
**ADDIS ABABA INSTITUTE OF TECHNOLOGY**

**SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING**

# SEISMIC BEHAVIOUR OF REINFORCED CONCRETE BUILDINGS WITH MASS AND STIFFNESS IRREGULARITIES

*A Thesis Submitted to School of civil and environmental Engineering in Partial Fulfilment of the Requirements for the Degree of Master of Science in civil Engineering by structural Engineering Stream*

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## ABSTRACT

Many buildings in the present situation have irregular configurations both in plan and elevation due to architectural consideration. The main objective of this research is to study the seismic behaviour of building structures with mass and stiffness irregularities. This is done by quantifying the effects of mass and stiffness irregularities on seismic demands of building structures. The seismic demands investigated are fundamental period, elastic demands (base shear, roof displacement, storey lateral displacement, storey drift and storey shear) and inelastic demands (roof displacement, storey drifts and storey ductility demands). These seismic responses are evaluated by response spectrum and non-linear static (pushover) analysis. A regular building is developed to be considered as base case structure, and cases that represent irregular structures are defined by modifying vertical distribution of mass and stiffness of the base case. The mass irregularity is created by increasing the mass of base case at bottom, middle and top floor by 200%, 300% and 400%, and the stiffness irregularity is created by increasing the height of ground floor of base case 150% and 200%.

The result of this research shows that, the seismic response of building structures is more sensitive to stiffness irregularities than to mass irregularities for mass and stiffness modification of 200%. For the cases of elastic analysis, the distribution of elastic demands over height of the building shows small changes for mass irregularity ( not more than 19%)when compared to stiffness irregularities ( more than 50%).On the other hand, inelastic demands distribution over height of building is non-uniform for both mass and stiffness irregularities. Generally, building structures with mass and stiffness irregularities shows maximum elastic and inelastic seismic demands studied in this thesis work when compared to regular ( base case) building.



**KEY WORDS:** *response spectrum analysis, pushover analysis, mass irregularity, stiffness irregularity*

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# Contents

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# List of Nomenclatures

| | |
|---|---|
| ATC | Applied Technology Council |
| CP | Collapse Prevention |
| EBCS | Ethiopian Building Code Standard |
| $F_b$ | Seismic Base Shear |
| FEMA | Federal Emergency Management Agency |
| $F_i$ | Horizontal Force Acting on Storey i |
| IBC | International Building Code |
| IO | Immediate Occupancy |
| LS | Life Safety |
| m | mass of storey |
| RC | Reinforced Concrete |
| T | period |
| q | Behaviour Factor |
| $\phi_i$ | Amplitude of the Elastic First Mode |

# CHAPTER ONE

# INTRODUCTION

## 1.1 General

During an earthquake, failure of structure starts at points of weakness. This weakness arises due to discontinuity in mass, stiffness and geometry of structure which is termed as irregularity. These weaknesses tend to emphasize and concentrate the structural degradation, often leading to complete collapse. Therefore the structural engineer needs a good understanding of the seismic response of different types and configurations of buildings. Uncertainties in both seismic demands and structural capacities should be considered in comprehensive design procedure that addresses the response of the structures with various configurations which might result from different constraints imposed by owners, architects and planners [1]. Many buildings in the present situation have irregular configurations both in plan and elevation due to aesthetic consideration and city regulation, which in future may be subjected to devastating earthquakes. Irregularities are not avoidable in construction of buildings; however the behaviour of structures with these irregularities during earthquake needs to be studied. Vertical irregularities are one of the major reasons of failures of structures during earthquakes. For example structures with soft storey and irregular distribution of mass were the most notable structures which collapsed during the past earthquakes [2]. Excess mass can lead to abrupt increase in lateral inertial forces, reduced ductility in column, and increased tendency of collapse due to P-delta effects [2].

Soft stories can result from omission of infill in single story, as often occurs in the first story, difference in height of floors, also from partial height infill which are also common in many buildings [3]. In many buildings, first storey columns are badly damaged as shown in fig 1.1 due to shear in earthquake [4]. So, the effect of vertical irregularities on the seismic performance of structures becomes really important. Height-wise changes in stiffness and mass make the dynamic characteristics of the buildings different from the regular building. Also such type of irregularities causes concentration of forces and stresses in some floor, where there is change in mass, stiffness and strength. When a structure has an irregularity in mass, stiffness, strength or vertical geometric irregularity, the method of analysis to be used is the most important point. Based on these, the current building design codes of many countries recommend method of analysis of the buildings. These analysis methods are broadly classified as linear static, linear dynamic, nonlinear static and nonlinear dynamic analysis. From these types of analysis, the first two is suitable only when the structural loads are small and at no point the load will reach to collapse load. The second two are

better than the first two to estimate seismic demand of buildings. In order to prevent collapse of building structures due to irregularity problem, seismic demand must be determined accurately.

**Definition of irregularity of buildings according to different building code**

According to International Building Code (IBC, 2003), building irregularity is to be considered, if the mass of any storey is more than 150% of the mass of an adjacent storey and if the storey drift at any storey is greater than 130% the storey drift at the storey above, and if the lateral stiffness is less than 70% of that in the storey above or less than 80% of the average lateral stiffness of the three storey's above. On the other hand, more strict criteria are proposed in Euro code 8, 2004 provisions. For instance, a structure can classify as regular if the mass of the individual stories remain constant or reduce gradually from the base to the top. A structure is irregular if the floor mass increases along the building height. According to Ethiopian Building Code Standard, EBCS 8, 1995, building is regular if, all lateral load resisting systems, like cores, structural walls or frames, run without interruption from their foundations to the top of the building or, when setbacks at different heights are present, to the top of the relevant zone of the building and both the lateral stiffness and the mass of the individual storey remain constant or reduce gradually, without abrupt changes, from the base to the top.

**Figure 1.1: Shear Failures of Ground Storey Columns [4]**

## 1.2 STATEMENT OF THE PROBLEM

Irregularity of building structure is major problem which leads to disaster during severe earthquake. Irregularities are not avoidable in construction of buildings; however the behavior of structures with these irregularities during earthquake needs to be studied. In order to prevent damages due to irregularity problem, seismic demands must be determined accurately. Several studies have focused on evaluating the response of regular structures. There is a lack of understanding of the seismic response of structure with irregularities in vertical distribution of masses and stiffness's. Most of the previous studies have focused on stiffness irregularities due to infill wall; therefore, a stiffness irregularity due to difference in height of stories needs to be studied. In this thesis work, a comprehensive evaluation of the effect of mass and stiffness irregularities on the seismic responses of reinforced concrete building structures is to be studied, and stiffness irregularity in this thesis work is introduced by varying height of story of buildings.

## 1.3 OBJECTIVE

### General objective

The general objective of this thesis work is to study the seismic behaviour of reinforced concrete building with mass and stiffness irregularities.

### Specific objective

- To study the effect of mass and stiffness irregularity on fundamental period of reinforced concrete buildings.
- To study the effect of mass and stiffness irregularities on the distribution of elastic and inelastic seismic demands over height of 6 and 10 story reinforced buildings.
- To evaluate the plastic hinge distribution of Reinforced Concrete buildings.

## 1.4 METHODOLOGY OF THE STUDY

For the accomplishment purpose of this thesis work there is a methodology that designed to achieve the general and specific objective of this thesis work. Among the methodology that used are listed as follows.

- Review of related literatures.
- Selection of types of reinforced concrete buildings used for analysis and Modeling of the selected buildings with appropriate software. In order to investigate the above-mentioned objectives, several three-dimensional reinforced concrete building models are formed and analyzed according to the Euro code 8, 2004.
- Performing linear and non-linear analysis on selected buildings.

## 1.5 SCOPE OF THE STUDY

In this thesis, only RC buildings with mass and stiffness irregularities are considered. Only linear dynamic and non-linear static analysis is used for all buildings studied in this thesis. Also the contribution of infill wall to the stiffness and effects of soil structure interaction were not considered

## 1.6 THESIS ORGANIZATION

This thesis composed of five chapters. Chapter 1 gives a general introduction, need for the investigation, objective and scope of the investigation and organization of the thesis. In Chapter 2, the review of the previous research on mass and stiffness irregularities are given. Chapter 3 presents the method of structural analysis use in this study and analytical models used in the analysis. The evaluation and comparison of the analyses results are given in Chapter 4. Finally, Chapter 5 contains the summary and the conclusions of the study, and the recommendations for future studies.

# CHAPTER TWO
# LITERATURE REVIEW

## 2.1 General

There are a lot of research works done by different researchers on irregularities of building both in plan and elevation. These works are summarized as follows.

Inel et.al. [6] Studied the evaluation of the buildings reflecting existing construction practice. The paper also covered some models with a soft story. It is concluded in that study that, (a) the increase in the confinement level increases the sustained level of damage, (b) the effect of infill are significant in low rise buildings with weaker members, (c) the main reason for a collapse is found to be weak columns and strong beams, (c) the structural irregularities like short column, soft story and heavy overhangs are quite dangerous but the soft story irregularity with a heavy overhang is the most dangerous one, (d) the irregularity effect are found to be more significant in mid-rise structures than the low rise ones, the soft story irregularity formed by the absence of infill at the ground story is found to be more dangerous than the stiffness based ones.

Esteva [7] studied the nonlinear response of buildings with excessive stiffness and strength above the first story. It is stated that the response of a building is quite sensitive to the stiffness variation along the height of the structure and the p-delta effects are significant on the response. The use of a safety factor to meet the local ductility demands in a soft story, which is dependent to the natural period of a structure, is offered.

Chang and Kim [8] investigated a 20-story building with a soft story by nonlinear time-history and nonlinear pushover analysis. It is stated that low strength reduction factor with perfectly yielding mechanisms are required for effective protection and it is also advised that an amplification factor must be applied to soft stories for which the displacements might be reduced by this way.

Chopra et al. [9] investigated the yielding point of a soft first story for the adequate protection of upper stories from significant yielding. It is concluded that, to limit the force transmitted to the adjacent story above, an elastic-perfectly plastic mechanism is needed as any residual stiffness increase the shear force transmitted. Even if the first story limits the forces transmitted to upper stories, the resulting shear wave propagates and any weakness of strength in an upper story may lead to collapse. In this paper it is also stated that the first soft story mechanisms must be designed according to very large displacements.

Fernandez [10] evaluated the effects of uneven distribution of mass and stiffness on the elastic and inelastic response of multi-storey buildings. It was noted that the type of earthquake record did not have an appreciable influence in the response of low rise (5story) buildings compared to high rise (10story) buildings. It was further observed that a reduction of first story stiffness by 17% to 67%

increased the first story drift by 20% to 100. In general it was noted that good behaviour of structure was obtained when the structure had a continuous variation of mass and stiffness along the height.

Costa, Oliveira and Duarte [11] studied the seismic behaviour of reinforced concrete buildings exhibiting vertical irregularities. Sixteen story buildings were studied for three different horizontal layouts and five vertical configurations. The buildings were idealized as a set of plane moment resisting frames connected to shear walls by rigid diaphragms. Based on their study, Costa et al., put forward the following observations. A discontinuity in the frame markedly increased the ductility demand in the shear wall. Further, the distribution of ductility demand was irregular in shear walls but was fairly regular in the frames, except for stories immediately above a discontinuity, where there was a significant increase in frame ductility demand. For irregular buildings, the ductility demands were observed to be nearly twice as high as those of regular buildings. In general, it was noted that if the irregularity occurred in the frame, the shear wall exhibited an increase in ductility demand and vice versa.

All-Ali et.al [12] showed that mass irregularity only had a limited influence on the seismic performance of buildings, they assumed beams were rigid and plastic hinge formed at all columns ends. Moreover they showed that increase of mass at the top floor produces a relatively larger effect on storey drifts than an increase at middle floor or at the bottom floor of the building.

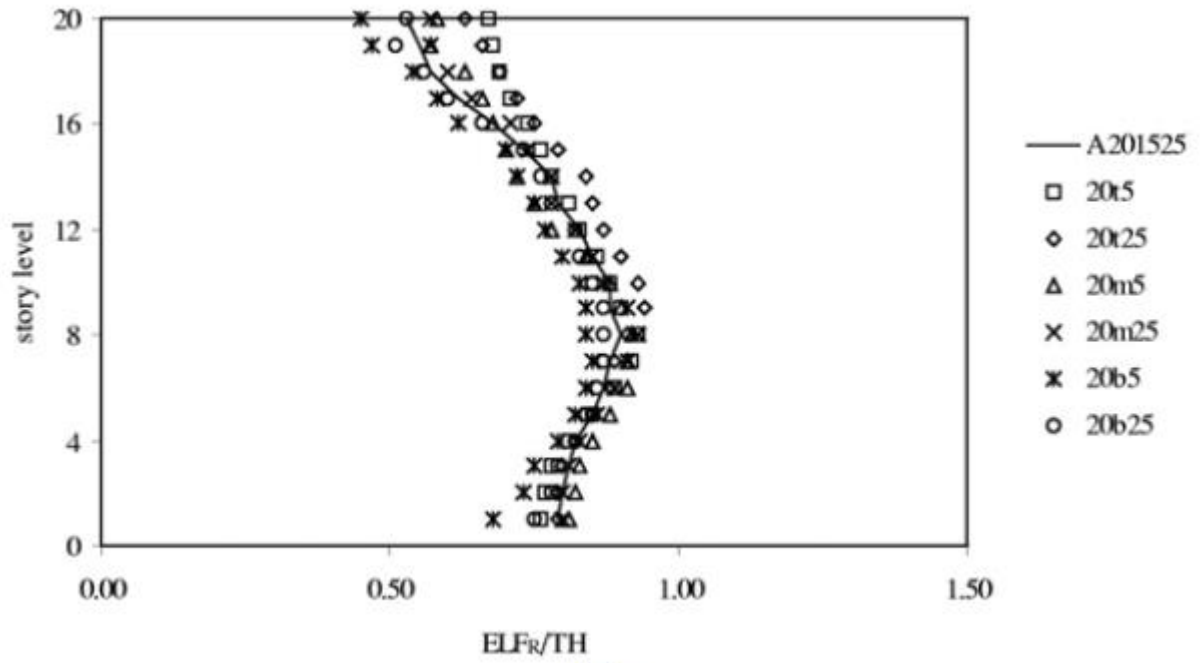
According to study of Sharany Haque et.al [13], open ground story buildings should not be treated as ordinary RC framed buildings. In this study, sway characteristics of RC framed buildings with open ground floor reveals that the columns of open ground floor demands much higher allowance for drift. Drift demand of these columns are, in general, about 75% higher than that predicted by conventional equivalent static force method. Thus special detailing of reinforcement, based on designing the building as special moment resisting frame, may be adopted to meet that high ductility demand of the ground floor columns. However, they feel that more research in this area is needed. It has been found that calculation of earthquake forces by treating the common RC framed buildings with open ground floor as ordinary frames results in an underestimation of design force and moment for ground floor columns. Result shows that, when RC framed buildings having brick masonry infill on upper floor with soft ground floor is subjected to earthquake loading, base shear can be more than twice to that predicted by equivalent earthquake force method with or without infill or even by response spectrum method when no infill in the analysis model. Since response spectrum method is seldom used in practice for the design of such buildings, it can be suggested that the design shear and moment calculated by equivalent static method may at least be doubled for the safer design of the columns of soft ground floor.

Davis and Menon [14] examined the presence of masonry infill panels modifies the structural force distribution significantly in an open ground story building. They considered verities of building case studies by increasing the storey heights and bays in open ground story buildings to study the change in the behaviour of the performance of the buildings with the increase in the number of storey and bays as well as the storey heights. They observed that with the total storey shear force increases as the stiffness of the building increases in the presence of masonry infill at the upper floor of the building. Also, the bending moments in the ground floor columns increase and the failure is formed due to soft storey mechanism that is the formation of hinges in ground storey columns.

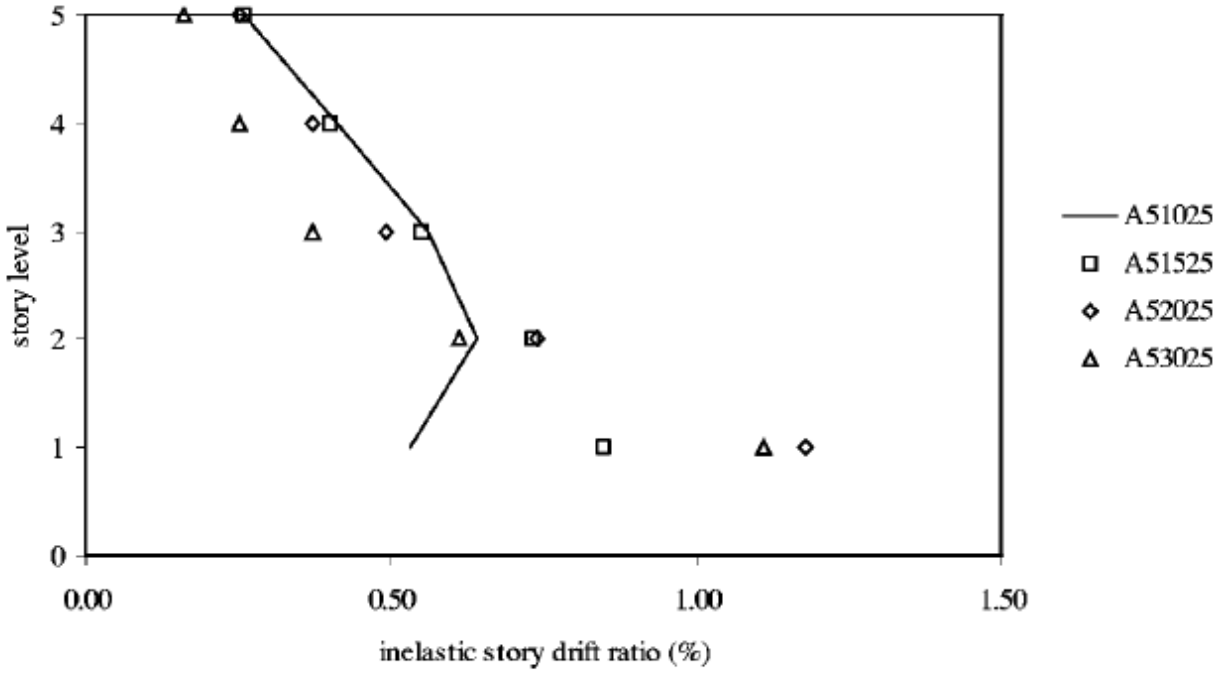
Al-Ali and Krawinkler [1] carried out evaluation of the effects of vertical irregularities by considering height-wise variations of seismic demands. They used a 10-story building model designed according to the strong-beam-weak-column (column hinge model) philosophy and a collective of 15 strong ground motions, recorded on rock or firm soil during Western U.S. earthquakes after 1983, for the parametric study. The effects of vertical irregularities in the distributions of mass, stiffness and strength were considered separately and in combinations, and the seismic response of irregular structures was assessed by means of the elastic and inelastic dynamic analyses. They found that the effect of mass irregularity is the smallest, the effect of strength irregularity is larger than the effect of stiffness irregularity, and the effect of combined-stiffness-and-strength irregularity is the largest. Roof displacement is not affected by the vertical irregularity.

Chintanapakdee and Chopra [14] studied the effects of stiffness and strength irregularities on story drift demand and floor displacement responses. They considered 48 frames, all 12-stories high and designed according to the strong-column-weak-beam (beam hinge model) philosophy. Three types of irregularities in the height-wise distributions of frame properties were considered: stiffness irregularity (KM), strength irregularity (SM), and combined-stiffness-and-strength irregularity (KS). They studied the influence of vertical irregularities in the stiffness and strength distributions, separately and in combination, on the seismic demands of strong-column-weak-beam frames. For this, they compared the median seismic demands of irregular and regular frames computed by nonlinear time history analyses for an ensemble of 20 large-magnitude-small-distance records (LMSR). The ground motion records were obtained for earthquakes from California, with magnitudes ranging from 6.6 to 6.9, and for firm ground sites at epicentral distances of 13 to 30 km. They found that introducing a soft and/or weak story increases the story drift demands in the modified and neighbouring stories and decreases the drift demands in other stories, as shown in Figure 3(a). On the other hand, a stiff and/or strong story decreases the drift demands in the modified and neighbouring stories and increases the drift demands in other stories, as shown in Figure 3(b). Irregularity in upper stories has very little influence on the floor displacements, as shown in Figures

3(c) and 3(d). In contrast, irregularity in lower stories has significant influence on the height-wise distribution of floor displacements, as observed in Figures 3I and 3(f). These results are found to be significantly different from those reported using less realistic column hinge models by Al-Ali and Krawinkler [1].




