shift in location of eccentrically placed mass irregularity from the bottom to the top level, when α increases by 5 times, T_i/T_r ratio and B_i/B_r ratio increases by 35% and 33% and in group C buildings, when α increases by 6 times, T_i/T_r ratio and B_i/B_r ratio increases by 40% and 44% respectively. Based on regression analysis, the best fit relations between the mass irregularity coefficient α, which incorporates the in-plan eccentricity, the natural period and the base shear ratios of irregular buildings are obtained as:

$$\frac{T_i}{T_R} = 1.365\alpha^{0.062} \tag{5.2}$$

$$\frac{B_i}{B_R} = 1.527\alpha^{0.085} \tag{5.3}$$

It can be observed that the base shear ratio has a better fit for the developed power equation, with respect to α, in comparison to the natural period ratio. Within a range of α from 0 to 0.5, which generally covers the irregularity of real buildings, both T_i/T_r ratio and B_i/B_r ratio have good compliance with α index and the equations developed can be used to predict the response of the buildings accurately. The base shear values in mass irregular buildings can be computed from the predicted values and the structural system can be properly planned for the proper location of irregular masses.

5.2.7. Summary

From the study carried out on buildings with irregular mass configurations, it can be inferred that the mass distribution in-plan and along the height is considerably influences the seismic response of a building. Therefore, the presence of heavy masses along the upper floor levels is highly critical, mainly when the masses are eccentric in in-plan distribution. Even if the masses are of smaller mass ratio of 1.5, if placed along the bottom half of the building with eccentricity, it can lead to increase of 10% to 22% in the natural period and 14 to 20% in base shear with respect to the regular frame building.

Based on the present investigations, a coefficient ‘α’ has been proposed which can be employed to quantify the mass irregularity in asymmetric buildings with in-plan eccentricity and thus determine the suitable placement of heavy eccentric masses without inducing high torsional coupling in such buildings located in earthquake prone areas. Also, the natural period and base shear values of any mass irregular building can be predicted with the help of the proposed mass irregularity coefficient and the structural system can be planned and designed suitably for the presence of eccentric masses in buildings located in seismically active areas.

5.3. Stiffness irregular buildings

The variations in seismic and torsional behaviour of 408 building frames due to inclusion of stiffness irregularities along the height as well as in plan were evaluated. Their absolute maximum responses of fundamental natural period, base shear ratio, roof rotation, torsional resultant, roof displacement and storey drift were obtained and their percentage difference with respect to the regular frames were also computed. A stiffness irregularity coefficient has also been proposed which quantifies the effect of in-plan eccentricity in buildings with soft stories.

5.3.1. Variation in fundamental natural period

The fundamental natural period of vibration is an intrinsic property of a structure and the natural periods of the stiffness irregular buildings were determined by carrying out Eigenvalue analyses on building frames. Figure 5.27 shows the variation of the fundamental natural period of 5, 10 and 15 storey frames represented as A, B and C group buildings with modified storey stiffness K_1 to K_4 as compared to that of the regular building frames. The natural periods of the regular buildings, 5R, 10R and 15R are obtained in the range of 0.578s – 1.85s and that of the IS0 configurations with vertical stiffness irregularity are in the range of 0.59s to 2.78s. Among the first set of buildings without any change in in-plan eccentricity or comparing among the IS0 configurations, it can be observed that the location of soft storeys with stiffness reductions at the base of the buildings increases the natural period. The buildings with soft stories at the ground floor level, has the highest variation in natural period in comparison to that of the regular frame buildings. Considering this variation, $CIS0_b$ with the modified storey stiffness K_4 at the bottom floor level has the highest variation in the natural period of 37% with respect to 15R.

Comparing the second set of buildings with in-plan eccentricity in combination with vertical stiffness irregularity, it can be observed that the inclusion of in-plan eccentricity increases the fundamental natural period in comparison to that of the in-plan regular IS0 buildings. This variation in natural period with respect to the regular buildings increases with increase in e_d/L of the configurations considered. Therefore, the variation of the natural period in buildings with stiffness irregularity along height as well as in-plan with respect to the in-plan regular buildings increases with reduction in storey stiffness which is further escalated due to change in e_d/L of the buildings. In the irregular frames with stiffness reduction up to 70% (K_1) and e_d/L less than 0.14, the variation of the natural period in buildings with soft stories at the bottom floor level is nominal and less than 20% with respect to IS0 frames and 44% with respect to regular frames. Whereas in the case of stiffness modification K_4 and e_d/L of 0.14, considerable variation up to 70% in natural period with respect to the regular buildings and 34% with respect to IS0 buildings can be observed. When the storeys at the bottom of the frame have reduction in stiffness along with in-plan eccentricity, natural period increases and become the highest in $IS8_b$ with a stiffness modification of K_4. Considering all the variations in stiffness and in-plan eccentricities, provision of modified inter-storey height and in-plan eccentricity at the bottom level of the buildings increases the natural period by maximum of 71.8 % in group A, 67% in group B% and 54.6% in group C, in comparison to corresponding IS0 buildings due to maximum torsional coupling. IS8 with highest e_d/L ratio has the highest natural period in all groups. It can thus be interpreted that with the inclusion of stiffness modification along the height of the building, the natural period increases and reaches a maximum when the stiffness modification is present in the bottom levels. When the columns at the upper floor level have modified stiffness in between K_3 to K_4, in combination with e_d/L of 0.3, maximum variation in natural period of 24% in group A, 38% in group B and 39 % in group C is observed with respect to the corresponding plan regular IS0 configurations. Hence, the introduction of in-plan eccentricity in soft stories even at the upper levels of buildings causes considerable variation in its natural period. The presence of stiffness irregularities tends to increase the overall flexibility of a structure and besides, the presence of torsion further amplifies the flexibility. Fundamental natural period being an inherent property of a structural system, any alterations in mass or stiffness in the buildings reflects directly in the change in the natural period of the building.

Frequency ratio, Ω which is the ratio of frequencies of the building in the torsional mode to that in the translational mode, decreases in all the three groups of buildings in the order of increasing ed/L. IS8 with the highest ed/L has the least Ω ratio among all the groups. Table 5.4 lists the frequency ratios of buildings of IS8 configuration of the three groups of buildings under 4 stiffness modifications K1 to K4. For the configurations from IS1 to IS8, Ω is obtained in the range of 1.215 to 0.730. Ω is less for the configurations with higher ed/L and hence the buildings with the maximum torsional coupling are AIS8b, BIS8b and CIS8b with modified stiffness K4 with frequency ratios of 0.828, 0.775 and 0.730 respectively. Furthermore, the general pattern which can be observed here is that the Ω values of the buildings tend to decrease with increase in the aspect ratio of the buildings and when the stiffness reduction of the particular storey increases, Ω consecutively increases.

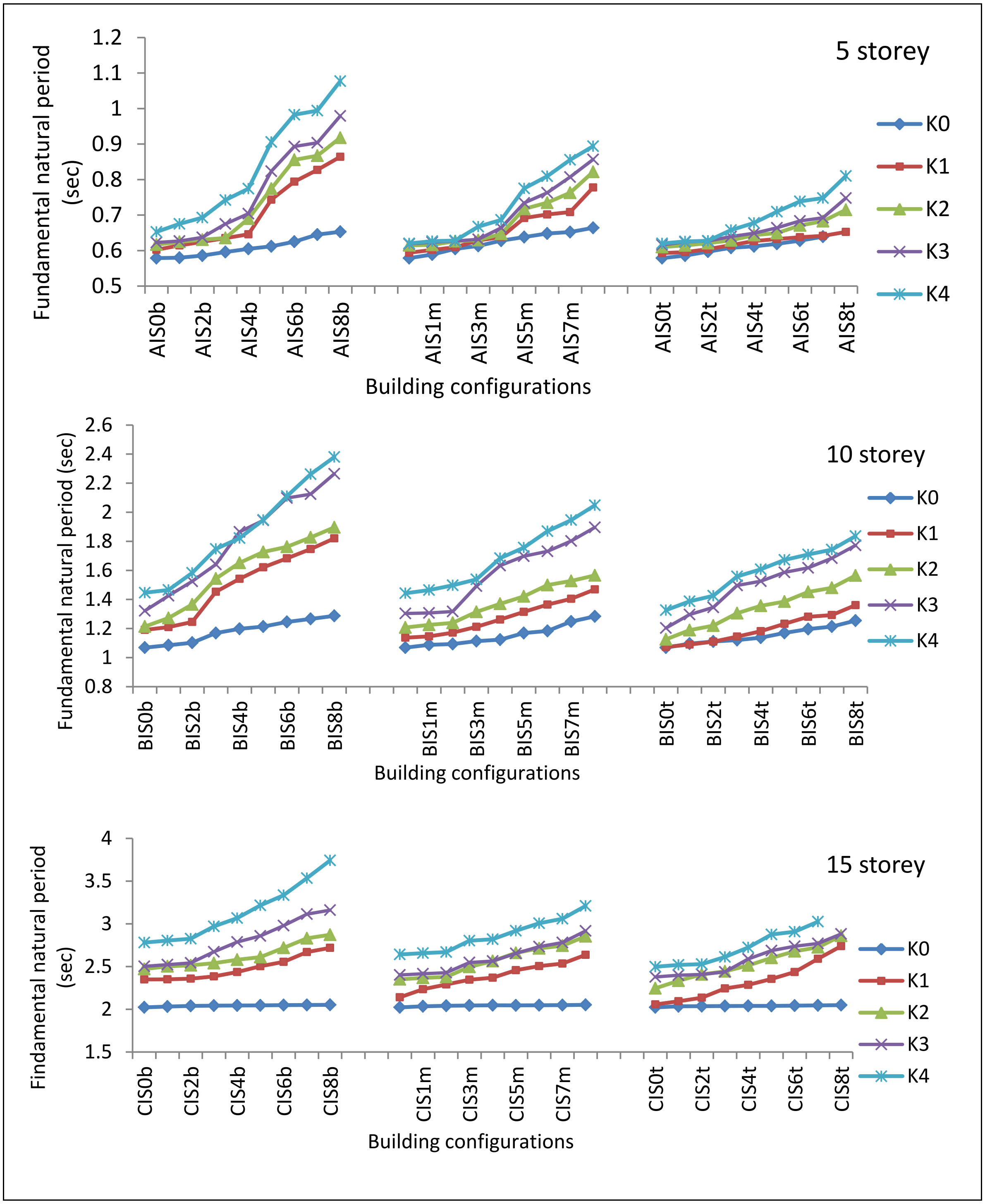


Figure 5.27 Variation of fundamental natural period in group A, group B and group C stiffness irregular building

Table 5.4 Frequency ratios of the stiffness irregular buildings

Frequency ratio (Ω)									
Building configuration	**K_1**	**K_2**	**K_3**	**K_4**	**Building configuration**	**K_1**	**K_2**	**K_3**	**K_4**
$AIS8_b$	1.04	0.925	0.838	0.828	$AIS8_b$	1.491	1.323	1.198	1.184
$AIS8_m$	1.07	0.904	0.845	0.845	$AIS8_m$	1.534	1.293	1.208	1.208
$AIS8_t$	1.11	0.863	0.869	0.86	$AIS8_t$	1.589	1.234	1.243	1.230
$BIS8_b$	1.00	0.936	0.821	0.775	$BIS8_b$	1.431	1.338	1.174	1.108
$BIS8_m$	1.00	0.881	0.834	0.811	$BIS8_m$	1.436	1.260	1.193	1.160
$BIS8_t$	1.04	0.869	0.841	0.846	$BIS8_t$	1.490	1.243	1.203	1.210
$CIS8_b$	0.95	0.925	0.732	0.73	$CIS8_b$	1.366	1.323	1.047	1.044
$CIS8_m$	0.99	0.904	0.752	0.749	$CIS8_m$	1.424	1.293	1.075	1.071
$CIS8_t$	1.00	0.863	0.811	0.78	$CIS8_t$	1.436	1.234	1.160	1.115

5.3.2. Variation in seismic base shear ratio

The seismic base shear ratio (the base shear expressed in terms of the total seismic weight (W) of a building) of all the buildings, subjected to El-Centro ground motion is as shown in Figure 5.28. It can be observed that the seismic base shear ratios of the buildings considered here decrease with an increase in the aspect ratio of the buildings. The base shear ratios of the 15R, 10R and 5R are obtained in the range of 0.061W to 0.118W. It can be observed that the location of the soft storey along the building height does not give significant variation in base shear ratio. The comparatively higher base shear ratio is observed in building with soft storeys or stiffness reduction at the lower floors of the building. However, due to the presence of soft storeys at various locations along the building height, a decrease in base shear ratio is observed in comparison to the regular frames. This decrease in base shear can be attributed to the increase in the natural period in buildings due to the presence of stiffness irregularity leading to increased flexibility in the buildings. While considering the initial set of buildings without eccentricity, due to the presence of the soft storey at the lower floor level, the base shear ratio decreases by a maximum of 25% in $CIS0_b$ in comparison to the regular buildings.

In the second set of irregular frames, considering the combination of vertical as well as in-plan stiffness irregularity in the bottom half of the frames, the variation in base shear ratio due to stiffness modification is amplified remarkably by the incorporation of in-plan eccentricity. The highest decrease in base shear due to in-plan eccentricity with respect to the in-plan regular IS0 frames are observed in $AIS8_b$, $BIS8_b$ and $CIS8_b$ configurations with a modified storey of stiffness K_4 as 18%, 22% and 36% respectively. The highest decrease of base shear in buildings with in-plan eccentricity with respect to the 15R is obtained in $CIS8_b$ as 51%. Considering the buildings with the soft storey of stiffness reduction K_1 and e_d/L of 0.14 at the lower floor levels, the maximum decrease in base shear ratio is observed as 27% with respect to regular buildings and 18% with respect to IS0 frames. When buildings with reduced storey stiffness K_1 at the lower levels which have e_d/L of 0.3, the maximum decrease in the base shear ratio is 35% with respect to the regular frame (R) and 25% with respect to plan regular frame (IS0). However, the presence of the same irregularities at the upper floor levels, leads to a maximum variation of 20% with respect to that of R configuration and 18% with respect to that of IS0 configuration. Whereas considering the combination K_4 with e_d/L of 0.3 at the upper floor levels, the maximum decrease in base shear becomes 28% with respect to the regular frame and 22% with respect to IS0 frame. Therefore the in-plan eccentricity can be said to cause a considerable decrease in the base shear of stiffness irregular buildings which have the soft storey at any floor level. Figure 5.29 shows the time history plots of base shear in $CIS0_b$ and $CIS8_b$ buildings in comparison to that of 15R building.

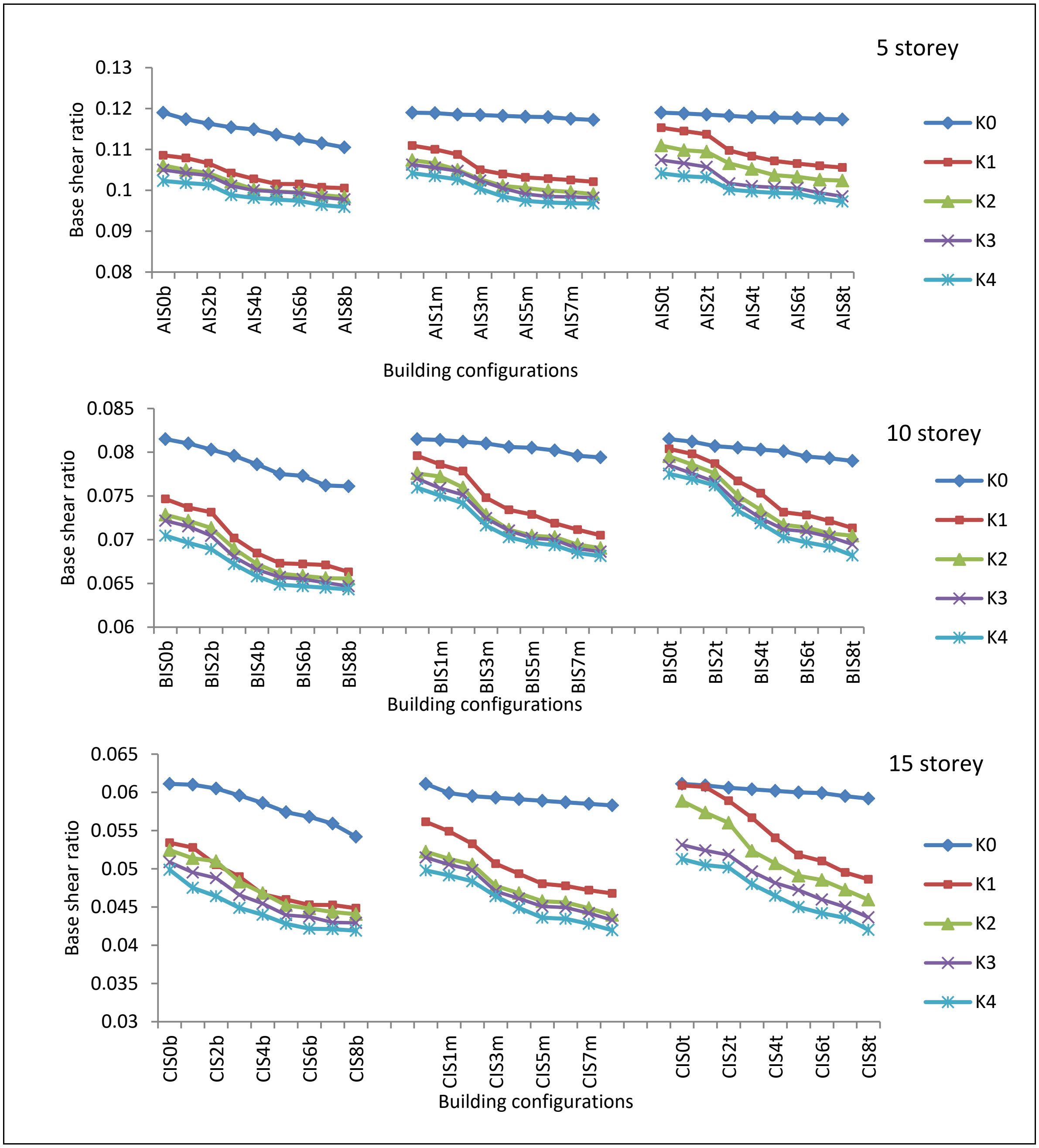


Figure 5.28 Variation in seismic base shear ratio in group A, group B and group C stiffness irregular buildings

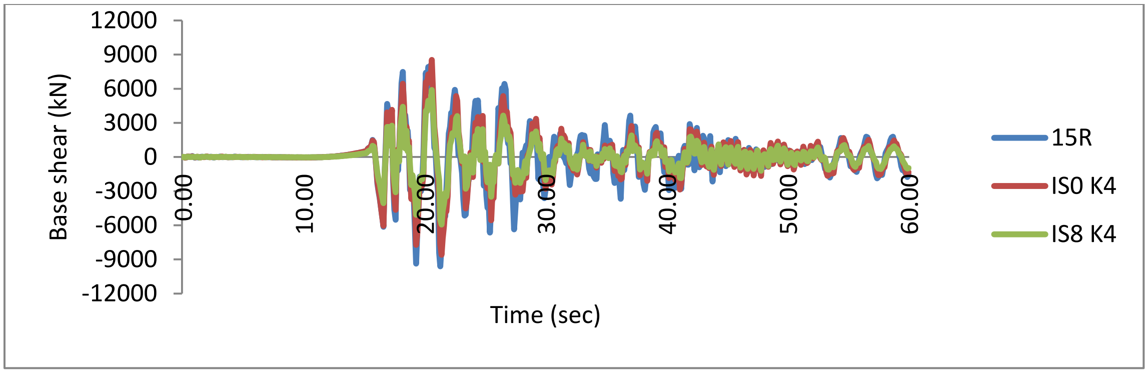


Figure 5.29 Time history of base shear of 15R and stiffness irregular buildings $CIS0_b$ and $CIS8_b$ with stiffness reduction K_4

5.3.3. Variation in roof deflection

The roof deflections of buildings with various stiffness irregularity locations under the application of El-Centro ground motion are represented in Figure 5.32. Roof deflection values are expressed in terms of the height of the buildings (H) and these roof deflection ratios of the 5, 10 and 15 storey regular buildings are in the range of 0.0052H to 0.0085H. It can be observed that due to stiffness reduction in storeys along the height of the building, roof deflection increases. The increase in roof deflection is proportional to the increase in aspect ratio and also to the location of irregularity. The maximum roof deflections of the frames are higher when the soft stories are present at the bottom of the frames in all the aspect ratio variants. The variation in roof deflection ratios of the stiffness irregular buildings with respect to that of the regular buildings increases as the location of irregularity shifts downwards from the upper floor level to the bottom floor level. Among the group C buildings, when the soft storeys with stiffness modification K_4 are located at the lower floor level, the maximum roof deflection ratio increases by 80% with respect to that of 15R building configuration. In group A, group B and group C buildings with stiffness reduction up to 70% (modified stiffness K_1) at the lower storey, 18%, 59% and 63% individual variations in roof deflection ratio are observed in comparison to that of the regular buildings. Considering the second set of buildings, the variation in maximum roof deflection due to stiffness modification along the height is further escalated by the incorporation of in-plan eccentricity up to a maximum of 68% in group C buildings. The highest roof deflection of 0.016H is observed in $CIS8_b$ building with a stiffness modification K_4. Nominal variations of 5% to 18% in group A, 4% to 20% in group B and 5% to 21% in group C in roof deflection ratios are observed in irregular buildings with stiffness modification K_1 at the upper storey with respect to the plan regular $IS0_t$ buildings. However, in buildings with stiffness modification K_1 at the lower floors, the roof deflection ratios increase by a maximum of 34% with respect to IS0 and 40% with respect to the regular frame.

