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M.Sc. in Civil Engineering

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UNIVERSITY OF GAZİANTEP

GRADUATE SCHOOL OF

NATURAL AND APPLIED SCIENCES

A NUMERICAL STUDY ON SEISMIC BEHAVIOR OF PLAN IRREGULAR BUILDINGS

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BY

IHSAN AHMED MUSTAFA

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A Numerical Study on Seismic Behavior of Plan Irregular Buildings

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Supervisor

Assoc. Prof. Dr. Esra METE GÜNEYİSİ

by

Ihsan Ahmed MUSTAFA

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This is to certify that we have read this thesis and that in our consensus opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

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Ihsan Ahmed MUSTAFA

ABSTRACT

A Numerical Study on Seismic Behavior of Plan Irregular Buildings

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The shape, size and geometry of the building affects critically its seismic performance. Buildings with simple geometry in plan had performed well during the past strong earthquakes. But buildings with plan irregularity are more vulnerable and expected to be experienced more damage during seismic excitation. In this study, an eight story reinforced concrete regular building having a typical 6 bay x 6 bay frame with plan dimensions of 36 x 36 m and twenty four irregular buildings with different irregularities of +, C, O, H, ┴, L and Z shapes were examined. The structures were modeled using a finite element method and evaluated by both nonlinear static and dynamic analyses. Three-dimensional structural models were utilized for the analysis. Capacity curves, variation of storey displacement, and roof displacement, base shear time history were computed for the regular and irregular buildings. The analysis of the results showed that the regular building had considerably greater capacity in comparison to the irregular buildings due to the fact that irregularity caused decreasing the capacity of the buildings and the structures were more doubtful when they were more irregularities. Moreover, the displacement demands of the buildings were sensitive to type and amount of irregularity, especially at roof level.

Keywords: Irregular structure, Nonlinear analysis, Performance assessment, Plan irregularity, Reinforced concrete building.

ÖZET

Düzensiz Binaların Sismik Davranışı Üzerine Sayısal Bir Çalışma

MUSTAFA, Ihsan Ahmed
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Binanın şekli, büyüklüğü ve geometrisi sismik performansını kritik derecede etkilemektedir. Planda basit geometriye sahip binalar geçmişteki şiddetli depremlerde iyi performans göstermişti. Ancak, planda düzensizliğe sahip binaların daha savunmasız olduğu ve sismik aktivite sırasında daha fazla hasar oluştuğu görülmüştür. Bu çalışmada, 36 x 36 m plan boyutlarına ve her iki yönde 6 açıklığa sahip 8 katlı düzenli bir betonarme bina ile +, C, O, H, $\perp$, L ve Z gibi farklı düzensizlikleri olan 24 düzensiz bina incelenmistir. Yapılar, sonlu elemanlar yöntemi kullanılarak modellendi ve hem doğrusal olmayan statik hem de dinamik analizlerle değerlendirildi. Analiz için üç boyutlu yapısal modellerden yararlanılmıştır. Düzenli ve düzensiz binalar için kapasite eğrileri, kat deplasman değişimi ve çatı yer değiştirmesi, taban kesme kuvveti zaman ilişkisi hesaplandı. Sonuçların analizi, planda düzensizliğin binaların kapasitesini düşürmesinden dolayı, düzenli yapının planda düzensiz yapılara kıyasla fazla kapasiteye sahip olduğunu gösterdi. Düzensizlik miktari çok olan yapıdan daha olumsuz sonuçlar elde edildi. Ayrıca, binaların yer değistirmesinin, özellikle çatı seviyesinde düzensizliğin türüne ve miktarına duyarlı olduğu gözlemlendi.

Anahtar Kelimeler: Düzensiz yapı, Doğrusal olmayan analiz, Performans değerlendirmesi, Plan düzensizliği, Betonarme yapı.

To my Wife…

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LIST OF SYMBOLS/ABBREVIATIONS

η_{bi}	Torsional irregularity factor
ASCE	American society of civil engineers
ATC	Applied Technology Council
BIS	Bureau of Indian standards
BST	Base shear torque surface
CD	Center of displacement
CP	Collapse prevention
CM	Center of mass
CR	Center of rigidity
CS	Center of stiffness
CV	Center of Strength
es, ev, em	Stiffness, strength and mass eccentricity
D	Roof displacement
Dt	Inelastic displacement
Dt	Target displacement
FEMA	Federal Emergency Management Agency
FEM	Finite Element Model

FEA	Finite Element Analysis
IBC	International building code
IO	Immediate occupancy
IS	Indian standard criteria for earthquake resistant design
LS	Life Safety
MDOF	Multi degree of freedom
NBCC	National Building Code of Canada
NIST	National Institute of Standards and Technology
NSP	Nonlinear static procedures
RC	Reinforced concrete
RSA	Response spectrum analyses
SDOF	Single degree of freedom
TSC	Turkish seismic code
TB	Torsionally balanced
TU	Torsionally unbalanced
UBC	Uniform building code
V	Base shear

CHAPTER 1

INTRODUCTION

Buildings are the complex framework and multiple items must to be studied. In the planning phase itself, structural and architects engineers would work together to promise that the unsuitable features are averted and suitable building configuration is selected. A desire to create a beautiful and functionally proficient structure drives architects to imagine wonderful structures. Occasionally the figure of premises catches the eye of visitor, or the structural system requests, and in other occasions each figure and structural system work together to create the structure a marvel. Each of these selections of shapes and building has significant impact on the performance of building during strong earthquake. For that the symmetry and regularity are generally recommended for a good design of earthquake resistant structure (Mohod, 2015).

The earthquake is shaking of the ground surface leads to an unexpected release of the energy in the ground surface. Earth's surface moves in every directions. Lateral movements caused the most destructive effects on buildings which interrupt the stability of the building, which leading to failure. Usually buildings were constructed to resist gravity loads. In recent year, the challenges facing structural engineer increased because of different kinds of irregularities involving in the structures were prepared by the architectural engineer (Alashker et al., 2015).