(a)

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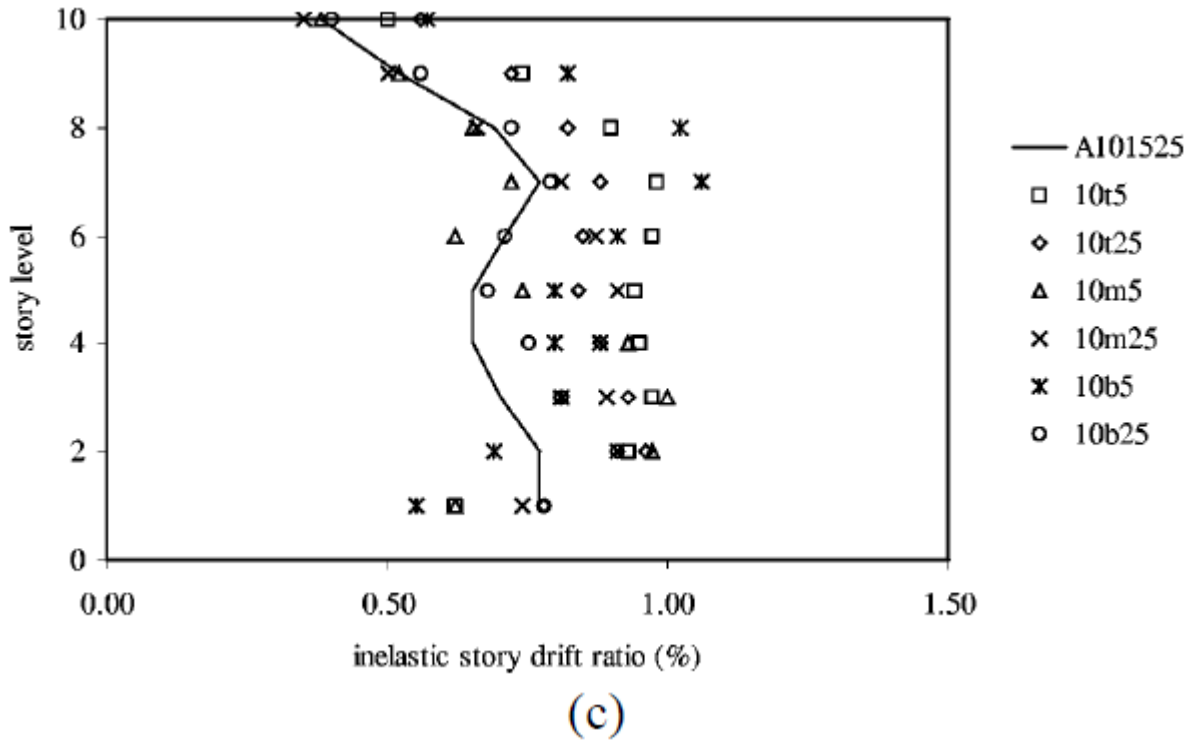



**Figure 2.1: (a)Ratio of elastic story drifts as predicted by ELFR and TH for the 20-story building with mass irregularity;** (b) Variation of inelastic story drift ratios for the 5-story building with mass irregularity; (c) Variation of inelastic story drift ratios for the 10-story building with mass irregularity (after Das and Nau, [12]).

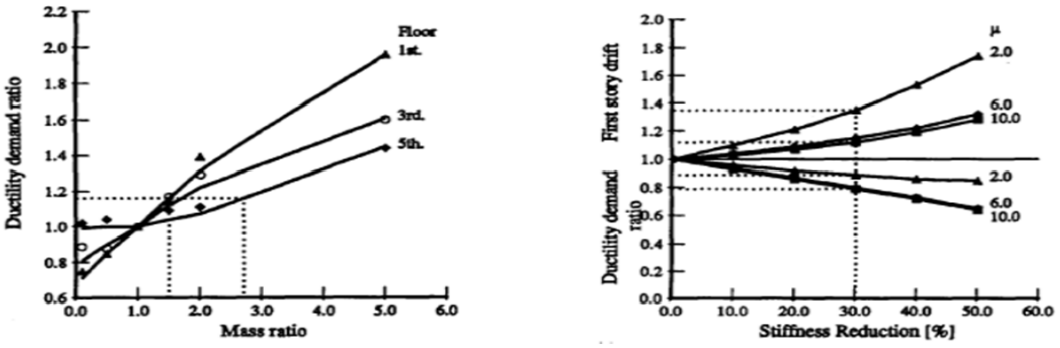
Moehle [41] studied the seismic response of four irregular reinforced concrete test structures. These test structures were simplified models of nine-story three bay building frames comprised of moment frames and frame-wall combinations. Irregularities in the vertical plane of these structures were introduced by discontinuing the structural wall at various levels. Based upon measured displacements and distributions of story shears between frames and walls, it was apparent that the extent of the irregularity could not be gaged solely by comparing the strengths and stiffnesses of adjacent stories in a structure. Structures having the same stiffness interruption, but occurring in different stories did not perform equally. It was observed that the curvature ductility demand in beams varied from 3.9 to 7.2 and for columns, from 1.8 to 2.9 for an abrupt termination of shear walls at different levels along the height.

Hidalgo, Arias and Cruz [21] presented an analytical study to determine the influence of vertical structural irregularities on the results of static and response spectrum analyses. Two shear-wall building models, typical of Chilean reinforced concrete construction, were used. The number of stories in each of these models was 20 and 15 respectively. Stiffness irregularities in these models were introduced by reducing the stiffness of one or more floors. The stiffness ratio studied were in

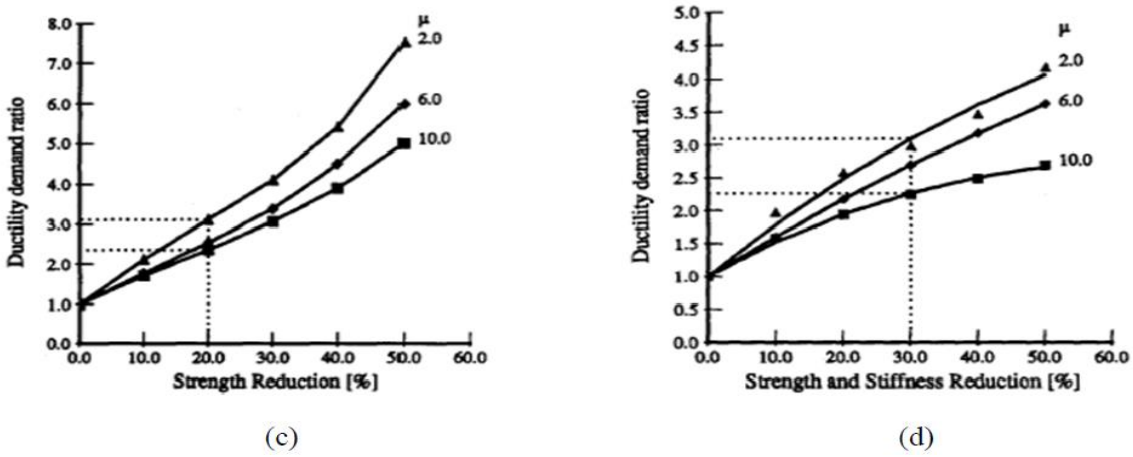
the range of 17% to 83% of the original stiffnesses the depth of the coupling beam in the case of the coupled shear-wall system was also varied. In order to study the variation of lateral strength over the height, it was assumed that changes in strength were usually associated with changes in stiffness. Effects of mass irregularities were also studied by either increasing or decreasing the mass of one floor with respect to the adjacent floor. The mass ratios studied were in the range of 25% to 175%. The locations of these stiffness, strength, and mass ratios were also varied along the height of the structure and were conceptualized as setbacks. The most relevant conclusions obtained from this study may now be summarized. Considering all cases of vertical structural irregularities, the most critical was that of a setback at mid-height of the building, involving simultaneous reductions in plan geometry, stiffness, and mass. The second most critical was that of reduction in stiffness in the lower stories. Also, an irregular distribution of strength could imply a larger demand of ductility at weak sections near the irregularity during severe earthquakes. It was also pointed out by the authors that for the type of buildings considered in their study, the UBC limitations for using the static analysis procedure were too stringent.

Aranda [39] studied irregular stiffness and mass distribution and setbacks in frames. He concluded that ductility demands increase by a factor of 2 in the vicinity of a sudden change in stiffness distribution.

Valmundsson*et.Al.* [24] studied the two dimensional building frames with 5, 10 and 20storey. They studied the earthquake response of these structures with non-uniform mass, stiffness and strength distributions. Response from time history and equivalent lateral force methods are being compared. Based on this comparison they evaluated the requirements under which a structure can be considered regular and ELF procedure are applicable.




(a) (b)




(c) (d)

**Figure2.2: (a) Maximum ductility demand for 5-story structure with mass irregularity and design ductility = 2**; **(**b) Maximum ductility demand and first story drift for 20-story structure with stiffness irregularity; (c) Maximum ductility demand for 20-story structure with strength irregularity; (d) Maximum ductility demand for 20-story structure with strength and stiffness irregularities [24].

Magliulo*et al.* [25] investigated 5 and 9-stories reinforced concrete buildings with mass irregularity that has been designed according to Europe building codes. They are concluded that international code can't realize building regularity and irregularity spatially in distribution of strength and mass.

Poncet and Trembly [26] concluded design of frame with vertical mass irregularity by linear static procedure gives lower performance than regular frame.

Sadashiva*et al*. [27] illustrated that the effect of irregularity depends on the structural model used, the location and amount of the irregularity, and the analysis method used.

A paper by Arlekar, Jain and Murty [29] highlights the importance of explicitly recognizing the presence of the open first story in the analysis of the building. The error involved in modelling such buildings as complete bare frames, neglecting the presence of infill in the upper story, is brought out through the study of an example building with different analytical models. This paper argues for immediate measures to prevent the indiscriminate use of soft first story in buildings, which are designed without regard to the increased displacement, ductility and force demands in the first storey columns. Alternate measures, involving stiffness balance of the open first storey and the storey above, are proposed to reduce the irregularity introduced by the open first storey. In this paper, stiffness balancing is proposed between the first and second storey of a reinforced concrete moment-resisting frame building with open first storey and brick infill in the upper story. A simple example building is analysed with different models. The stiffness effect on the first storey is demonstrated through the lateral displacement profile of the building, and through the bending moment and shear force in the columns in the first storey.

The study of Nagae, Suita, and Nakashima [30] focuses on the seismic performance of soft-first-storey buildings, which are demanded specially in urban areas. Six-story reinforced concrete buildings are focused on and the seismic response of the soft-first-storey structures and typical frame structures are statistically assessed based on the results of dynamic response analyses. The mean annual frequency of the maximum interstory drift ratio exceeding the specified value is computed, and the mean annual frequencies for the plural values are shown as a seismic hazard curve for each case. Eventually the probabilities of the maximum interstory drift ratios exceeding safety limit states are computed and compared. The soft-first-story buildings with the yield strength coefficient of more than 0.7 showed the same level of safety in comparison with typical frame structures, on the condition that the same deformation capacities are given to the main structural members.

S.Haque, Khan Mahmud Amanat [31] studied the behaviour of the columns at ground level of multi-storeyed buildings with soft ground floor subjected to dynamic earthquake loading. The structural action of masonry infill panels of upper floors has been taken into account by modelling them as diagonal struts. Finite element models of six, nine and twelve storied buildings are subjected to earthquake load in accordance with equivalent static force method as well as response spectrum method. It has been found that when infill is incorporated in the FE model, modal analysis shows different mode shapes indicating that dynamic behaviour of buildings changes when infill is incorporated in the model. Natural period of the buildings obtained from modal analysis are close to values obtained from code equations when infill is present in the model. This indicates that for better dynamic analysis of RC frame buildings with masonry walls, infill should be present in the model as well. Equivalent static force method produces same magnitude of earthquake force regardless of the infill present in the model. However, when the same buildings are subjected to response spectrum method, significant increase in column shear and moment as well as total base shear has been observed in presence of infill. In general, a two fold increase in base shear has been observed when infill is present on upper floors with ground floor open when compared to the base shear given by equivalent static force method. The study suggests that the design of the columns of the open ground floor would be safer if these are design for shear and moment twice the magnitude obtained from conventional equivalent static force method. Study of the sway characteristics also reveals significantly high demand for ductility for columns at ground floor level. Presence of in filled wall on upper floors demands significant enhancement of column capacity or ductility to cope up with increased sway or drift.

Samir Helou& Abdul RazzaqTouqan [32], illustrates the importance of the judicious distribution of shear walls. The selected building is analyzed through nine numerical models which address the behaviour of framed structures. The parameters discussed include, the fundamental period of

vibration, lateral displacements and ending moment. It is noticed that an abrupt change in stiffness between the soft storey and the level above is responsible for increasing the strength demand on first storey columns. Extending the elevator shafts throughout the soft storey is strongly recommended.

Tuladhar and Kusunoki [33], investigated the seismic performance and design of the masonry infill reinforced concrete fame structure with the soft first storey under a strong ground motion. The study also highlighted the error involved in modeling of the RC frame buildings as completely bare frame neglecting stiffness and strength of the masonry wall in the upper floors. The attempt was made to determine the strength increasing factor to account the effect of the soft storey through various 2D analytical models using capacity spectrum method and established its relationship with initial stiffness ratio. In this study, the nonlinear dynamic time history analysis was also carried out with a 3D practical model in order to verify the proposed strength increasing factor.

According to Yong Lu,Tassios, Zhang and Vintzileou [34] two six-story, three-bay, reinforced concrete frames, one having a tall first story, and the other a discontinuous interior column, were designed in accordance with Euro code 8,2004, and their models were constructed and tested on an earthquake simulator. The main objectives of the investigation were to study the structural effects of these particular irregularities and to check the relevant design code provisions. During tests, Frame having tall first storey performed in a reasonably regular manner. For discontinuous interior column frame, the response during moderate earthquakes was strongly influenced by the increased flexibility in the direction towards the missing column side, combined with the gravitational effects on the suspended beam spans. The response of discontinuous interior column frame to strong earthquakes was dominated by an apparent soft first-story mechanism. Both frames exhibited a weakness at the fifth story where discontinuity in stiffness occurred. A base shear over strength factor, with respect to the design required base shear, approximately of the order of three and two, was achieved for tall first storey frame and discontinuous interior column frame, respectively.

R.K.L. Su [35], studied how to improve the general understanding of the seismic response of concrete buildings with transfer structures in low-to moderate seismicity regions. This paper summarizes and discusses the existing codified requirements for transfer structure design under seismic conditions. Based on the previous shaking table test results and numerical findings, the seismic effects on the inelastic behaviours of transfer structures are investigated. The mechanisms for the formation of a soft storey below transfer floors, the abrupt change in inter-storey drift near transfer storey and shear concentration due to local deformation of transfer structures are developed. Design principles have been established for controlling soft-storey type failure and minimizing shear concentration in exterior walls supported by transfer structures. The influence of the vertical positioning of transfer floors on the seismic response of buildings has also been reviewed.

A paper of Yong Lu [36], highlights a comparative study of the nonlinear behaviour of reinforced concrete (RC) multi-storey structures is carried out on the basis of measured response of four six story, three-bay framed structures, namely a regular bare frame, a discontinuous column frame, a partially masonry-infill frame and a wall-frame system. The structures were designed for seismic requirements in accordance with Euro code 8, 2004, and their models were subjected to similar earthquake simulation tests. Experimental observations and numerical analyses show that the distribution of the story shear over strength is a rather stable indicator of the general inelastic behaviour of frames and hence can be employed as a characteristic parameter to quantify the frame irregularity for design purposes. Abrupt discontinuity of the geometry or arrangement of structurally effective elements, where unavoidable, may be compensated by strength enhancement targeting a smoothed over strength profile to allow for distributed inelastic deformation, and this principle applies as well to non-uniformly masonry infill frames. For the wall-frame system, adequate countermeasures against rocking of the RC wall are shown to be a key to maintaining the effectiveness of the system at advanced inelastic response.

According to paper of Poonam et al. [38], results of the numerical analysis showed that any storey, especially the first storey, must not be softer/weaker than the story above or below. Irregularity in mass distribution also contributes to the increased response of the buildings. The irregularities, if required to be provided, need to be provided by appropriate and extensive analysis and design processes.

Bariola [40] investigated the influence of strength and stiffness variation on seismic behaviour of structures. He studied the nonlinear response of an 8 story building, with five bays per floor, subjected to five different earthquakes. Three different categories of building periods were considered-low, medium, and high. For every building, two cases were considered, one weak building and one strong. The weak building had base shear strength of 15% of the total weight of the building, while the strong building had base shear strength of 30% of the total weight. The results of this study indicated that the period of a structure increases during an earthquake, with larger period elongation for weaker structures. He stated that if this increases in period is considered along with an increase in damping, a standard linearly-elastic response spectrum can be used to estimate the building response.

Sharooz and Moehle [43] studied the effects of setbacks on the earthquake response of multi-storey buildings. In an effort to improve design methods for setback structures, an experimental and analytical study was undertaken. In the experimental study, a six story moment-resisting reinforced concrete space frame with 50% setback in one direction at mid-height was selected. The analytical study focused on the test structure and on several analytical representations of setback buildings. The

following were the observations from the experimental study. The displacement profiles were relatively smooth over height. Relatively large interstory drifts at the base of the tower were accompanied by a moderate increase in damage at that level. Overall, the predominance of the fundamental mode on the global translational response in the direction parallel to the setback was clear from the lateral displacement and inertia force profiles. Furthermore, the distribution of lateral forces was almost always similar to the distribution specified by UBC. The abrupt reduction in forces at the setback exceeded the static analysis threshold of the UBC; no significant peculiarities in dynamic response were detected. To investigate further, an analytical study was also carried out on six generic reinforced concrete setback frames. These frames were designed by the UBC static method and by a modal analysis procedure. All the setback frames were classified as having an irregular configuration according to the current building code (UBC-1998). The following observations were made from the analytical study. For each of the six setback configurations, all the frames indicated a similar amount and distribution o ductility demand. For all six frames, the floor plan dimensions and mass ratios ranged between 300% and 900% respectively. These were well above the threshold limits for applicability of the static design approach. Nevertheless, the response of only a few frames incurred damage concentration in the tower as indicated by relatively high rotational ductility's. Furthermore, frames having identical plan dimensions and mass ratios but setback at different heights did not experience the same degree of damage. Thus, the approach (UBC 1998) in which regular and setback structures were differentiated according to plan dimension or mass ratio appeared to be insufficient.

Wood [44] investigated the seismic behaviour of reinforced concrete frames with setbacks. Two small-scale reinforced concrete test structures with setbacks were constructed and subjected to simulated ground motion. The displacement, acceleration, and shear responses of the setback frames were compared with those of seven previously tested frames with uniform profiles. Each structure considered by Wood in this study was comprised of two identical planar frames. The tower structure was a symmetrical arrangement with a seven story tower and a two story base. The stepped structure was an unsymmetrical arrangement of a three-story tower, a three-story middle section, and a three story base. The first story height was approximately 1.4 times the height of the upper stories. These nine structures were classified using the UBC-1998 definitions for vertical structural irregularities. Based on this study, the following conclusions were drawn. The displacement and shear responses of the setback frames were governed primarily by the first mode. Acceleration response at all levels exhibited the contribution of higher modes. The linear mode shapes for setback frames exhibited kinks that were not present in uniform frames; however, there was no evidence to suggest that kinks adversely influenced the dynamic response of the setback frames. Distributions of maximum story

shear were also well represented by the equivalent lateral force distributions for all frames. Further based on observations, Wood pointed out that the difference between the nonlinear behaviour of regular and setback frames do not warrant different design procedures required in building codes (UBC-1998; BOCA-1989).

Pinto and Costa [42], evaluated the nonlinear seismic behaviour of setback buildings with reinforced concrete frames. In the study, a set of 17 different buildings were considered: nine 4 story, four 8 story and four 20 story buildings. All of these structures had the same plan configuration. Hover, with regard to elevation, some were regular but others were irregular with different degrees of setback. The fundamental frequencies of these buildings ranged from 0.49 Hz to 3.20 Hz. This covered all of the key frequencies of the design response spectrum in the Portuguese Code. The main conclusions suggested by the authors may be summarized as follows. For most buildings, it was evident that a greater concentration of the largest values of ductility demands occurred in lower stories. However, some critical zones at intermediate heights were also observed. For buildings with similar frequencies and different heights, the tallest exhibited the greatest values either for the ductility demands or for the story forces at the floor level. The consequences of the irregularities were evident on the shear forces for all of the buildings and on the ductility demands of the 4 story buildings. However, the influence of the irregularities was not evident on the displacements of the 8 and 20 story buildings. The influences of the characteristics of ground motion on the response parameters of the buildings analyzed were also observed.

Aranda [39] studied the nonlinear response of irregular reinforced concrete frames. Two reinforced concrete frames, irregular in height, were idealized as single stick models with masses lumped at the floor levels. It was found that irregularities in elevation increase the ductility demand by a factor of 2. This effect was more pronounced where there was a sudden change in the stiffness distribution along the vertical height of the building.

Moehle and Alarcon [41] presented a combined experimental and analytical study to examine the seismic response behaviour of reinforced concrete frame-shear wall structures. In one of the models, vertical irregularity in the frame-shear wall system was introduced by interrupting the shear wall at the first story level. Inelastic dynamic analysis was capable of adequately reproducing measured displacement waveforms, but accurate matching of responses required a trial and error approach to establish the best behaviour assumptions. It was observed that in the vicinity of the discontinuity, the elements exhibited a curvature ductility demand 4 to5 times higher in the case of the model without any interruption of the shear wall.

Wong and Tso [48], who studied the elastic response of set-back structures by means of response spectrum analysis, found that the modal weights of higher order modes for setback structures are

large, leading to a seismic load distribution that is different from static code procedures. They also found that for setback structures, although higher order modes may contribute more to the base shear than the fundamental mode, the first mode still dominates the displacement response.

Humar and Wright [45], using one ground motion record in their study, found that the difference in elastic and inelastic interstory drifts between setback and regular structures depends on the level of the story considered. For the tower, interstory drifts were found to be larger than for regular structures. For the base, interstory drifts were found to be smaller in "setback" structures than in the regular ones.

Ruiz and Diederich [47] also studied structures with soft and weak first story subjected to the same soft soil ground motion record from the 1985 Mexico earthquake. They concluded that the behaviour of these structures depends on the ratios of the dominant period of the excitation to the effective period of the nonlinear response. They also noted that their results are based on one soft soil record and that different results might be obtained for other kinds of ground motions.

Nassar and Krawinkler [46] and Seneviratna and Krawinkler [49] studied an extreme case of a "weak" first story structure, in which all the stories in the MDOF structure stay elastic except for the first story. Both studies concluded that extreme strength discontinuities, such as the "weak" first story they studied, lead to large amplifications in ductility and overturning moment demands and should be avoided whenever possible.