In group C buildings, roof deflection ratio has a maximum variation of 115% with respect to that of the regular buildings and 68% with the IS0 buildings due to the effect of the in-plan eccentricity of 0.3L in combination with soft storey of stiffness reduction K_4 at the lower floor level. Buildings with e_d/L of 0.14, has a maximum variation of 45% and 23% with respect to that of IS0 frames when the stiffness irregularities are present at the bottom and the top storey levels respectively. However, when the same in-plan irregularities have an e_d/L of 0.3, the maximum variation in roof deflection becomes 68% and 34% with respect to that of IS0 frames. Figure 5.31 shows the time history plot of roof deflection in $CIS0_b$ and $CIS8_b$ buildings with stiffness reduction K_4.

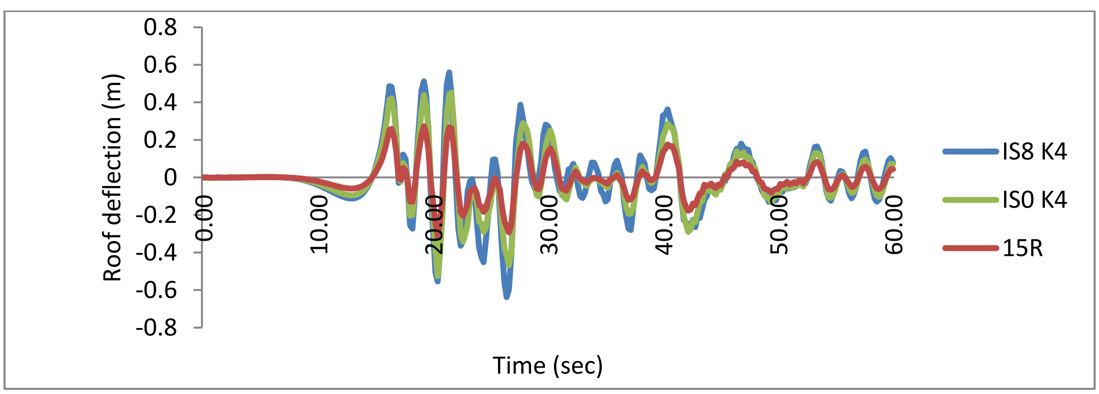


Figure 5.31 Time history of roof deflection of 15R and stiffness irregular buildings $CIS0_b$ and $CIS8_b$ with stiffness reduction K_4

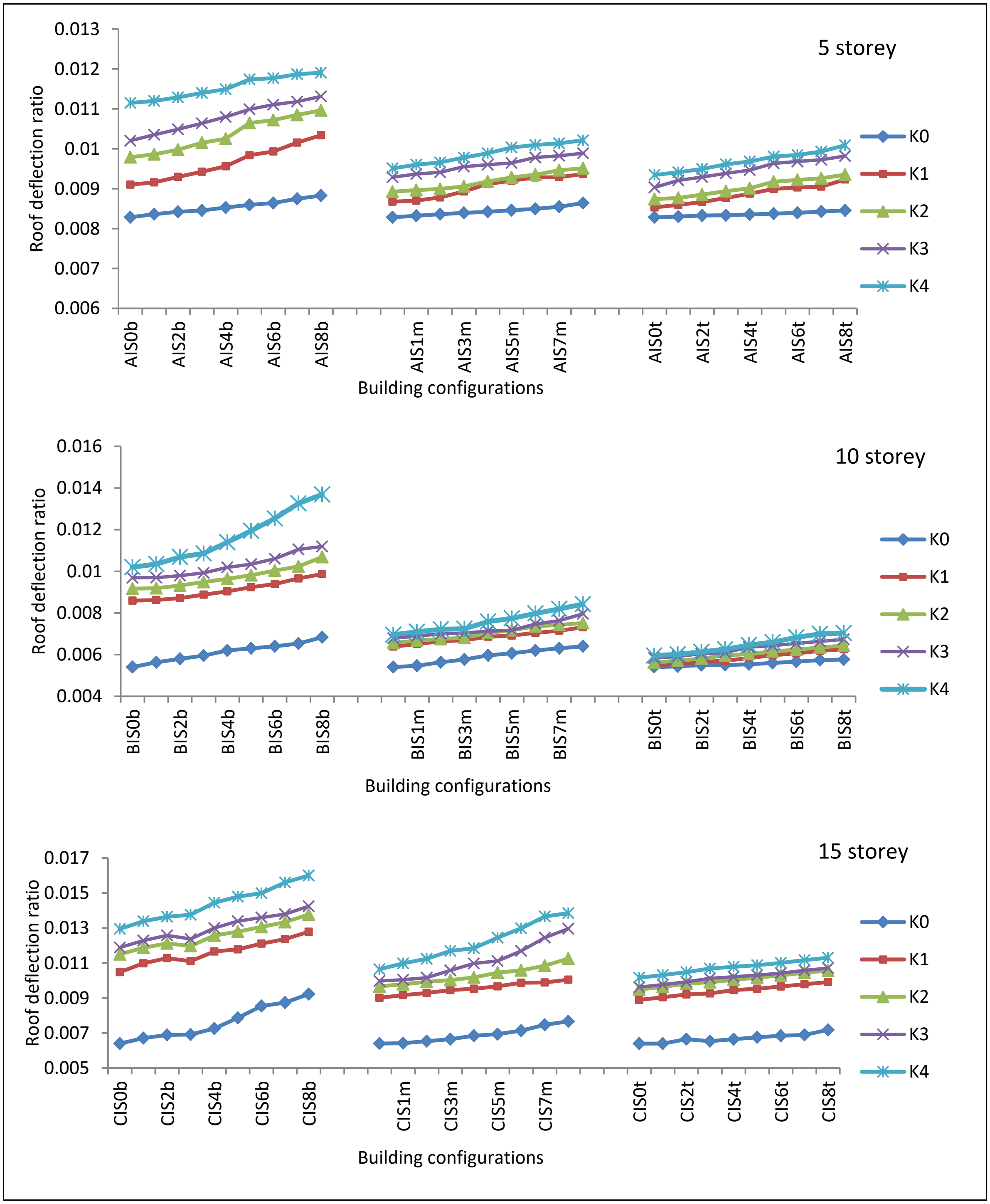


Figure 5.32 Variation in roof deflection ratio in group A, group B and group C stiffness irregular buildings

5.3.4. Variation in storey drift

Storey drift relates to the lateral deflections within a building or can be defined as the lateral displacement of one level of a multi-storey building relative to the level below it. The greater the storey drift, the higher the likelihood of damage in the building. IS 1893:2016 specifies that storey drift should not be greater than 0.004 of the storey height under the action of the design base shear. IBC 2015 sets the maximum drift for regular buildings between 0.7% and 2.5% of storey height, while EC 8 specifies between 1% and 1.5%. The inter-storey drift or the difference in the displacements of two consecutive floor levels normalized by the inter-storey height of the IS0 and IS4 configurations belonging to all the three groups of buildings are shown in Figures 5.33 to 5.35. The IS4 configuration has an e_d/L of 0.14 which is the mean of the dynamic eccentricities of the stiffness irregular patterns considered here. It can be observed that the storey drift pattern shows a significant variation at the location of stiffness irregularity and this variation is substantially accentuated when in-plan eccentricity is also present in the buildings. The presence of soft storey remarkably increases the storey drift demand in the particular storey level and reduces the drifts in the others. Therefore storey drift demand is susceptible to the variation in stiffness along the height of a building. The pattern of variation of storey drifts at the vicinity of the stiffness irregularity in the buildings is similar in all the groups considered here. For the in-plan regular models with soft stories, increase in the first-storey height has great influence on storey drift response. The storey drifts are higher in 5 storey buildings in comparison to the 10 and 15 storey buildings. The maximum storey drifts of IS0 buildings in 5, 10 and 15 storey variants are 1.25%, 0.89% and 0.85% respectively.

In the case of plan irregular buildings, the in-plan stiffness eccentricity increases the storey drift demand in comparison with the IS0 buildings. The maximum storey drift of IS4 building with in-plan eccentricity are 1.5%, 1.15% and 0.92% in group A , group B and group C buildings respectively.

Considering the buildings with soft storeys having stiffness modification K_4 at the lower floor levels, the variation of storey drift in IS4 for 5 storey, 10 storey and 15 storey buildings are 38%, 42% and 52% in comparison to that of IS0 buildings. This variation increases to 72%, 85% and 89% in the case of IS8 buildings for an e_d/L of 0.3. In comparison to the storey drift demands of the regular buildings, the drift demand of the IS4 configuration with stiffness modification K_4 at the lower floor level increases by 86% to 273%, by 72% to 115% in the case of soft storey at the middle floor level and by 75% to 156% in the case of soft storey at the top floor level. When the stiffness modification K_1 is present at the lower floor levels, increase of 51% with respect to IS0 building and 108% with respect to regular buildings is observed. Comparing the variation of storey drifts obtained in buildings with stiffness modification, K_1 and K_4, the highest variation of 56% is obtained in the frames with soft stories at lower floor levels. Storey drift pattern along with the roof displacement comprises the global deformation of the building and therefore even slight variation in the pattern of stiffness eccentricity especially in the ground floor is found to cause considerable changes in the seismic response of the building in totality. Hence it can be observed that storey drift of the group A buildings with stiffness irregularities along the bottom floor level exceeds 0.004 times the storey height as per limited by IS 1893:2016. When in-plan eccentricity is present along with the soft storey effects, storey drifts further increases. Whereas in the case of group B and group C buildings the limits are exceeded when soft storey is present at the bottom floor level in combination with stiffness eccentricity greater than 0.14L.

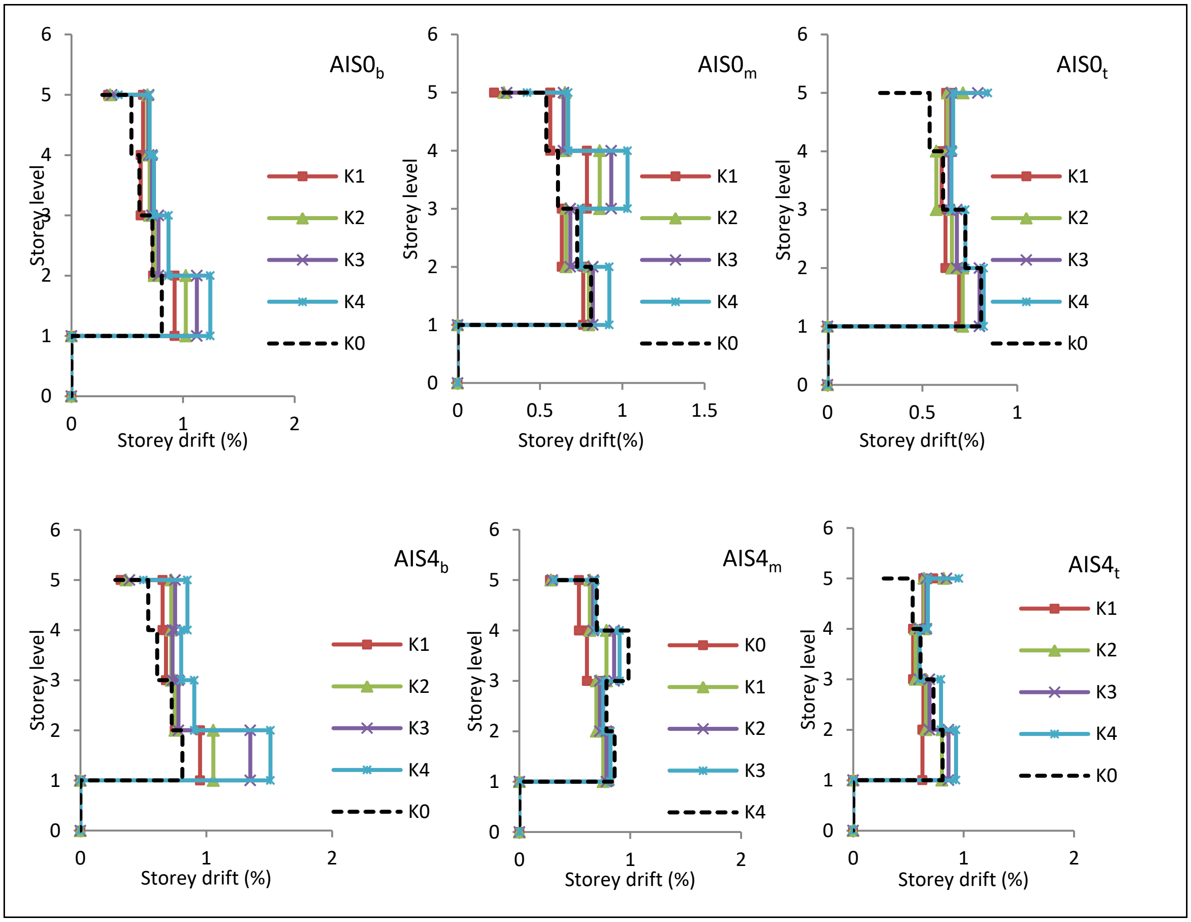


Figure 5.33 Variation in storey drifts patterns on IS0 and IS4 in group A buildings

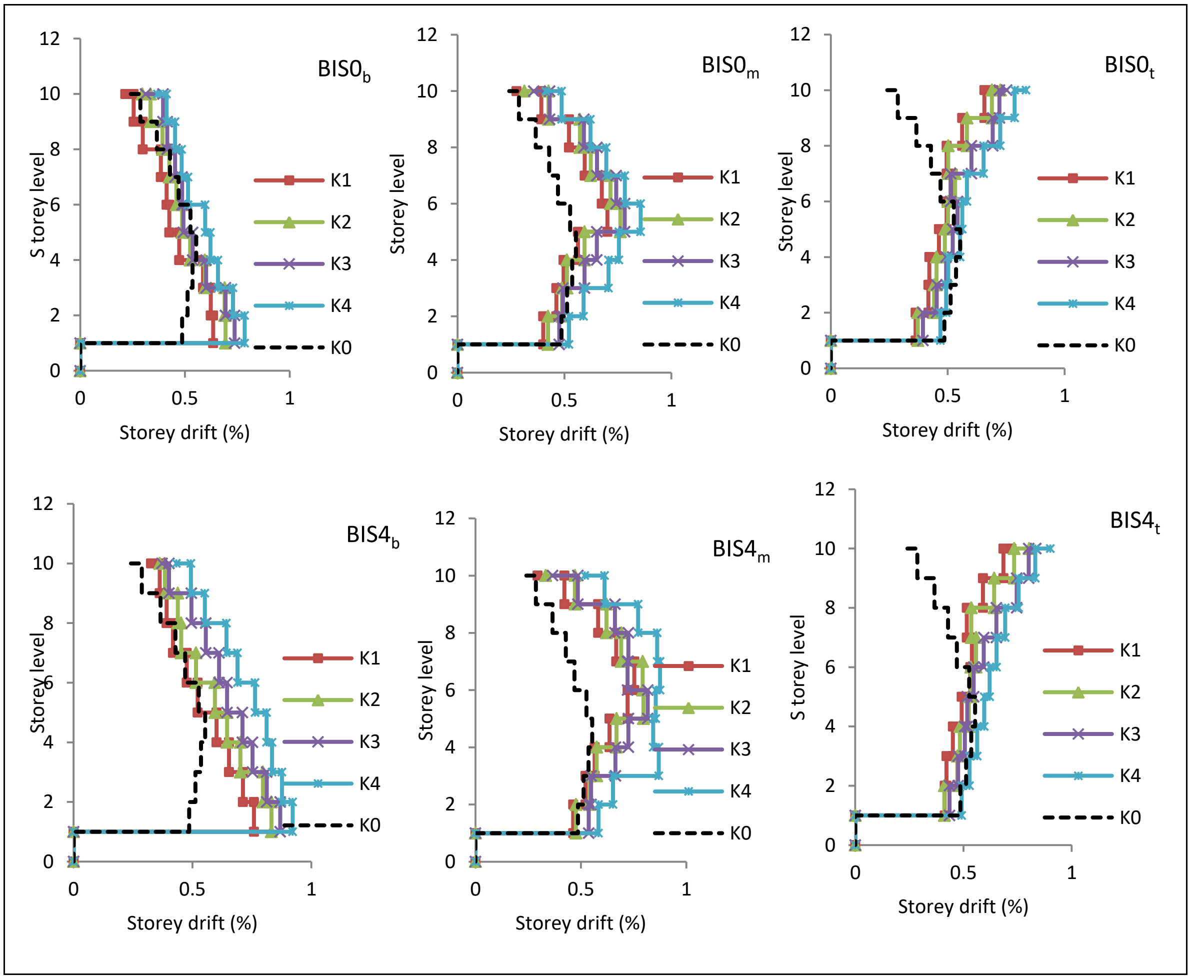


Figure 5.34 Variation in storey drifts patterns on IS0 and IS4 in group B buildings

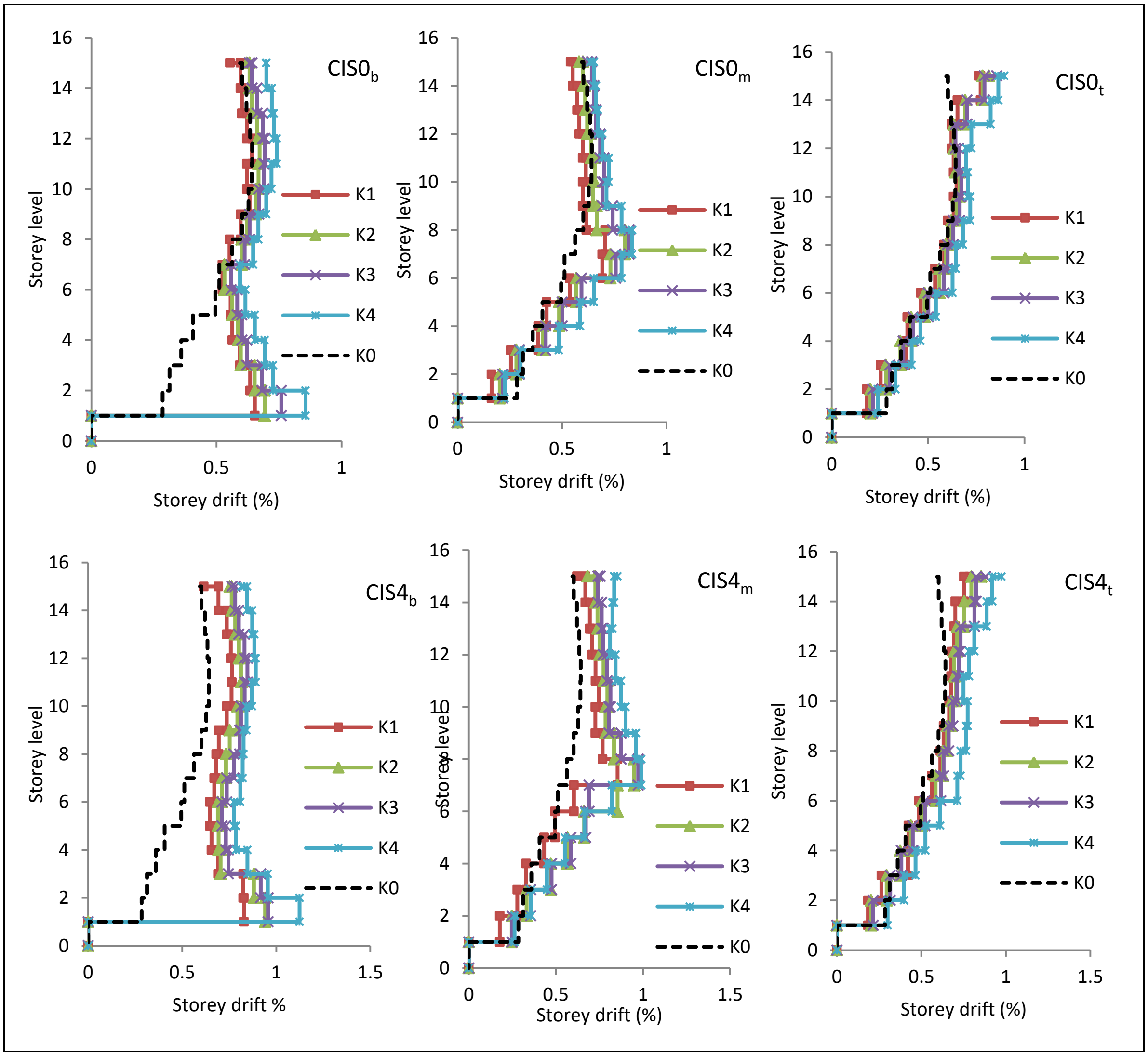


Figure 5.35 Variation in storey drifts patterns on IS0 and IS4 in group C buildings

5.3.5. Variation in roof rotation

The roof rotation is estimated by considering the highest storey displacements of the extreme corners of the roof of the building. Further, roof rotation in radians is obtained by dividing the relative displacement at the corners of the roof by the plan width of the building. From the initial set of vertically stiffness irregular buildings, it can be observed that the maximum roof rotation increases with increase in the aspect ratio of the buildings and correspondingly the 15 storey buildings have the highest roof rotation. It can be seen from Figure 5.36 that when the soft storeys are present in the bottom floor level of the building, the roof rotation is the highest due to the increased flexibility at the base of the buildings giving rise to torsional moments which leads to rotation of the building. The roof rotation of group C buildings with a modified stiffness K_4 at the bottom floor level increases by 2.5 times in comparison with that of the regular frames. Whereas, in the case of modified stiffness K_1 at the lower floor level, the maximum roof rotation increases by 150% with respect to that of the regular building.