One major reason of failure in RC structures is the torsional response of the frame buildings, produced by the earthquake and/or by the structural irregularities. Seismic provisions are usually set standards to design and build of new buildings exposed to earthquake movements with three aims: (i) reduce the risk to life associated to all kinds of structures, (ii) rise the anticipated performance of buildings having a substantial public hazard because of the specific use, and (iii) increase the ability of fundamental facilities to work after an earthquake (Landingin et al., 2013).

A building is consider to be a regular when the building configurations are roughly symmetrical about the axis and it is consider to be the irregular if it losses symmetry and discontinuity in the geometry, mass or load resisting elements. In the multistory frame buildings damages due to earthquake are usually starts at the weak points. This weakness grows due to discontinuity in mass, stiffness and geometry of structure. The frame structures containing this discontinuity are indicated as irregular structures. So if a structure can perform well in earthquake implies it should have sufficient strength, stiffness, ductility and simple configuration. So these kinds of structures must be good designed under earthquake loading accounting the specified seismic design. Therefor that they can tolerate moderate to strong earthquakes (Sakale et al., 2014).

The analysis of the earthquake reaction of irregular frames is complex and more difficult than that of regular structures due to nonlinear and inelastic response. The dynamic nonlinear analysis procedure is the best option for solving these issues because they supply further realistic models of the structural response to the strong earthquake and supply further reliable evaluation of earthquake performance than other methods. However, such an approach is not feasible for most practical applications (Mahdi and Gharaie, 2011).

There are different types of plan irregularity which are torsion irregularity, re- entrant corners and diaphragm discontinuity which are specified in the codes (Dubey and Sangamnerkar, 2011).

Irregular structures usually display unfavorable seismic behavior, described by the concentration of plastic demand in a limited part of the structure; this matter can cause collapse under moderate or strong seismic motion. Every modern building codes supply rules to verify the regularity of structures in plan; if the rules aren't obeyed, some “penalties” in the design specification are provided (CEN, 2004). The evidence from past earthquakes obviously indicated that the irregularity in plan is the most important sources of the damage, which can be caused by asymmetric distributions of the mass, strength and stiffness, so it causes torsional floor rotations localizing the seismic demand in small parts of the structure (De Stefano and Pinutucchi, 2008).

1.1 Objective of the Thesis

The main purpose of this study is to investigate the nonlinear response of a regular (RC) frame building in plan compared with twenty four irregular reinforced concrete (RC) frame buildings in plan. They were divided into six groups, namely +, C, O, H, ⊥, L and Z shapes. Analytical modeling of the regular and irregular frame buildings were performed by using static nonlinear analysis and dynamic nonlinear analysis. The three-dimensional model of frame structures were performed by using finite element analysis software SAP2000. The result of the analysis were obtained in terms of capacity curve, displacement time history and base shear time history.

1.2 Outline of the Thesis

Chapter 1-Introduction: Aim and objective of the thesis were presented.

Chapter 2 Literature review: Literature review depending on this study is given. It contains a review of the relevant literature that covers previous studies conducted on plane irregular buildings.

Chapter 3-Case study: This chapter provides a description of models, analysis of the regular and different types of plan irregular buildings. The analysis methods and the characteristics of the ground motion record are given also.

Chapter 4-Discussion of the results: Results obtained from the seismic analysis of the regular building and the irregular buildings was presented. A discussion of the results were given.

Chapter 5-Conclusions: The conclusion of the thesis were given in this chapter.

CHAPTER 2

LITERATURE REVIEW

2. 1 Introduction

Horizontal irregularities is one of the major causes for the failure amplification under earthquake action. Previous earthquakes, actually, showed that structures with asymmetrical distribution of the structural characteristics or irregular arrangement are exposed to a rise in the seismic demand, producing larger failures. Causes of irregularity in the configuration of structure will be various types and multiple and are generally divided within two mainly kinds: horizontal and vertical irregularities. First kind is correlated to asymmetrical plan strength, mass and stiffness distributions, producing a significant rise in the torsional influences once the building is exposed to lateral loads. Second kind includes difference in the structural and geometrical characteristics along the height of the structure which usually leads to a rise in the seismic demand in exact storeys. Frequently these kinds of irregularity involve the advancement of brittle collapse mechanisms as a result of local rise of the seismic demand in exact parts that aren't supplied with adequate ductility and strength (De Stefano and Mariani, 2014).

These two kind can be divided to five different kinds like torsional, re-entrant corners, diaphragms discontinuity, out of plane offset and non parallel system for plan irregularity also vertical irregularity like stiffness (soft storey), mass, elevation geometric, in plane discontinuity in vertical parts resisting discontinuity and lateral load incapacity (weak storey) (IS 1893(Part I): 2002).

Starting with the irregularity of a plan, evaluation of building performance during previous earthquakes indicates this kind of irregularity, that is because of asymmetric distributions of the mass, strength and stiffness, is the greatest common causes of serious destruction, because it effects in floor rotations and floor translations (De Stefano and Pintucchi, 2008).

The effects of torsion on buildings produced by irregularity in strength, mass or stiffness distribution has been tackle various approach. The most commonly used and simplest kind, the methodology assumed by code where by moving the application point of the applied static load, on the plane of every floor, the torsion effect must be taken into account. Eccentricities are employed to determine that movement and concept was introduced to evaluate the seismically induced displacement for the two critical edges of a building, which are called the flexible edges and the stiff edges. This methodology has been advanced depend on single story models. The results of this methodology can be extended to regular multi-story models; However, it's doubled that analogy could be extended to irregular models, fundamentally because the static force methods doesn't represent for complex dynamic response and high order effects. The expressions displayed on designing codes may not represent the torsional effects, there have been many attempts to produce methodologies that cover the defects from the above procedure, a number of which until now study one floor systems and try to conclude the results to multistory irregular systems (Lam et al., 1997).

Many researchers studied passive control that seems to be an appropriate option to traditional design with the purpose of mitigate the effects of torsion. Like this researchers have been dedicated generally to viscous and visco-elastic damping and base isolation devises (De Stefano and Pintucchi, 2008).