Sarkar*et.Al.*[50] proposes a new method of quantifying irregularity in stepped building frames, which accounts for dynamic characteristics i.e. mass and stiffness. This paper discusses some of the key issues regarding analysis and design of stepped buildings. They proposed a new approach for quantifying the irregularity in stepped building. It accounts for properties associated with mass and stiffness distribution in the frame. This approach is found to perform better than the existing measures to quantify the irregularity. Based on free vibration analysis of 78 stepped frames with varying irregularity and height, this study proposes a correction factor to the empirical code formula for fundamental period, to render it applicable for stepped buildings. They proposed a measure of vertical irregularity, called 'regularity index', accounting for the changes in mass and stiffness along the height of the building as a ratio of $\Gamma_1$ and $\Gamma_{ref}$, Where, $\Gamma_1$ is the 1$^{st}$ mode participation factor for the setback building frame under consideration and $\Gamma_{ref}$ is the 1$^{st}$ mode participation factor for the similar regular building frame without steps. The regularity index is mathematically expressed as follows:

$$\eta = \frac{\Gamma_1}{\Gamma_{1,ref}} \qquad (2.1) \quad \text{Ref. [50]}$$

They concluded that regularity index increases with increase in number of story, and the rate of increase being stiffer when the number of story per step increases. Also, the concluded that, for any given number of story, the regularity index is least when the number of story per step is largest. Thus the proposed irregularity index is able to capture effectively the irregularity caused due to the various geometrical stepped configurations. In continuation of the study they propose to improve the code based empirical formula for estimating the fundamental period to render it applicable for stepped building. They defined a correction factor *k* for the empirical formula of IS 1893:2002 and modified it, as shown:

$$T = 0.075 h^{0.75} * k \qquad (2.2)$$

$$k = \frac{T}{T_{ref}} = [1 - 2(1-\eta)(2\eta - 1)] \quad for\ 0.6 \le \eta \le 1.0 \quad \text{Ref.} \quad [50]$$

Where, *h* = overall building height in meter.

Rajeev and Tesfamariam [51] studied the seismic performance of non-code conforming RC buildings designed for gravity loads. The analysis highlights the need for reliable vulnerability assessment and retrofitting. The vulnerability is compounded since the RC buildings are subject to different irregularities such as weak storey, soft storey, plan irregularities sand other types Fragility based seismic vulnerability of structures with consideration of soft storey(SS) and quality of construction(CQ) is demonstrated on three-, five-, and nine-storey RC frames designed prior to 1970s. Probabilistic seismic demand model (PSDM) for considered structures is developed, by using the nonlinear finite element analysis. Further, the fragility curves are developed for the three structures considering SS, CQ and of their interactions. Finally, confidence bounds on the fragilities are also presented as a measure of their accuracy for risk-informed decision-making. With the PSDM models the corresponding fragility curves are generated in the analysis. They concluded that the vertical irregularities and construction quality in seismic risk assessment have a significant influence in the decision making phase. The proposed approach of developing a predictive tool can enhance regional damage assessment tool, such as HAZUS, to develop enhanced fragility curves for known SS and CQ.

Scarlet [52] identified the qualification of seismic forces of OGS buildings. A multiplication factor for base shear for OGS building was proposed. Modelling the stiffness of the infill walls in the analysis was focused. The effect of in Multiplication factor with the increase in storey height was studied. He observed the multiplication factor ranging from 1.86 to 3.28 as the number of storey increases from six to twenty.

Patel [53] proposed both linear as the Equivalent Static Analysis and Response Spectrum Analysis and the nonlinear analyses as the Pushover Analysis and Time History Analysis for Low-rise open ground storey framed building with infill wall stiffness as an equivalent diagonal strut model. The effect of the infill wall is studied considering the Indian standard code IS 1893 2002 criteria mention for OGS buildings. She observed that the analysis results shows that a MF of 2.5 is too high to be multiplied to the beam and column forces of the ground storey of the buildings. Their study conclude that the problem of open ground storey buildings cannot be identified properly through elastic analysis as the stiffness of open ground storey building and a similar bare-frame building are almost same.

Karavasilis*et.Al.* [54] presented a study on the inelastic seismic response of plane steel moment resisting frames (MRF) with setbacks.

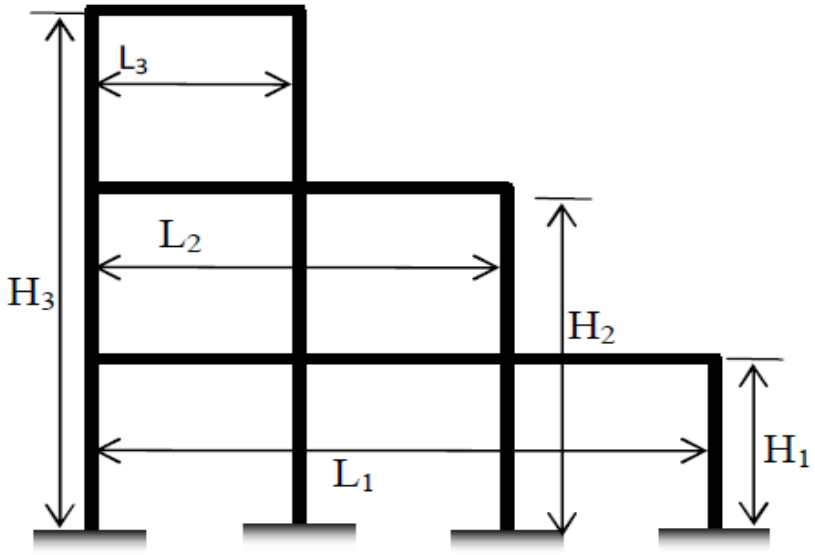



**Figure 2.5: Frame geometry for definition of irregularity indices** [54]

The statistical analysis of the created response databank indicates that the number of stories, beam-to-column strength ratio, geometrical irregularity and limit state under consideration strongly influence the height wise distribution and amplitude of inelastic deformation demands.

Nonlinear regression analysis is employed in order to derive simple formulae which reflect the above-mentioned influences and offer, for a given strength reduction (or behaviour) factor, three important response quantities, i.e. the maximum roof displacement, the maximum inter storey drift ratio and the maximum rotation ductility along the height of the structure. They proposed an alternative approach to quantify the irregularity in a building frame due to the presence of steps. They defined two irregularity indices for stepped buildings, $\phi_s$ and $\phi_b$ (for storey-wise and bay wise stepping respectively) as follows:

$$\phi_s = \frac{1}{n_s - 1}\sum_{1}^{n_s - 1}\frac{L_i}{L_{i+1}}$$

$$\phi_b = \frac{1}{n_b - 1}\sum_{1}^{n_b - 1}\frac{H_i}{H_{i+1}} \qquad \text{(2.3) Ref. [54]}$$

Where, $n_s$ is the number of storeys of the frame and $n_b$ is number of bays at the first storey of the frame. $H_i$ and $L_i$ are height and the width respectively of the $i^{th}$ storey as shown in Fig 2.5

These indices represent the irregularity in stepped frame in an improved manner compared to the code procedures. However, it is not convenient to use two indices to represent the irregularity of the same stepped building. Moreover, these indices are based on geometrical considerations alone. It was assumed that the column and beam sizes are uniform throughout their length and masses are uniformly distributed along the height and width of the frame. But this may not be the case in practical buildings. However unlike design code provisions, the irregularity indices prove to quantify the degree or amount of irregularity in a setback building.

Young [56] presented a study in the determination of the fundamental period of vibration of structures with geometric irregularities. This study investigated the fundamental periods of three different types of steel earthquake-resistant building structures: moment resisting frames (MRF), concentrically braced frames (CBF), and eccentrically braced frames (EBF) with varying geometric irregularities. A total of 24 MRFs, 12 CBFs, and 12 EBFs are designed and analyzed with ETABS v.9.7.2. The fundamental periods based on vibration theory for each example were compared with empirical equations, including design code equations as well as equations proposed in published literature. Based on the results obtained from vibration theory (Rayleigh equation), equations for the approximate fundamental periods are put forth for MRFs, CBFs, and EBFs which take into account vertical and horizontal irregularities.

They proposed two factors as $H_{av}/H$ and $D_{av}/D$, and performed a regression along with overall building height to compute the fundamental period of the building. They defined $H_{av/H}$ as the ratio of weighted average height of building to the overall height of the building and $D_{av}/D$ as the ratio of weighted average width of building to the overall width of the building. They proposed an equation for the calculation of fundamental period the irregular steel MRF building of the form as shown below:

$$T = 0.07(H)^{0.75}\left(\frac{H_{av}}{H}\right)^{0.35}\left(\frac{D_{av}}{D}\right)^{0.2} \qquad \text{(2.4) Ref. [56]}$$

Through statistical comparison, it was found that a 3-variable power model which is able to account for irregularities resulted in a better fit to the Rayleigh data than equations which were dependent on height only. The proposed equations were validated through a comparison of available measured

period data. For braced frames, the proposed equations were also compared with a database of examples from literature.

In general, there are a significant number of research studies that investigates the effect of vertical irregularities, performance of structures with irregularities and many other works on irregularities. Among those studies, many of them evaluate the seismic behaviour of buildings focusing only on two-dimensional analysis of the buildings. Also in many of the studies, stiffness irregularity studied was due to masonry infill wall. Therefore this research investigates the seismic behaviour of 3D Reinforced Concrete buildings with mass and stiffness irregularities. Stiffness irregularity to be studied in this thesis work is introduced by varying height of story of buildings.

# CHAPTER THREE
# METHODS OF STRUCTURAL ANALYSIS

## 3.1 General

Seismic analysis is a major tool in earthquake engineering which is used to predict the response of buildings due to seismic excitations in a simpler manner. In the past decades, the buildings were designed just for gravity loads and seismic analysis is a recent development. Now a day's seismic analysis is a part of structural analysis and a part of structural design where earthquake is prevalent [20].

Among the engineering community involved with the development of seismic design procedures, there is a general belief that the conventional elastic design and analysis methods cannot capture many important aspects that control the seismic performance of structures in severe earthquakes. Moreover, another powerful tool, inelastic time-history analysis is computationally expensive and not feasible for many cases. Nowadays, engineers are seeking a technique, which would solve the drawbacks described above. The search for a more useful and rational design process is a big issue for the future. Design has always been a compromise between simplicity and reality. The latter term, reality, seems to be very complex due to big uncertainties in imposed demands and available capacities. The first term, simplicity, is a necessity driven by computational cost and at the same time the limited ability to implement complexity with available knowledge and tools [16].

The estimation of demands can be accomplished using a variety of available procedures. The primary objective is to determine forces and deformations both at the global and at the local level when the structure is subjected to seismic loads that characterize the hazard at the building site. As such, there are four possible methods to analyse a mathematical model of a building structure. They may be classified into two broad categories depending on the treatment of the response or the treatment of the loads. The former category results in the distinction between linear and nonlinear methods of analysis, while the latter distinguishes static and dynamic application of the seismic loads.

**Linear static and dynamic procedures:** These procedures are recommended for regular buildings where issues such as torsion and high-mode effects are negligible. The expectation is that the computed displacements using linear equivalent elastic stiffness are approximately equal to the actual displacements that may occur inelastically under the design loads [24].

**Nonlinear static and dynamic procedures:** Nonlinear procedures are generally applicable for all buildings with the exception that nonlinear static procedure is limited to buildings where high-mode effects are small. The FEMA document has explicit guidelines to determine if higher modes play an important role in the response: high-mode effects are deemed significant if the shear in any story

resulting from a modal analysis considering modes required obtaining 90% mass participation exceeds the corresponding story shear considering first-mode response only by a factor of 1.3 [24].

### 3.2 EQUIVALENT STATIC ANALYSIS

The equivalent static analysis procedure is essentially an elastic design technique. It is simple to apply than the multi-modal response method. According to Euro code 8, 2004, this type of analysis may be applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction. This requirement is deemed to be satisfied in buildings which fulfil the following conditions.

- They have fundamental periods of vibration $T_1$ in the two main directions which are smaller than $T_1 \leq \{4T_c, 2s\}$.
- All lateral load resisting systems, such as cores, structural walls, or frames, shall run without interruption from their foundations to the top of the building or, if setbacks at different heights are present, to the top of the relevant zone of the building.
- Both the lateral stiffness and the mass of the individual storeys shall remain constant or reduce gradually, without abrupt changes, from the base to the top of a particular building.
- In framed buildings the ratio of the actual storey resistance to the resistance required by the analysis should not vary disproportionately between adjacent storeys.

The equivalent static analysis procedure consists of the following steps:

- The fundamental mode shapes in the horizontal directions of analysis of the building may be calculated using methods of structural dynamics or may be approximated by horizontal displacements increasing linearly along the height of the building.
- The seismic action effects shall be determined by applying, to the two planar models, horizontal forces $F_i$ to all storeys.

$$F_i = F_b \cdot \frac{s_i . m_i}{\sum s_i . m_i} \qquad (2.5)$$

$F_i$ is the horizontal force acting on storey $i$;

$F_b$ is the seismic base shear

$s_i$, $s_j$ are the displacements of masses $m_i$, $m_j$ in the fundamental mode shape;

$m_i$, $m_j$ are the storey masses computed in accordance with

- The horizontal forces $F_i$ determined in according to the above shall be distributed to the lateral load resisting system assuming the floors are rigid in their plane [56].

### 3.3 RESPONSE SPECTRUM METHOD

According to Euro code 8, 2004 [56], this type of analysis shall be applied to buildings whose response is significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction. The response of all modes of vibration contributing significantly to the global response shall be taken into account. These requirements may be deemed to be satisfied if:

- The sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure;
- All modes with effective modal masses greater than 5% of the total mass are taken into account.

Symmetrical buildings with uniform mass and stiffness distribution behave in a fairly predictable manner, whereas buildings that are asymmetrical or with areas of discontinuity or irregularity do not. For such buildings, dynamic analysis is used to determine significant response characteristics such as (1) the effect of the structure's dynamic characteristics on the vertical distribution of lateral forces; (2) the increase in dynamic loads due to Torsional motions; and (3) the influence of higher modes, resulting in an increase in story shears and deformations.

Static method specified in building codes are based on single-mode response with simple corrections for including higher mode effects. While appropriate for simple regular structures, the simplified procedures do not take into account the full range of seismic behaviour of complex structures. Therefore, dynamic analysis is the preferred method for the design of buildings with unusual or irregular geometry.

Two methods of dynamic analyses are permitted: (1) elastic response spectrum analysis and (2) elastic or inelastic time history analysis. The response spectrum analysis is the preferred method because it is easier to use. The time history procedure is used if it is important to represent inelastic response characteristics or to incorporate time dependent effects when computing the structure's dynamic response.

In this research work, equivalent static analysis is not used, because, the building to be used for this study is not regular which does not fulfil the Euro code 8, 2004 requirement to use this method. Therefore, to evaluate the elastic responses, linear dynamic analysis (response spectrum analysis) is used, and for inelastic responses, non-linear static analysis (pushover) is used.

### 3.4 PUSHOVER ANALYSIS OF BUILDINGS

Pushover analysis is a non-linear static analysis carried out under conditions of constant gravity loads and monotonically increasing horizontal loads. It may be applied to verify the structural performance of newly designed and of existing buildings for the following purposes:

- To verify or revise the over strength ratio values $\alpha_u/\alpha_1$
- To estimate the expected plastic mechanisms and the distribution of damage;
- To assess the structural performance of existing or retrofitted buildings
- As an alternative to the design based on linear-elastic analysis which uses the behaviour factor *q*.

Nonlinear static pushover analysis has become the most commonly used method to determine the nonlinear behaviour of the building structures in the recent years. In this simplified method, a capacity curve is obtained which shows the relation of base shear and roof displacement. This curve represents the behaviour of the building structure under increasing base shear forces. As the capacities of the members of the lateral force resisting system exceed their yield limits during the increasing of the base shear forces, the slope of the force-deformation curve will change, and hence the nonlinear behaviour can be represented. In the pushover analysis, the applied lateral forces to a model are increased in a regular manner depending on the initial load pattern. Member forces are calculated for each step and the stiffness of the members whose capacities are exceeded is changed according to the hinge properties in the next step of the analysis. This process ends when the structure becomes unstable.

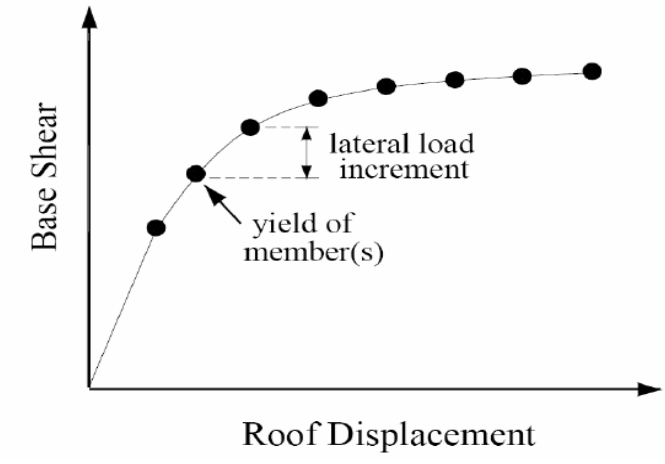



**Figure 3.1: An example pushover curve of a building structure** [26]

The pushover analysis can be performed considering the control over the force or displacement. Force control option is useful when the magnitude of the load is known clearly, and the structure is expected to support that load. The displacement control is useful when the magnitude of the load is unknown and displacements are searched.

### 3.5 Purpose of Non-linear Static Push-over Analysis

The purpose of pushover analysis is to evaluate the expected performance of structural systems by estimating its strength and deformation demands in design earthquakes by means of static inelastic analysis, and comparing these demands to available capacities at the performance levels of interest. The evaluation is based on an assessment of important performance parameters, including global drift, interstory drift, inelastic element deformations (either absolute or normalized with respect to a yield value), deformations between elements, and element connection forces (for elements and connections that cannot sustain inelastic deformations), The inelastic static pushover analysis can be viewed as a method for predicting seismic force and deformation demands, which accounts in an approximate manner for the redistribution of internal forces that no longer can be resisted within the elastic range of structural behaviour. The pushover is expected to provide information on many response characteristics that cannot be obtained from an elastic static or dynamic analysis. The following are the examples of such response characteristics: [12]

- The realistic force demands on potentially brittle elements, such as axial force demands on columns, force demands on brace connections, moment demands on beam to column connections, shear force demands in deep reinforced concrete spandrel beams, shear force demands in unreinforced masonry wall piers, etc.
- Estimates of the deformations demands for elements that have to form inelastically in order to dissipate the energy imparted to the structure.
- Consequences of the strength deterioration of individual elements on behaviour of structural system.
- Consequences of the strength deterioration of the individual elements on the behaviour of the structural system.
- Identification of the critical regions in which the deformation demands are expected to be high and that have to become the focus through detailing.
- Identification of the strength discontinuities in plan elevation that will lead to changes in the dynamic characteristics in elastic range.
- Estimates of the interstory drifts that account for strength or stiffness discontinuities and that may be used to control the damages and to evaluate P-Delta effects.

- Verification of the completeness and adequacy of load path, considering all the elements of the structural system, all the connections, the stiff non-structural elements of significant strength, and the foundation system. The last item is the most relevant one as the analytical model incorporates all elements, whether structural or non-structural, that contribute significantly to the lateral load distribution.

Load transfer through across the connections through the ductile elements can be checked with realistic forces; the effects of stiff partial-height infill walls on shear forces in columns can be evaluated; and the maximum overturning moment in walls, which is often limited by the uplift capacity of foundation elements can be estimated.

### 3.6 Lateral load patterns used in pushover analysis

According to Euro code 8, 2004, at least two vertical distributions of the lateral loads should be applied.

1. Uniform lateral load pattern

The lateral force at any story is proportional to the mass at that story, i.e.

$$F_i = \frac{m_i}{\sum m_i} \qquad (2.6)$$

Where $F_i$: lateral force at $i^{th}$ story

$m_i$: mass of $i^{th}$ story

2. modal lateral load pattern

The lateral force at any story is proportional to the product of the amplitude of the elastic first mode and the mass at that story, i.e.

$$F_i = \frac{m_i \phi_i}{\sum m_i \phi_i} \qquad (2.7)$$

Where $\phi_i$: amplitude of the elastic first mode at $i^{th}$ story

**Steps of static nonlinear/pushover analysis using ETABS software**

Nonlinear computer programs like, SAP2000 and ETABS are able to perform a pushover analysis directly. In static non-linear analysis of ETABS, several types of non- linear behavior can be considered. This includes material nonlinearity at discrete, user-defined hinges in frame/line elements, material nonlinearity in the link elements, and geometric nonlinearity in all elements and so on.

Static nonlinear analysis cases are completely independent of all other analysis types in ETABS. The distribution of load applied on a structure for a given static nonlinear case is defined as a scaled combination of one or more of the following:

- Any static load case.
- Uniform acceleration acting in any of the three global directions. The force at each joint is proportional to the mass tributary to that joint and acts in the specified direction.
- A modal load for any Eigen or Ritz mode. The force at each joint is proportional to the product of the modal displacement, the modal circular frequency squared (w2), and the mass tributary to that joint, and it acts in the direction of the modal displacement.

Depending on the physical nature of the load and the behavior expected from the structure, two different types of control are available for applying the load: force and displacement controls. The Procedure for performing static non- linear analysis by ETABS will be given in the following paragraph:

- Creation of a building model as it would be done for any analysis.
- Definition of frame hinge properties and assignment to the frame/cable elements.
- Definition of static and dynamic analysis cases that may be needed for steel or concrete design of the frame elements, particularly if default hinges are used.
- Running the analysis cases needed for design (if required).
- If any concrete hinge properties are based on default values to be computed by the program, concrete design will be performed so that reinforcing steel is determined.
- Definition of the load cases that are needed for use in the push over analysis, including gravity loads and other loads that may act on the structure before the lateral seismic loads are applied.
- Definition of lateral loads that will be used to push the structure.
- Definition of the nonlinear static analysis cases to be used for pushover analysis, including:
  - ✓ A sequence of one or more cases that start from zero and apply gravity and other fixed loads using load control. These cases can include geometric nonlinearity.
  - ✓ One or more push over cases that start from this sequence and apply lateral pushover loads. These loads should be applied under displacement control. The monitored displacement is usually at the top of the structure and will be used to plot the push over curve.
- Running of the pushover analysis cases.
- Revision of the pushover results. Plot of the push over curve, the deflected shape showing the hinge states, force and moment plots, and print or display any other results needed.
- Revision of the model if necessary.