Among the second set of buildings with in-plan eccentricity, it is observed that distinct variation is present between the roof rotations of the building configurations IS1-IS8 with respect to that of IS0 buildings. IS8 configuration with the highest e_d/L has the highest roof rotation, in all the aspect ratio variants. The maximum rotation among all the buildings with in-plan eccentricity is observed in $CIS8_b$ with a stiffness modification K_4 at the lower floor as 0.077 rad; whereas the minimum rotation was observed in $AIS1_t$ with a modified stiffness K_1 at the upper floor level as 0.0115 rad. The roof rotation of $CIS8_b$ becomes 3.5 times as that of $CIS0_b$ due to an e_d/L of 0.3 in in-plan stiffness distribution and stiffness reduction K_4at the bottom floor levels. However, $CIS8_t$ with stiffness modification K_1 at the upper floor levels increases the roof rotation considerably by a maximum of 150% with respect to the CIS0 buildings. Considering buildings with an in-plan eccentricity of 0.14L at the upper floor level, roof rotation has a variation of 60-120% and at the lower floor level, it has a variation of 96-196% with respect to the corresponding IS0 building. When soft storey with stiffness reduction K_1 is present at the bottom half of the buildings along with e_d/L less than 0.1, the increase in roof rotation reaches a maximum of 85% in comparison to that of the vertically irregular IS0 buildings. This implies that incorporation of in-plan eccentricity significantly escalates variation in roof rotation due to stiffness irregularities especially when the soft storeys are located at the lower floor levels. Therefore in order to reduce the torsional effects due to stiffness irregularities it is recommended to avoid any change in stiffness at the bottom floor of a building. Also if any variation in stiffness is unavoidable at the ground floor levels, it should be ensured that they are kept clear of any in-plan eccentricity since even the slightest of eccentricity can induce root rotations due to torsion.

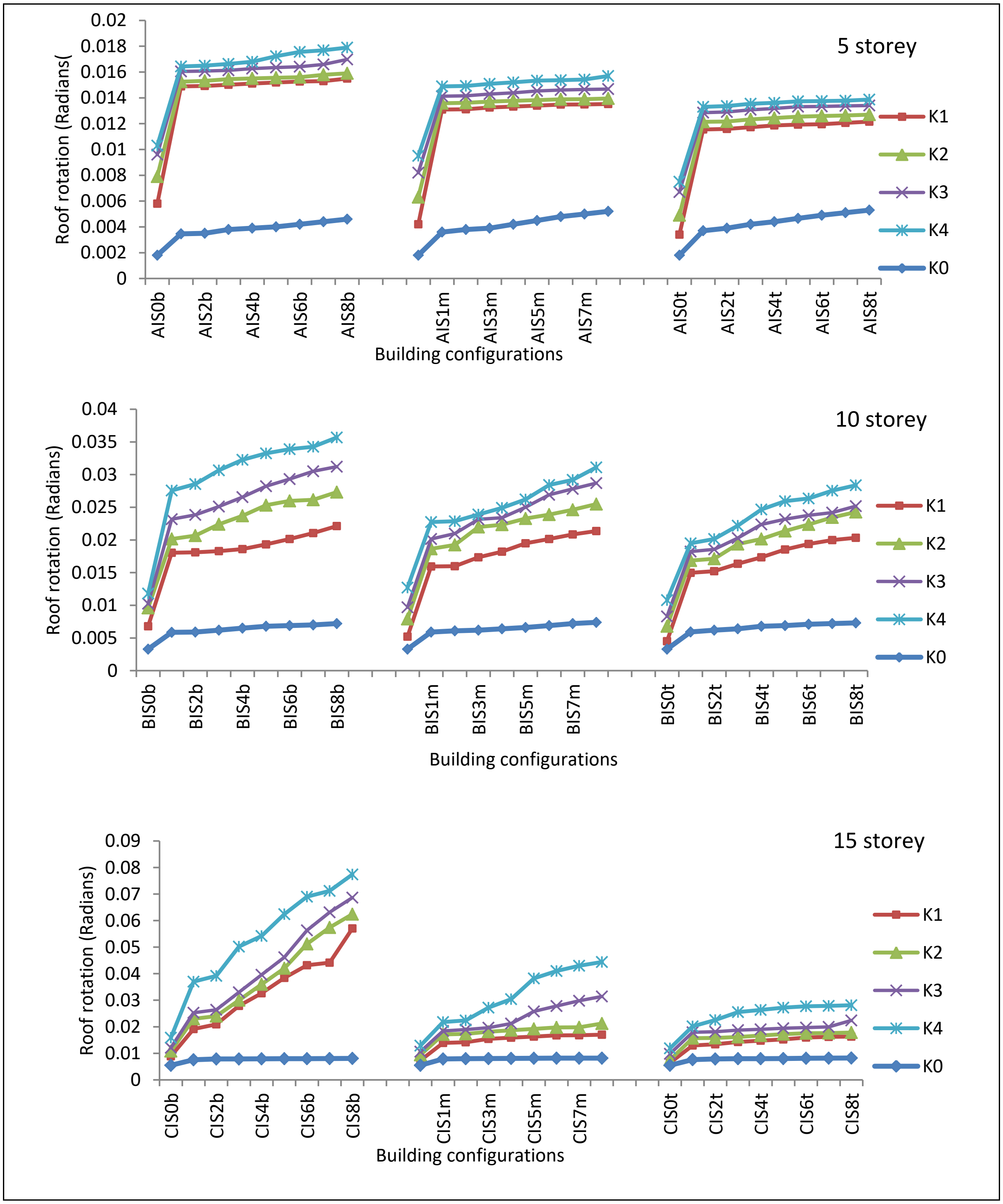


Figure 5.36 Variation in roof rotation in group A, group B and group C stiffness irregular buildings

5.3.6. Variation in torsional resultant

An accurate evaluation of the torsional response is quite complex because the coupled lateral-torsion vibration modes of the entire building are to be taken into account by carrying out 2D or 3D response assessment. Approximately, the torsional resultant or the highest torsional moments in the corner columns are obtained from the transient analysis. The static torsional responses are determined by computing the twist in the buildings implied by the roof rotation characteristics and the torsional moment induced in the columns. The torsional resultants are observed to increase with an increase in the aspect ratio of the buildings. The variations of torsional moments in the corner column of the stiffness irregular building configurations are shown in Figure 5.37. The torsional moments of the buildings with soft storeys at the lower floor levels have a maximum variation of 51% with respect to that of the buildings with soft storeys at the upper floor levels. The torsional moments of the corner columns are the highest in the buildings with soft storey at the bottom floor level. The torsional moment increases with an increase in stiffness reductions of the irregular buildings considered. The torsional resultants of the buildings with stiffness reduction K_1 is higher by a maximum of 45% with respect to the K_0 buildings. Considering $IS4_b$ with an e_d/L of 0.14 has a maximum increase in torsional moment by 69% with respect to that of $IS0_b$. Comparing $IS4_b$ and $IS8_b$ buildings, torsional moments are higher in the latter as compared to the former by a maximum of 140%. In the case of buildings with irregularities at the upper floor level, $IS4_t$ with an e_d/L of 0.14 has a maximum increase in torsional moment by 48% with respect to that of $IS0_t$. Torsional resultants in the columns increase significantly due to the presence of in-plan eccentricity. Therefore any variation in stiffness should be distributed throughout the floor without causing in-plan stiffness eccentricities. In unavoidable circumstances, the columns of the irregular building should be adequately designed so as to minimize the occurrences of any structural damages due to the torsional moments developed. Figure 5.38 shows the times history of torsional moments in the corner columns of $CIS0_b$ and $CIS8_b$ buildings with stiffness reduction K_4 in comparison with that of 15R.

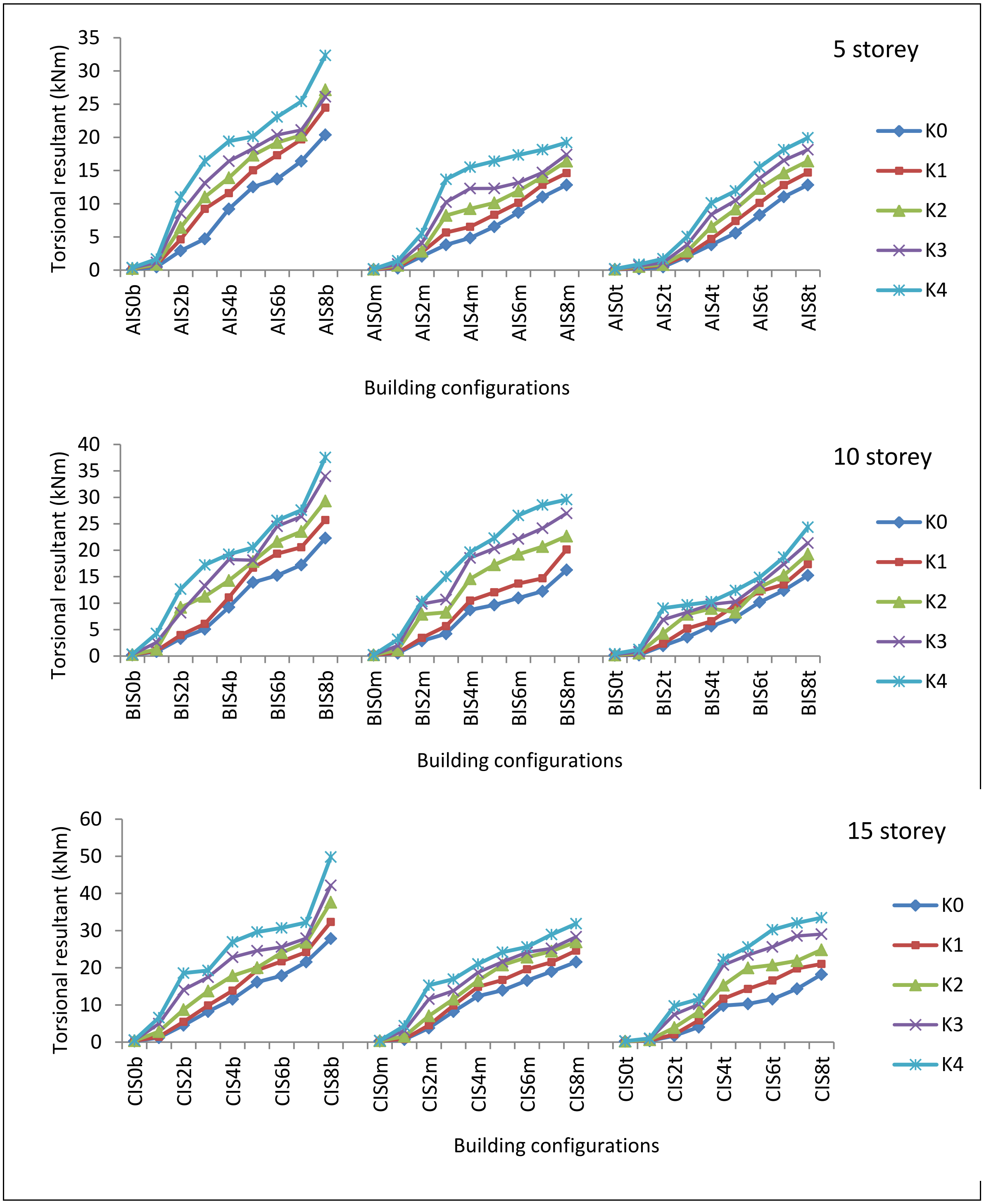


Figure 5.37 Variation in torsional resultant in group A, group B and group C stiffness irregular buildings

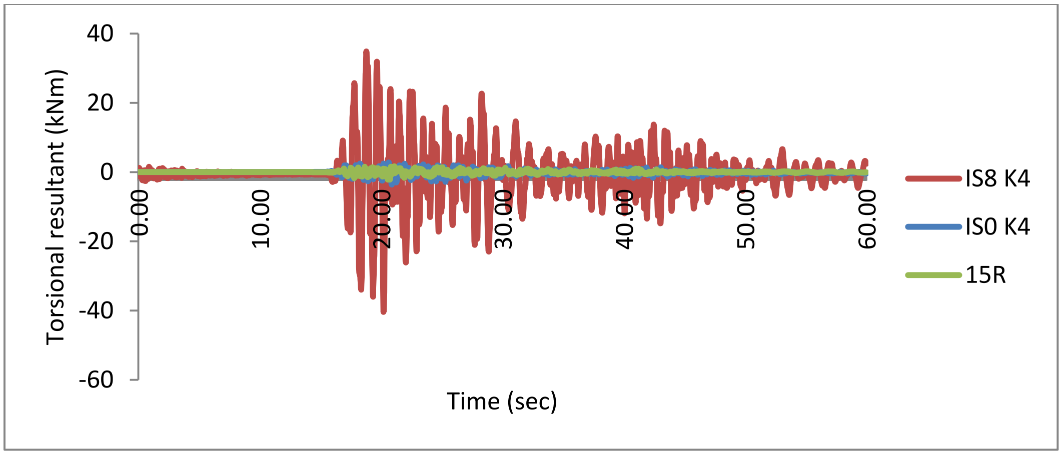


Figure 5.38 Time history of torsional resultant of 15R and stiffness irregular buildings CIS0$_b$ and CIS8$_b$ with stiffness reduction K$_4$

5.3.7. Stiffness irregularity coefficient

In the present work, to study the combination of in-plan eccentricity and stiffness irregularity and the variation of the different parameters associated, a stiffness irregularity coefficient 'β' is proposed as given below,

$$\text{Stiffness Irregularity Coefficient}, \beta = \frac{R_i}{h_{ti}} \frac{e_{di}}{S_{ri}} \tag{5.4}$$

At any storey level i, S_{ri} is the stiffness modification ratio or the ratio of the stiffness of the ith storey with irregularity due to increased inter-storey height to that of the regular storey, e_{di} defines the in-plan dynamic eccentricity, R_i denotes the fraction of the height over which the irregularity is considered and h_{ti} indicates the height of the soft storey from the ground level. This coefficient 'β' includes the effects of in-plan eccentricity and stiffness irregularity for buildings with equal dimensions along the direction of seismic excitation and in the transverse direction. This coefficient is applicable for buildings with eccentric stiffness irregularity at any height from the base of the building.

Considering the set of stiffness irregular buildings, β varies from 0.0016 to 0.7 and increases with eccentricity as well as the change in location of irregularity from the upper level to the lower floor level. The coefficient is obtained in the range of 0.014-0.7 for group A buildings, 0.0036-0.36 for group B buildings and 0.0016-0.24 for group C buildings. 'β' is lowest for buildings with stiffness irregularity present at the top floor level and becomes prominent when the irregularity is present in combination with the in-plan eccentricity at the bottom floor level of the frame. The value of β for the buildings with eccentric stiffness irregularities at the upper floor level is 70-90% lower in comparison to the buildings with stiffness irregularities at the lower floor levels. β also varies remarkably with the stiffness reduction of the buildings considered and the magnitude of increase of β in buildings with stiffness reduction K_4 with respect to those with stiffness reduction K_1 in between 77-120%. The pattern of variation of β with respect to the eccentricity of the buildings is similar in all three aspect ratio variants for the particular storey level considered. β of IS8 configuration is 3.28 times as that of IS1 configuration and 5 times as that of IS0 configuration. Figure 5.39 shows the variation of irregularity indices of the building configurations considered in this study. Buildings up to 15 storey height with soft storey at the ground floor level with a stiffness reduction up to 70%, have β in the range of 0.027 to 0.08 and if in-plan eccentricity is also present, β is in the range of 0.16 to 0.48.

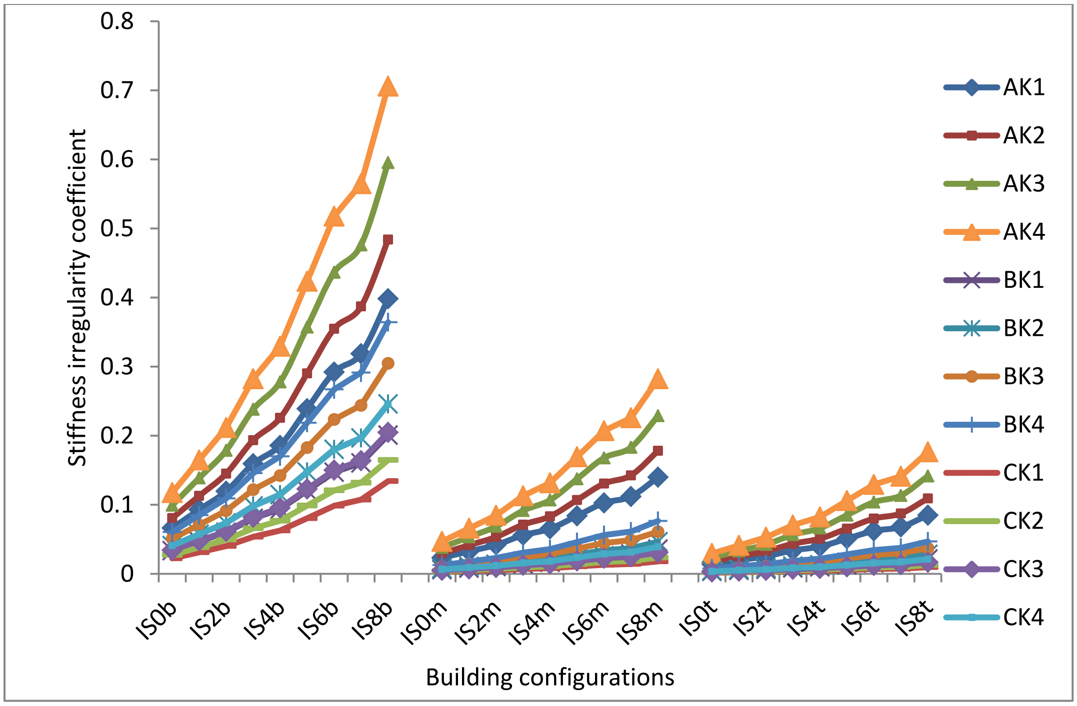


Figure 5.39 Variation of stiffness irregularity coefficient for all the buildings

As shown in Figure 5.39, AK_1 to AK_4 correspond to the group A buildings with stiffness reduction K_1 to K_4 and similarly in group B and group C buildings. In the present study, buildings with stiffness reduction in the range of 80% to 40% along with the highest dynamic eccentricity of 0.3L have β in the range of 0.0016 to 0.7. Within this range of β, natural period, base shear ratio, and roof deflection ratio of the irregular buildings are higher by 98%, 51%, and 115% respectively with respect to that of the regular frame buildings. Further, the variation of the ratio of natural periods (T_i/T_r) and base shear ratios (B_i/B_r) of the stiffness irregular buildings to that of the regular one with respect to the proposed stiffness irregularity coefficient β was evaluated. T_i and B_i correspond to the natural period and base shear ratio of irregular building and T_r and B_r correspond to that of the regular one. Considering buildings with e_d/L in the range of 0.05-0.3 in combination with soft stories in the lower floor level with a stiffness reduction of 80% and β in the range of 0.022-0.39, T_i is obtained in the range of 1.04-1.17 times T_r and B_i in the range of 0.91-0.73 times B_r. Similarly, for a stiffness reduction of 40%, T_i is obtained in the range of 1.12-2.3 times T_r and B_i in the range of 0.85-0.68 times B_r. Based on non-linear regression analysis, the best fit relations to represent T_{ip} (predicted natural period of irregular building) in terms of T_r and β and B_{ip} (predicted base shear ratio of irregular building) in terms of B_r and β is obtained as,

$$T_{ip} = \beta^{0.067} {T_r}^{1.71} \tag{5.5}$$

$$B_{ip} = \beta^{-0.021} {B_r}^{0.807} \tag{5.6}$$

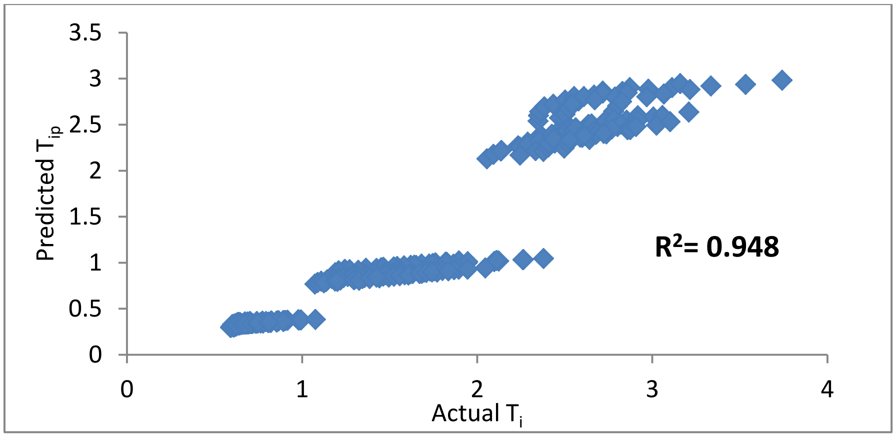


Figure 5.40 Actual T_i from dynamic analysis versus predicted T_{ip}

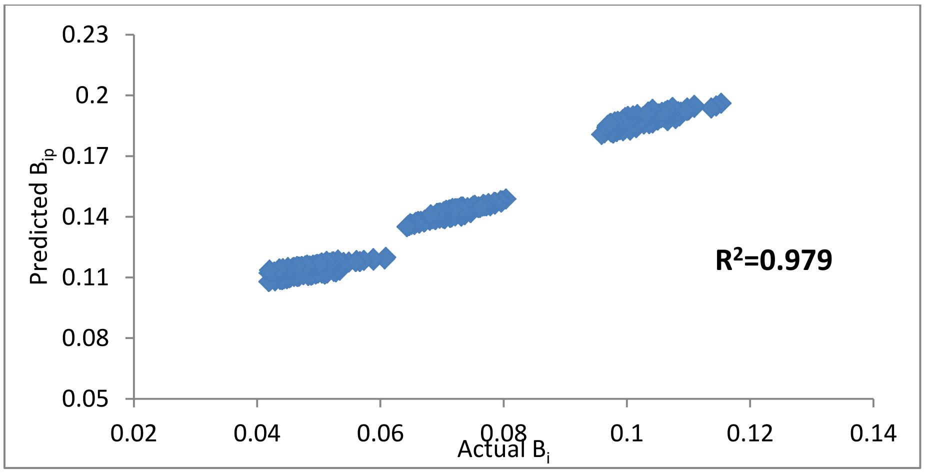


Figure 5.41 Actual B_i from dynamic analysis versus predicted B_{ip}

Figures 5.40 and 5.41 show the plots between the actual values of T_i and B_i obtained from the seismic analysis of the irregular buildings and the predicted values of T_i and B_i obtained from the proposed Equations 5.5 and 5.6. The model developed for predicting the natural period and base shear ratio of an irregular building on the basis of the proposed stiffness irregularity coefficient β has a well-fit plot (R^2>0.9) with the natural period and base shear ratio of the irregular building configurations considered in the study. Therefore this proposed model can be put into application to predict the responses of buildings having a combination of stiffness irregularity, along the height as well as in plan at any floor level.