The beginning of the studies about the effects of torsion including irregular structures, the history back to the 30s of the previous century because of a growing knowledge of the difficulty of the reaction of non-symmetric structures to earthquake movements which isn't clearly translational, however it includes torsional distortions which in many cases negatively effect on their seismic behaviour (Ayre, 1938).

Differences in structural properties, random distribution of the live load mass and the possible torsional ground motion are three causes why both irregular and regular frames should be designed for the casual torsional forces. So, lateral forces for the regular building do not burn torsional modes. One proposed technique is to conduct a mass with many various dynamic analysis at different positions. This concept is not feasible because the dynamic base shears and the basic dynamic characteristics of the building should be variant for every analysis (Abd-el-rahim and Ferghaly, 2010).

Plan asymmetric frame buildings submitted to lateral input earthquake motions are influenced by torsional coupling, for example, floor translations and rotations, which usually outcomes in larger lateral loads and buckles experienced by resisting parts (shear walls, frames). Moreover, the structures are planned to undergo inelastic behaviour under strong ground motions, torsional motions is one of the very important reason of failure and severe damage, because they lead to further displacements and higher ductility demand in the resisting parts (Abd-el-rahim and Ferghaly, 2010).

In earthquakes torsional effects can appear even when static center of mass coincide with static center of resistance. For example, earthquake waves performing with a skew with regard to the structure axis also yielding and cracking in a nonsymmetrical mode could produce torsion. Also these influences can amplified the torsion because eccentricity between static centers. Therefore structures should be classified as irregular if they have an eccentricity between the static centers of resistance and mass excess 10 % of the structure dimension perpendicular to the earthquake load direction. Plan configurations like H-shapes possess a symmetrical geometry should be categorized as irregular due to the wings reaction. Substantial variances in stiffness between parts of a diaphragm at a level are categorized as irregularities because they could produce a variation in the distribution of the earthquake loads to the vertical elements and produce torsional loads not occurred in the distribution observed for the regular frame (Abd-el-rahim and Ferghaly, 2010).

A frame structure would have the maximum opportunity of surviving an earthquake, if a) the force bearing members were distributed uniformly; b) every walls and columns were continuous and without offsets from footing to roof; c) every beams were free of offsets; d) beams and columns were co-axial; e) beams and columns are approximately the same width; f) no major change in members section suddenly; g) the frame structure is as homogeneous and continuous as possible. Some elementary rules for vertical frames in a seismic structures. If at every un-symmetry was unavoidable, then seismic joints would be supplied between them. Seismic joint is special joint designed to avoid hammering of adjacent dissimilar structures. It based on the relative displacement of floors as known “Drift” (Abd-el-rahim and Ferghaly, 2010).

2.2 Irregularities in elevation

The buildings irregular configuration has been defined clearly in the current version of IS 1893 (Part 1)-2002 (BIS, 2002). Five kinds of irregularity in elevation documented as displayed in Figure 2.1. They are:

- Mass irregularity,
- Stiffness irregularity (soft storey),
- Discontinuity in capacity (weak storey),
- Vertical geometric irregularity or irregularity in elevation (set-back),
- In-plane discontinuity in lateral-load-resisting vertical components, and
- In-plane discontinuity in lateral-load-resisting vertical components.

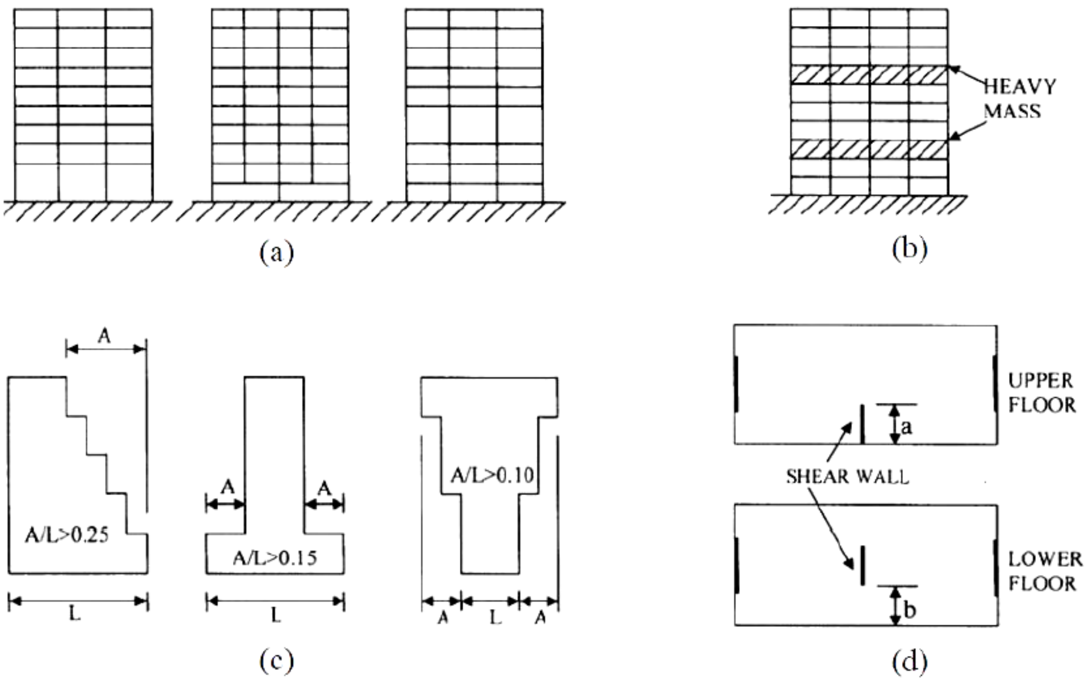


Figure 2.1 a) Irregularity of stiffness/strength; b) irregularity of mass; c) elevation geometric irregularity or setback; and d) in-plane discontinuity in lateral-load-resisting vertical components when b > a: plan view (BIS, 2002)

2.3 Irregularities in plan

There are three kinds of irregularity in plan depending on Turkish Seismic Code (TSC, 2007):

- **Torsional irregularity:**

  The situation where the torsional Irregularity Factor η_{bi}, that is determined for any of the two orthogonal seismic directions if the proportion of the greatest storey drift at any storey to the average storey drift in the same direction at the same storey, is more than 1.2 (see Figure 2.2).