Several types of outputs can be obtained from the static nonlinear analysis of ETABS software. These include: Plot of Base Reaction versus Monitored Displacement, tabulated values of Base

Reaction versus Monitored Displacement at each point along the pushover curve, Plot of Base Reaction versus Monitored Displacement in the ADRS format, tabulated values of the capacity spectrum (ADRS capacity and demand curves), Graphical view of the sequence of hinge formation and the color-coded state of hinge, and graphical view of the member forces and stresses on a step by step basis.

### 3.7 Nonlinear behaviour of structural elements

The nonlinear behaviour of a building structure depends on the nonlinear responses of the elements that are used in the lateral force resisting system. Therefore, before applying any nonlinear analysis method on a building structure, the nonlinear behaviour of such elements must be clearly described and evaluated.

ATC-40 and FEMA-356 codes define the acceptance criteria depending on the plastic hinge rotations by considering various performance levels. In Figure 3.2, the five points (A, B, C, D and E) which are used to define the hinge rotation behaviour of RC members and the acceptance criteria on a force versus deformation diagram are given. In this diagram, points marked as IO, LS and CP represent immediate occupancy, life safety and collapse prevention, respectively [54].

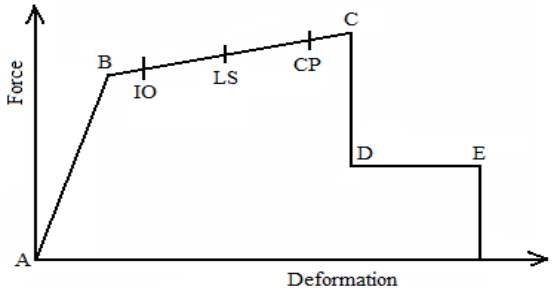



**Figure 3.2: Acceptance criteria on a force versus deformation diagram** [54].

The load-deformation relation is defined by linear response (or elastic response) until point B. At point B, the member yields and again a linear response is observed with a reduced stiffness between the points B and C. At point C, a sudden reduction in the load resistance of the element occurs and the graph drops to point D. The residual resistance is observed until point E, where the final loss of resistance takes place.

The initial slope of this diagram between points A and B defines the elastic stiffness of the structure. In the analyses carried out in this study, the second slope between points B and C is taken as 10% of the initial slope. Point C in this diagram represents the ultimate strength of the element where the significant stiffness degradation begins.

The above-mentioned nonlinear response of the structural member is called hinge property which is defined symmetrically in order to include the reversals to the calculations. For the modelling of

nonlinear response of an element, ATC-40 [2] and FEMA-356 [21] express the parameters A, B and C in Figure 3.2 by defining plastic rotation angles.

### 3.8 ANALYTICAL MODELS

In order to evaluate seismic response of the building structures, special concrete moment resisting 3D frames with mass and stiffness irregularity are analyzed. The considered buildings are 6 and 10 story buildings. The buildings are symmetrical in plan, so torsion is not to be considered, all floors are assumed to be rigid. Mass irregularities are introduced at different location along the height of the building (bottom, middle and top). Also mass ratio of 200%, 300% and 400% are used to increase mass of irregular floor. The building has identification numbers selected as S6-M2-LB, S6-M2-LM, S6-M2-LT, S6-M3-LB, S6-M3-LM, and S6-M3-LT for 6 stories building with mass irregularity, S6-H1.5 and S6-H2 for building with stiffness irregularities. For 10 storey building, the identification number of buildings is S10-M2-LB, S10-M2-LM, S10-M2-LT, S10-M3-LB, S10-M3-LM and S10-M3-LT for building with mass irregularities and S10-H1.5 and S10-H2 for building with stiffness irregularities. For example, S6-M2-LB stands for 6 storey building with mass of bottom floor is 2 times other floors and S6-H1.5 stands for 6 storey building with height of ground floor is 1.5 times other floors. S10-M2-LB stands for 10 storey building with mass of bottom floor is 2 times other floors and S10-H1.5 stands for 10 storey building with height of ground 1.5 times other floors. All of the building structures are analyzed according to Euro code 8, 2004. ETABS software is used for the analysis. The building is to be constructed at Addis Ababa.

Table 3.1 the material and parameter used in this analysis

| | | |
|---|---|---|
| Peak ground acceleration $a_g$ | 0.1g | |
| Concrete | C-30 | |
| steel | S-500 | |
| Modulus of elasticity of concrete | 33Gpa | |
| Modulus of elasticity of steel | 200Gpa | |
| Importance factor | 1 | |
| Earthquake zone | Zone 2 | |
| Damping ratio | 5% | |
| Soil class | C | |
| Building type | Special moment resisting RC frame | |
| Number of storey of building | Beam dimension(mm) | Column dimension(mm) |
| 6 | 250x450 | 400x400 |
| 10 | 400x500 | 600x600 |

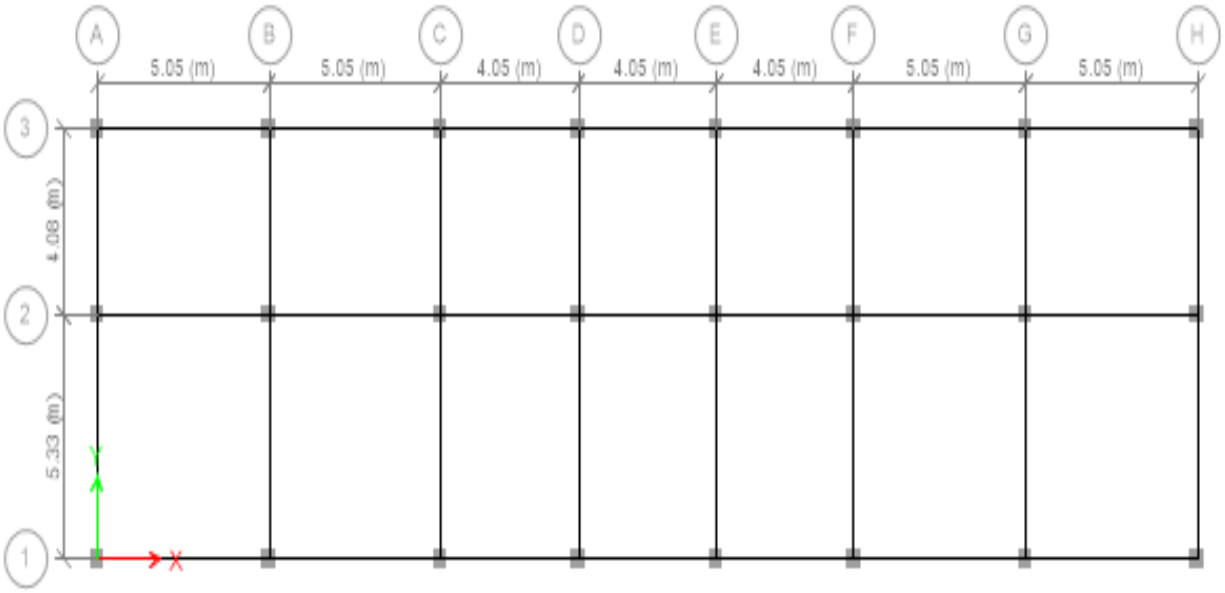



**Figure3.3: Plan of 6 storey building**

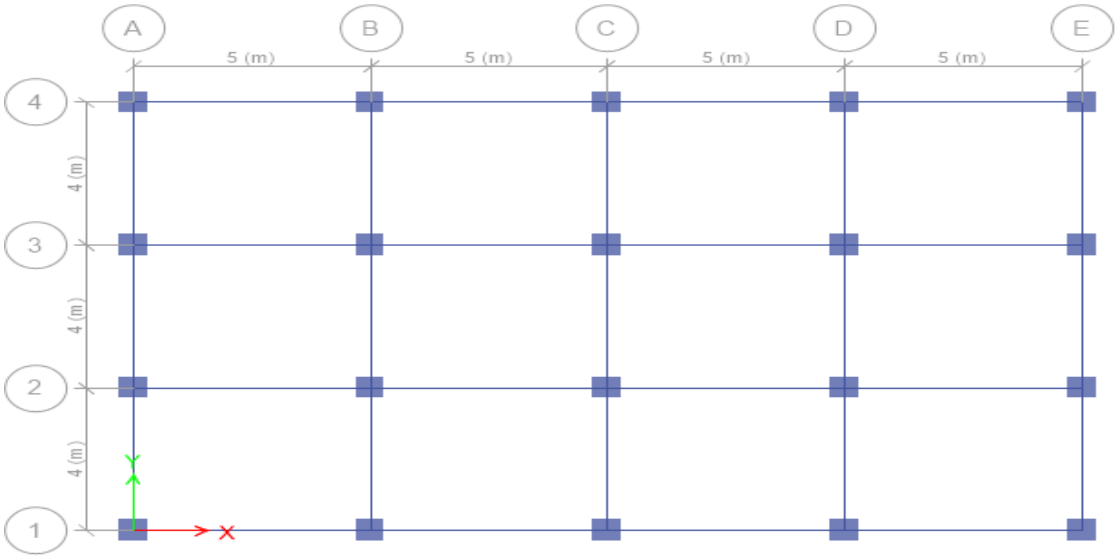



**Figure3.4: plan of 10 storey building**

# CHAPTER FOUR

## ANALYSIS AND DISCUSSION OF RESULTS

### 4.1 General

The response of reinforced concrete building with mass and stiffness irregularities have been evaluated. In order to study the effect of mass and stiffness irregularity on seismic demand of the buildings, soft story is introduced at ground floor by increasing the height of ground floor 150% and 200% of other floors. Mass irregularity is introduced at bottom, middle and top of the building by increasing mass of irregular floor 200%, 300% and 400% of other floors.

The effects of stiffness and mass irregularities on elastic seismic demands (story lateral displacement, story drift and story shear) are evaluated by response spectrum analysis. Effects of stiffness and mass irregularities on inelastic seismic demands are evaluated by static non-linear (pushover analysis).

### 4.2 Effect of mass and stiffness irregularity on fundamental period

The objective of this section is to evaluate the effects of irregularities on the fundamental natural period of buildings. In this research work, the variation of the fundamental period is evaluated for mass modification factor of 2, 3, and 4, and stiffness modification factor of 1.5, and 2. To define the irregularity considered, parameter $L_H$ represents the level at which irregularity starts for mass irregularity case. For example, $L_H$=0.5 represents a structure with an irregularity which starts at mid-height of the structure.

Varying the mass by a factor of 2 and 3 does not have significant effect on fundamental period, except when the mass modification factor is 4 which is 37.2% high for 6 story irregular building and 32.39% for 10 story irregular building compared to the base case. Although the effect is small, there is a pattern in relationship between the level of the irregularity $L_H$ and the change in fundamental period. As shown in figure 4.1 the change in fundamental period increases as the floor that experiences mass irregularity moves towards top (increase in $L_H$). On the other hand, modifying stiffness of ground story by a factor of 1.5 and 2 shows significant change in fundamental period of the buildings when compared to mass irregularity. The increase in percentage is 45.2% for 6 story building and 41.51% for 10 story buildings. Generally the effect of stiffness irregularity on fundamental period of building is larger than effect of mass irregularities.

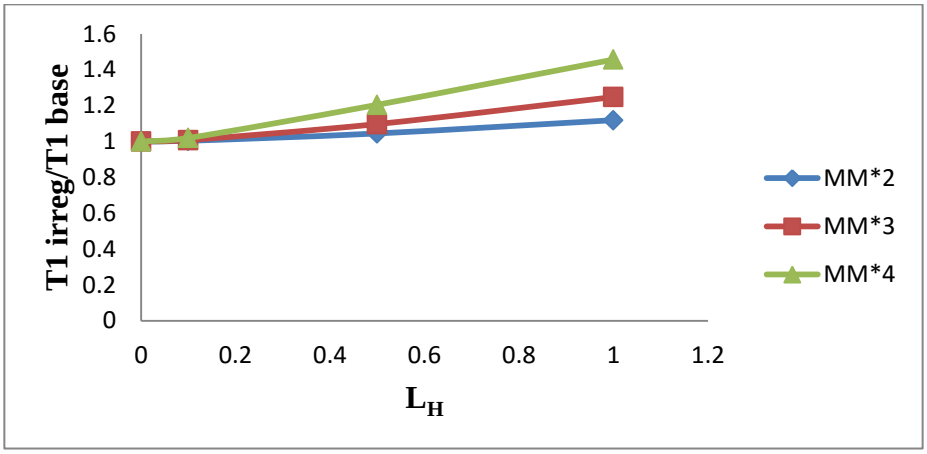



**Figure 4.1: Change in fundamental period case with mass irregularity for 6 story building**

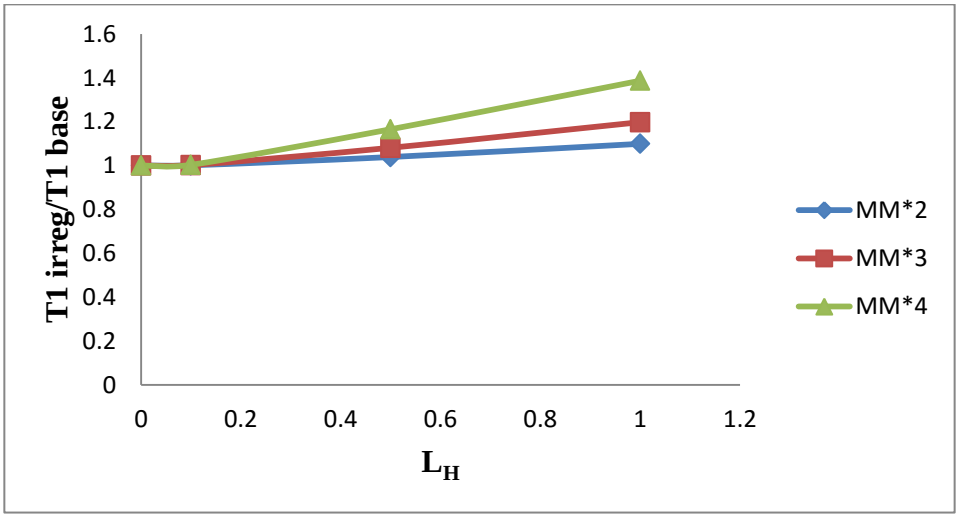



**Figure 4.2: Change in fundamental period case with mass irregularity for 10 story building**

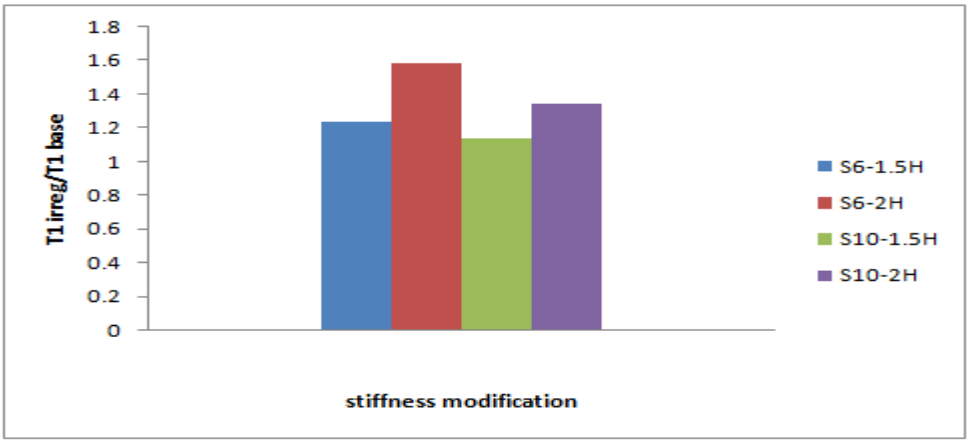



**Figure 4.3 Change in fundamental period case with stiffness irregularity**

### 4.3 Effect of mass irregularity on elastic seismic demands

The effect of mass irregularity on storey shear, storey drift and storey lateral displacement and their distribution over height is evaluated. Response spectrum was performed on base case and case with irregularities to evaluate elastic seismic demands. Three cases with mass irregularities are studied, with mass modification factor of 2, 3, and 4 at different location along the height of buildings (bottom, middle and top). The figures 4.4 and 4.5 presents the base shear demands for the cases with irregularities normalized by the base shear demand for the base case, which is plotted against variation in $L_H$. Since the fundamental period for the cases studied are not the same, the variations of elastic demands presented reflects the effect of the change in fundamental period due to irregularity. The total mass of each case with mass irregularity studied does not equal to the total mass of the corresponding base case. Therefore an increase in mass increases the base shear value, though the increase is not more than 15% for both 6 and 10 story buildings.

Figures 4.4and 4.5 shows the variation of elastic base shears with the level of mass irregularity for the cases with mass modification of 2,3 and 4. For the cases studied here, mass and fundamental period are the two primary parameters that control change in base shear. As can be seen in figures, the ratio of base shears ( irregular case to regular) is greater than one in all cases except when the mass irregularity with mass modification of 2 is at bottom ($L_H$=0.1). This implies that the effect of increasing the mass is larger than the effect of change in fundamental period. However, as the irregularity moves to higher floors (increasing $L_H$), the effect of the change in fundamental period increases until it overrides the effect of mass increase, because, the fundamental period of structure increases as the level of irregularity increases.

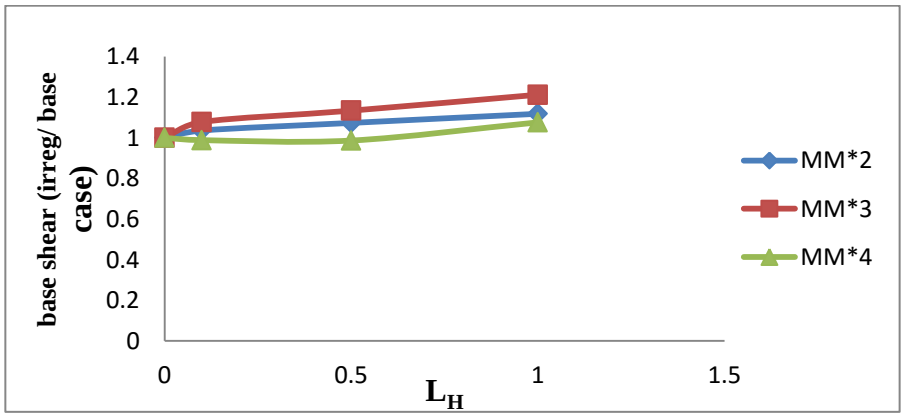



**Figure 4.4: Variation of elastic base shear with location of mass irregularity for 6 story building**

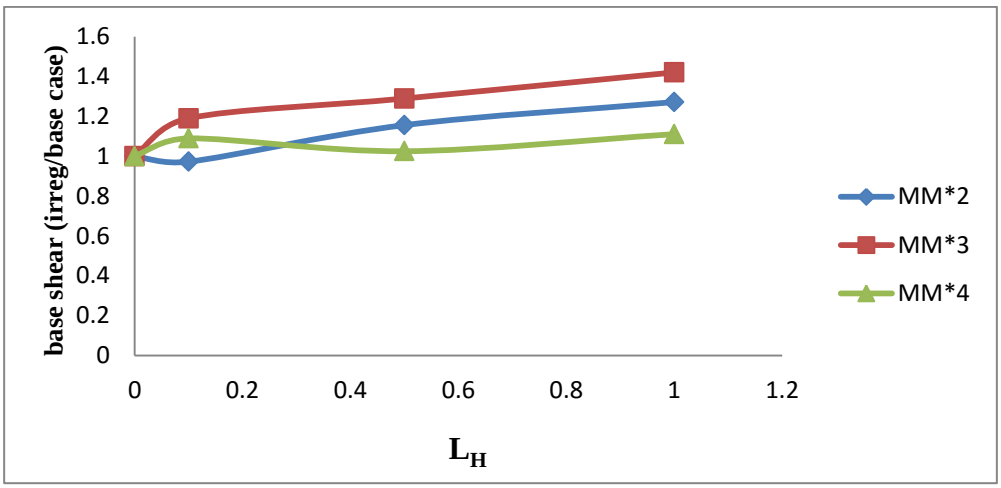



**Figure 4.5: Variation of elastic base shear with location of mass irregularity for 10 story building**

Figures 4.6 and 4.7 present story shear demands for cases with mass irregularities and base case. Storey shear demands are normalized by the total weight of each case in order to provide the bases for comparing the results for cases with different mass irregularities. As shown in figures, increasing mass by 2 and 3 does not change significantly the base shear coefficient between the base case and the cases with irregular mass (less than 15%) for both 6 and 10 stories buildings. However, mass modification factor of 4 shows significant change, more than 50%. Even though, there is no significant change for mass modification of 2 and 3, the distribution of the elastic storey shear demand over height changes considerably. Story shear demands increase at the stories below the floors with large masses. For example, the mass irregularity in case of mass modification 4 results in a storey shear distribution that if different from the base case, with highest difference being at transition level (stories 3 and 4). For all cases the base shear coefficient is minimum at top stories and increases down to lower stories.