5.3.8. Summary

Based on the response parameters evaluated in the study, it can be inferred that the stiffness at the base of a building is highly influential of the overall stability and response of the building when subjected to seismic loading. Therefore the presence of soft stories with longer columns primarily to provide for unobstructed, free space at the ground floor level is highly critical particularly, when the building is located in earthquake prone areas. Even if the stiffness reduction is by 80% and if the soft storeys are present in combination with in-plan eccentricity in the range of 0.05-0.3L at the lower floor levels, it can lead to an increase in natural period up to 70%. Based on the present investigations, a coefficient 'β' purely based on the geometric dimensions of the building has been proposed which can be utilized to assess the combination of vertical and in-plan stiffness eccentricity in buildings. Employing this coefficient, the natural period and base shear values of any stiffness irregular building can be predicted. From these predicted values, the base shear of a building having soft stories at any height from the base can be computed. Consecutively the structural system can be planned and designed suitably for better performance under the action of earthquake forces.

5.4. Prediction of natural period of irregular buildings

The irregular buildings considered in the study includes the mass irregular buildings with mass ratios 1.25 to 5 in combination with e_d/L in the range of 0.094 to 0.144, stiffness irregular buildings with a stiffness reduction of 0.79 to 0.43 in combination with e_d/L in the range of 0.07to 0.3 and shear wall buildings with a combination of mass and stiffness variation with e_d/L in the range of 0.052 to 0.57. The natural periods of all these irregular buildings (T_i) were evaluated through free vibration analysis or Eigenvalue analysis and were related to the natural period of the corresponding regular buildings without any irregularity (T_r) with respect to the varying e_d/L of each configuration. Through non-linear regression analysis of 630 such irregular buildings, a well fit model was developed to predict the natural period of any irregular building in terms of the natural period of the corresponding regular configuration and its dynamic eccentricity ratio as given in Equation 5.7 below. Figure 5.42 shows the plot between the natural periods of irregular buildings T_{ip} predicted based on the proposed equation and T_i of the buildings obtained through dynamic analysis.

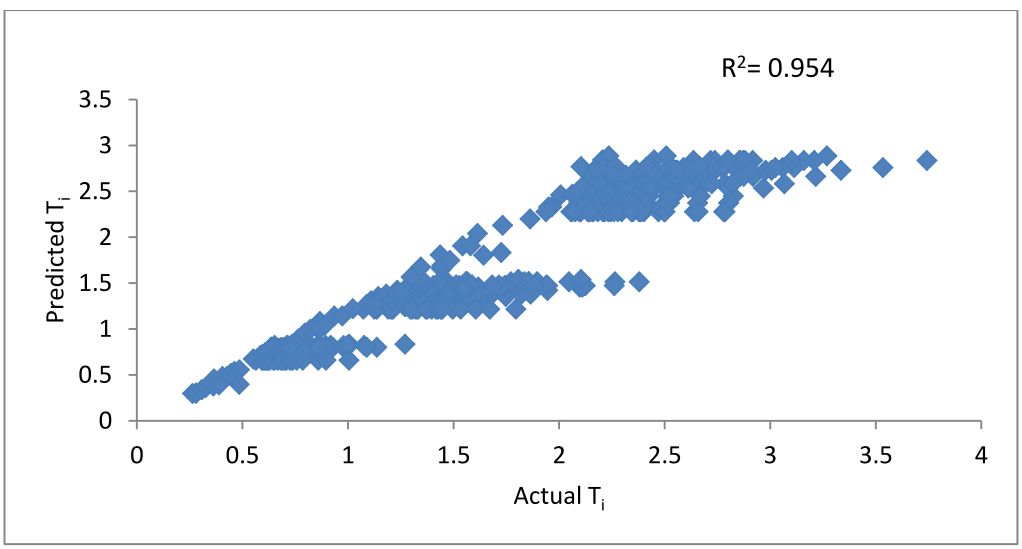


Figure 5.42 Actual T_i from dynamic analysis versus predicted T_{ip}

$$T_{ip} = 1.64T_r\left(\frac{e_d}{L}\right)^{0.122} \tag{5.7}$$

Many researchers have proposed empirical as well as numerical relationships to estimate the fundamental period of RC buildings based on their height and structural type. A semi-empirical expression was employed in the report of Applied Technological Council (ATC3-06, 1978) to estimate the natural period of RC buildings based on their height. The expression had the form 0.75 T = C_t H where C_t is taken to be 0.03 for RC moment-resisting frames and H represents the building height measured in feet. This expression, or slight variations of it, was been subsequently adopted by the European seismic design regulation, Eurocode 8, and the National Building Code of Canada (NBCC, 2005) International Building code (IBC, 2015) for moment-resisting frames. UBC-97 adopted a formula based on only the material, the type of structure and the building height. However, other characteristics such as the presence of varying mass and stiffness, which could influence the dynamic behaviour of buildings under seismic loads, are not considered. The approximate fundamental translational natural period of oscillation of an RC MRF building as per IS 1893: 2016, NBCC 2005, Eurocode 8 and IBC 2015 is given by

$$T_{a\ IS} = 0.075h^{0.75} \tag{5.8}$$

As per ASCE 7-16, FEMA 450, IBC 2015, the approximate natural period is obtained from the equation:

$$T_{a\ ASCE} = C_T {h_n}^x \tag{5.9}$$

Where, h_n is the structural height and the metric equivalents of coefficients C_t and x are determined for a concrete MRF system as 0.0466 and 0.9 and therefore, Equation 5.9 can be rewritten as:

$$T_{a\ ASCE} = 0.0466h^{0.9} \tag{5.10}$$

A non-linear regression analysis to find the relation between the height of the building and natural period has been carried out considering all regular buildings with symmetric distributions of mass and stiffness considered in this study. The natural period of regular buildings are predicted as T_{rp}, as follows:

$$T_{rp} = 0.014h^{1.29} \tag{5.11}$$

Substituting Equation 5.11 for T_r in 5.7, the natural period of irregular buildings can be predicted as:

$$T_{ip} = 0.022h\left(\frac{e_d}{L}\right)^{0.122} \tag{5.12}$$

5.4.1. Modification factor for natural period of regular building

Modification factors 'γ_{IS}' and 'γ_{ASCE}' have been proposed based on the non-linear regression analysis of natural period of irregular buildings to estimate the natural period of irregular buildings from the formulae for the estimation of natural period of irregular buildings given in IS 1893:2016 and ASCE 7-16. The relation between the natural period of the regular frame building in the study and the approximate natural period as per IS 1893:2016 ($T_{a\ IS}$) and ASCE 7-16 ($T_{a\ ASCE}$) are obtained as in Equations 5.13 and 5.14.

$$T_r = 0.186h^{0.54}T_{a\ IS} \tag{5.13}$$

$$T_r = 0.299h^{0.39}T_{a\ ASCE} \tag{5.14}$$

These equations are substituted for the value of T_r in Equation 5.7 to obtain the relation to estimate the natural period of any irregular building in terms of the approximate natural period of the regular building as per IS 1893:2016 and ASCE 7-16. To estimate the natural period of any irregular configuration with respect to the natural period of the corresponding regular configuration in terms of the approximate natural period as per IS

1893:2016 and ASCE 7-16, height of the building and in plan dynamic eccentricity is given by :

$$T_{ip\ IS} = T_{a\ IS} \times \gamma_{IS} \quad \text{where} \quad \gamma_{IS} = 0.31\ h^{0.54} \frac{e_d}{L}^{0.122} \tag{5.15}$$

$$T_{ip\ ASCE} = T_{a\ ASCE} \times \gamma_{ASCE} \quad \text{where} \quad \gamma_{ASCE} = 0.49\ h^{0.54} \frac{e_d}{L}^{0.122} \tag{5.16}$$

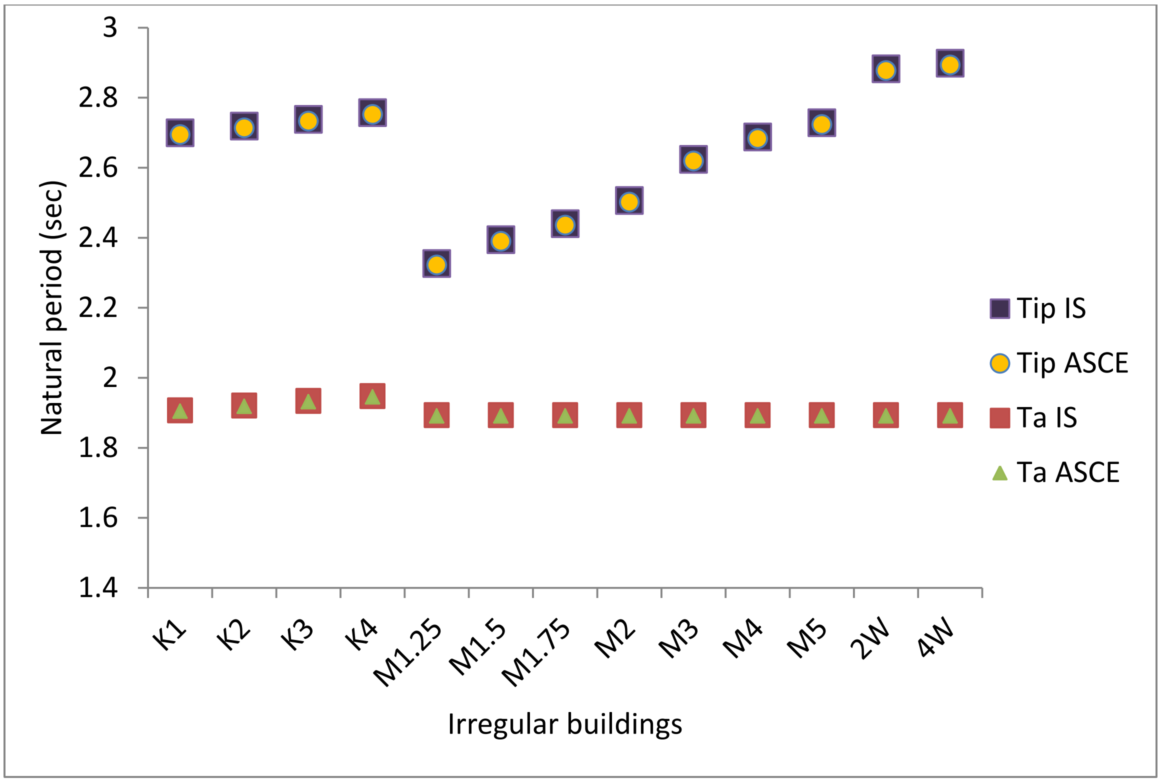


Figure 5.43 Natural periods of critical irregular buildings in group C

These relations 5.15 and 5.16 give the proposed equations for the natural period of a building considering the in-plan irregularities. The natural periods of the critical buildings with mass, stiffness as well as shear wall irregularities belonging to group C are given in Figure 5.43. The natural period of the buildings considering the irregularities computed as per the proposed equations and the natural period of the same buildings calculated as per the approximate code equations without considering the irregularities have quite a remarkable variation. Therefore these proposed equations can be put into use to predict the accurate fundamental natural period of any building with irregularity and can hence be used to re-plan the building to modify the natural period for reduced base shear and rotations.

5.5. Transient Analysis of critical cases under three different ground motions

The natural frequencies of all the buildings considered in the present study are in the range of 0.267 to 3.86Hz. El- Centro ground motion has high amplitude contents in the frequency range of 1Hz to 2.5Hz. In addition to El-Centro, two more ground motions with frequency contents in different ranges were selected for further analyses. The buildings with critical cases of irregularities were selected and subjected to Kobe (1995) earthquake which has high amplitude frequency contents in the range of 0.5Hz-1Hz and Koyna (1967) earthquake with high amplitude frequency contents in the range of 3Hz-4.5Hz whose frequency contents do not exactly match with that of the frequency range of considered buildings. The critical cases under each category of irregularity as considered for analysis under Kobe and Koyna ground motions and the variations in responses obtained for different ground motions are discussed herewith.

5.5.1. Mass irregular buildings

Transient analysis was carried out on vertically mass irregular buildings wherein the irregularity is present in combination with in-plan eccentricity at the top, middle and bottom floor levels using El-Centro ground motion data in the initial part of the study. However, the critical case of asymmetry in mass distribution along the height of the building is when the mass irregularities are present in combination with eccentricities in the upper half of the building height. Therefore buildings with irregularities of mass ratio 1.25, 2 and 5 located at the upper floor levels were selected and subjected to acceleration time history of Kobe and Koyna earthquakes and the seismic responses were evaluated. Under each mass ratio, the in-plan eccentric configurations IM1 and IM3 along with the plan regular configuration IM0 were considered. The variations in base shear ratio, roof deflection ratio and roof rotation responses of the mass irregular building under the application of the El-Centro, Koyna and Kobe ground motion data were evaluated and compared. The variation of seismic responses of the buildings due to the influence of the mass irregularities and their eccentricity in placement when subjected to Koyna and Kobe ground motions is similar to those obtained initially for the El-Centro ground motion. However, individual variations in the responses of the same buildings subjected to the three earthquakes can be observed, which is probably due to change in their high amplitude frequency contents.

Figures 5.44 to 5.46 show the variations in seismic base shear ratios of the mass irregular buildings belonging to group A, group B and group C. The highest base shear response recorded among the buildings is represented by the data label in the figures and its font color stands for the ground motion responsible for the response. In general, the variation in base shear ratios due to the increase in magnitude, location and in-plan eccentricity of mass irregularity is similar in all the three ground motions considered in the study. However, slight variations can be observed in the base shear values generated by the Koyna and the Kobe ground motions as compared to that of the El-Centro due to the difference in their frequency contents. Koyna earthquake generated lower base shear in the mass irregular cases considered in comparison to Kobe and El-Centro earthquake data. The seismic bases shear generated by Koyna earthquake in these cases had maximum variation of 15% with respect to that of El-Centro earthquake. Comparing the seismic base shear generated by Kobe and El-Centro earthquakes, the former generated higher base shear in few building configurations with mass irregularities in comparison to the latter. This increase in base shear in the case of Kobe earthquake in comparison to El-Centro earthquake varies from 0.3-12%. In general, group B buildings subjected to Kobe earthquake are observed to have higher base shear ratio with respect to that of El-Centro earthquake. The highest variation, in this case, is observed in $BIM0_t$ with mass ratio 2. Further in the case of most of the group B buildings with mass ratio 5 and group C buildings, El-Centro earthquake is observed to generate highest base shear among the three ground motions considered. The $CIM0_t$ building with a mass ratio of 1.25 subjected to El-Centro ground motion has a maximum variation of 23% with respect to base shear generated by Koyna ground motion and 18% with respect to Kobe ground motion.

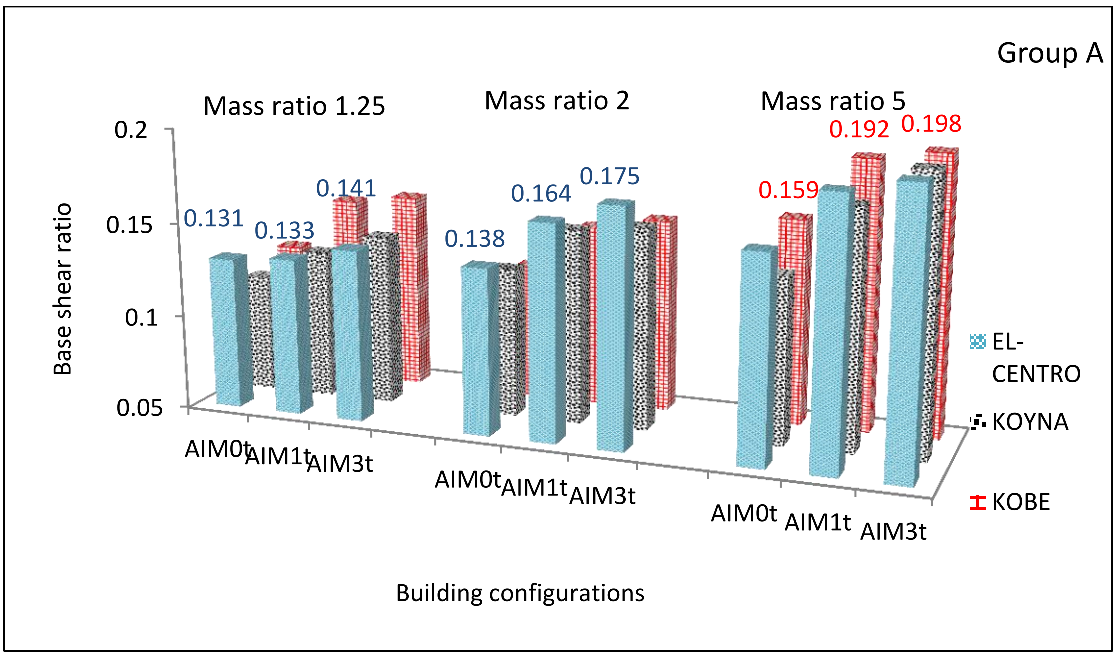


Figure 5.44 Variation in seismic base shear ratios of group A mass irregular buildings subjected to three ground motions

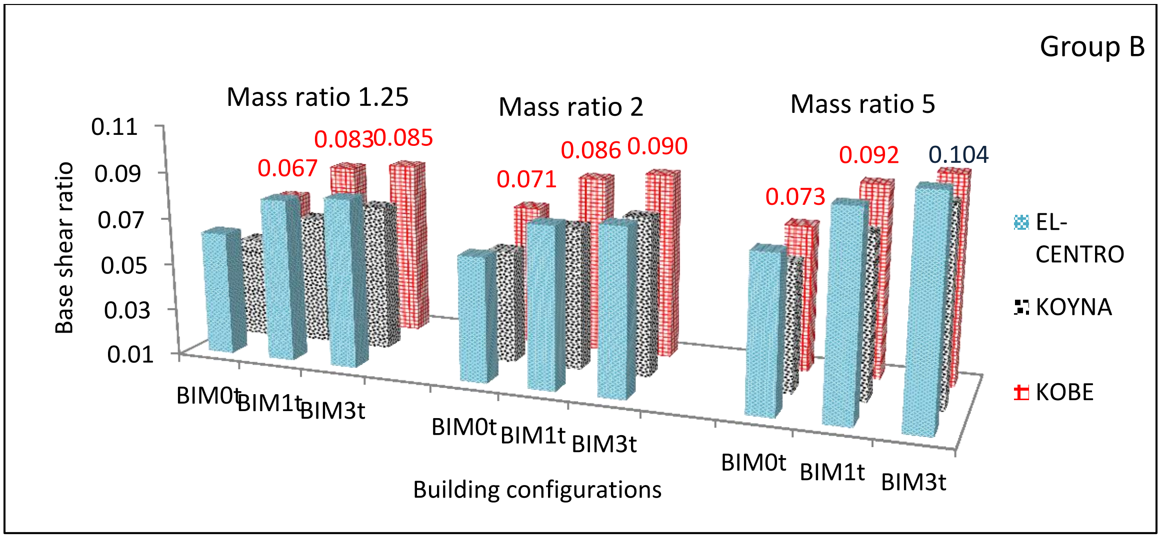


Figure 5.45 Variation in seismic base shear ratios of group B mass irregular buildings subjected to three ground motions

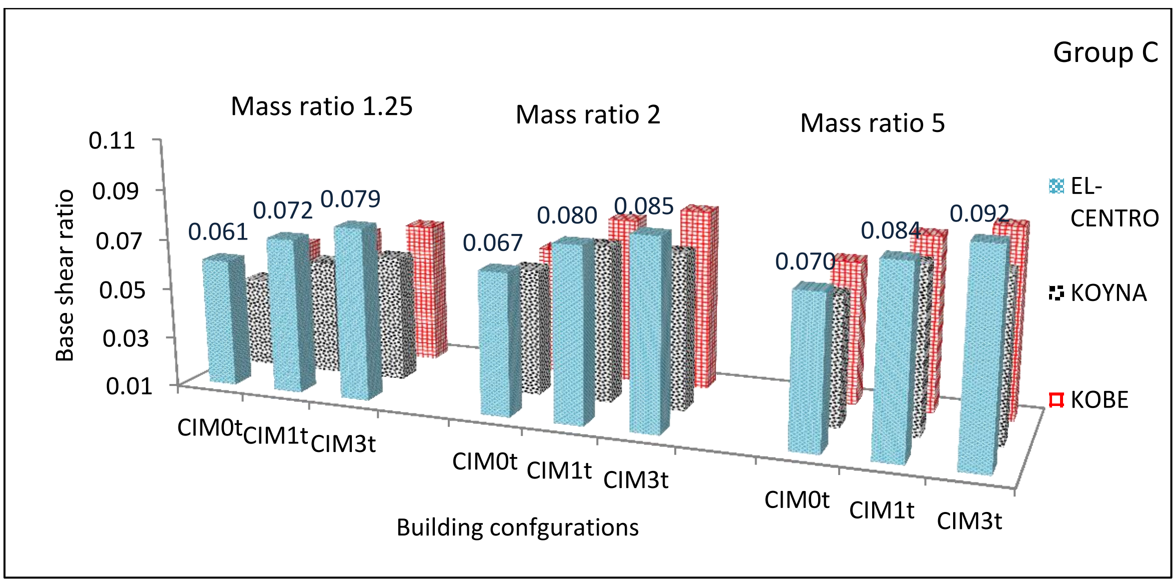


Figure 5.46 Variation in seismic base shear ratios of group C mass irregular buildings subjected to three ground motions

Considering the variation of roof deflection, as shown in Figures 5.47 to 5.49, it can be observed that in certain mass irregular buildings, Kobe earthquake generated higher roof deflections as compared to that generated by El-Centro ground motion. Group B buildings with a frequency range of 0.53- 0.88Hz, have higher roof deflection ratios when subjected to Kobe earthquake with the highest variation of 10% with respect to El-Centro ground motion in $BIM0_t$ with a mass ratio of 1.25. El- Centro earthquake generated the highest deflection ratios in the case of 15 storey irregular buildings with mass ratio 2 and 5. $CIM3_t$ subjected to El- Centro ground motion has the highest variation of 12% in roof deflection ratio with respect to that obtained when the same building was subjected to Kobe earthquake. In the case of roof deflection, Koyna earthquake generated lower responses in all the buildings as compared to the responses generated by El-Centro and Kobe ground motions.