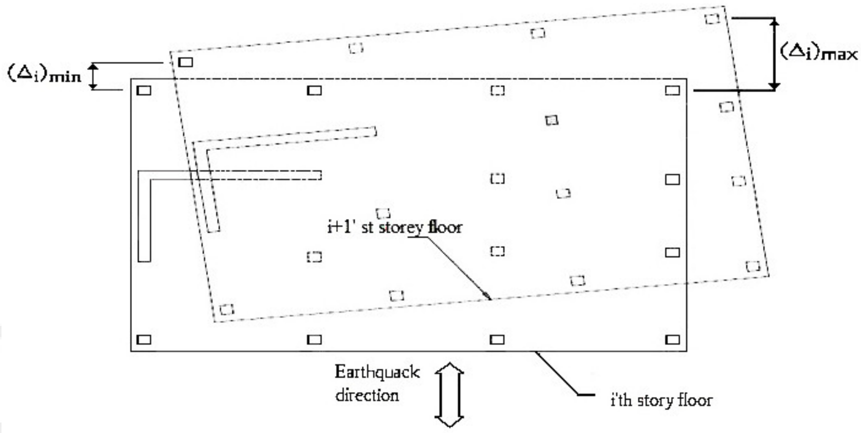
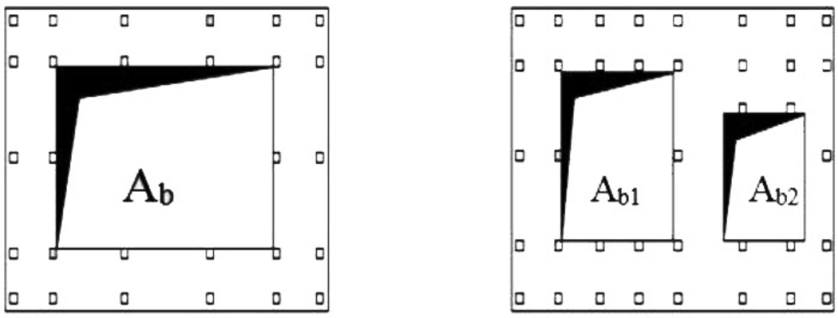


Figure 2.2 Torsional irregularity (TSC, 2007)

- **Floor discontinuities:**

i. The situation where the total area of the openings containing those of elevator and stairs shafts is more than (0.3) of the total floor area, A_b / A > 0.3 where A is gross floor area and A_b is the sum of opening areas as shown in Figure 2.3.



$A_b = A_{b1} + A_{b2}$

Figure 2.3 Floor discontinuities type i (TSC, 2007)

ii. The cases where the safe transfer of earthquake forces to vertical structural parts is difficult because of local floor openings as shown in Figure 2.4.

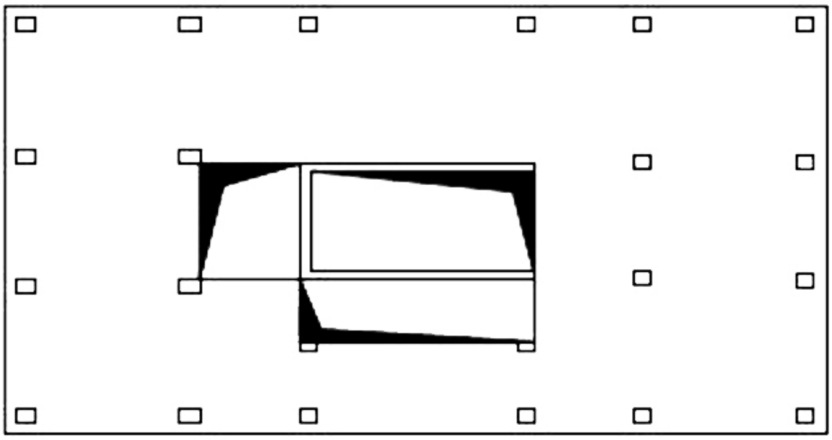


Figure 2.4 Floor discontinuities type ii (TSC, 2007)

iii. The cases when sudden decreases in the in-plane strength and stiffness of floors as shown in Figure 2.5.

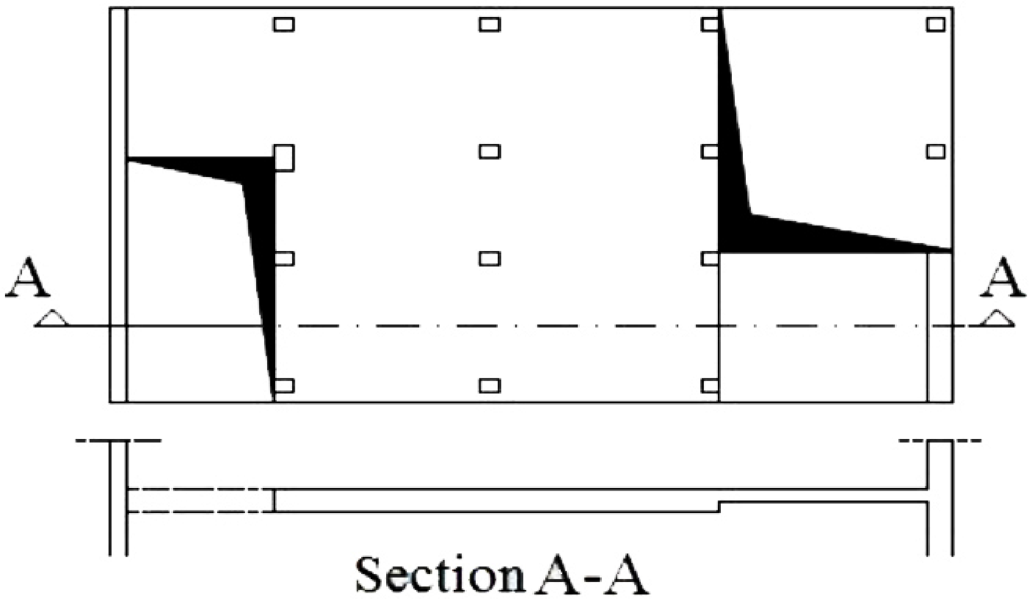


Figure 2.5 Floor discontinuities type iii (TSC, 2007)

- **Plan projections:**

The situations where projections behind the re-entrant corners in each of the two principal directions in plan override the entire plan dimensions of the structure in the respective directions by more than 20% $a_y > 0.2\ L_y$ and at the same time $a_x > 0.2\ L_x$ as shown in Figure 2.6 (TSC, 2007).