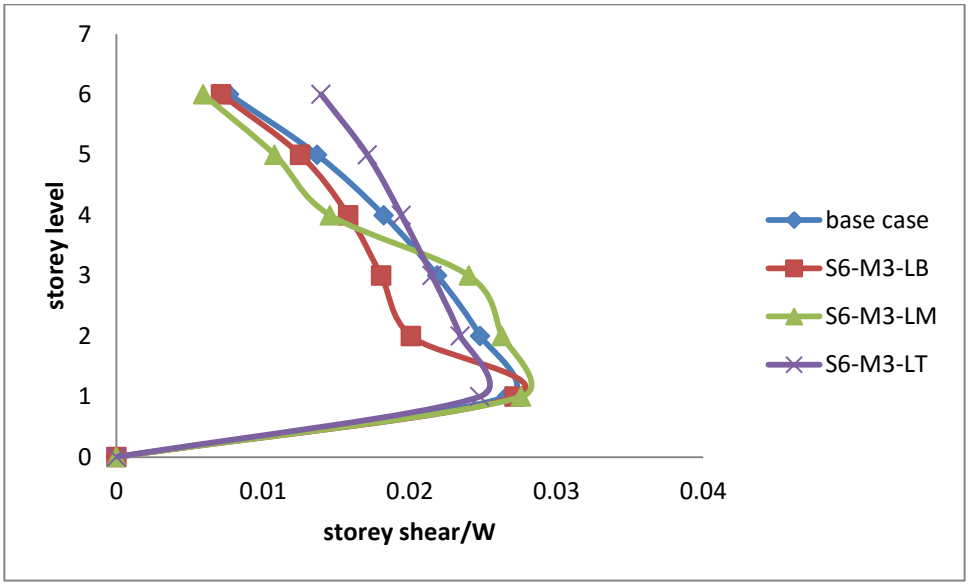



**Figure 4.6: Storey shear distribution over height for 6 story buildings case with mass irregularity**

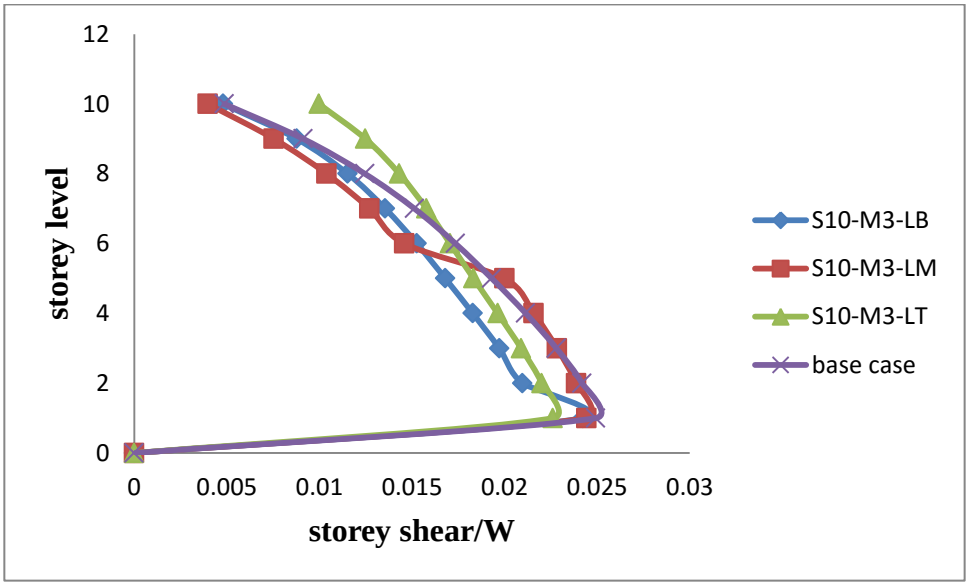



**Figure 4.7: Storey shear distribution over height for 10 storey buildings case with mass irregularity**

Figures 4.8 and 4.9 shows that elastic storey drift demands distribution over height for the base case and cases with selected irregularities. The selected irregular buildings shown on figure are buildings with irregular mass at top floor with mass modification of 2, 3 and 4 for comparison purpose. As can be seen in figures, there is a variation in storey drift demand between irregular and base case which is not this much significant, 17.39%. The distribution over height for both cases has similar shape. Though the difference in storey drift demand is not significant, maximum storey drift value occurred at floor with irregular mass and adjacent floors. Generally, the elastic storey drift demand of the

entire floors was lower than allowable drift according to Euro code limit (drv≤0,005h) for both 6 and 10 stories buildings.

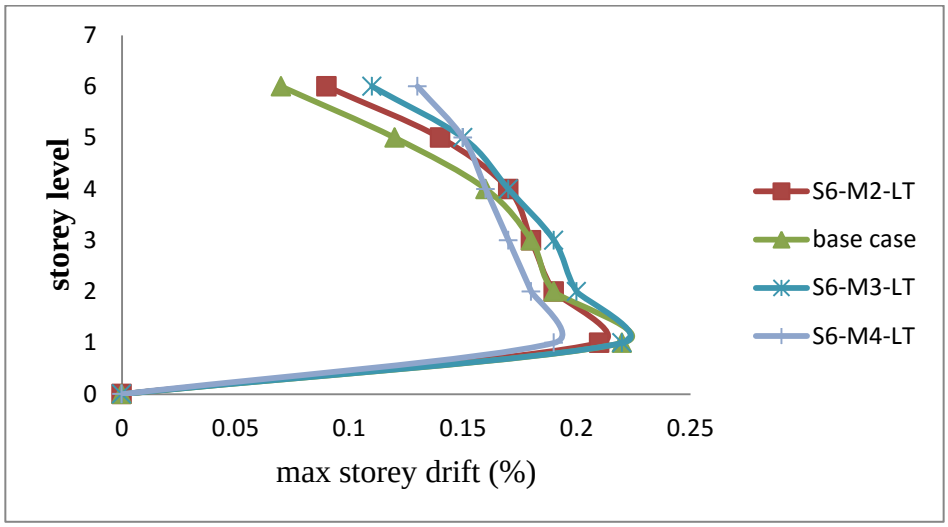



**Figure 4.8: Storey drift demand distribution over height for 6 story buildings case with mass irregularity**

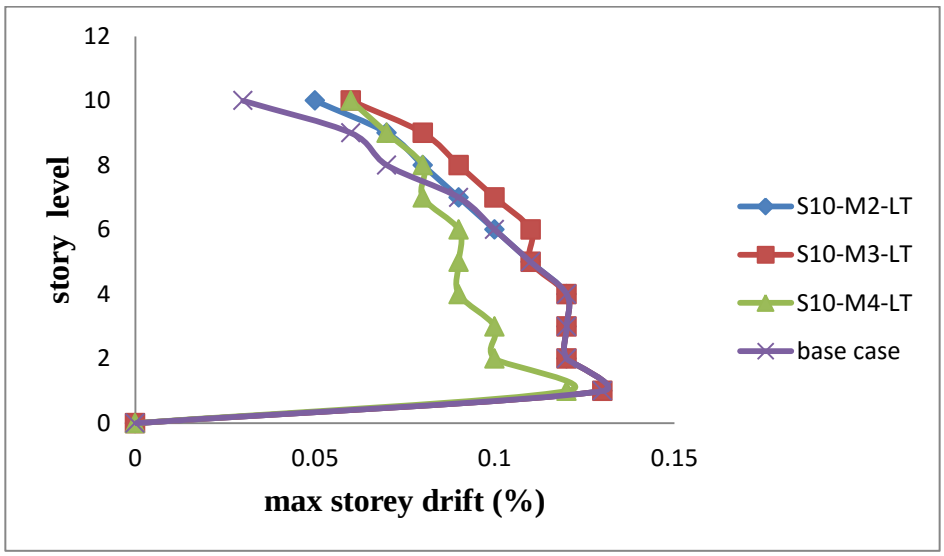



**Figure 4.9: Storey drift demand distribution over height for 10 story buildings case with mass irregularity**

Figures 4.10, 4.11, 4.12 and 4.13 shows the variation of ratio of lateral roof displacement with mass modification and location of irregular mass. Ratio of lateral roof displacement is calculated by roof displacement of irregular case divided by base case. As presented in figure, mass modification as well as location of mass affects roof displacement ratio. The ratio is maximum when the irregular floor mass is at the middle height of the structure for both 6 and 10 stories buildings. The ratio is greater than one for irregular mass at middle and top, and less than one when irregular mass is at the bottom, this show effect of location of floor with irregular mass on lateral displacement of the structure.

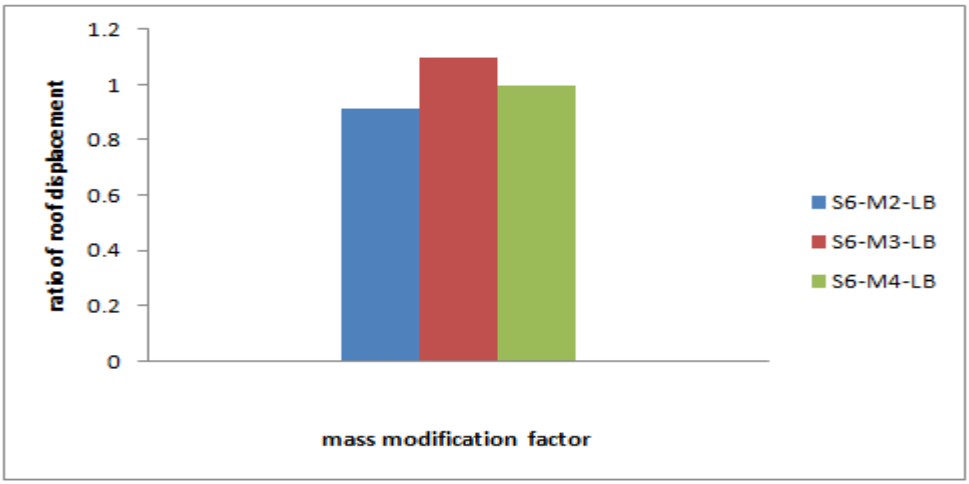



**Figure 4.10: Variation of roof displacement ratio with mass modification for 6 story building**

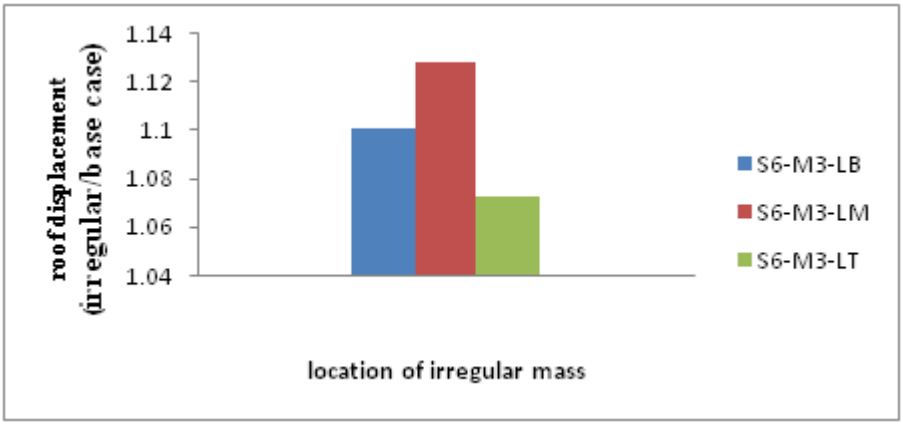



**Figure 4.11: Variation of roof displacement ratio with location of mass irregularity for 6 story building**

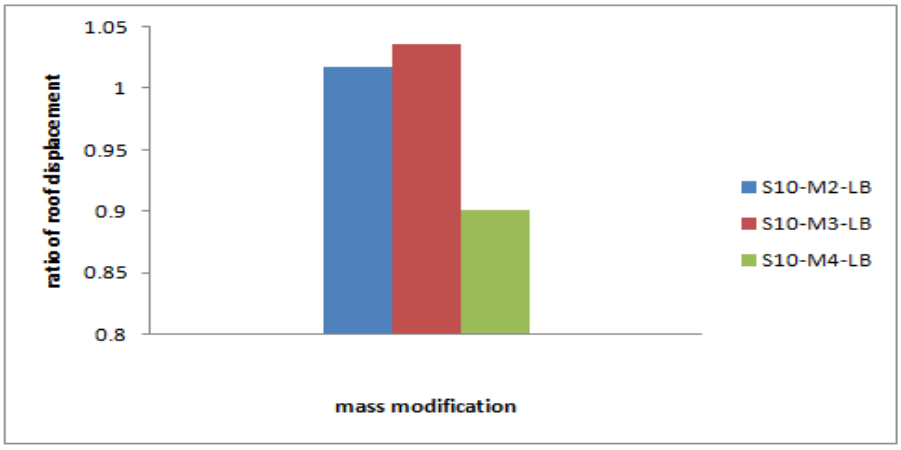



**Figure 4.12: Variation of roof displacement ratio with mass modification for 10 story building**

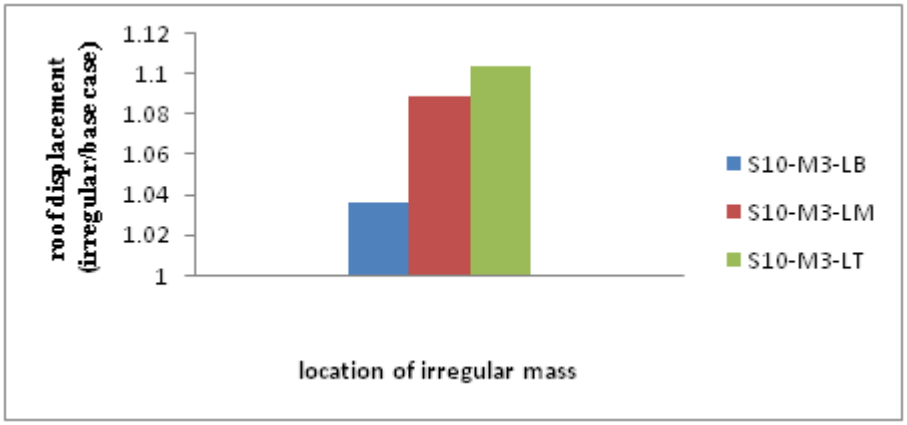



**Figure 4.13: Variation of roof displacement ratio with location of irregular mass for 10 story building**

Figures 4.14 and 4.15 shows distribution of storey lateral displacement over height for buildings with mass modification factor of 3 and the base case buildings. The storey displacement is normalized by the height of the building. The distribution of storey lateral displacement over height exhibits similar profile for all buildings. Storey lateral displacement increases with increase in mass ratio even though the effect is not significant this much (not more than 12.5% for 6 story buildings and 9.2% for 10 story buildings). Generally the effect of mass irregularity on the storey lateral displacement is significant on 6 story buildings when compared to the 10 story buildings.

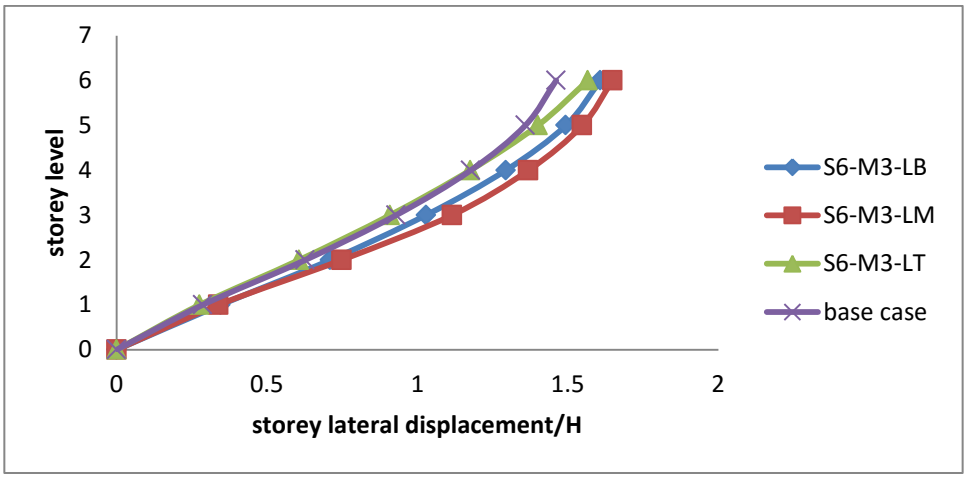



**Figure 4.14: Lateral displacement distribution over height for 6 story buildings**

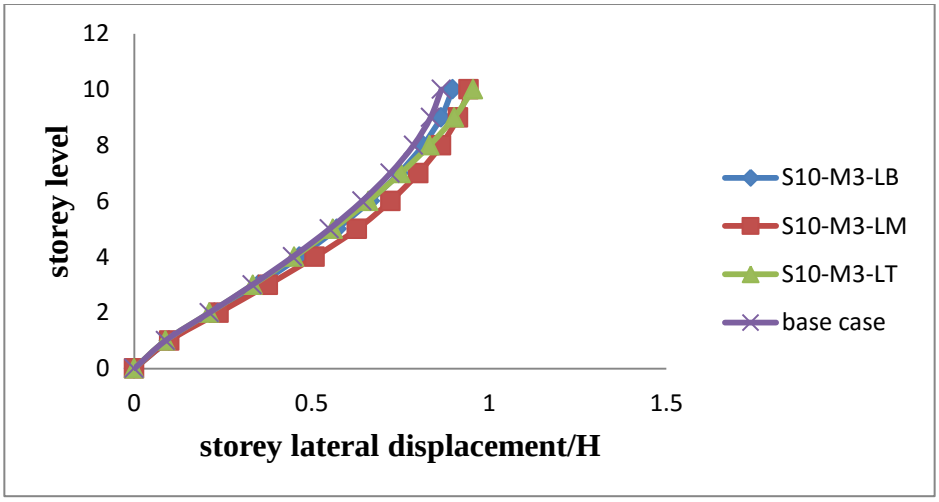



**Figure 4.15: Lateral displacement distribution over height for 10 story buildings**

### 4.4 Effect of stiffness irregularity on elastic seismic demands

In this section, the effect of stiffness irregularity on elastic demands are studied considering two modification factor of stiffness, the height of ground floor is increased by 150% and 200% of other floors. The figure 4.16 presents the base shear demands for the cases with stiffness irregularities normalized by the base shear demand for the base case, which is plotted against stiffness modification factor. Introducing stiffness irregularity with stiffness modification factor of 150% decreases the base shear value by 12.18% from base case which is not this much significant when compared to stiffness modification factor of 200% which decreases the base shear value by 28.61% for 10 story building. For 6 story buildings, the change in base shear value is significant when compared to 10 story building; the decrease in base shear for stiffness modification of 150% is 20.73% and 43.13% for stiffness modification of 200%.

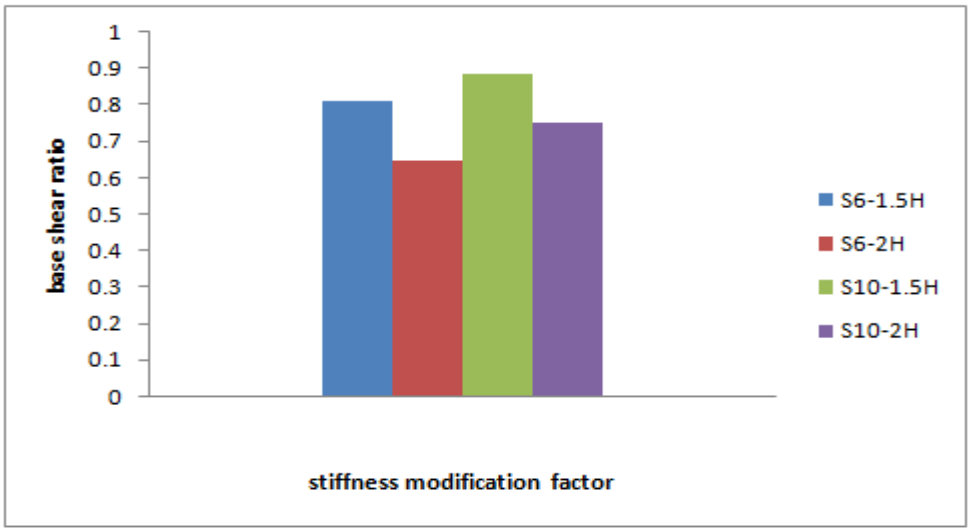



**Figure 4.16: Variation of base shear with stiffness modification factor**

Figure 4.17 and 4:18 shows normalized story shear demands for irregular cases in which ground floor height is increased by 1.5 and 2 of other floors. The shape of story shear distribution does not vary much from the base case for the cases studied. However, the base shear coefficient decreases when compared to the base case by 25.36% for 10 story building and also the shear demands at top floor decreases, which is the result of the first mode dominating the response. There is an increase in shear demand at lower stories due to the increase in the contributions of higher modes for irregular cases.

By comparing the story shear demands, it is concluded that normalized story shears are more sensitive to mass irregularities than to stiffness irregularities.