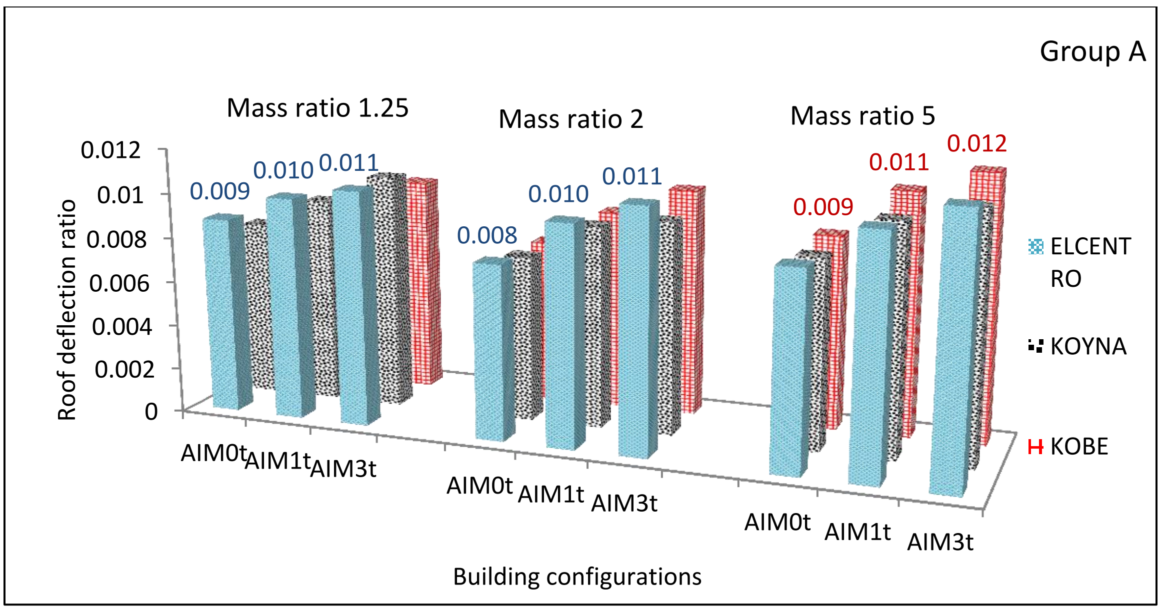


Figure 5.47 Variation in roof deflections of group A mass irregular buildings subjected to three ground motions

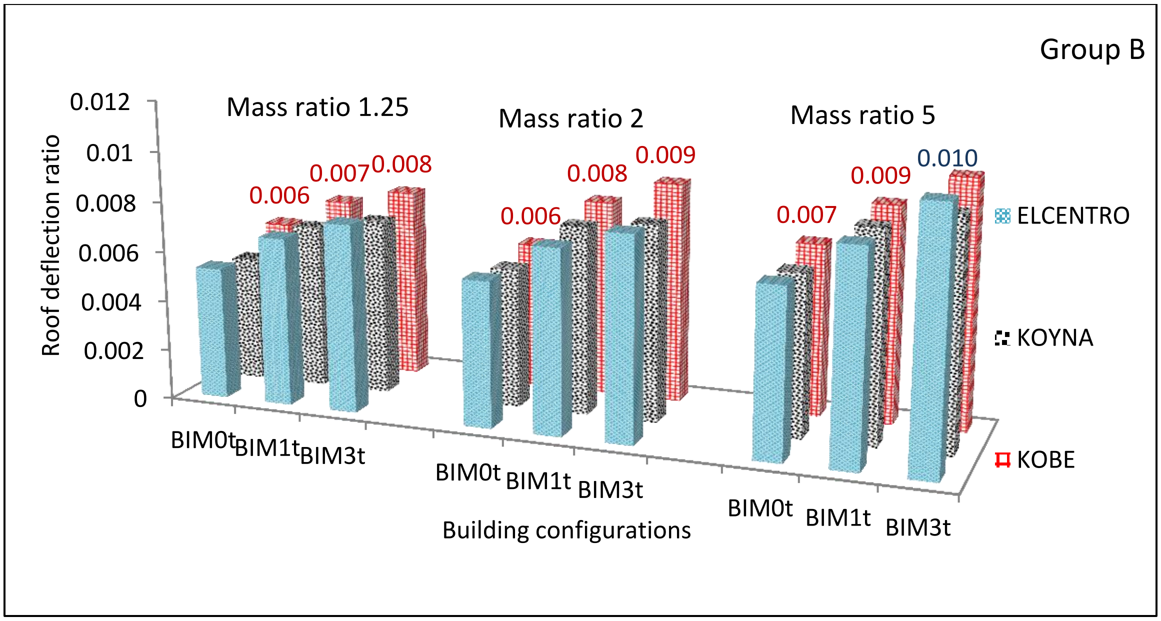


Figure 5.48 Variation in roof deflections of group B mass irregular buildings subjected to three ground motions

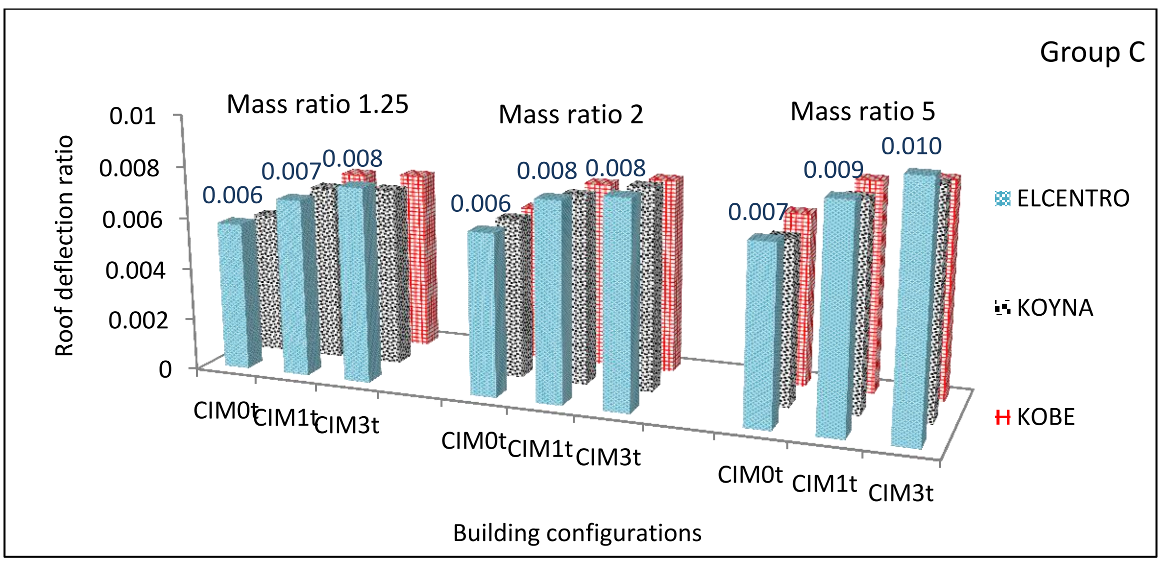


Figure 5.49 Variation in roof deflections of group C mass irregular buildings subjected to three ground motions

The roof rotations as shown in Figures 5.50 to 5.52 imply the torsional behavior of the same set of buildings generated by the three ground motions and the variations are due to the effect of different amplitude frequency contents of the acceleration time history data. Koyna earthquake comparatively generated lesser torsion or roof rotation in comparison to the El-Centro and Kobe earthquake in all the buildings. In group A buildings with mass ratio 5 as well as group B buildings, Kobe ground motion generated roof rotations higher by 3-18% with respect to that of El-Centro. In most of other mass irregular buildings, El-Centro ground motion causes higher roof rotations in comparison to Kobe and Koyna earthquakes. The roof rotations generated in the buildings by El-Centro ground motion are higher by a maximum of 15% and 18% in comparison to Kobe and Koyna earthquakes respectively.

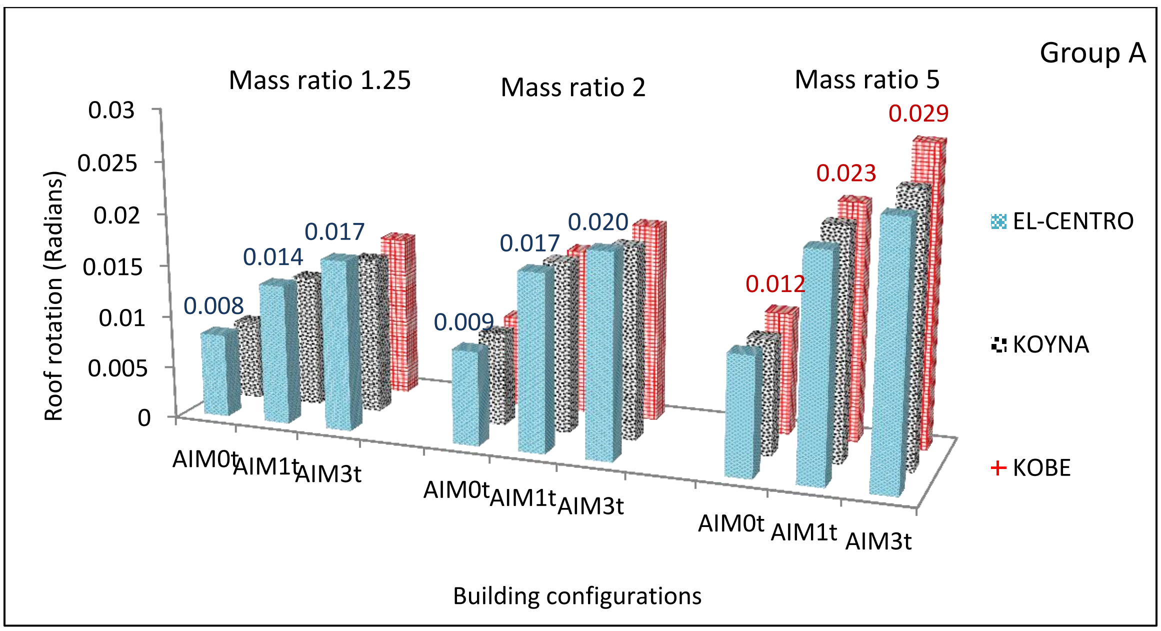


Figure 5.50 Variation in roof rotations of group A mass irregular buildings subjected to three ground motions

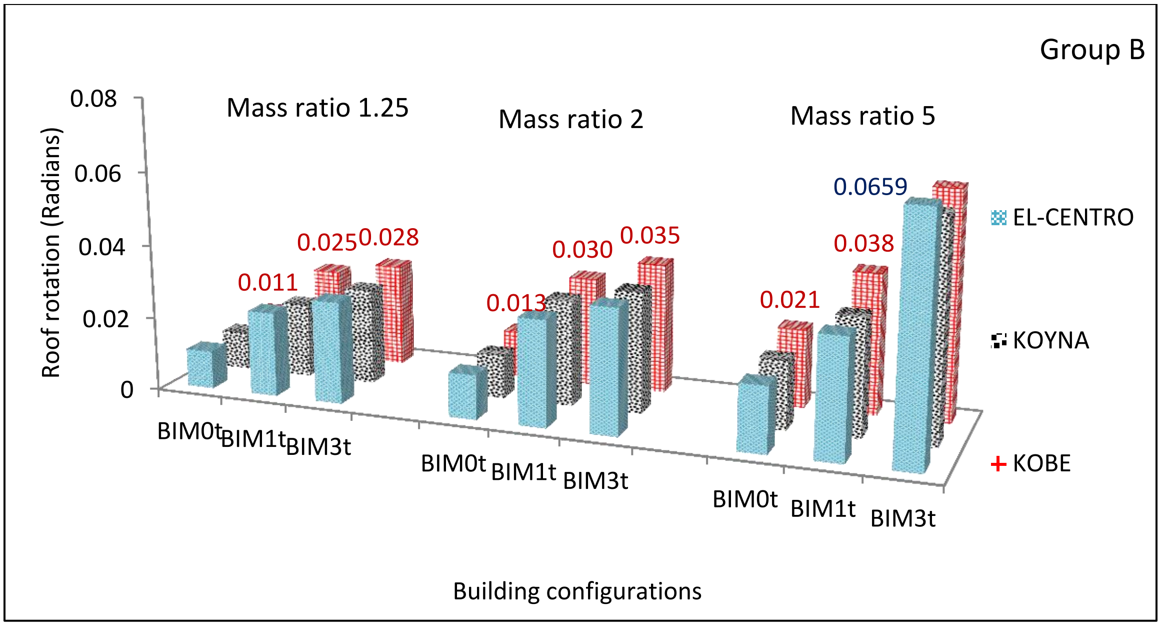


Figure 5.51 Variation in roof rotations of group B mass irregular buildings subjected to three ground motions

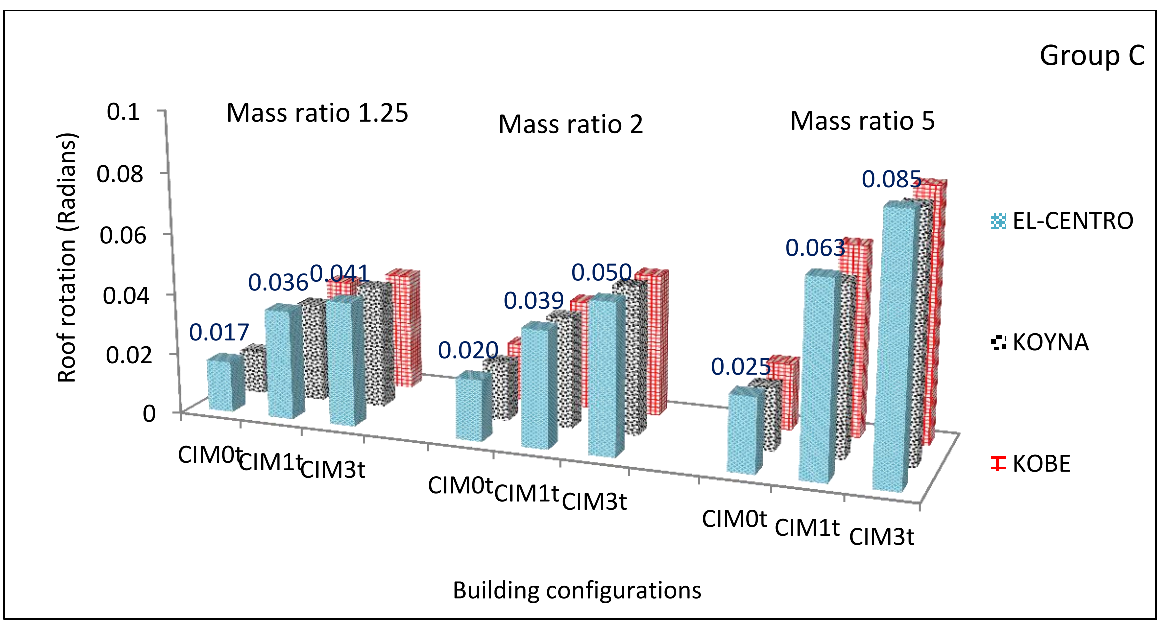


Figure 5.52 Variation in roof rotations of group B mass irregular buildings subjected to three ground motions

The discrepancies in the seismic responses under the influence of the different ground motions are mainly due to the variation in the frequency content of the ground motions. Koyna has intense amplitude contents in a higher frequency range as compared to the range of frequencies of the mass irregular buildings considered here. Whereas Kobe has its high amplitude contents in the lower frequency range 0.5 to 1Hz. The higher responses in few of the group B buildings under the application of Kobe ground motion is probably due to the concurrency of the natural frequency of those buildings with the frequency range of Kobe ground motion with high amplitude contents. It can be observed that Kobe and El-Centro ground motions have higher amplitudes as compared to Koyna ground motion within the range of frequencies of the mass irregular buildings considered and therefore have higher seismic responses than the latter. Figure 5.53 shows the Fourier spectrum of the three ground motions in the range of 0.01Hz to 5Hz along with the identifiers for the mass irregular buildings which have higher responses for each of the ground motion considered in the study.

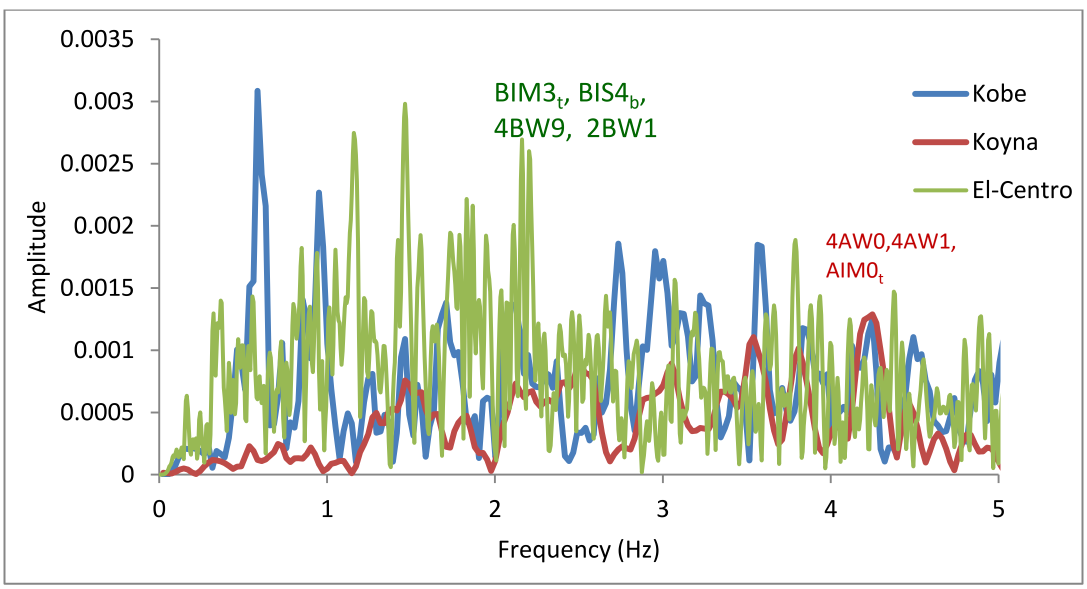


Figure 5.54 Fourier spectrum of Kobe, Koyna and El-Centro ground motions of mass irregular buildings with higher responses

5.5.2. Stiffness irregular buildings

Time history analysis was carried out on vertically stiffness irregular buildings with irregularity present in combination with in-plan eccentricity at the top, middle and bottom floor levels using El-Centro ground motion. From the initial study on stiffness irregular buildings it was comprehended that the critical cases of asymmetry in stiffness distribution are when the irregularities are present in combination with in-plan eccentricities in the lower half of the building height or in specific when the soft storey is present in the ground floor level. Therefore, buildings with stiffness modifications K_0, K_1 and K_4 along with buildings which do not have stiffness reduction but having in-plan eccentricity at the ground floor level were selected and subjected to Kobe and Koyna earthquakes and the seismic responses were evaluated and compared with the responses in the case of El-Centro earthquake. Among each of these cases of stiffness modifications, the in-plan eccentric configurations IS1, IS4 and IS8 were considered. The variations in base shear ratio, roof deflection ratio and roof rotation of the stiffness irregular buildings under the application of El-Centro, Koyna and Kobe ground motion were compared.

It is observed that the seismic responses of the stiffness irregular buildings generated by Koyna and Kobe ground motions are similar to the responses obtained for the El-Centro ground motion data in terms of their variation with respect to in-plan eccentricity. However, slight discrepancies are observed in the responses of the same buildings subjected to the three earthquakes probably due to the presence of difference amplitude frequency contents.

The variation in seismic base shear ratios of the selected stiffness irregular buildings belonging to group A, group B and group C subjected to the three different ground motions are nominal as shown in Figures 5.55 to 5.57. It can be observed that Koyna earthquake generated comparatively lower base shear than El- Centro earthquakes but has a higher base shear as compared to that of Kobe earthquake in group A buildings with stiffness modifications K_1 and K_4 and natural period in the range of 0.6 to 0.7 sec. In the case of group B buildings, Kobe earthquake generated higher base shear by 3-9% with respect to that of El-Centro ground motion. In the case of group A and group C buildings, El-Centro earthquake generated base shear which is higher by a maximum of 10% with respect to that of Koyna earthquake and 7% with respect to that of Kobe earthquake.