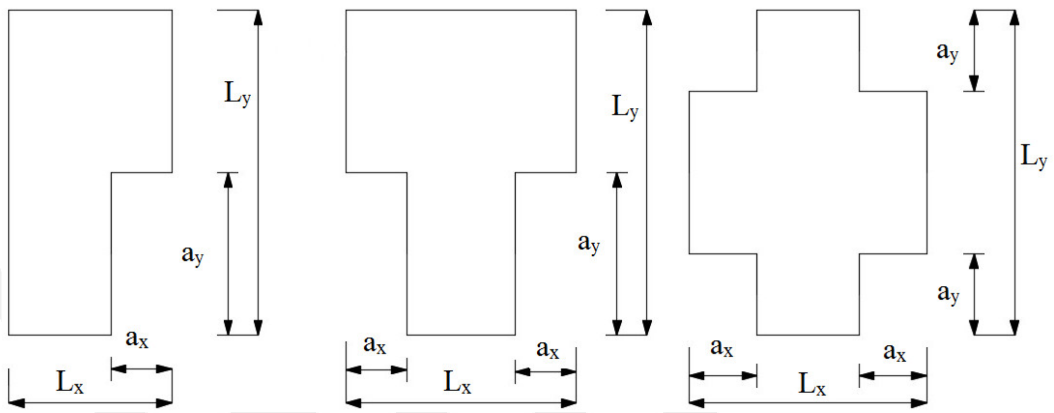
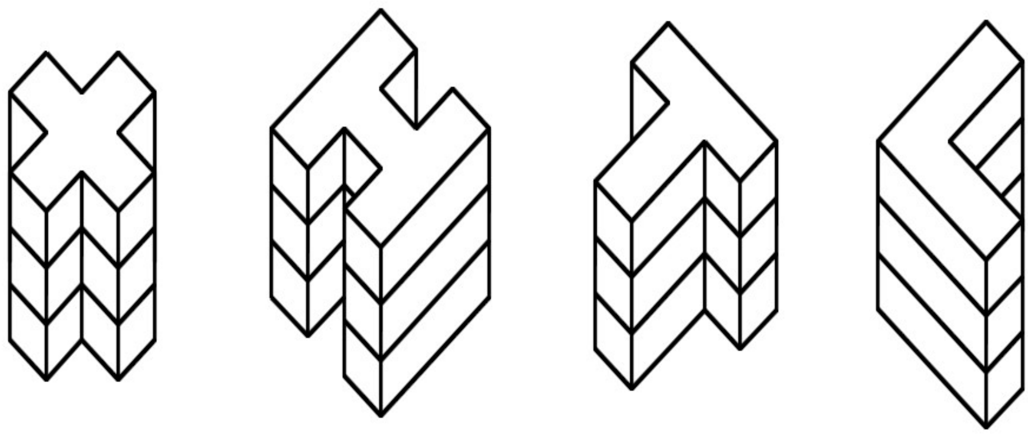


Figure 2.6 Projections in plan (TSC, 2007)

2.4 Floor plan variables

The term rectangular identifies plan shape represented by polygons with reentrant corners which sides meet orthogonally (Figure 2.7) (Abd-el-rahim and Ferghaly, 2010).



(a) Symmetry (b) Proportion (c) Reentrant corners

Figure 2.7 Irregular rectangular floor plan shapes (Abd-el-rahim and Ferghaly, 2010)

The major variables that determine the characteristics of floor plan shapes are: (a) Symmetry, (b) proportion, and (c) reentrant corners. Nevertheless, even when buildings possess floor plan shapes that belong to the same family (for example, all those in the H shape family), they don't necessarily have the same degree of vulnerability to earthquakes. The vulnerability will based on:

(i) Number of axes of symmetry,

(ii) The location within the shape of reentrant corners,

(iii) Ration of the rectangular components of the floor plan figure,

(iv) (Torsional eccentricity) the displacement of corners of rigidity with respecting to the center of mass; according to the study of Abd-el-rahim and Ferghaly (2010).

Structural asymmetry leads to eccentricity and then to torsional effects which are produce if the center of rigidity doesn't coincide with the center of mass. The frame then turns about its rigidity instead of its center of mass. If this rotation appear, the weakest portions fail and the building might collapse. The more eccentricity, the greater the twisting or torsional effect on the frame building and, then lead to, the greater damage (Abd-el-rahim and Ferghaly, 2010).

The length to depth ratio is not depend on the dimensions of the rectangular storey plan but in the case of rectangular irregular figures the ratio of length to depth in every wing (for example long wings produce serious diaphragm deformations that lead to torsional effect). Under the impact of earthquake loads, each wing will possess different dynamic behaviour due to its particular stiffness and position relative to the direction of horizontal loads. The movement of the different portions of the frame can be very complicated, producing considerable torsional effects, diaphragm deformation and concentration of stress and strain at the vertices of reentrant corners (Abd-el-rahim and Ferghaly, 2010).

Evaluation of the performance of the frame buildings throughout earthquakes proposed that horizontal irregularities are one of the significant reasons of failure throughout appearance of an earthquake. Horizontal irregularity may appear because of irregular distribution of mass, strength and stiffness alongside the plan (Varadharajan, 2014).