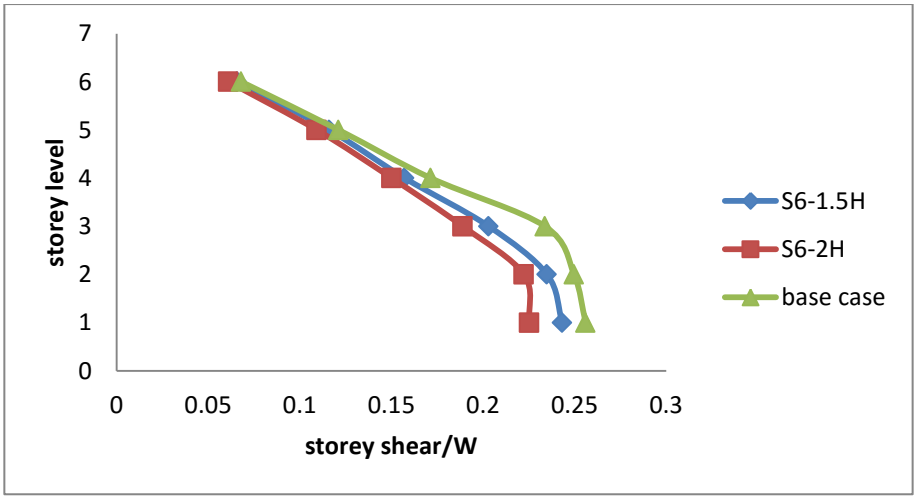



Figure 4:17 Distribution of story shear over height case with stiffness irregularity for 6 story buildings

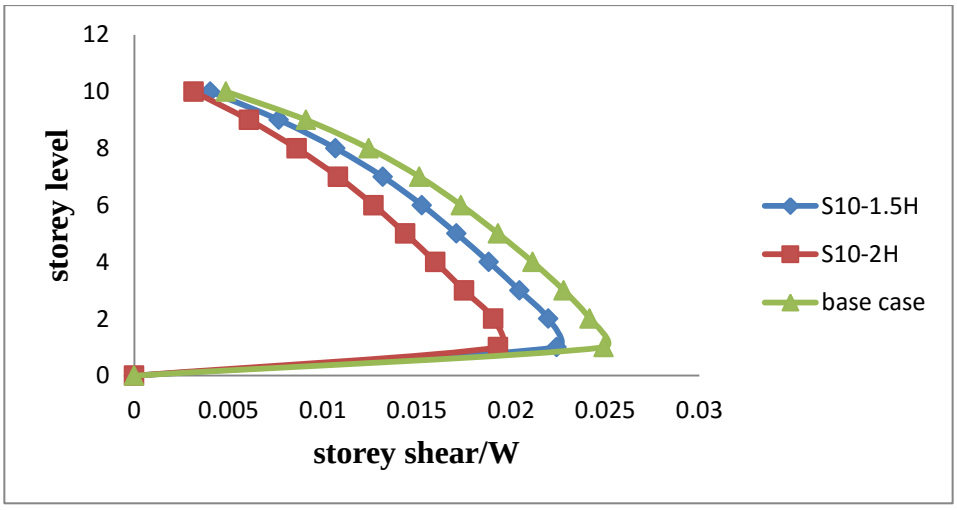



**Figure 4.18: Distribution of story shear over height case with stiffness irregularity for 10 story buildings**

The distributions of story drift demands over height for the cases with stiffness irregularities are very different from the base case. Figures 4.19 and 4.20 shows the values of maximum story drifts in percent for the case studied. The presence of a soft story drastically increases the drifts in the soft story and decreases the drifts in the other stories. The abrupt change in the slope of the profile indicates the stiffness irregularity. All drift profiles corresponding to models having stiffness irregularity have a sudden change of slope at ground floor level; however, the regular building shows smooth displacement profiles. The buildings with excess height at ground storey are weaker than the regular building, because lateral stiffness at soft story is quite less than the regular building. The maximum increase in drift for floor with stiffness irregularity studied in this case is 63.16% for stiffness modification of 200%. Though there is an increase in drift demand, drift of all floors are less than the allowable drifts according to Euro code limit ($drv \leq 0{,}005h$). Generally, storey drift demands are more sensitive to stiffness irregularity than mass irregularities.

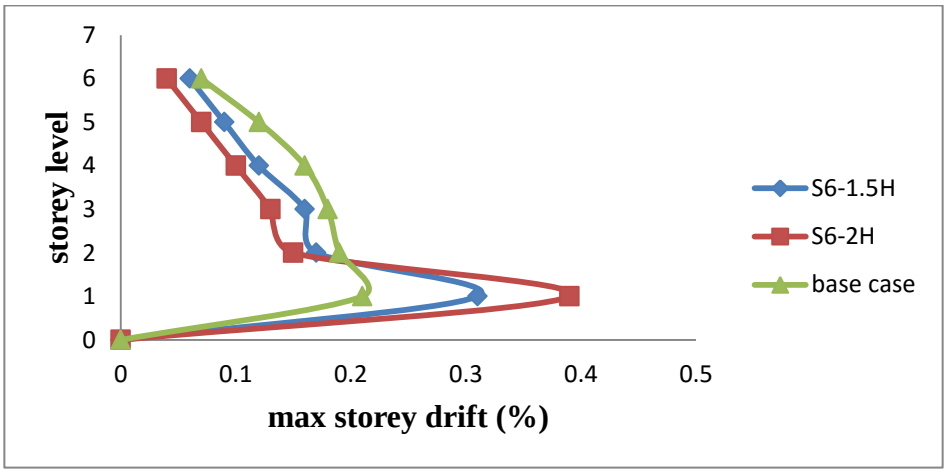



**Figure 4.19: Story drifts demand distribution over height case with stiffness irregularity**

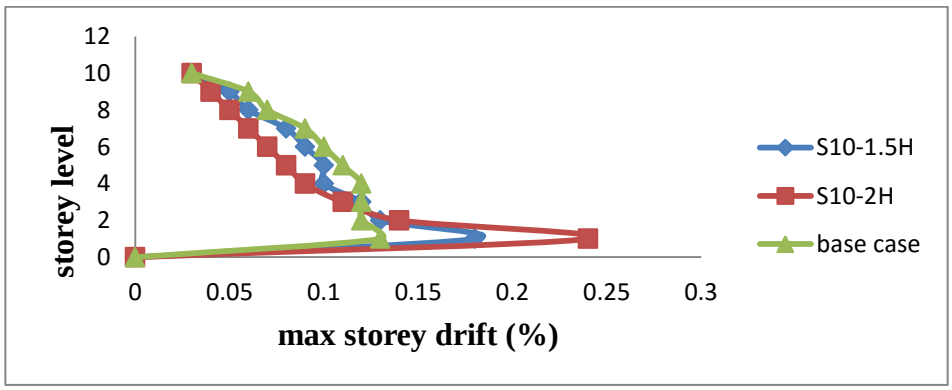



**Figure 4.20: Story drifts demand distribution over height case with stiffness irregularity**

Figure 4.21 shows the variation of ratio of lateral roof displacement with stiffness modification. Ratio of lateral roof displacement is calculated by roof displacement of irregular case divided by base case. As presented in figure, stiffness modification affects the value of roof displacement ratio. This ratio increases with increase in stiffness modification, which is significant for 6 story building.

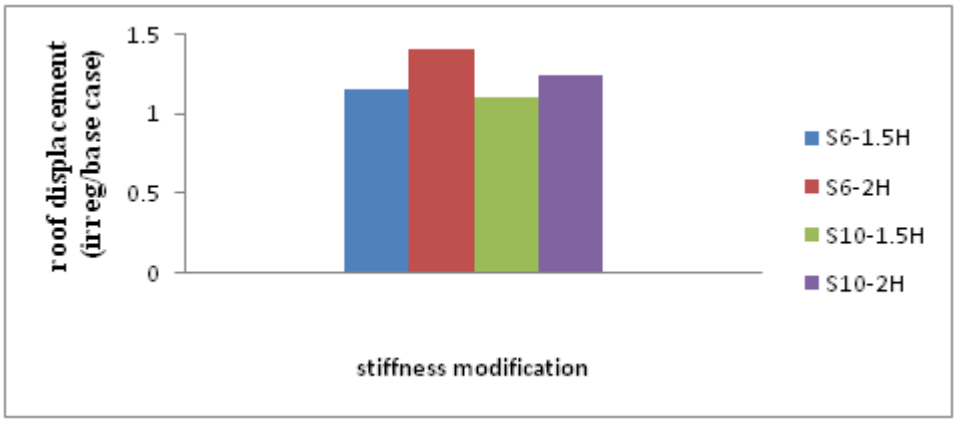



**Figure 4.21: Variation of ratio of roof displacement with stiffness modification**

Figures 4.22 and 4.23 shows distribution of story lateral displacement over height for buildings with stiffness modification factor of 1.5 and 2 and the base case building. The story displacement is normalized by the height of the building. The distribution of story lateral displacement over height exhibits similar profile for all buildings. Story lateral displacement increases with increase in stiffness ratio even though the effect is not significant this much. Generally, effect of stiffness irregularity on story lateral displacement is significant for 6 story building when compared to 10 story building.

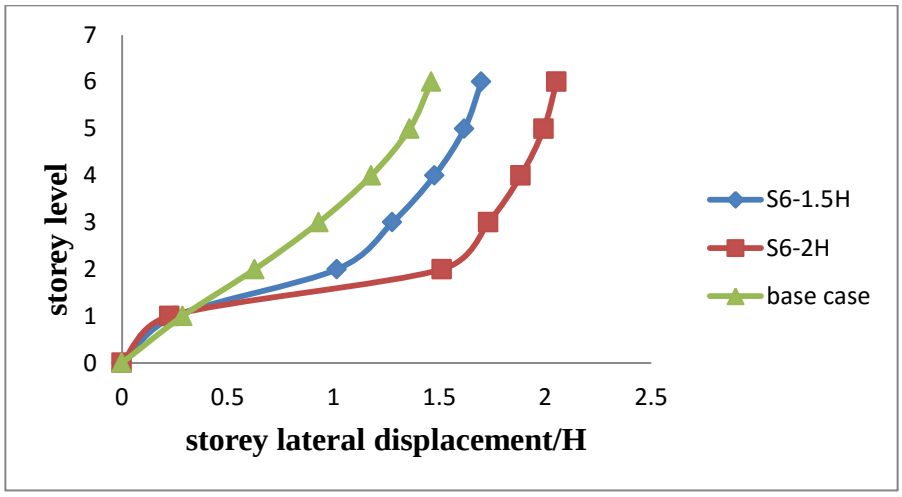



**Figure 4.22: lateral displacement distribution over height case with stiffness irregularity for 6 story building**

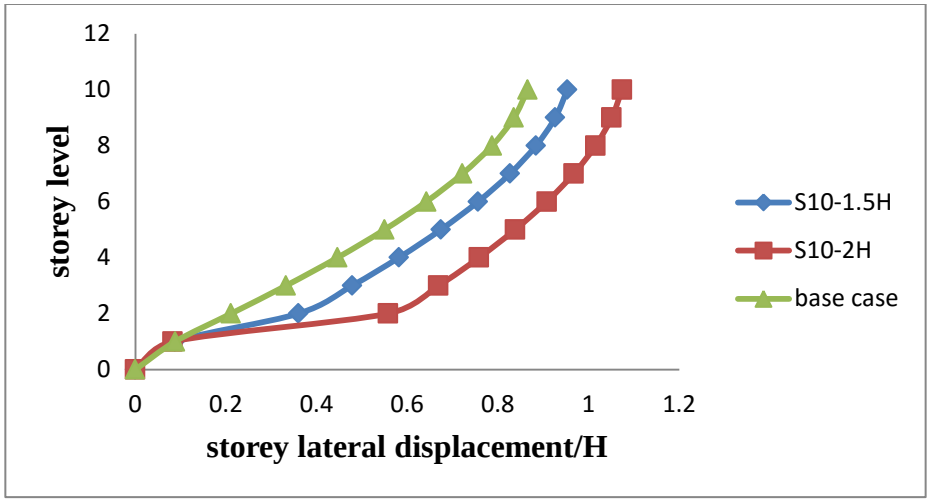



**Figure 4.23: lateral displacement distribution over height case with stiffness irregularity for 10 story building**

**Summary on results of elastic demands**

Varying mass of the buildings by a factor of 2 and 3 changes fundamental period of building (less than 13%), except for mass modification factor of 4 which is 37.2% for 6 story irregular building and 32.39% for 10 story irregular building compared to the base case, but, modifying stiffness of ground story by a factor of 1.5 and 2 shows significant change in fundamental period of the buildings when compared to mass irregularity. The increase in percentage is 45.2% for 6 story building and 41.51% for 10 story buildings. Generally the effect of stiffness irregularity on fundamental period of building is larger than effect of mass irregularities.

Base shear is not increased more than 15% for increase in mass by 200%, 300% and 400%, but, for the case of stiffness irregularity, base shear decreases by 28.16% for 10 story building and 43.13% for 6 story building.

Storey shear demand distribution shows the same profile for both irregular and regular cases for both mass and stiffness irregularities. Variation in storey drift demand between irregular and base case is 17.39% for mass irregularity and 63.16% for stiffness irregularity. The distribution over height for both cases has similar shape. Generally, the elastic storey drift demand of the entire floors was lower than allowable drift according to Euro code limit (drv≤0,005h) for both 6 and 10 stories buildings.

Storey lateral displacement increases only by12.5% for case of mass irregularity and 18.15% for the case of stiffness irregularity, which shows both irregularities doesn’t have significant effect on storey lateral displacement when compared to drift demand.

### 4.5 Effect of mass irregularity on inelastic global displacement

Roof displacement is a convenient measure of the global drift of the structure; it is also a quantity needed to perform a demand assessment by means of nonlinear static (pushover) analysis. The parameter defined here $\alpha_{irreg}$ is used to represent the roof displacement of irregular structure. This parameter relates the maximum roof displacement of an inelastic irregular structure to the maximum roof displacement of an inelastic base case.

Figures 4.24 and 4.25 presents the variation of parameter $\alpha_{irreg}$ for the cases with mass irregularity. As shown on figures, mass irregularities do not cause significant changes in global drifts. The values of $\alpha_{irreg}$ for the cases studied are between 0.99 and 1.09. Though there is no significant change on roof displacement, the ratio of maximum roof displacement of inelastic irregular structure to maximum roof displacement of inelastic base shows changes with mass modification and irregular mass location. As can be seen in figures, $\alpha_{irreg}$ increases with increase in mass modification and location of irregular mass. The value of $\alpha_{irreg}$ is maximum when the irregular mass is at top floor and small at when irregular mass is at lower stories. The conclusion that can be drawn here is that for the types of mass irregularities discussed here, the effect of the irregularities on the global displacement demand is relatively small.

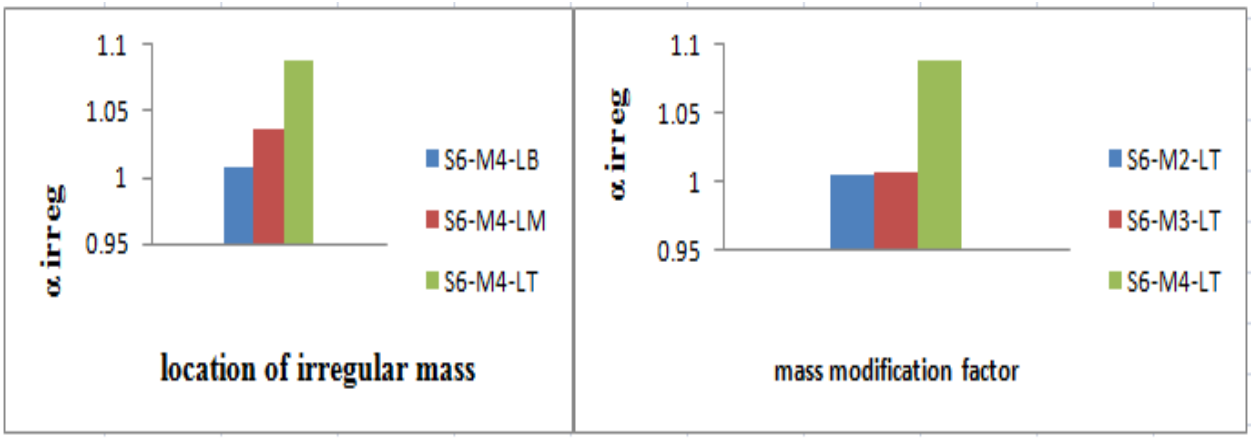



**Figure 4.24: Variation of α irreg with location of irregular mass and mass modification**

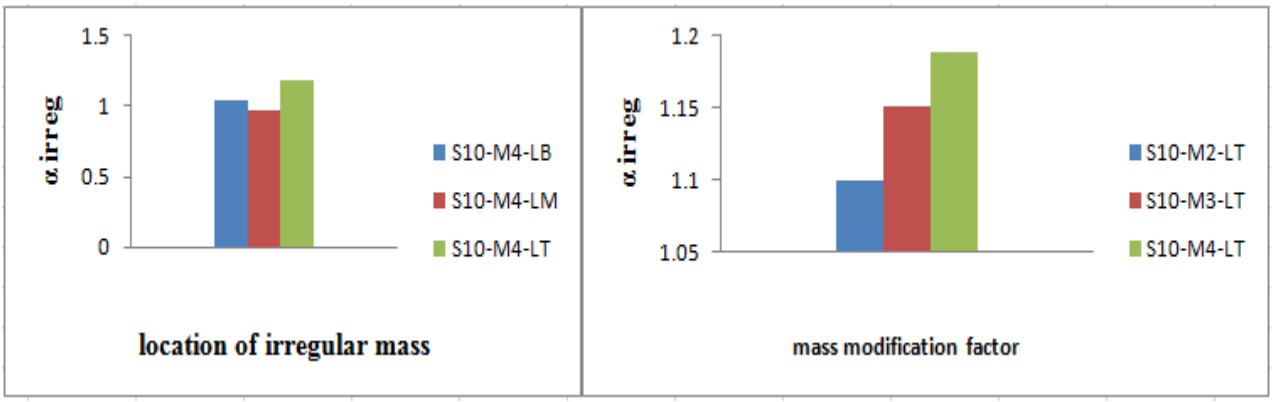



**Figure 4.25: Variation of α irreg with location of irregular mass and mass modification**

### 4.7 Effect mass irregularities on story ductility demand

Ductility is defined as maximum story displacement over story yield displacement. The height wise variation in ductility demands on multistory buildings depends on the relative yield strength of the beams and columns and the relative yield strength of various stories. The yield displacement is obtained from a pushover analysis under triangular lateral load pattern.

The figure 4.26 shows distribution of ductility demands over height for mass irregularity cases with mass modification of four. As can be presented in figure, the distribution is non-uniform, because, there is a variation of shear strength in each story. For all cases with mass irregularity studied here, the effect of irregularity on story ductility demand is small except when irregular floor mass is at top. The change in ductility demand is maximum for 10 story buildings when compared to 6 story buildings, which indicates that, the effect of increase in number of stories on ductility demand. The ductility demand for irregular buildings is 35.4% higher than for base case for 10 story building. Though mass irregularity studied here does not have much significant change on ductility demand, the value of ductility demand is maximum at floor with irregular mass which shows increase in ductility demand with mass modification increase.

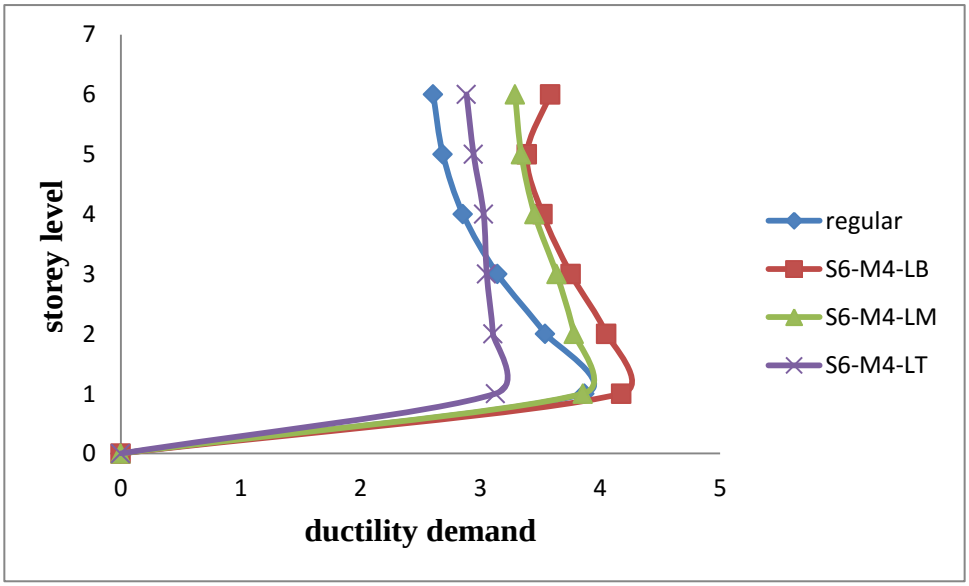



Fig : 4.26 Distribution of ductility demand over height for 6 story building

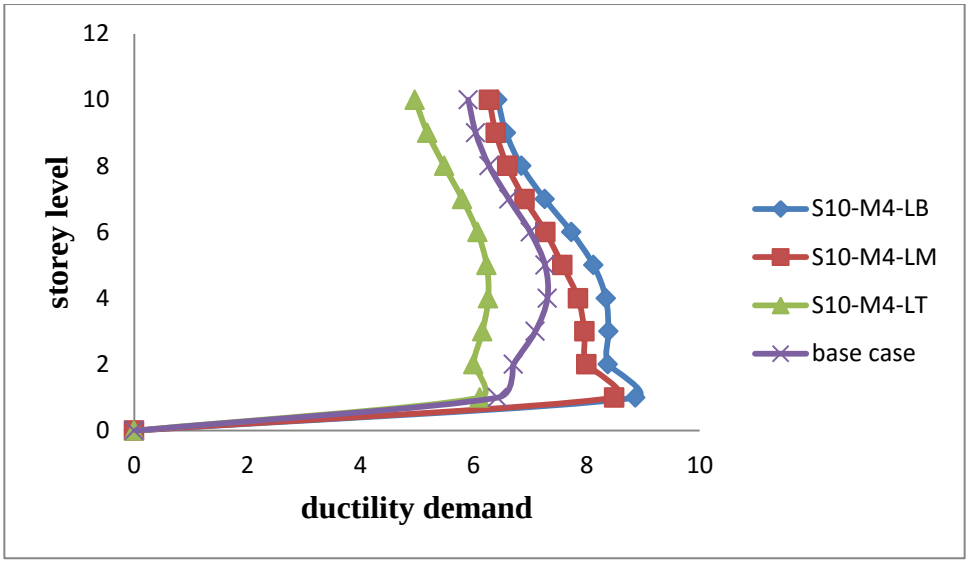



**Figure 4.26: Distribution of ductility demand over height for 10 story buildings**

### 4.8 Effect of stiffness irregularity on inelastic global displacement

Effect of stiffness irregularity on inelastic global displacement is studied by considering stiffness modification factor of 150% and 200% for both six and ten story buildings.

Figure4.25 present values of the parameter $\alpha_{irreg}$ plotted against stiffness modification factor. For the cases studied, stiffness irregularities do not change roof displacements when compared to base case for 10 story buildings, less than 10%, but for 6 story building, the change in value of $\alpha_{irreg}$ is significant when compared to 10 story buildings with 27.87% difference from base case.