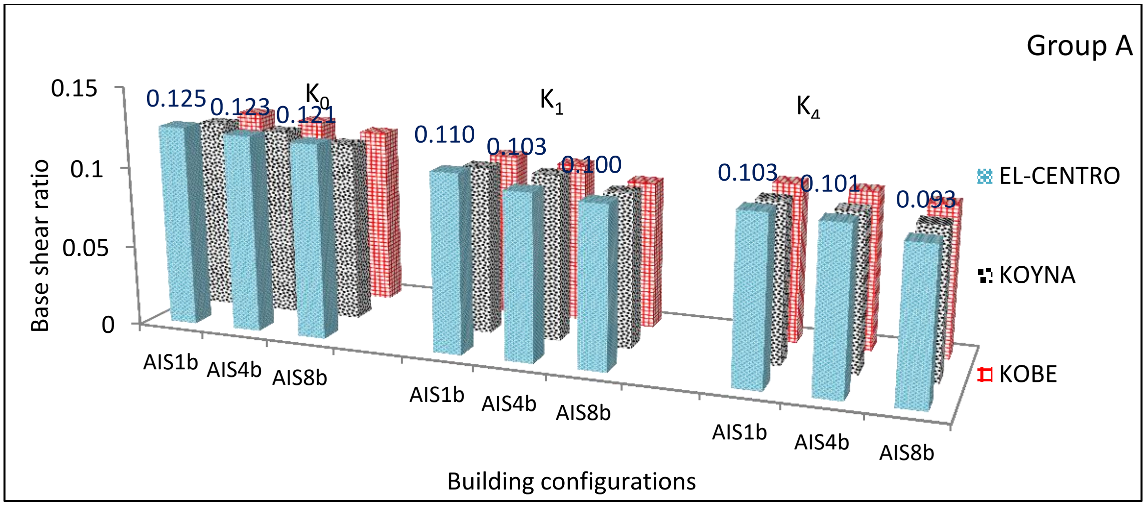


Figure 5.55 Variation in base shear ratio of group A stiffness irregular buildings subjected to three ground motions

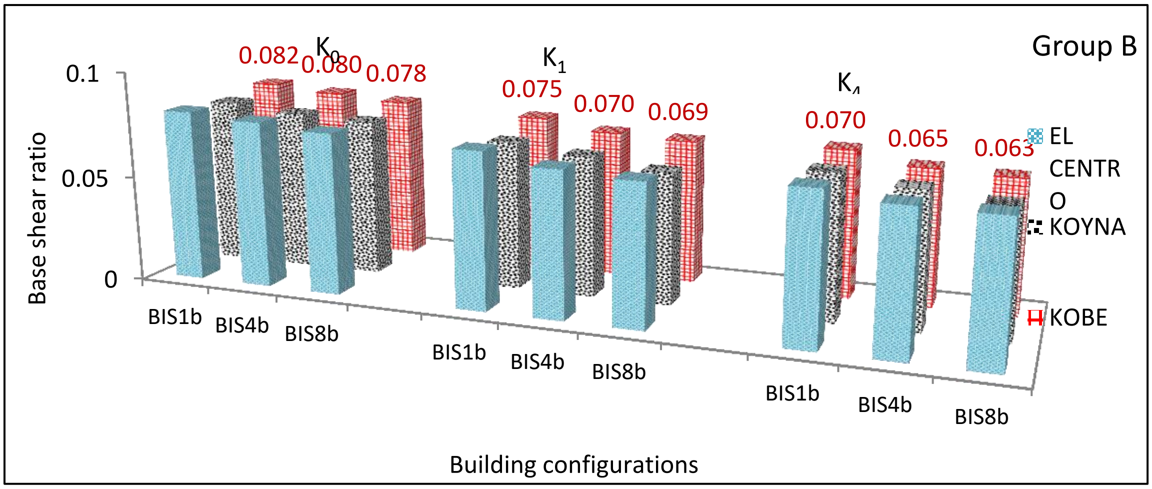


Figure 5.56 Variation in base shear ratio of group B stiffness irregular buildings subjected to three ground motions

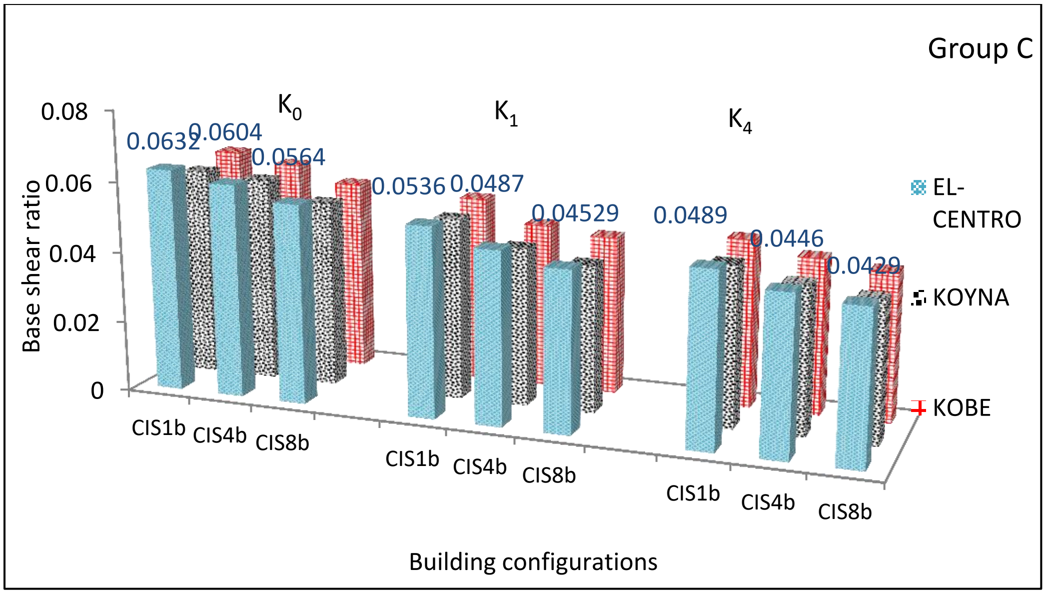


Figure 5.57 Variation in base shear ratio of group C stiffness irregular buildings subjected to three ground motions

Comparing the roof deflection responses of the stiffness irregular buildings generated by the three earthquakes, it can be observed from Figures 5.58 to 5.60 that the Kobe earthquake generated higher roof deflections in Group B buildings with a maximum increase of 12% with respect to El-Centro ground motion in BIS4$_b$ with stiffness modification of K$_4$. In group A and group C stiffness irregular buildings with the natural period in the range of 0.5-0.7 sec and 2.3-3.7 sec, El- Centro ground motion generated the highest roof deflection response. Koyna earthquake generated lowest roof deflection responses among all three ground motions considered.

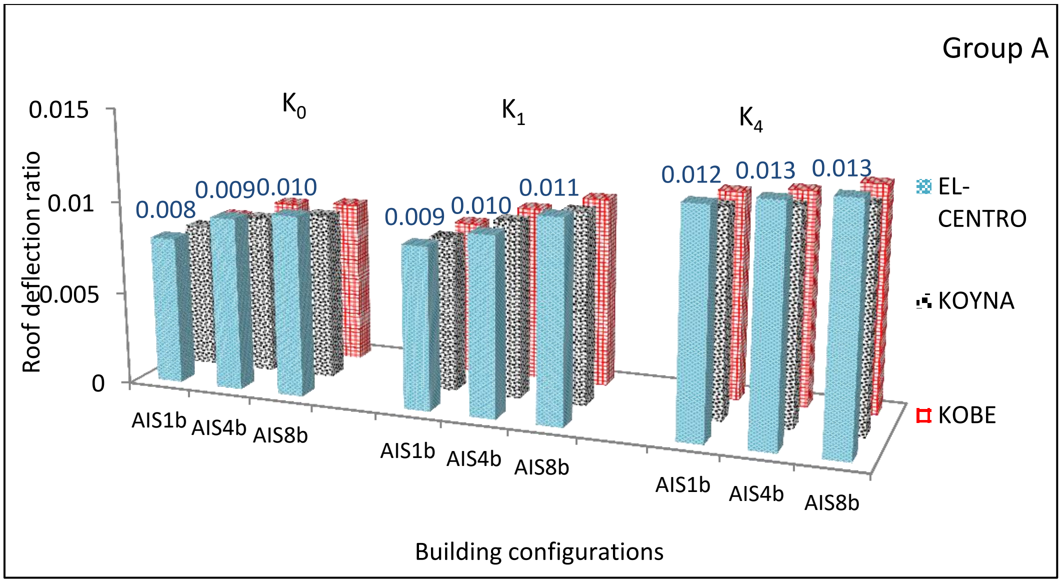


Figure 5.58 Variation in roof deflections of group A stiffness irregular buildings subjected to three ground motions

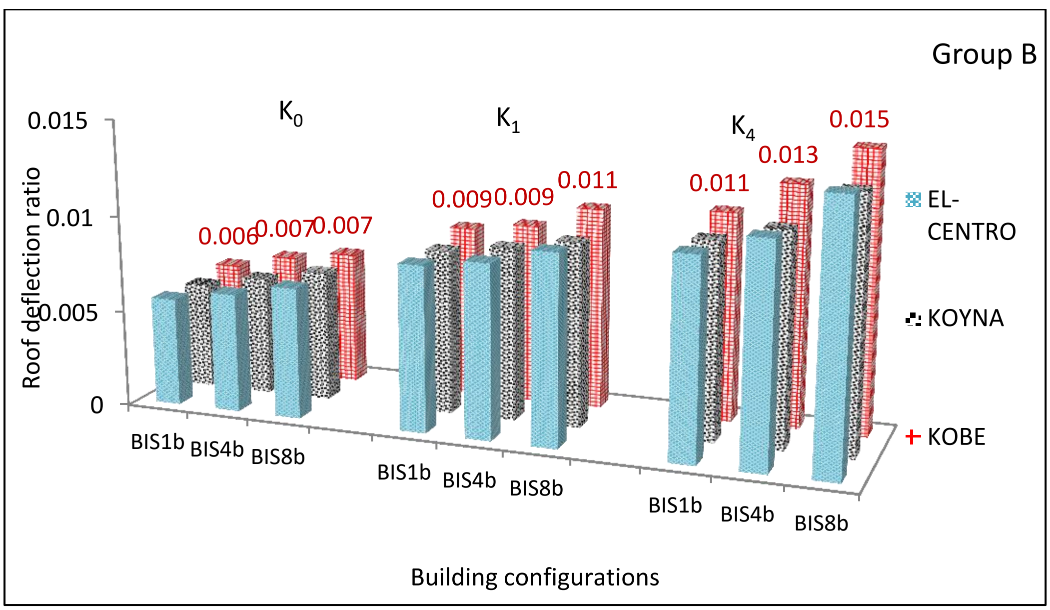


Figure 5.59 Variation in roof deflections of group B stiffness irregular buildings subjected to three ground motions

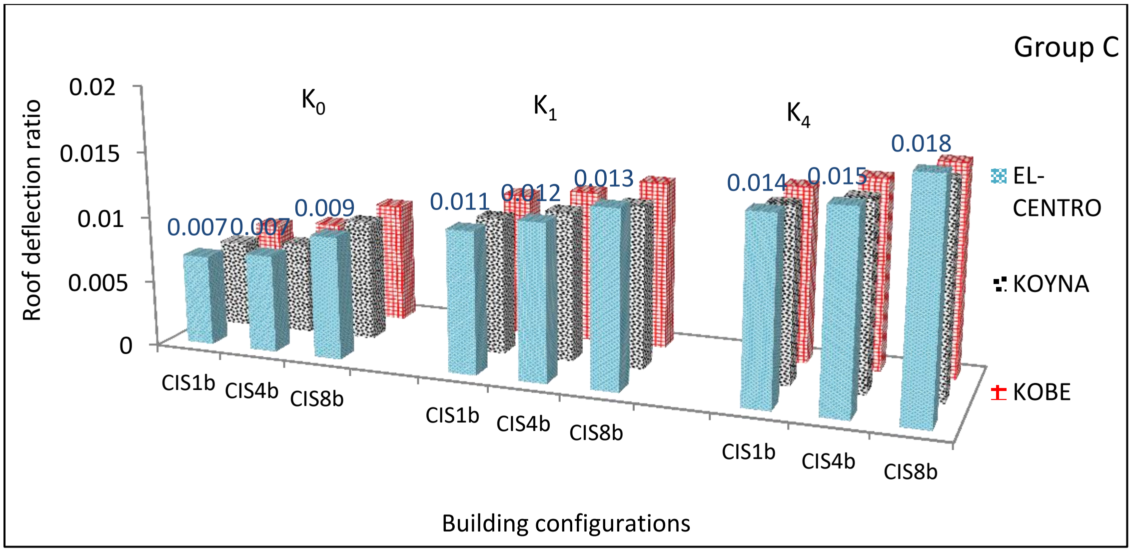


Figure 5.60 Variation in roof deflections of group C stiffness irregular buildings subjected to three ground motions

The roof rotation generated by the buildings on the application of the three ground motions as shown in Figures 5.61 to 5.63 have variation due to the change in amplitude of the frequency content of the three ground motions which were scaled equally to 0.343g. Koyna earthquake caused comparatively lesser roof rotations in the case of group A buildings as compared to El-Centro ground motion, but higher rotation response in comparison to Kobe earthquake. Among all the three ground motions, the Kobe earthquake generated higher roof rotation in the case of group B buildings and similarly El-Centro earthquake data in the case of group C buildings. The maximum increase in the roof rotation generated by Kobe earthquake with respect to El-Centro earthquake is 11% in BIS4_b with a stiffness modification of K_1 whereas the maximum reduction in roof rotation generated by Kobe earthquake with respect to that by El-Centro earthquake is 10% in CIS8_b with stiffness modification of K_4. Whereas in the case of application of Koyna ground motion, the maximum variation in roof rotation as compared to the El-Centro ground motion is obtained as 13% in BIS4_b with stiffness modification K_1.

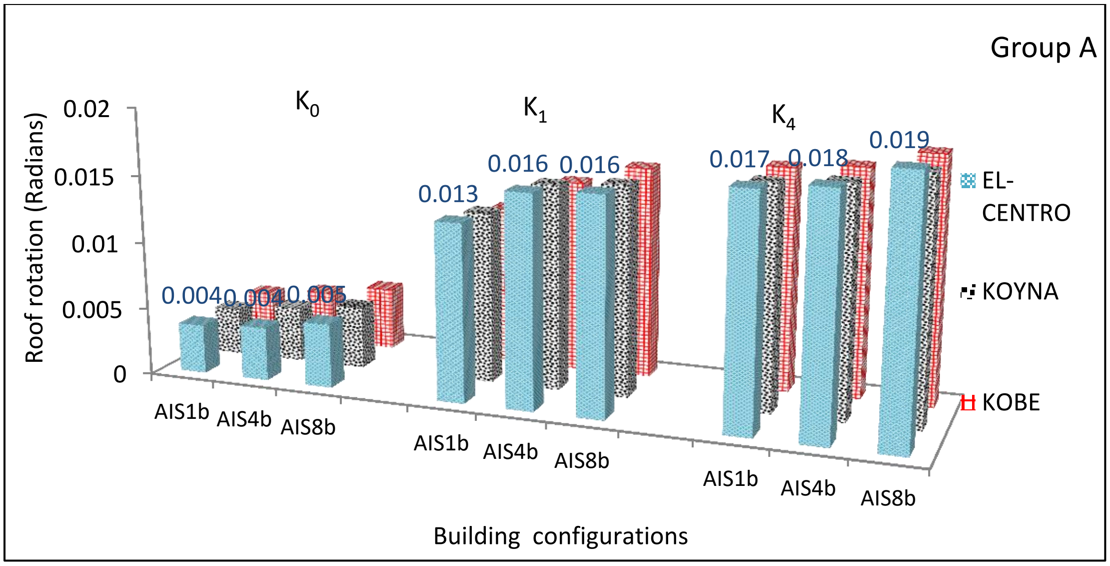


Figure 5.61 Variation in roof rotations of group A stiffness irregular buildings subjected to three ground motions

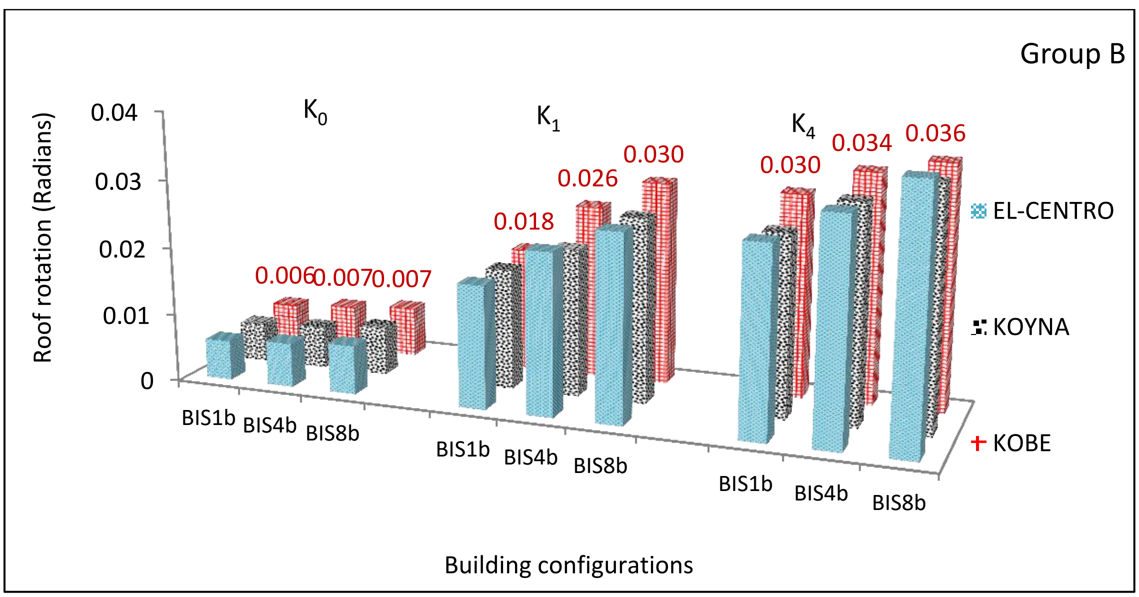


Figure 5.62 Variation in roof rotations of group B stiffness irregular buildings subjected to three ground motions

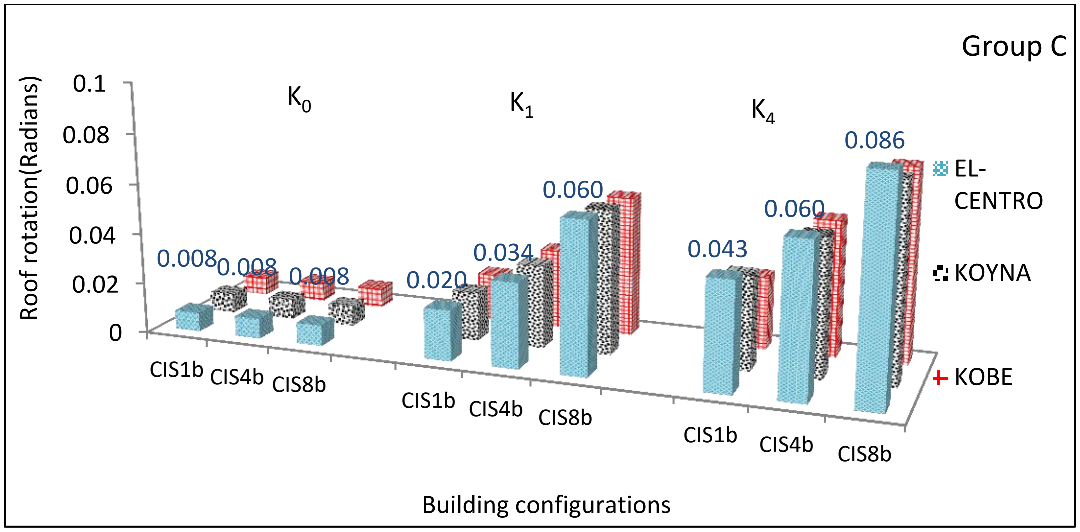


Figure 5.63 Variation in roof rotations of group C stiffness irregular buildings subjected to three ground motions

The variations of the seismic responses of stiffness irregular buildings subjected to the three ground motions are nominal and less than 12%. Koyna earthquake caused lower responses in all the stiffness irregular buildings considered Most of the group B irregular buildings fall in the frequency range of 1Hz to 2Hz, which matches with the frequency range of Kobe earthquake. This might be the cause for the higher responses of group B buildings which are subjected to the Kobe earthquake. Figure 5.64 shows the Fourier spectrum of the three ground motions along with mappings of the frequencies of stiffness irregular buildings in the range of 0.01Hz to 5 Hz, which have higher responses for each of the ground motion considered in the study.