2.5 One storey models

Previous investigations considered the torsional influences on horizontal irregular structure systems with single storey frames. The most important cause for selecting one storey frames was their simplicity. Those frames defined the effect of torsion on parameters of earthquake reaction and those outcomes were also developed design procedures for horizontal irregular structures. Nevertheless in current time multistory frames used to calculate the accurate inelastic torsional response of horizontal irregular structure systems. However, the use of multi-storey structure models is restricted due to the complexities, and it is one of the main causes that one storey frames are still favoured by several investigators. Earlier investigators on horizontal irregularities used one storey models generally concentrated on difference of locations of CS or CM with regard to each other to produce eccentricity. The essential purpose was to calculate the torsional reaction of the structure systems because of eccentricity. To produce eccentricity several investigators changed location of CR (center of rigidity) or CS keeping location of CM constant, the eccentricity produced was named as stiffness eccentricity (es).

Several investigators changed location of CM keeping location of CS as constant, the produced eccentricity was named as mass eccentricity (em) (Tso and Myslimaj 2003). Differing from earlier approaches several investigators generated variance in strengths of resisting parts to differ location of CV with regard to CM, and the produced eccentricity named as strength eccentricity (ev). The eccentricity descriptions defined as displayed in Figure 2.8 (Varadharajan, 2014).

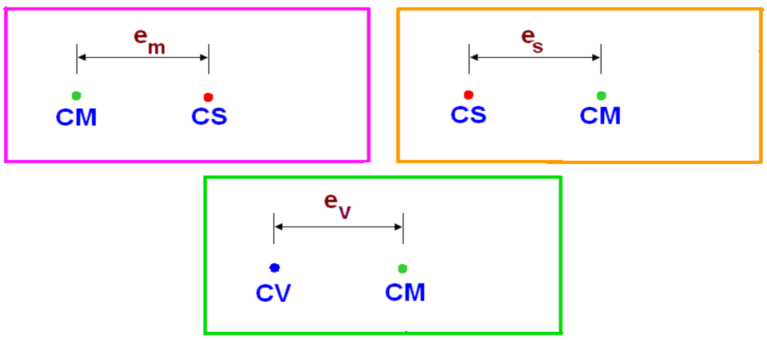


Figure 2.8 Eccentricity kinds: (a) Mass e_m, (b) Stiffness es, (c) Strength e_v (Varadharajan, 2014)

The horizontal component of seismic force shall normally prompt the lateral components of the motion and rotational component about a vertical axis, when the centers of resistance and mass of the buildings don't coincide. These structures will be denoted as possessed of eccentric structures. Furthermore, when every points of the base of the building aren't motivated at the same time, there could be torsional responses to earthquake loading stimulated in symmetric buildings, even in the purely translational component of ground irritation. This is due to the velocity of propagation of the ground irritation is finite. Essentially with respect to buildings that are torsionally flexible, meditations made of the influence of strong earthquake loading upon structures with eccentricity in centers of mass demonstrate that a large percentage of the destruction in this structures appears because of the nature of the reaction of lateral force-resisting parts situated at the structural edges. By an exemption, every prior outcomes of research on this subject are related with eccentric structures, with primary consideration specified to elastic response (Ayre, 1938).

Studies on horizontal irregular structures commenced in the early 1980's with Tso and Sadek (1985) who evaluated the difference in ductility demand by carrying out seismic inelastic response of single storey mass eccentric with stiffness degradation by the use of bi-linear hysteric model and Clough's stiffness degradation model.

Results noted that the time period had predominant influence on the demand of ductility following the elastic range. The rapprochement of outcomes demonstrated a 20 % between bilinear and Clough's model. Irregular distributions of stiffness and strength are the important reasons of damages during the ground motions. Tso and Bozorgnia (1986) estimate the seismic inelastic response of asymmetric plan structure models with stiffness and strength eccentricity (as shown in Table 2.1) using curves suggested by Tso and Dempsey (1980). Outcomes of the study demonstrated the effectiveness of the curves suggested by Tso and Dempsey (1980) excepting for torsionally stiff structures with low yield strength.

Tso and Sadek (1989) carry out inelastic analyses of mono-symmetric structure systems as shown in Table 2.1 with strength eccentricity. The center of strength was explained under name yield strength of resisting pats. It noted that the code explained eccentricities depended on stiffness criteria were valuable in expecting the seismic elastic response.

Pekau and Guimond (1990) tested the sufficiency of accidental eccentricity to compute for the torsion prompted because of the difference of stiffness and strength of the resisting parts which was approved by the use of elasto-plastic force-deformation relationship. Outcomes observed occurrence of torsional amplification as a result of stiffness and strength variation.

Table 2.1 Descriptions of different models adopted by (Tso and Sadek, 1989)

Sample No	Model Name	Description
1	M_c	Mass eccentric model with all three resistance elements having equal yield deformation
2	M_{c1}	Stiffness eccentric model with identical yield strength
3	M_{c2}	Stiffness eccentric model with identical yield deformation

Chandler and Hutchinson (1992) calculated the influences of torsional coupling on single storey stiffness eccentric structures. Outcomes was observed a strong reliance of torsional coupling influences on the natural time period of the building. Likewise, the effectiveness of torsional design requirements as stated by variant codes was determined by conducting inelastic and elastic analyses on single storey stiffness eccentric structure systems. Outcomes showed smaller displacement of stiff edge as compared to flexible edge.

Chandler and Hutchinson (1986) provide a detailed study of the coupled torsional and lateral response of a partially symmetric one storey structure model submitted to earthquake base loadings and steady state. It was noted that the specific influences of the controlling factors on the greatest torsional and translational responses of the coupled system weren't influenced by the nature of the loading. The peak edge lateral displacement of the structure increasing from the combined response influences was studied. The related shear lateral loads in vertical resisting parts placed on the periphery of the building could be meaningfully raised in evaluation with the corresponding amounts for a symmetric building. It is decided that for particular ranges of the key factors defining the structural system, typical of the properties of many actual structures, torsional coupling prompts an important amplification of earthquake loads which would be considered in their design.

Ferhi and Truman (1996) defined earthquake reaction of structure systems with the occurrence of strength and stiffness eccentricity. They were studied both inelastic and elastic behaviour. It noted that the seismic reaction demonstrated larger dependence on stiffness eccentricity in the elastic range. Furthermore, the effect of strength eccentricity on seismic reaction was noted to be in the inelastic range.