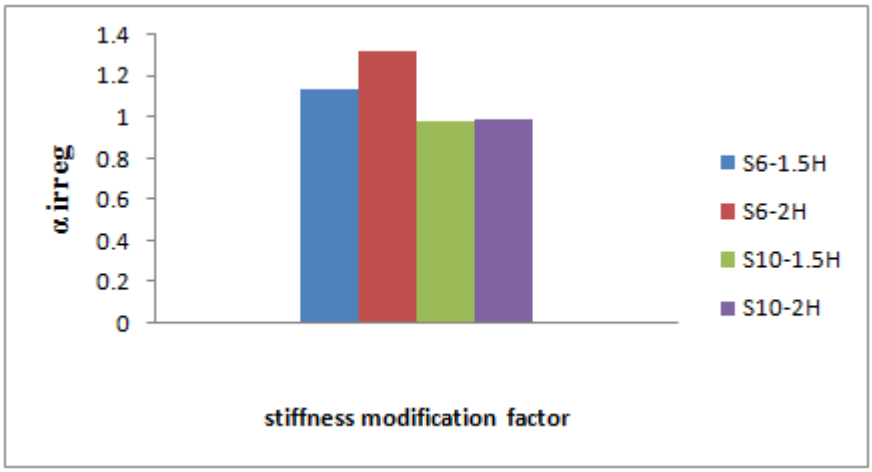



**Figure 4.27: Variation of α irreg with stiffness modification for 6 and 10 story buildings**

### 4.10 Effect of stiffness irregularities on storey ductility demand

For case with stiffness irregularities, change in stiffness of a story result in changes in storey yield drift. Since ductility represents story drift normalized by yield drift, this change in story yield drift will be reflected in the ductility distribution over height. In general, the values of ductility demands are reduced in soft story. This situation happens despite the increase in story drift at soft storey. The reason is that reducing the stiffness of soft story results in increasing the yield drift, which results in reduced value for storey ductility demands. The presence of soft story increases the value of ductility demands in most other stories. The change in ductility demand increases with increase in height of the building. For example the difference in ductility demand at ground floor for base case and irregular case is 52% for 10 story building and 22.1% for 6 story building. Generally, for building with stiffness irregularity, drift demands are amplified at stories with decreased stiffness, but because of the change in story yield values, ductility demands are decreased at these stories. Therefore, in evaluating the performance of this type of building structures, both drift and ductility demands have to be considered to ensure that the structure can withstand the demands imposed by the ground motion.

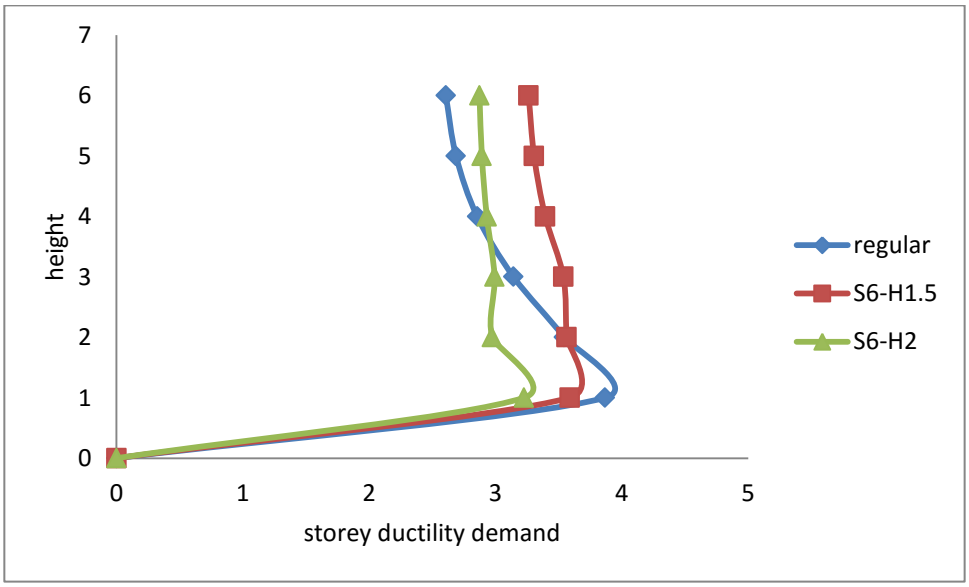



**Figure 4.28: Distribution of ductility demand over height for 6 story buildings**

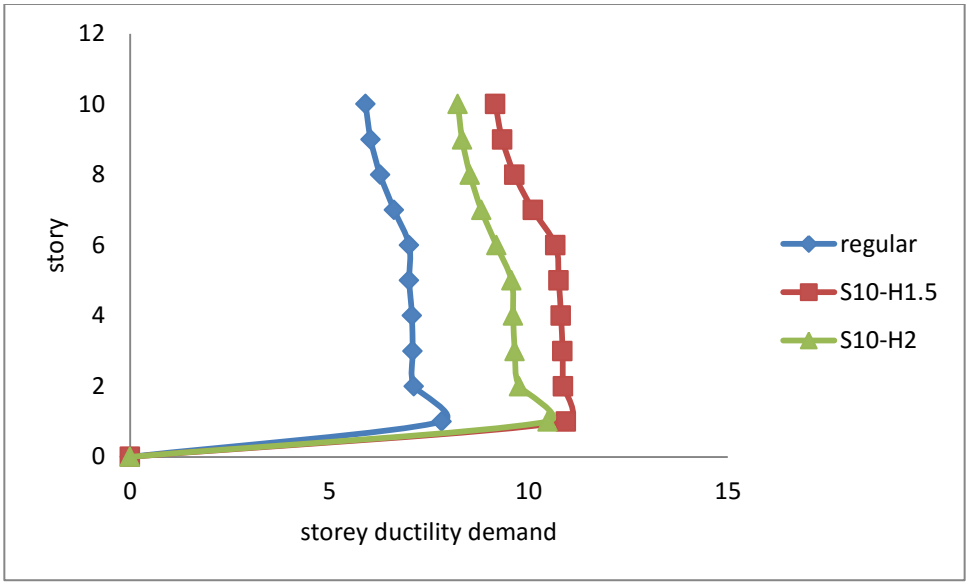



**Figure 4.29: Distribution of ductility demand over height for 10 story buildings**

### 4.11 Variation of Ductility demand ratio with mass and stiffness modification factor

Ductility demand ratio is calculated by ratio of ductility demand of irregular buildings to ductility demand of base case and these ratios are plotted against mass and stiffness modification factor. As presented on figures 4:30 and 4:31, for the case of mass irregularity, ductility demand ratio increases with increase in mass ratio and maximum value of this ratio is located at the floor with irregular mass. Considering the location of mass, when the irregular floor is at the middle of the building, the ductility demand ratio is maximum compared to the other two locations (bottom and top). Also ductility demand ratio increases with height of the building; for example, maximum ductility demand ratio for 6 story building is 0.94 and 1.23 for 10 story building. Figure shows ductility demand ratio variation with stiffness modification factor. As in the case of mass irregularity, ductility demand ratio for stiffness irregularity case doesn't increase with stiffness modification factor. For 150% stiffness modification factor, the ductility demand ratio is maximum when compared to 200% stiffness modification factor, but, for both cases maximum ductility demand is occurred at story with stiffness irregularity. Generally ductility demand is more sensitive to stiffness irregularity than mass irregularity.

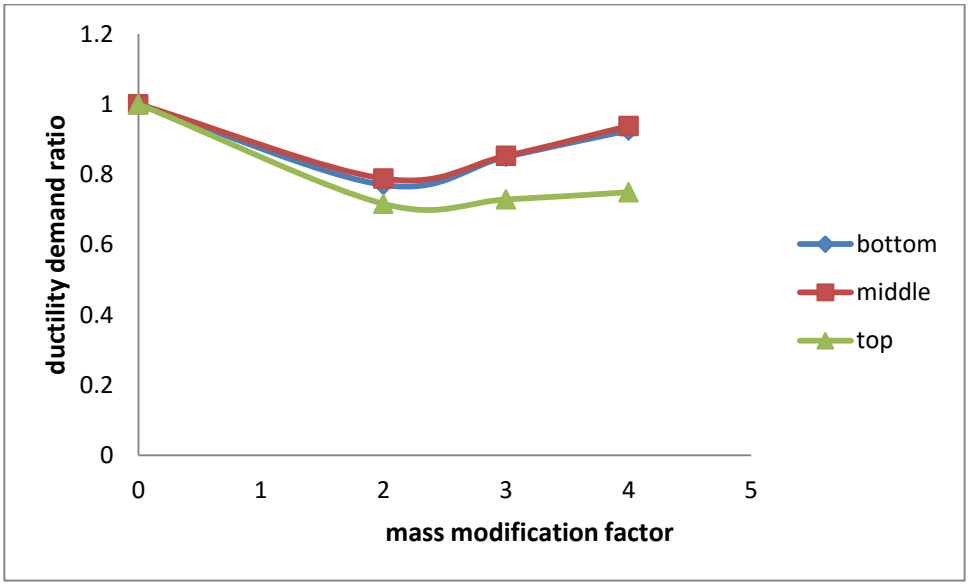



**Figure 4:30 ductility demand ratio case with mass irregularity for 6 story buildings**

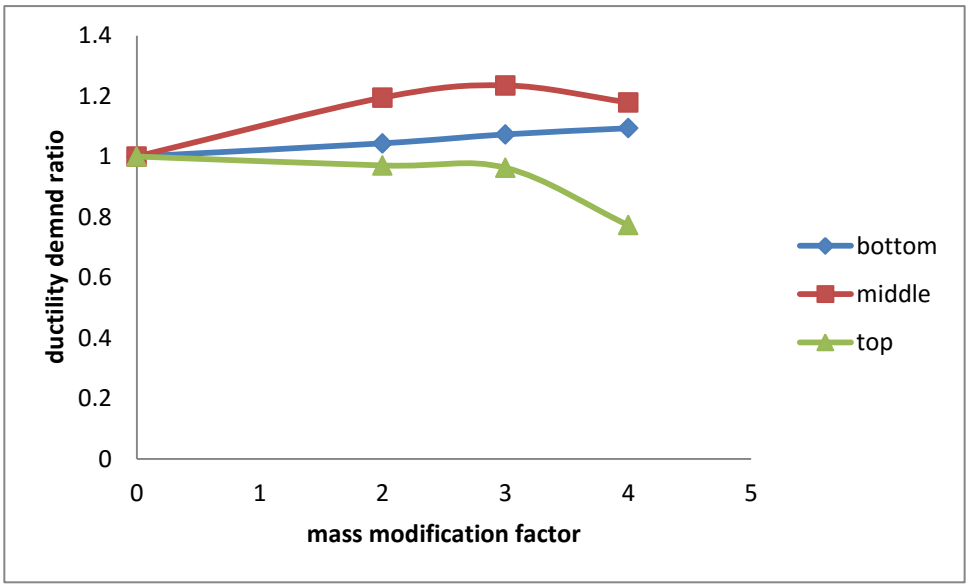



**Figure 4:31 ductility demand ratio case with mass irregularity for 10 story buildings**

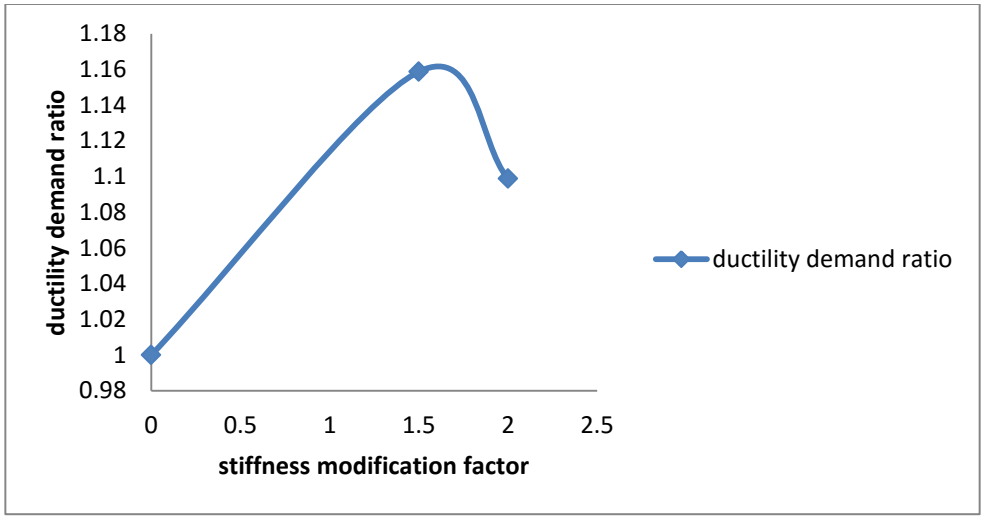



**Figure 4:32 ductility demand ratio case with stiffness irregularity for 6 story buildings**

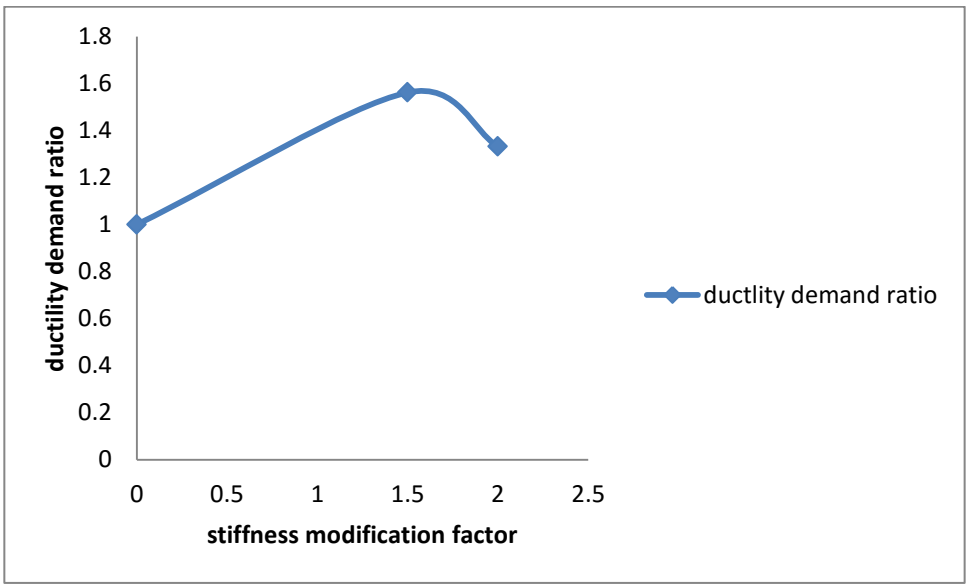



**Figure 4:33 ductility demand ratio case with stiffness irregularity for 10 story buildings**

### 4.11 Plastic hinge formation

The hinging patterns are plotted at different levels as shown in figures 4.36 to 4.39. Most of the hinges formed in the selected buildings are in the linear range at performance level B which shows that the buildings are safe for expected earthquake forces. Plastic hinges formation starts with beam ends and base columns of lower stories. All hinges are formed at lower stories. Number of plastic hinge formed greater than collapse prevention at the last step of pushover analysis is minimum. Maximum number of plastic hinge formed at CP is six which is for building with stiffness irregularity implying the column at ground floor needs to be retrofitted.

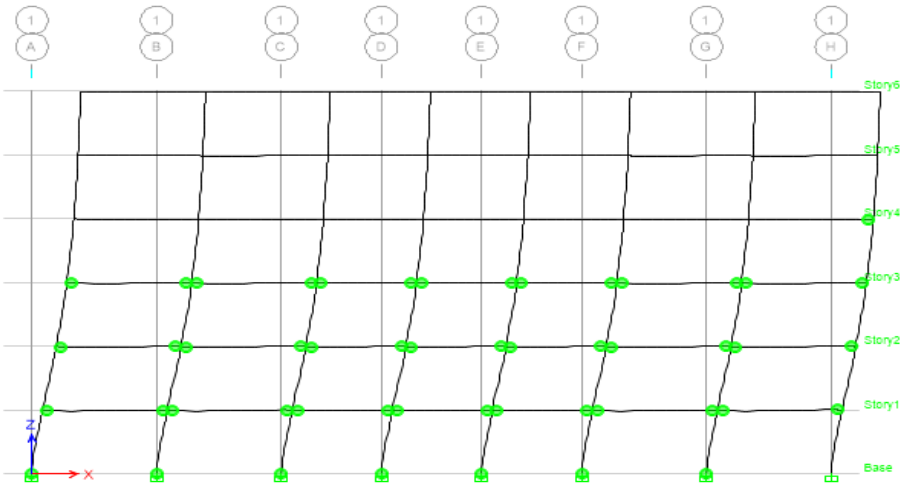



**figure 4.36: plastic hinge distributions for S6-M2-LB**

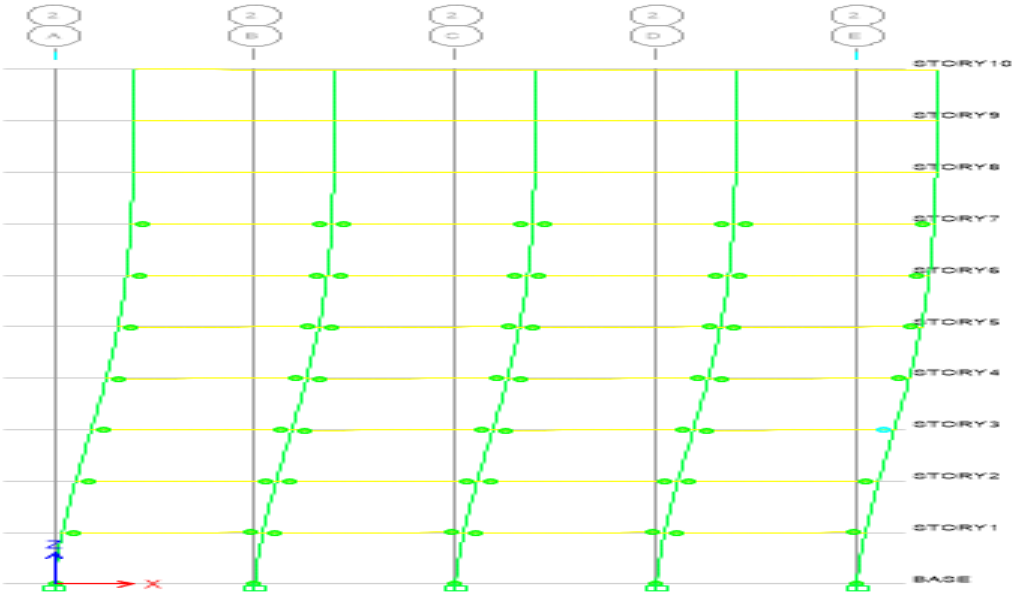



**Figure 4.37: plastic hinge distribution for S10-M2-LB**

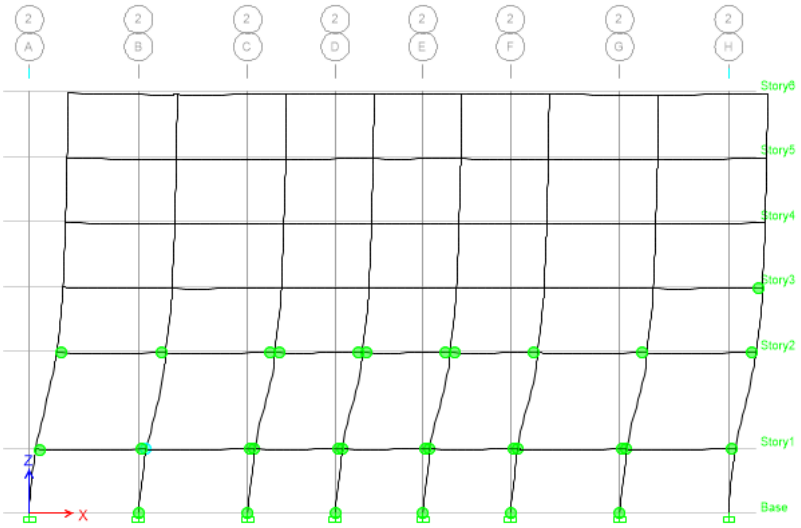



**figure 4.38: plastic hinge distribution for s6-h1.5**

**Summary of inelastic seismic demands**

For inelastic demands studied here, a mass irregularity does not cause significant changes in global drifts. The values of $\alpha_{irreg}$ for the cases studied are between 0.99 and 1.09. On other hand, change in ductility demand for cases of mass and stiffness irregularities increases with number of stories. For the cases studied in this thesis work, ductility demand change is maximum (35.4%) for 10 story building when compared to 6 story building which is 27.3% for case of mass irregularity and, 52% for 10 story building and 22.1% for 6 story building for stiffness irregularity case. Also, ductility demand ratio increases with mass modification factor increase and for stiffness modification factor, ductility demand ratio is maximum when the stiffness modification factor is 150%.The change in roof displacement due to stiffness irregularity for irregular buildings are less than 10% for 10 story building and 27.87% for 6 story buildings from the regular base case roof displacement.

Most of the hinges formed in the selected buildings are in the linear range at performance level B which shows that the buildings are safe for expected earthquake forces. Plastic hinges formation starts with beam ends and base columns of lower stories.

# CHAPTER 5

# CONCLUSION AND RECOMMENDATION

## 5.1 conclusions

In this research work, the seismic behaviour of 6 and 10 stories building with mass and stiffness irregularities are studied. Mass irregularities are introduced at different location along the height of the building (bottom, middle and top) with mass modification of 200%, 300% and 400%. The structural models are 3D and symmetric in plan. Stiffness irregularity is created at ground floor by increasing the height of floor 150% and 200% of other floors. The seismic responses of the buildings are evaluated by response spectrum and non-linear static pushover analysis. The effect of mass and stiffness irregularities are assessed by evaluating fundamental period of buildings, elastic seismic demand (base shear, roof displacement, storey lateral displacement, story drift and story shear), inelastic seismic demands (roof displacement, storey drift and storey ductility demands).Therefore, conclusions from this study are drawn as follows:

- The effect of irregular mass on fundamental period of building structure is not significant when compared to stiffness irregularity, fundamental period changes by 45.2% for stiffness irregularity and 11.21% for mass irregularity for mass and stiffness modification factor of 200%.
- Base shear demand of irregular building due to mass increases 23.89% when compared to increase in fundamental period which is only 11.2%. Therefore, base shear is more sensitive to mass irregularity than fundamental period. But for stiffness irregularity, base shear decreases by 43.13% from regular building which is opposite to fundamental period with increase in 45.2%.
- Mass irregularities show relatively small effects on distribution of storey shear, lateral displacement and storey drift demands than stiffness irregularity of the same magnitude.
- For the cases of inelastic analysis, roof displacement demands are not sensitive to the presence of either mass or stiffness irregularities. For the cases of all mass modification and stiffness modification factor, roof displacement demands do not change by more than 15% from the base case.
- The change in distribution of story drift and story ductility demands over height due to mass and stiffness irregularities are non-uniform for all cases studied. Story drifts demands increase in the soft story and decrease in most of the other stories. On the other hand, ductility demand is maximum at story with decreased stiffness.The behavior of assumed well detailed

reinforced concrete frame building is adequate as indicated by the distribution of hinges in the beams and the columns. Most of the hinges developed in the beams and few in the columns but with limited damage.