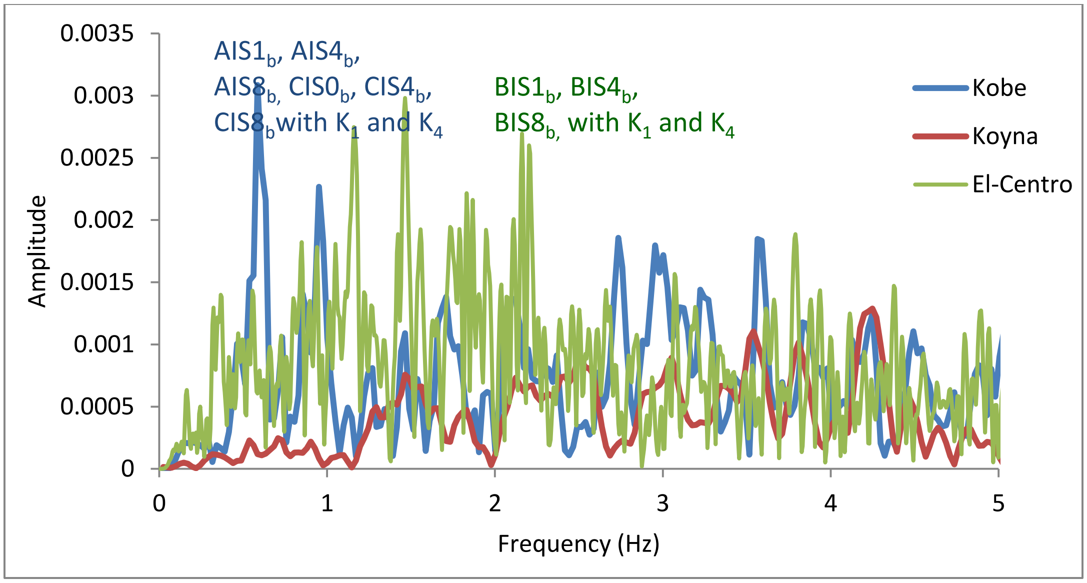


Figure 5.64 Fourier spectrum of Kobe, Koyna and El-Centro ground motions of stiffness irregular buildings with higher responses

5.5.3. Buildings with combination of mass and stiffness irregularities

In the set of shear wall buildings considered, irregularity was incorporated by changing the shear wall arrangement. Shear walls contribute to the mass and stiffness of a framed structure and hence with change in the positioning of shear walls, the centre of mass as well as centre of stiffness shifts inducing a combination of mass and stiffness irregularity in the building. Two sets of these shear wall buildings were subjected to Kobe and Koyna ground motion data, the first set comprising of 2W0, 2W1, 2W5 and 2W9 and the second set comprising of 4W0, 4W1, 4W4 and 4W9 in group A, group B and group C buildings. Variations were observed in the seismic responses of the shear wall buildings generated by the three ground motions due to the difference in their frequency contents. The range of frequencies of the shear wall buildings in all the groups were in-between 0.57 to 4Hz.

The base shear responses of the shear wall buildings for the three ground motions have variations with each another, as shown in Figures 5.65 to 5.67. The variation in the base shear generated by Koyna earthquake is highest in the case of 4AW1 building with a variation of 24% with respect to that of El-Centro ground motion and similarly, 4AW0 has variation of 13%. The Koyna earthquake has intense frequency contents in the range of 3Hz to 4.5Hz and shear wall buildings in group A falling in this range is observed to have generated higher base shear in comparison to Kobe and El-Centro earthquakes. Among the group B buildings, the Kobe earthquake generated higher base shear in the case of 4BW4 and 4BW9 configurations. In the case of group C buildings, Kobe ground motion had higher base shear ratios in the case of 4CW0, 4CW1 and 4CW4 buildings. The maximum variation in base shear generated by Kobe ground motion with respect to that of El-Centro is 18% in the case of 4CW0. It can be observed that El- Centro had the higher seismic response in terms of base shear in 2W buildings belonging to group B and group C.

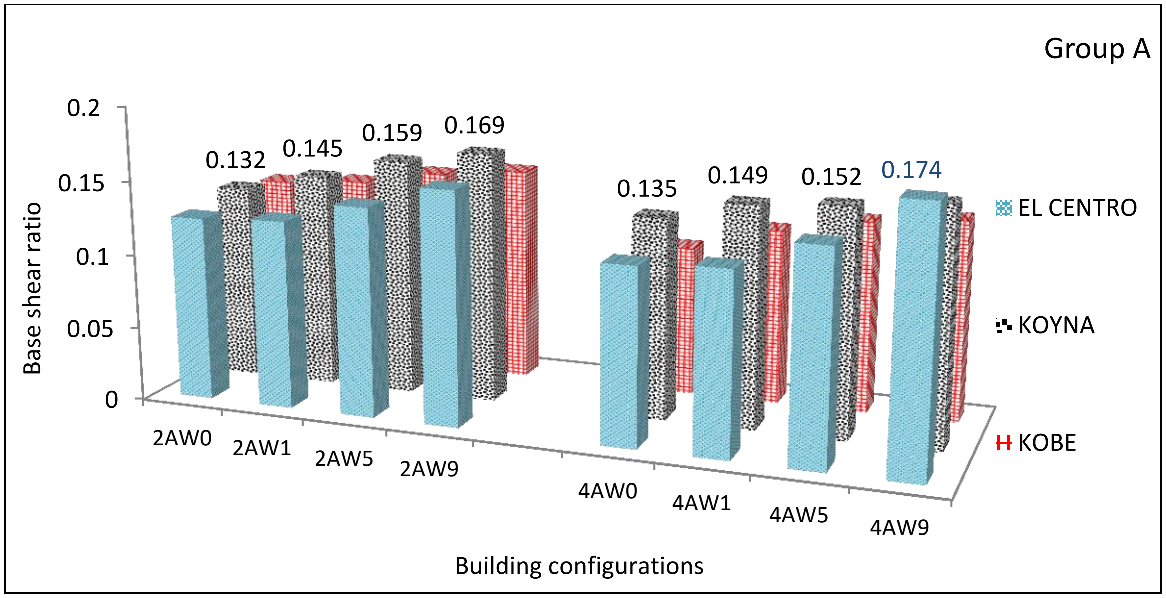


Figure 5.65 Variation in base shear ratio of group A shear wall buildings subjected to three ground motions

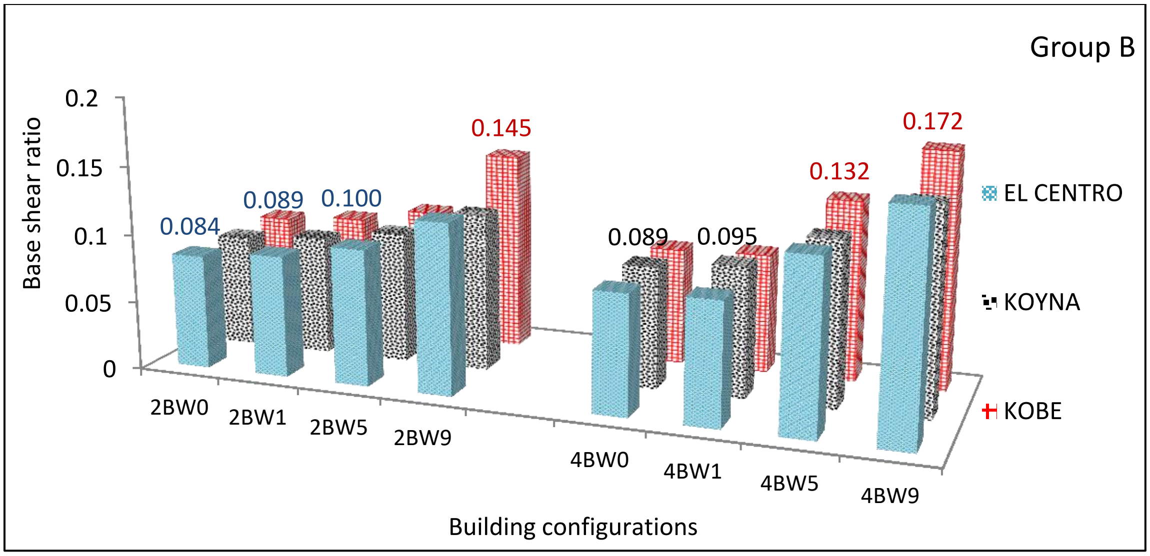


Figure 5.66 Variation in base shear ratio of group B shear wall buildings subjected to three ground motions

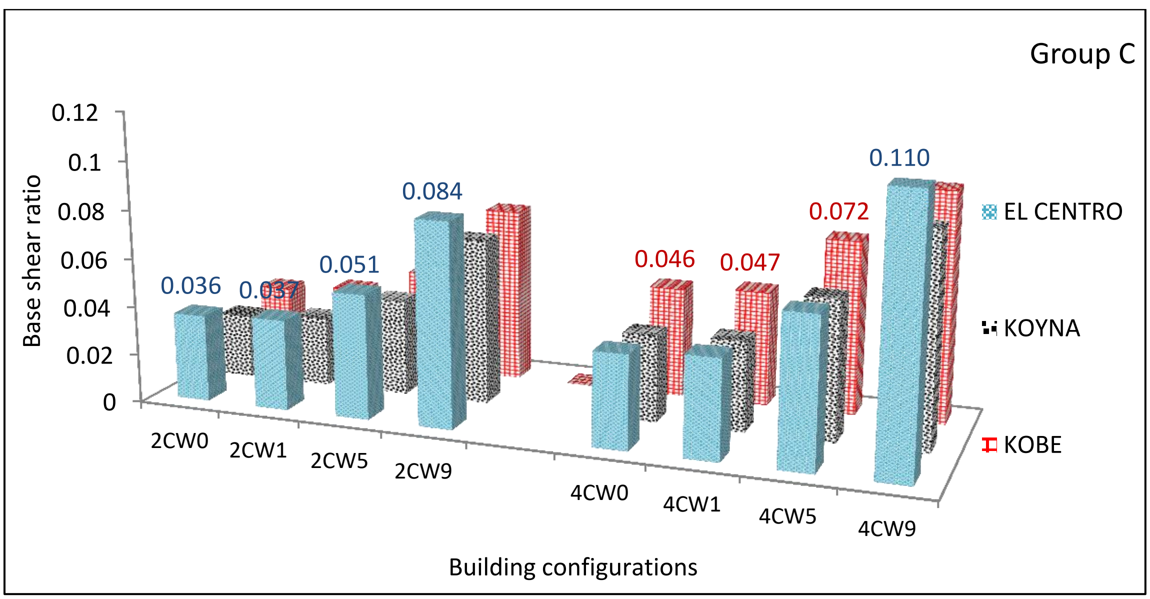


Figure 5.67 Variation in base shear ratio of group C shear wall buildings subjected to three ground motions

The roof deflection responses of the irregular shear wall buildings also have considerable divergence or variation for different ground motions with respect to the applied ground motion as shown in Figures 5.68 to 5.70. The maximum variation in the roof deflection ratio of the buildings subjected to Koyna earthquake with respect to that of El-Centro is 11% in the case of 4AW0 building. El-Centro ground motion caused a deflection response higher by 17% with respect to that caused by Koyna ground motion in 4CW9 building. As observed in the case of base shear Kobe earthquake generated higher roof deflection response in group C and few group B buildings with the highest variation of 20% in 4CW4 with respect to that generated by El-Centro earthquake. In the case of 4CW9, El-Centro ground motion generated higher roof deflection response with respect to Kobe by 13%. This is probably due to the variation in the natural period by 30% due to increase in e_d by 0.4L.

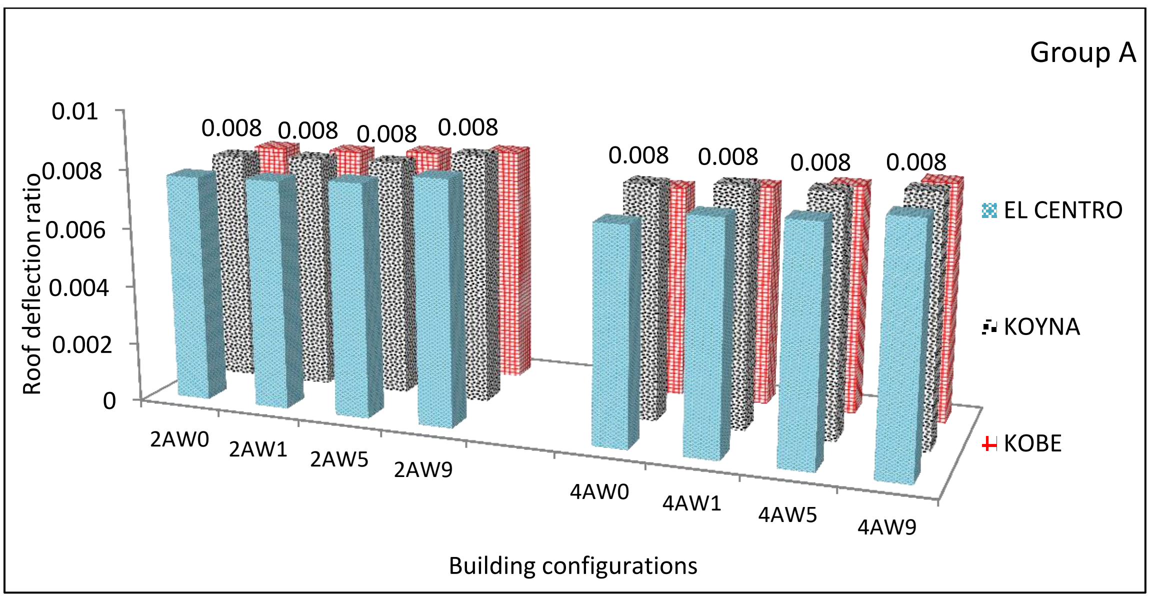


Figure 5.68 Variation in roof deflections of group A shear wall buildings subjected to three ground motions

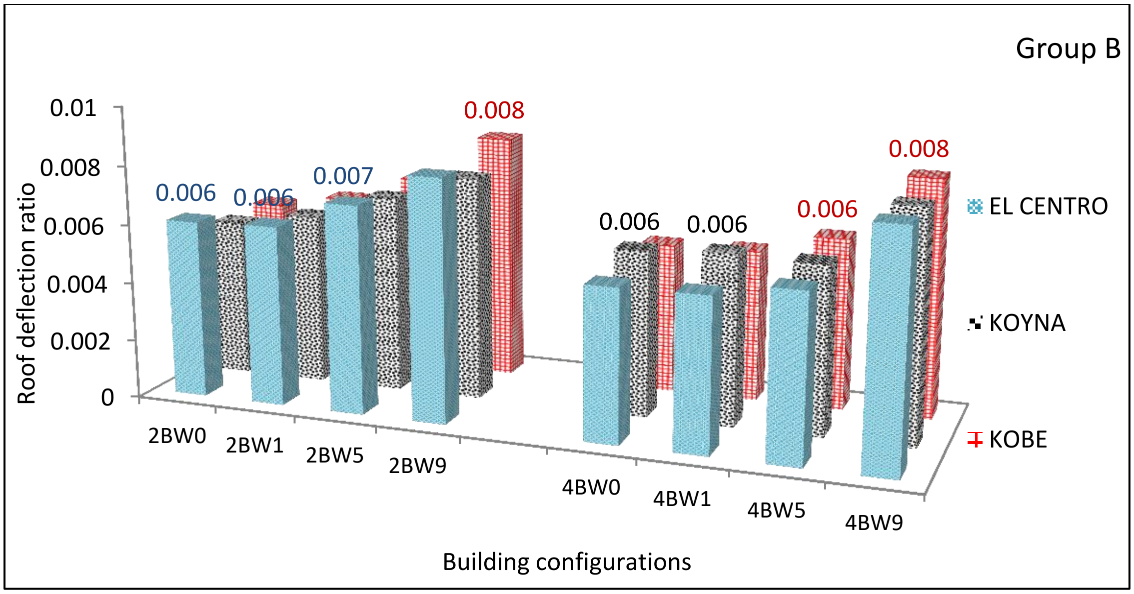


Figure 5.69 Variation in roof deflections of group B shear wall buildings subjected to three ground motions

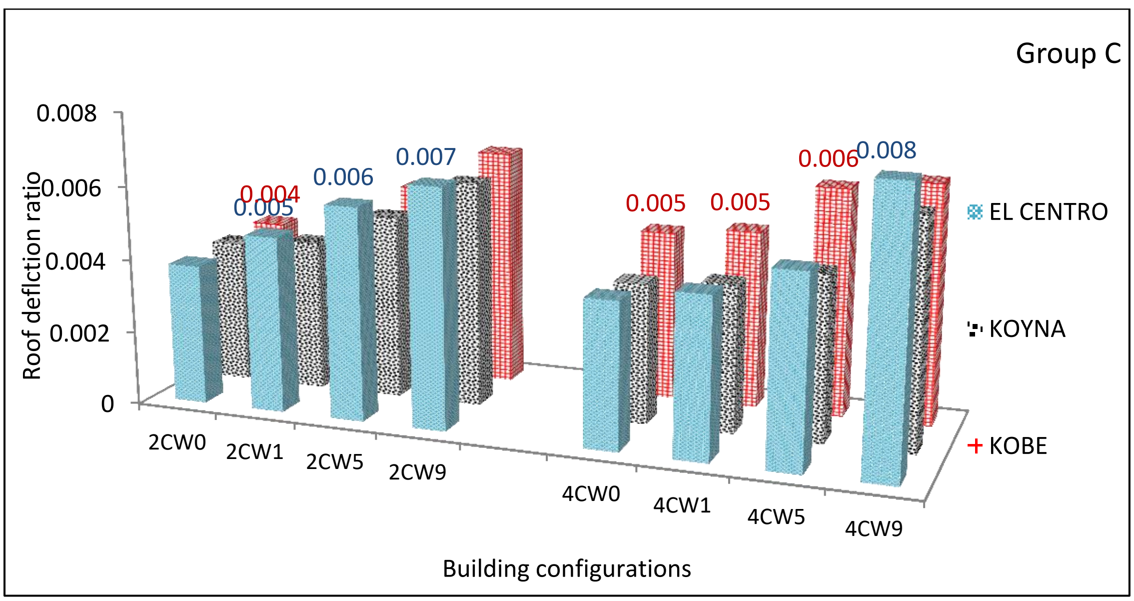


Figure 5.70 Variation in roof deflections of group C shear wall buildings subjected to three ground motions

The varying locations of shear walls within the plan of the buildings generate torsion and roof rotation directly gives this behavior. The roof rotation responses of the regular 4W0 and 2W0 buildings in all the three groups of buildings are minimal in comparison to the irregular shear wall buildings, as shown in Figures 5.71 to 5.73. The variation in roof rotation response of the shear wall buildings due to different high amplitude frequency content of the applied ground motions is observed to be in agreement to the variation in base shear responses and roof deflection responses of the buildings. Koyna earthquake generated a variation of 16% in roof rotation in the case of 4AW1 with respect to response due to application of El-Centro earthquake. Similarly, the Kobe earthquake generated a maximum variation of 22% in roof rotation with respect to that caused by the El-Centro earthquake in the case of 4CW4 building. El-Centro earthquake is observed to have generated 18% higher roof rotation in the case of 2BW0 building with respect to that caused by Kobe earthquake. In most of the other cases, El-Centro earthquake generated higher roof rotation in comparison to the Koyna and Kobe earthquake by 6-12%.