Chandler and Duan (1997) established an optimized technique for defining the seismic reaction of TB and TU structures. They were included factors such as force reduction factor (R), eccentricity (e), normalized radius of gyration (Pk) and uncoupled lateral period (Ty) in the suggested optimization procedure. The writers suggested design eccentricity and over strength factor expressions and evaluated it with code defined expressions. This study was performed on torsionally unbalanced and torsionally balanced. Results demonstrated that the over strength factor was found to be noticeably lower when evaluated to UBC-94 and NBCC-95 but greater than EC8 for entire range of Pk. The parameters e, R, pk, Ty were found to influence the seismic response.

De-La-Colina (1999) considered the influences of torsion on the simple TU systems studding the earthquake constituents in two perpendicular directions. The influences of the following factors were studied, (a) design eccentricity, (b) seismic load reduction factor, (c) natural time period. The structural model presented for the study is displayed in Figure 2.9. Depend on the outcomes of the investigation it was determined that, with a raise in the load reduction factor value, the ductility demand reduced for the flexible element. Concerning the influence of time period, it was noted that for (TU) stiff elements the ductility demand raised with the time period and vice versa for (TU) flexible elements. The increasing in the value of stiffness eccentricities reducing the normalized ductility demand. Depend on these outcomes it noted that stiffness eccentricity had smaller influence on seismic reaction when evaluated to strength eccentricity.

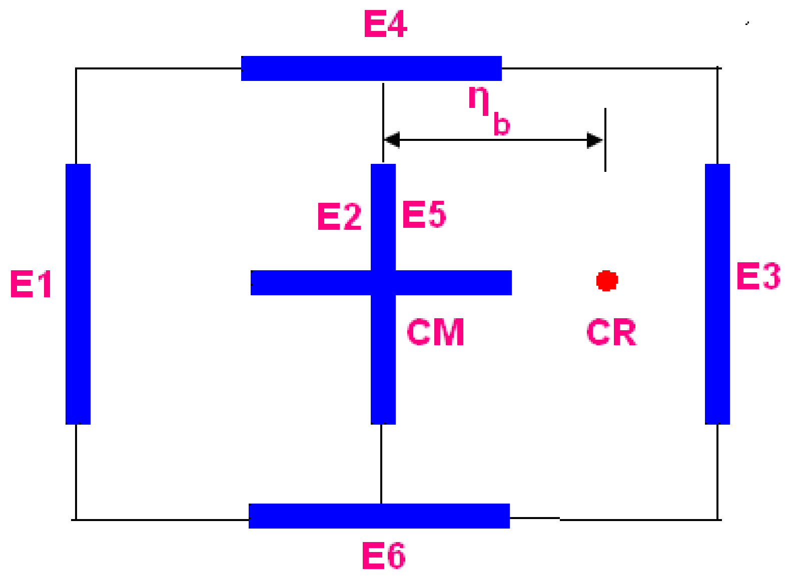
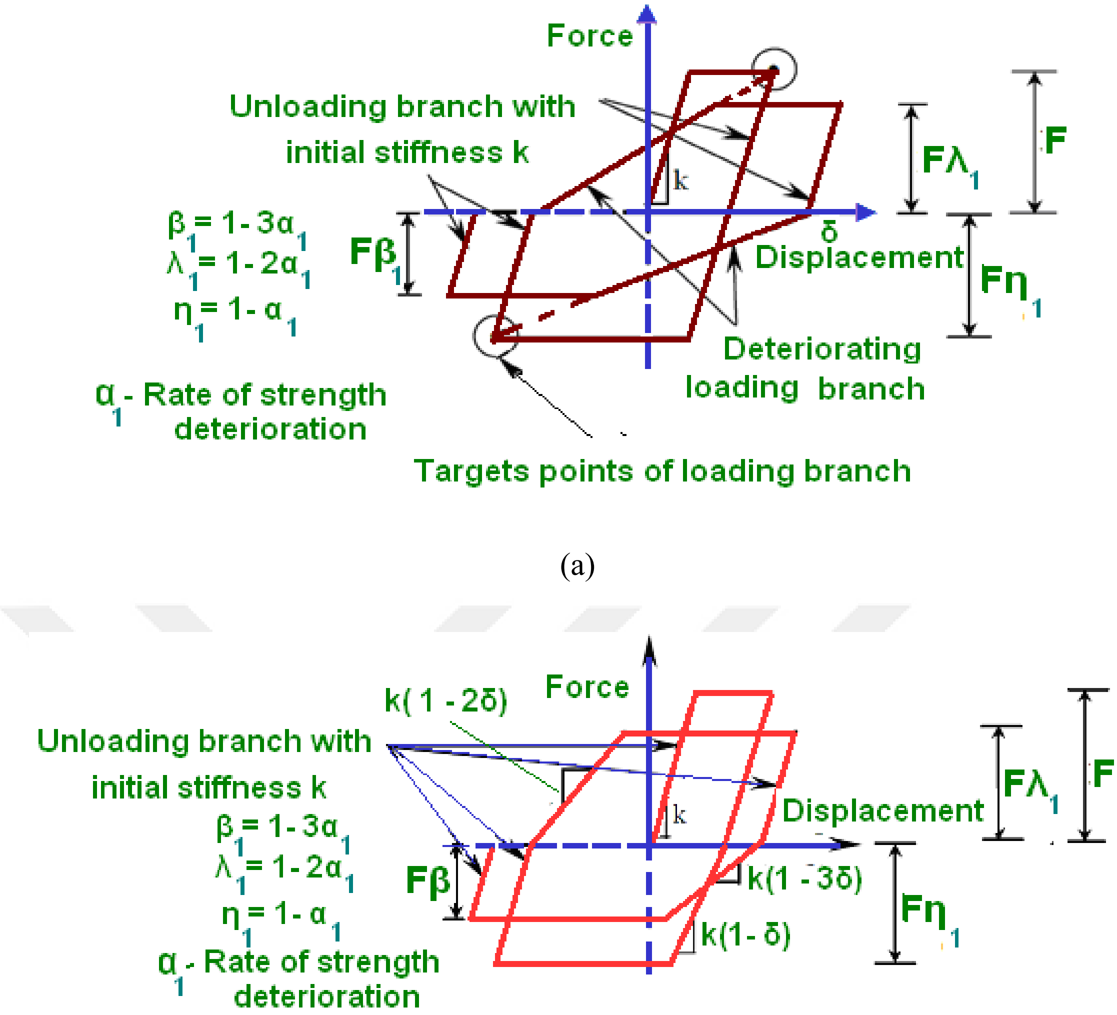


Figure 2.9 Structural model considered by (De-La colina, 1999)

Ghersi and Rossi (2001) calculated the effect of bi - directional seismic agitations on the seismic reaction of the stiffness eccentric of single storey structure systems by the use of inelastic and elastic analyses. The seismic reaction from the elastic analysis was evaluated with the outcomes of inelastic analysis.