## 5.2 Recommendations for Future Study

- In this study, mass irregularity is introduced only at single floor, so future study will cover mass irregularity at many floors over height of the building.
- The structures considered in this study are regular in plan and the lateral forces are assumed to be applied at the mass center, therefore, future study can cover unsymmetrical building with significant torsion.

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## APPENDIX: A

### Values of Different Responses of RC Buildings in Tabular Form

Table A: 1 storey ductility demands for 6 story buildings case with mass irregularity

| storey | regular | S6-M2-LB | S6-M2-LM | S6-M2-LT | S6-M3-LB | S6-M3-LM | S6-M3-LT |
|---|---|---|---|---|---|---|---|
| 1 | 3.868024 | 2.980108 | 2.872816 | 2.45017 | 3.294732 | 2.951912 | 2.823083 |
| 2 | 3.541543 | 3.186149 | 3.13112 | 2.67506 | 3.431905 | 3.203084 | 3.078717 |
| 3 | 3.142245 | 3.098019 | 3.052177 | 2.65389 | 3.295466 | 3.094519 | 3.070361 |
| 4 | 2.854766 | 2.933536 | 2.89527 | 2.544392 | 3.107502 | 2.935592 | 2.959765 |
| 5 | 2.687285 | 2.814369 | 2.780075 | 2.444956 | 2.980142 | 2.822638 | 2.847489 |
| 6 | 2.606605 | 2.758841 | 2.727212 | 2.385418 | 2.920233 | 2.770137 | 2.773064 |

Table A: 2 storey ductility demands for 6 storey buildings case with stiffness irregularity

| storey | regular | S6-H1.5 | S6-H2 |
|---|---|---|---|
| 1 | 3.868024 | 3.588235 | 3.225806 |
| 2 | 3.541543 | 3.849345 | 3.673131 |
| 3 | 3.142245 | 3.536818 | 3.126325 |
| 4 | 2.854766 | 3.393621 | 2.930832 |
| 5 | 2.687285 | 3.305439 | 2.891886 |
| 6 | 2.606605 | 3.262169 | 2.87307 |

Table A: 3 storey ductility demands for 10 story buildings case with mass irregularity

| storey | regular | S10-M2-LB | S10-M2-LM | S10-M2-LT | S10-M3-LB | S10-M3-LM | S10-M3-LT |
|---|---|---|---|---|---|---|---|
| 10 | 5.90692 | 6.304723 | 6.333346 | 6.23649 | 6.36671 | 6.505276 | 6.178852 |
| 9 | 6.03783 | 6.444742 | 6.468636 | 6.43242 | 6.50801 | 6.641111 | 6.435333 |
| 8 | 6.27560 | 6.699159 | 6.713009 | 6.73833 | 6.76139 | 6.886697 | 6.807396 |
| 7 | 6.62631 | 7.051715 | 7.067183 | 7.09958 | 7.00850 | 7.248828 | 7.252592 |
| 6 | 7.00470 | 7.398684 | 7.436479 | 7.40799 | 7.46368 | 7.651654 | 7.661276 |
| 5 | 7.26453 | 7.608590 | 7.666875 | 7.56060 | 7.66991 | 7.928320 | 7.987212 |
| 4 | 7.30545 | 7.599448 | 7.698102 | 7.49604 | 7.66296 | 8.025345 | 8.206372 |
| 3 | 7.09245 | 7.328155 | 7.4662 | 7.11508 | 7.42917 | 6.266062 | 8.340103 |
| 2 | 6.70370 | 6.901710 | 7.061851 | 6.53946 | 7.06291 | 7.317340 | 8.460454 |
| 1 | 6.42362 | 6.6956767 | 6.811462 | 6.120754 | 6.890395 | 7.0382129 | 9.1841748 |

Table A: 4 story ductility demands for 10 story buildings case with stiffness irregularity

| storey | regular | S10-H1.5 | S10-H2 |
|---|---|---|---|
| 10 | 5.906921 | 9.16642 | 8.225219 |
| 9 | 6.037835 | 9.338232 | 8.332551 |
| 8 | 6.275603 | 9.646216 | 8.531152 |
| 7 | 6.626313 | 10.10959 | 8.813685 |
| 6 | 7.004701 | 10.67294 | 9.192153 |
| 5 | 7.264537 | 11.1656 | 9.573759 |
| 4 | 7.305454 | 11.35398 | 9.747451 |
| 3 | 7.092459 | 11.42194 | 9.462274 |
| 2 | 6.703707 | 13.45511 | 8.553908 |
| 1 | 6.423624 | 10.93636 | 10.47566 |

**Capacity curves for 6 and 10 story buildings**

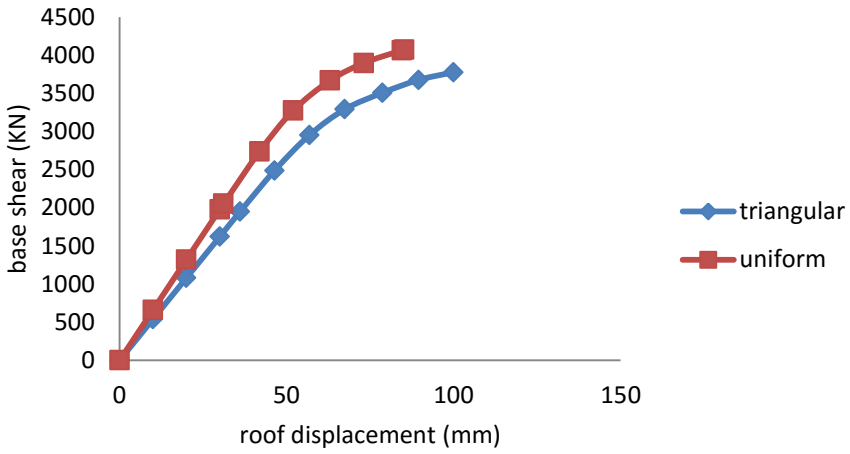


Figure A: 1 capacity curve for S6-M3-LB

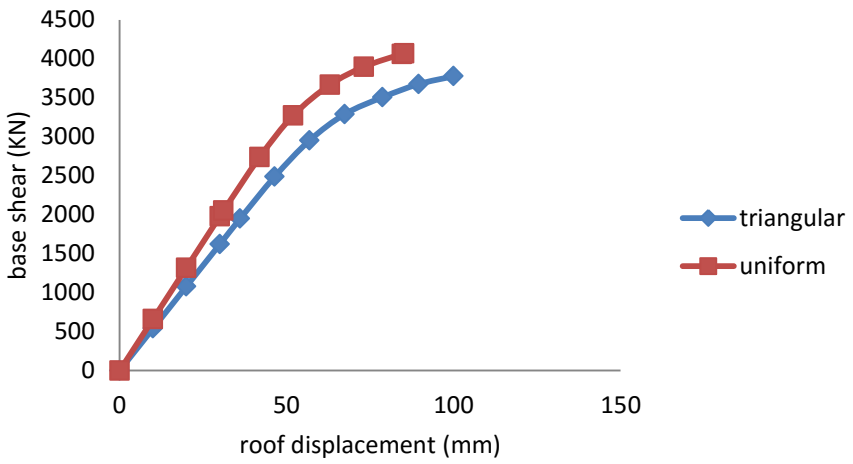


Figure A: 2 capacity curve for S6-M2-LM

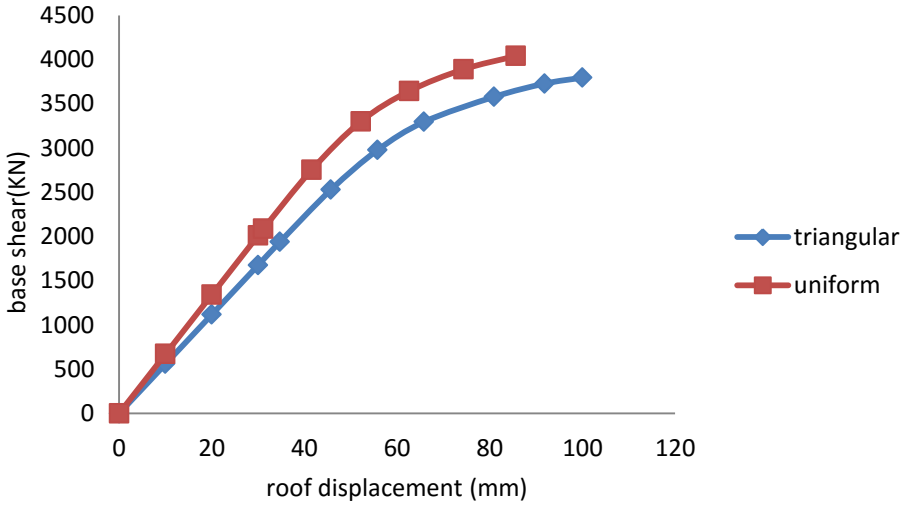


Figure A: 3 capacity curve for S6-M3-LM

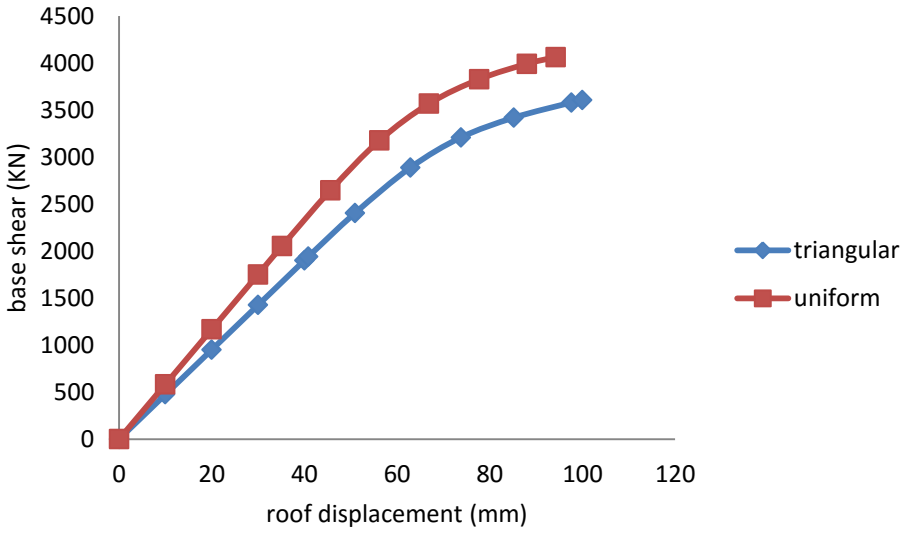


Figure A: 4 capacity curve for S6-M2-LT

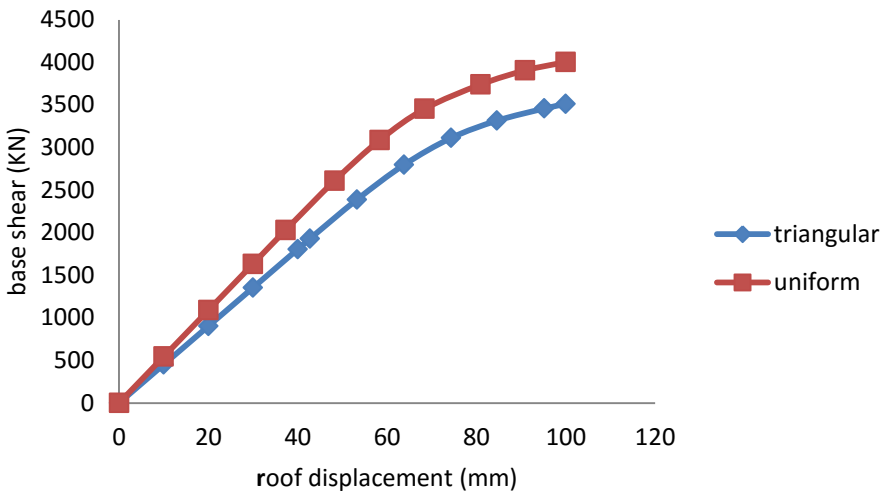


Figure A: 5 capacity curve for S6-M3-LT

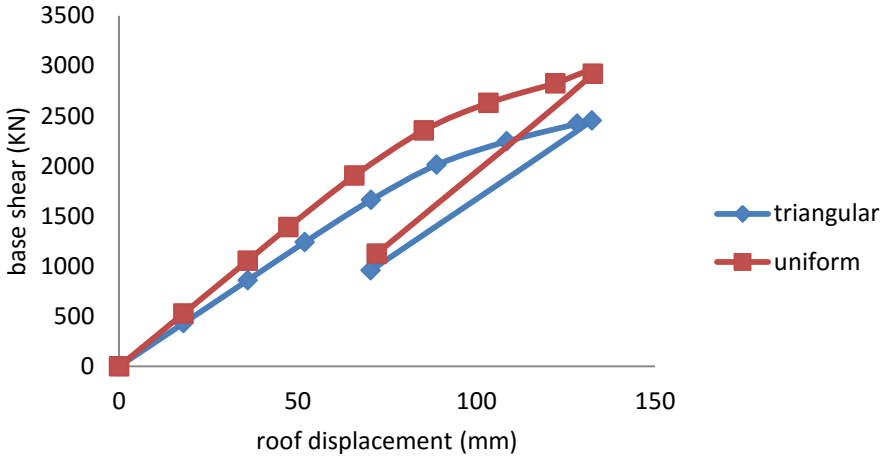


Figure A: 6 capacity curve for S6-H2

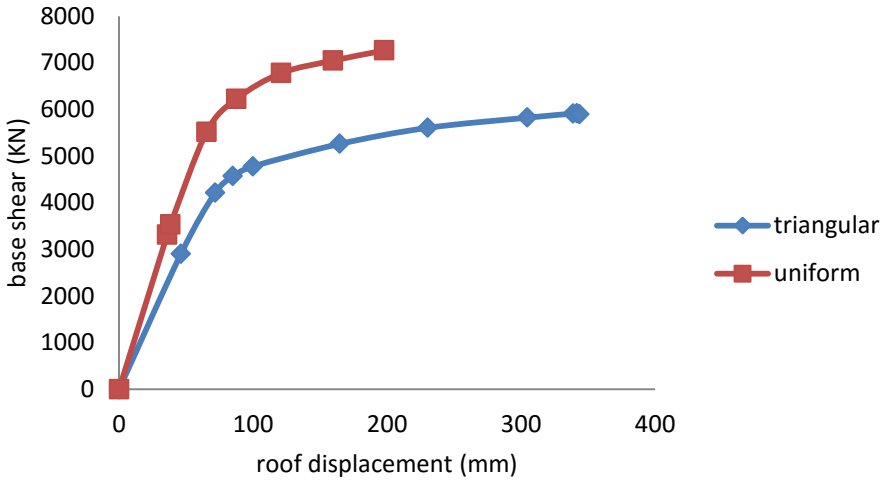


Figure A: 7 capacity curve for S10-M3-LB

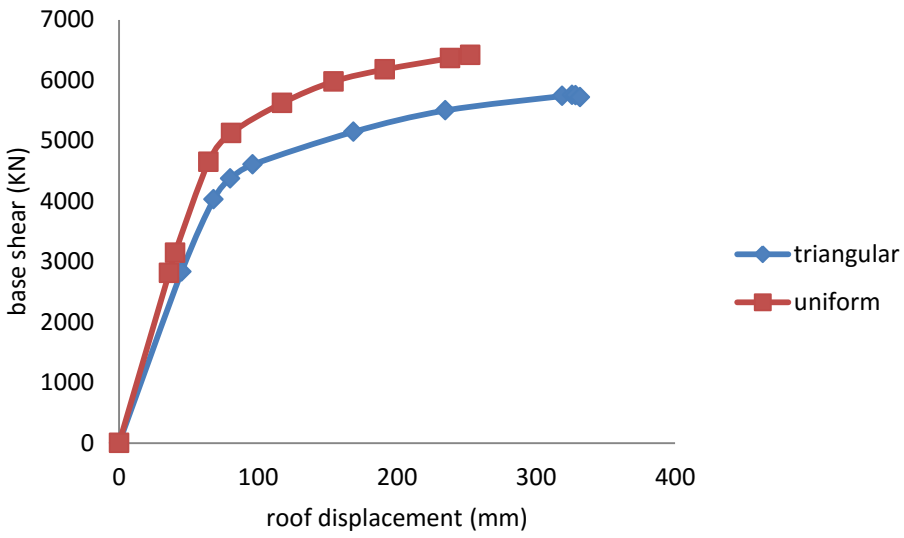


Figure A: 8 capacity curve for S10-M2-LM

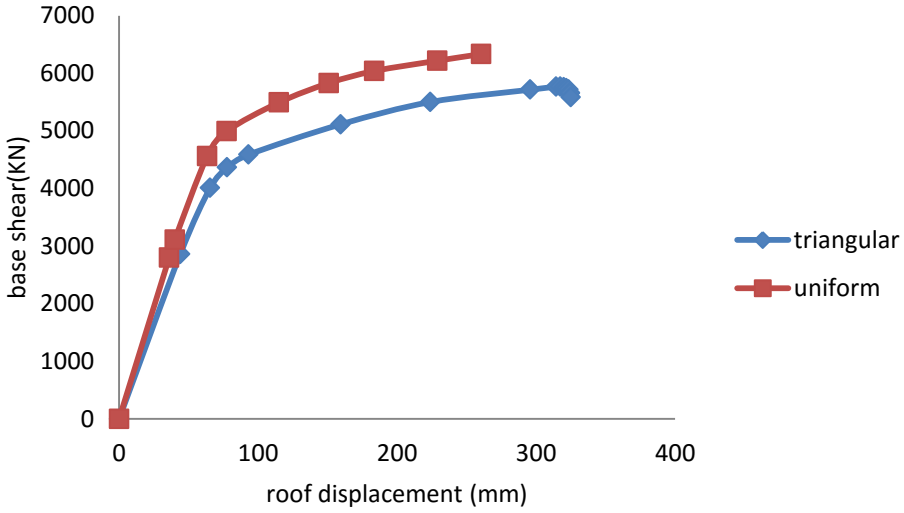


Figure A: 9 capacity curve for S10-M3-LM

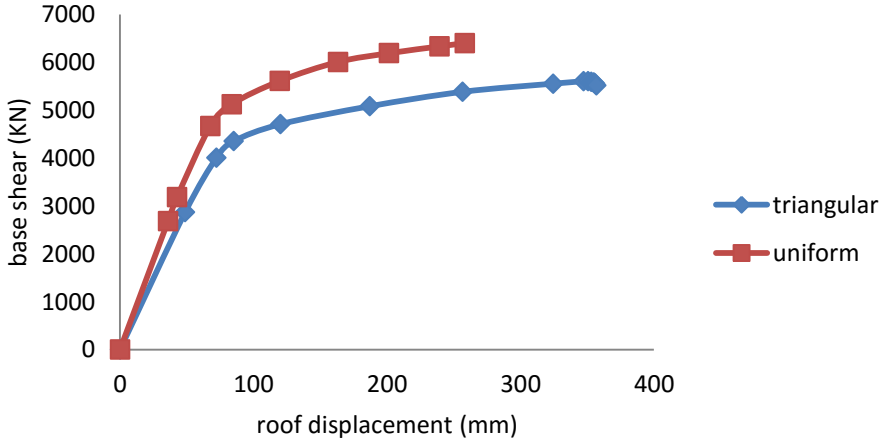


Figure A: 10 capacity curve for S10-M2-LT

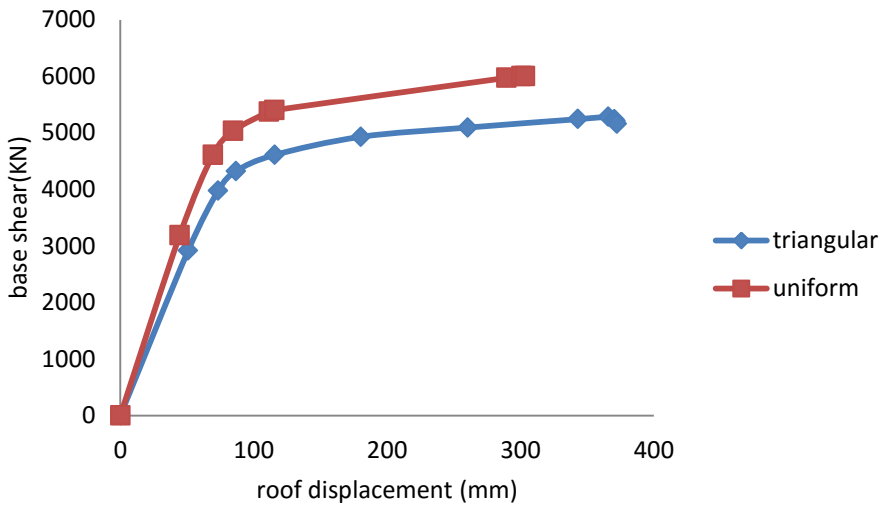


Figure A: 11 capacity curve for S10-M3-LT

**Declaration**

I hereby declare that the work presented in this thesis which is entitled "Seismic Behaviour of Reinforced Concrete Building with Mass and Stiffness irregularity" is my original work and has not been presented in any other University and that all sources of material used for the thesis have been properly acknowledged.

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Kuleni Fekadu