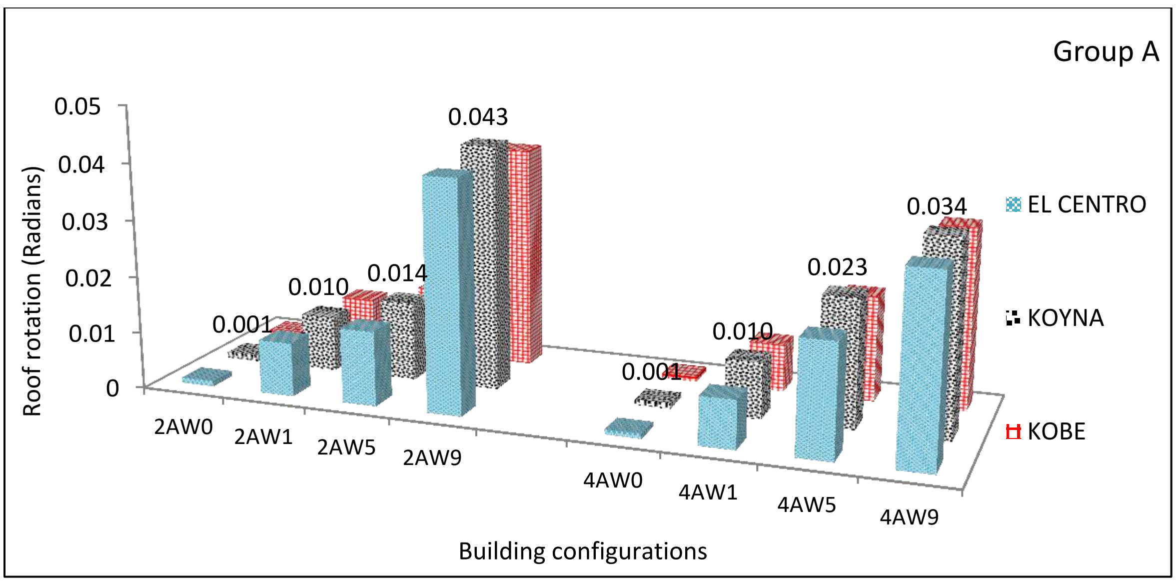


Figure 5.71 Variation in roof rotations of group A shear wall buildings subjected to three ground motions

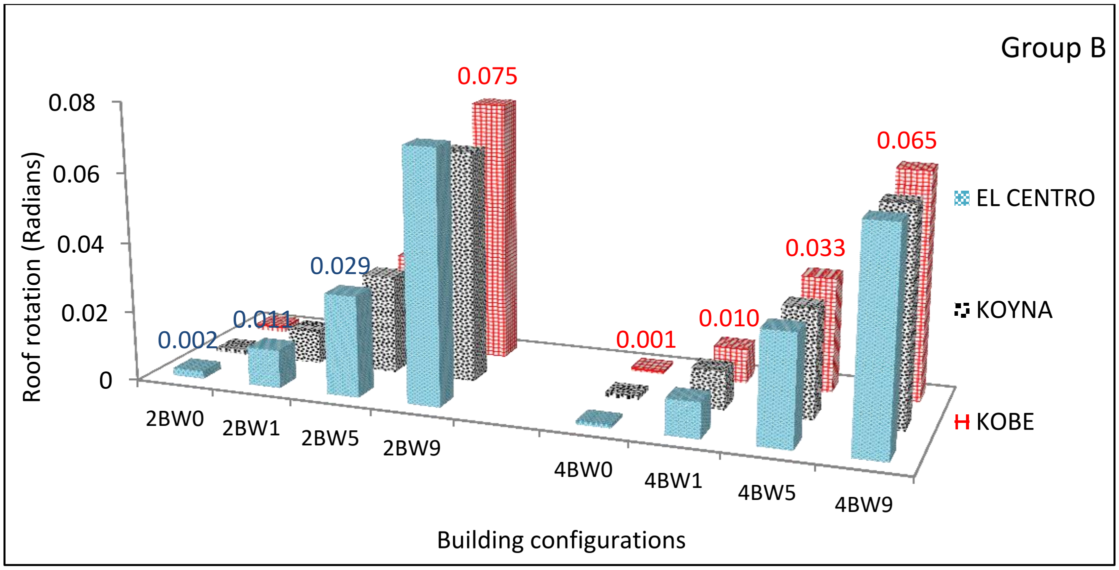


Figure 5.72 Variation in roof rotations of group B shear wall buildings subjected to three ground motions

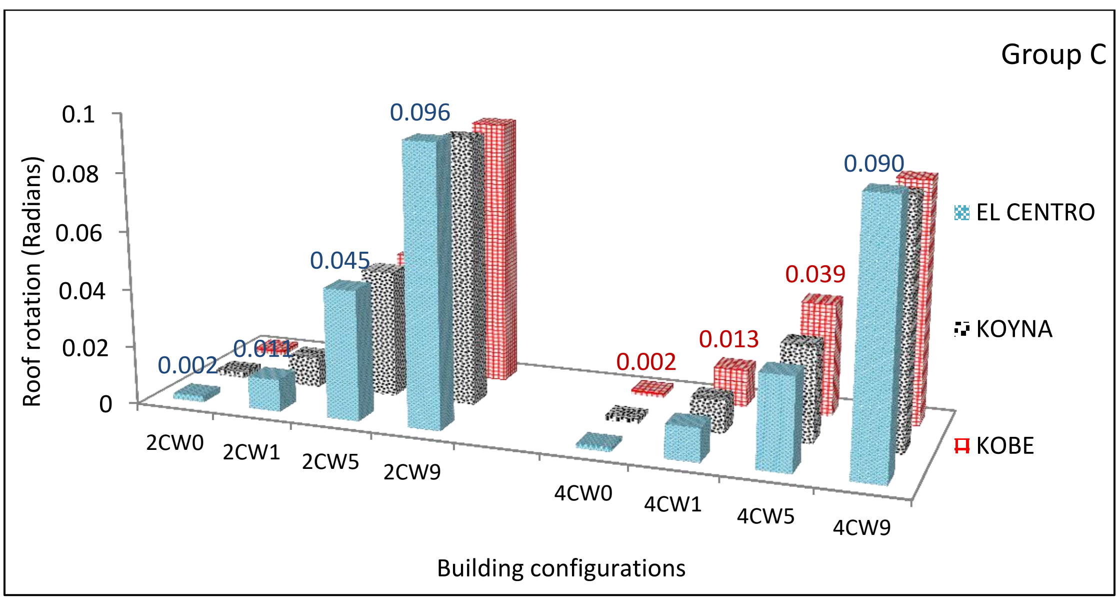


Figure 5.73 Variation in roof rotations of group C shear wall buildings subjected to three ground motions

In the case of shear wall buildings of group A with natural period in the range of 0.25 to 0.48sec, Koyna earthquake caused higher responses in terms of base shear, roof deflection as well as roof rotations. In buildings of this frequency range, Kobe earthquake generated responses lower than that due to the application of the Koyna and El-Centro ground motions. Also, in the case of shear wall buildings, variations, up to 24% is observed, which is higher compared to the case of mass and stiffness irregular buildings. Figure 5.74 shows the Fourier spectrum of the three ground motions along the mapping of frequencies for the shear wall buildings which have higher responses for each of the ground motions considered in the study.

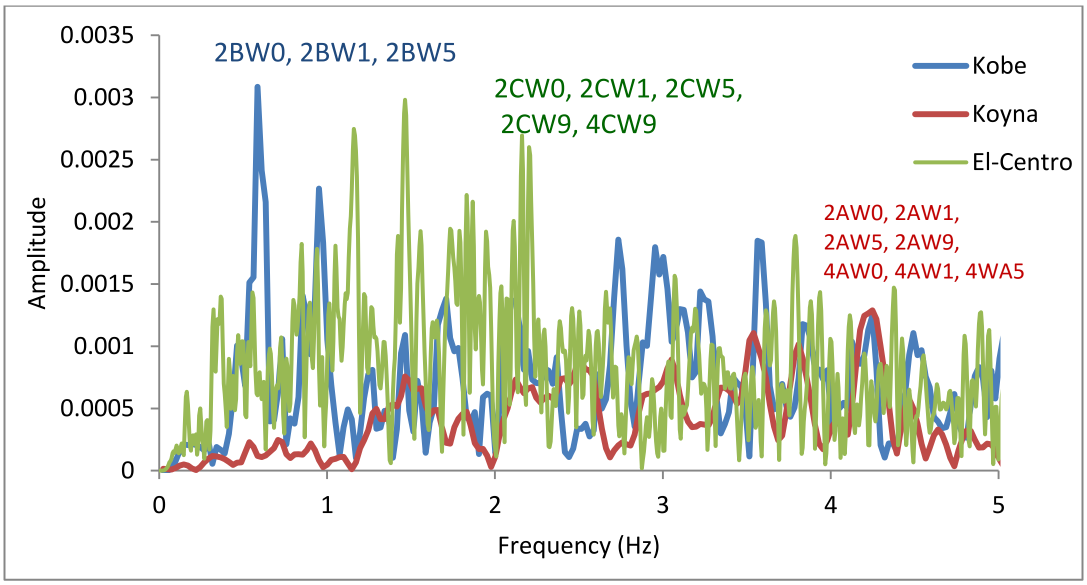


Figure 5.74 Fourier spectrum of Kobe, Koyna and El-Centro ground motions of shear wall irregular buildings with higher responses

The significant observations are that the seismic response of an irregular building with its natural frequencies falling within the high amplitude frequency content range of ground motion has considerable variation with that of the other two ground motions. The maximum increase of 24% was observed in the case of responses of 4AW0 configuration subjected to Koyna ground motion with respect to that generated by El-Centro. The pattern of variation in responses due to the change in eccentricity of the configurations remains the same in all the three ground motions. The El- Centro ground motion, which has high intense frequency contents throughout a broader range of frequencies in comparison to Kobe and El-Centro generated higher responses in the majority of the buildings. Therefore it can be stated that the initial study carried out, on all the buildings with different types of irregularities (considering El-Centro ground motion) and the indices proposed to quantify them holds accurate and valid for the other ground motions also.

5.5.4. Summary

The variations in the seismic responses generated by the three ground motions are mainly due to the variation in their frequency contents. The pattern in responses due to the change in eccentricity of the irregular configurations holds the same for all the three ground motions. The differences in the responses due to the different high amplitude frequency contents of the earthquakes are higher in the case of shear wall buildings in which the presence of shear wall modifies the lateral load-carrying capacity of the building as well as its frequency range.

CHAPTER 6

CONCLUSIONS

The irregular building frames with the eccentric location of shear walls, masses and stiffness were analysed and parametric studies were conducted to determine the effect of in-plan eccentricity. In the case of mass and stiffness irregularities, the effect of in-plan eccentricity, magnitude and the location of the irregularity at the bottom, middle or top level of the frames on their seismic responses were analysed. The dynamic characteristics and seismic responses of structures are expressed in terms of fundamental natural period, absolute maximum responses of base shear, roof deflection, storey drifts, torsional resultant and roof rotation.

The seismic responses of the shear wall buildings 2W0-2W9 and 4W0-4W9 with in-plan eccentricity in the range of 0.05L to 0.57L in this study were evaluated and the following conclusions are drawn:

- Shear wall improves the seismic behavior of buildings when symmetrically arranged in plan. However, as eccentricity of the configurations increases, the seismic responses increases and becomes the same or even higher than that of the bare frame buildings.

- The in-plan eccentricity incorporated was the highest in 2W9 as 0.57L and the highest variation of the same configuration with respect to the building with symmetrically configured shear walls (2W0) are 60% in natural period, 89% in base shear ratio, 62% in roof deflection ratio and 98% in roof rotations.

- The eccentricity to the plan width ratio has high compliance to the torsional irregularity coefficient and can be used to represent the torsional irregularity of buildings with irregularity in mass and stiffness.

Considering the mass irregular buildings belonging to group A, group B and group C with different locations of the additional masses along the height of the building and varying in-plan eccentricities from 0.05L to 0.144L, the seismic responses are studied and the following conclusions are made:

- Vertical mass irregularity at the top level of the frames increases the natural period by 8% to 44% as compared to masses at the bottom level of frames and this is amplified due to e_d/L of 0.14 by a maximum of 31% with respect to the plan regular IM0 buildings. In buildings with a mass ratio of 1.5 at the upper level, there is an amplification of natural period by 18% in Group A, 28% in Group B and 38% in Group C buildings due to in-plan as well as vertical mass irregularity with respect to the regular frame.
- Base shear ratio increases by 9% to 29% in all three groups of buildings due to positioning of the masses at the top levels in comparison to that at the bottom levels. The base shear ratio increases by 35% due to the high in-plan eccentricity at the top level with respect to IM0 buildings. Even a mass ratio of 1.5 placed as $3CIM3_t$ increases the base shear by 22% with respect to regular frame.

- Roof deflection ratio is higher when the masses are at the top level of the frame, by 7% to 46%, as compared to bottom levels. A maximum increase in roof deflection in $3CIM1_t$ is 29% with respect to 3CIM0 building due to in-plan eccentricity. Even though the mass ratio of 1.5 is within the IS code limits, it leads to an increase of roof deflection by 57% with respect to regular frame due to in-plan eccentricity at the upper levels.

- Roof rotation is more when masses are placed at the upper levels by 16% to 67% as compared to the placement of masses at bottom levels. In-plan eccentricity leads to an increase of 138.5% in roof rotation with respect to the plan regular frames. Due to in-plan eccentricity, roof rotation increases by a maximum of 88.8% with respect to the regular frame even in the case of irregular buildings with a mass ratio of 150% (M1.5) at the upper floor levels.

- Proposed mass irregularity coefficient α can be applied to quantify irregularity in mass irregular buildings with in-plan eccentricity and location of irregularity along the building height. The natural period and base shear ratio of the irregular buildings of a mass ratio less than 3, has a good correlation with α in the range of 0 to 0.5. It is suggested that buildings with even additional eccentric masses of ratio of 1.5 also should be planned in such a way that α is not more than 0.3.

The stiffness irregular buildings with modified inter-storey height leading to reduced stiffness reduction up to 43% and in- plan eccentricity ranging from 0.05L to 0.3L were analysed using El-Centro data and the following inferences are made:

- Vertical stiffness irregularity along the height of the frames increases the natural period by 11% to 37% in comparison to that of the regular frames and this is further amplified due to in-plan eccentricity by a maximum of 68% $CIS8_b$ with a stiffness modification K_4. Even in case of K_1 and e_d/L of 0.3 at the lower level, there is an amplification of natural period by 53% in group A, 43% in group B and 35% in group C buildings with respect to the corresponding regular frames due to in-plan stiffness eccentricity.

- Base shear ratio decreases due to stiffness reduction which is further supplemented by the presence of in-plan eccentricity. Considering stiffness reduction K_1 and e_d/L of IS8, base shear ratio of the irregular building decreases by a maximum of

35% due to in-plan eccentricity, whereas considering stiffness reduction K_4, the maximum reduction in base shear ratio becomes 51% as compared to the regular frame.

- Variation in maximum roof deflection with respect to that of the regular frame is higher when the soft stories are present at the bottom level of the frame. The maximum increase of 68% in roof deflection is observed in $CIS8_t$ with respect to CIS0 due to in-plan eccentricity. Considering stiffness reduction K_1 along with e_d/L of 0.3 at the lower floor level, roof deflection ratio increases by 40% with respect to that of regular frame.

- Vertical stiffness irregularity increases the storey drift demands at the vicinity of the irregularity along the building height. Storey drift escalates further due to in-plan eccentricity as compared to regular frame buildings by a maximum variation of 72%, 85% and 89% in the case of $AIS8_b$, $BIS8_b$ and $CIS8_b$ buildings respectively, each with stiffness modification K_4.

- Stiffness irregularities with the highest in-plan eccentricity of 0.3L increase the maximum roof rotation by 2 to 3.5 times when they are present along with soft stories at the lower floor level. The roof rotation increases by a maximum of 85% with respect to IS0 building due to in-plan eccentricity of 0.1L at the lower floor levels along with stiffness modification K_1.

- The proposed stiffness irregularity coefficient β can quantify the stiffness irregularity of any building with soft stories in terms of the in-plan eccentricity, stiffness reduction and location of the soft storey along the height. Soft storeys should be avoided in the lower half of a building in seismically active areas, and in case of any stiffness in-plan eccentricity, β should be possibly maintained within the range of 0 to 0.4.

The proposed expression for natural period with modification factor γ can be used to calculate the natural period of the irregular buildings rather than using the approximate natural period expression as per the seismic code. The expression can be put into use to re-plan the building to modify the natural period for reduced base shear and rotation.

Analysing the seismic responses of the buildings when subjected to three ground motions, the responses vary due to the different frequency content of the earthquakes. However, the pattern of responses with the in-plan eccentricity holds the same for all the three ground motions. The highest variation of 24% was observed in the case of group A shear wall buildings subjected to Koyna ground motion.

In summary, in-plan eccentricity needs to be accounted for while checking the irregularity of a structure. Building systems with mass irregularity at the top levels even within the mass ratio of 1.5, which is permissible as per IS 1893:2016, should be avoided. Similarly any type of stiffness irregularity at the bottom levels of the buildings also should be avoided. The proposed irregularity indices based on geometric dimensions α, β and γ can be employed for the better planning of a building to reduce the seismic effects.

CONTRIBUTIONS FROM THE STUDY

- The extent of change in responses due to variation in the dynamic eccentricity ratio of the buildings with a combination of in-plan eccentricity and vertical irregularities is presented highlighting the significance of accounting in-plan eccentricity while planning buildings.
- A new mass irregularity coefficient α, to quantify mass irregularity and predict the response of vertically mass irregular buildings with in-plan eccentricity based on the geometric dimensions of buildings, location of masses and its in-plan eccentricity has been proposed.
- A new stiffness irregularity coefficient β, to quantify stiffness irregularity and predict the response of vertically stiffness irregular buildings with in-plan eccentricity based on the geometric dimensions of buildings, location of soft storey and its in-plan eccentricity has been proposed.

- Modification factor γ to the approximate natural period expression given in IS 1893:2016 and ASCE 7-16 to estimate the natural period of irregular buildings as an accurate alternative has been proposed.

SCOPE OF FUTURE WORK

This study can be extended to evaluate:

- Experimental investigation on the seismic response of irregular buildings
- The effect of soil-structure interaction on the behavior of irregular buildings

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LIST OF PUBLICATIONS

Journals

1. Archana J. S., Jayalekshmi B.R. and Venkataramana K. (2019), "Effect of in-plan eccentricity in vertically mass irregular buildings irregular RC framed buildings under seismic loads." Asian Journal of Civil Engineering, Springer Publishers, 20(5), 713-726.]10.1007/s42107-019-00138-w

2. Archana J. S., Jayalekshmi B.R. and Venkataramana K. (2018), "Torsional Response of Plan Asymmetric Shear Wall Buildings under Earthquake Loading." International Joint conferences on Advances in Engineering and Technology (AET), 84-90, Grenze Scientific society, McGraw Hill Education(India).

3. Archana J. S., Jayalekshmi B.R. and Venkataramana K. (under review), "Effect of in-plan eccentricity in vertically stiffness irregular buildings irregular RC framed buildings under seismic loads." Engineering Structures, Elsevier

Conferences

1. Archana J. S., Jayalekshmi B.R. and Venkataramana K. (2018), "Torsional behavior of buildings with stiffness irregularity under earthquake load." Proceedings of 8th International Symposium (IES 2019), Paper no: C6-5, 1-6, Kumamoto University, Japan.

2. Archana J. S., Jayalekshmi B.R. and Venkataramana K. (2018), "Torsional Response of Vertically Irregular Building Frames." Proceedings of 16th Symposium on Earthquake Engineering (16SEE), Paper no: 173, Department of Earthquake Engineering, IIT Roorkee, 20-22 Dec 2018.

3. Archana J. S., Jayalekshmi B.R. and Venkataramana K. (2018), "Torsional Response of Plan Asymmetric Shear Wall Buildings under Earthquake Loading."

Proceedings of Second International Conference on Architecture Materials and Construction Engineering, Kerala, IDES and Association of Civil and Environmental Engineers (AMCE), 84-90, Jan 19-20, 2018.

4. Archana J. S., Jayalekshmi B.R. and Venkataramana K. (2018), "Torsional Response of Buildings with Plan Irregularity under Seismic Loading." Proceedings of International Conference on Global Civil Engineering Challenges in Sustainable Development and Climate Change, (ICGCSC-2017), ISBN 978-93-5267-355-1, 5-9, MITE, Moodbadri, Karnataka, March 17-18, 2017.

5. Archana J. S., Jayalekshmi B.R. and Venkataramana K. (2018), "Torsional Behaviour of Asymmetric Shear Wall Buildings." Proceedings of International Conference on Recent Trends in Engineering and Material Sciences, 1-IC-3003, JNU, Jaipur, March 17-19, 2016.

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ACADEMIC PROFILE

Ph.D/ Structural Engineering	*Under the guidance of Dr. B.R. Jayalekshmi and Prof. Katta Venkataramana, Department of Civil engineering, National Institute of Technology Karnakata, Surathkal, 2015-2019.* (on-going) Thesis: Torsional behavior of buildings under seismic loading
M. Tech/ Structural Engineering	Mar Baselios College of Engineering and Technology, Kerala University, 2012-2014 (CGPA 8.77)
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WORK EXPERIENCE

Assistant Professor:	SCMS School of Engineering and Technology, Angamaly, Ernakulam July 2014-December 2014

SOFTWARE SKILLS

Professional Software: AUTOCAD, LS Dyna, STAAD Pro, ANSYS, ETABS, SAP2000, MATLAB

PUBLICATIONS

1. Archana.J.Satheesh, B.R. Jayalekshmi and Katta Venkataramana (2019), Effect of in-plan eccentricity in vertically mass irregular buildings irregular RC framed buildings under seismic loads, Asian Journal of Civil Engineering, Springer Publishers.

2. Archana.J.Satheesh, B.R. Jayalekshmi and Katta Venkataramana (2018), Torsional behavior of buildings with stiffness irregularity under earthquake load, Proceedings of 8th International Symposium (IES 2019), Kumamoto University, Japan.

3. Archana.J.Satheesh, B.R. Jayalekshmi and Katta Venkataramana (2018), Torsional Response of Plan Asymmetric Shear Wall Buildings under Earthquake Loading, International Joint conferences on Advances in Engineering and Technology(AET), McGraw Hill Education(India).

CONFERENCES AND WORKSHOP

1. Presented paper titled 'Torsional Response of Vertically Irregular Building Frames' in 16th Symposium on Earthquake Engineering (16SEE) conducted by Department of Earthquake Engineering, IIT Roorkee on 20-22 Dec 2018.
2. Presented paper titled 'Torsional Response of Plan Asymmetric Shear Wall Buildings under Earthquake Loading' in the Second International Conference on Architecture Materials and Construction Engineering, Kerala, jointly conducted by IDES and Association of Civil and Environmental Engineers (AMCE) on Jan 19-20, 2018.
3. Presented paper titled 'Torsional Response of Buildings with Plan Irregularity under Seismic Loading' at the International Conference on Global Civil Engineering Challenges in Sustainable Development and Climate Change, (ICGCSC-2017), conducted by MITE, Moodbadri, Karnataka on March 17-18, 2017.
4. Attended the 'Literature Review Workshop' conducted by National Information Centre of Earthquake Engineering, IIT Kanpur on June 2-11, 2016.

5. Presented paper titled ‘Torsional Behaviour of Asymmetric Shear Wall Buildings’, at the International Conference on Recent Trends in Engineering and Material Sciences conducted by JNU, Jaipur on March 17-19, 2016.
6. Presented paper titled ‘Flexural and Durability Performance of Self Compacting Rubberized Concrete’, at the National Conference on Emerging Technologies (NCET 2014) conducted by GEC, Barton Hill, Trivandrum on August 02-03, 2014.
7. Attended the ‘National Seminar on Advances in Structural Engineering’ (NASE 2013) conducted by Mar Baselios College of Engineering and Technology on December 16-17, 2013.
8. Published a paper ‘Corrosion Influence on Concrete of different Compressive Strengths’, in the proceedings of ‘National Seminar on Advances in Structural Engineering’ (NASE 2013).
9. Underwent 15 days training on detailed design and analysis of residential buildings using STAAD Pro at Muralee & Associates, Trivandrum in the month of September, 2013.
10. Attended the “International Conference on Green Technologies” conducted by Mar Baselios College of Engineering and Technology from December 18-20, 2012.
11. Attended the national colloquium on ‘Concrete Construction for Costal Conditions-Causes, Concerns & Challenges’ (7Cs), on December 8, 2012.

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