Dutta (2001) studied the torsional inelastic behavior of the RC asymmetric buildings using single storey models. He noted these features may hugely increase the displacement and the ductility demand in the structural parts because of the successive unsymmetrical localized yielding and the progressive strength deterioration; causing in continuous shifting of the center of strength and raising strength eccentricity. Generally this amplification effect was noted to be raising with the proportion of strength deterioration.

Dutta and Das (2002) investigated the influences of strength degradation on the bi-directional. The seismic reaction of a one storey asymmetric plan structures exposed to bi - directional seismic agitations. They suggested two simple hysteresis models as characterized in Figure 2.10 (a, b**)**. Two models can account for stiffness and strength deterioration of the reinforced concrete structural parts exposed to cyclic loading. From the outcomes it noted that local buckle demands at flexible and stiff edge demonstrated difference when strength deterioration was taken into account. The deliberation of unidirectional seismic agitation obtained in the smaller amounts of buckle demands at flexible and stiff edge.



(a)

(b)

Figure 2.10 Hysteresis model proposed by Dutta and Das (2002)

Hao and Gong (2002) studied the inelastic response of single storey with two-way eccentricities and submitted to bi-directional spatial ground motion. For the analysis, twenty sets of bi-directional spatially changing horizontal ground motion time histories were mathematically simulated. The simulated movements were compatible individually with Newmark-Hall design response spectrum with 5% damping and normalized to 0.5g, and were compatible with an empirical coherency loss function between each other. Ensemble major responses of the system to 20 sets of earthquake motions are estimated. Influences of system factors like stiffness eccentricities in both directions, uncoupled torsional-to-lateral frequency ratios, the spatial ground movement wave passage influence, on coupled torsional-lateral inelastic reactions were studied. Outcomes were obtainable in dimensionless form also they were evaluated to the code torsional provisions.

Tso and Myslimaj (2003) suggested a new method named yield distribution based approach for the distribution of stiffness and strength. The writers modeled a one storey frame by a rigid rectangular deck reinforced by five resisting parts in Y direction and two resisting parts in X. The resisting parts were modeled by the use of elasto-plastic, the hysteresis bilinear and Clough‟s models for deformation – force relationship.

Myslimaj and Tso (2005) considered the problem of the resisting parts possessing strength-dependent stiffness in term of single storey below excitations of bi-directional. They determined a suitable distribution of stiffness, strength and mass to decrease torsional response finds the centers of stiffness and strength on the opposite sides of the center of mass, the situation denoted as 'balanced CV–CR position". They suggested two strength design techniques to realize the defined CV–CR position, the first depend on the use of the static equilibrium analysis. The other depend on a strength distribution equivalent to the yield displacements. An assessment with outcomes from code-designed systems (Figure 2.11) showed the effectiveness of the balanced location.

Fujii et al. (2004) proposed a simplified technique of non-linear analyses for asymmetric horizontal frames with stiffness eccentricity labeled SDOF‟s and MDOF‟s. Outcomes demonstrated that torsionally flexible systems showed smaller fluctuations in the first mode when evaluated with torsionally stiff structure.

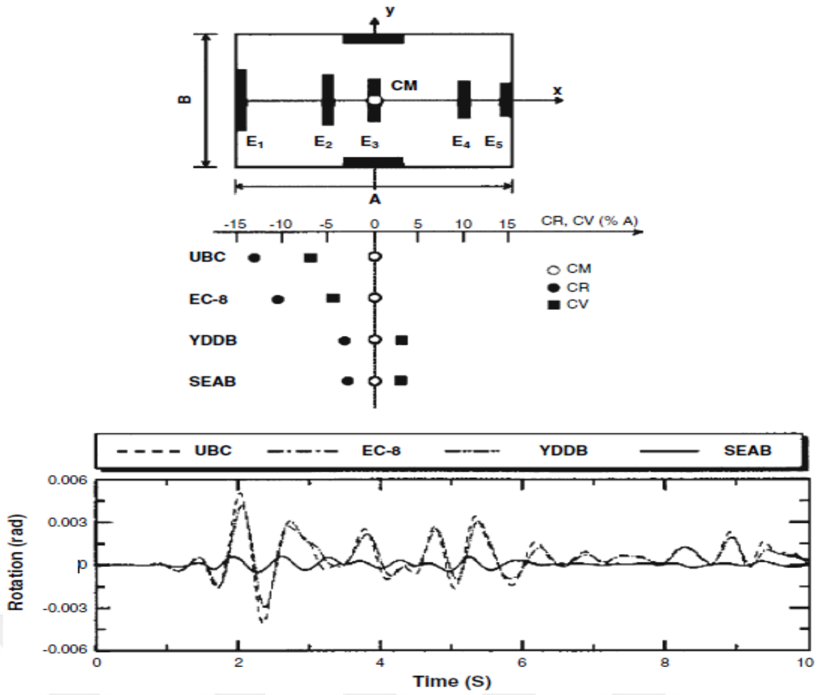


Figure 2.11 Evaluated four models of CR and CV positions and their deck rotation time histories under agitation of El Centro earthquake (Myslimaj and Tso, 2005)

Moghadam and Aziminejad (2005) achieved (Performance based design) of asymmetric buildings. The writers assessed the earthquake reaction of one storey frames with irregular arrangement for optimizing stiffness, strength and mass centers arrangements matching to variant levels of the plastic hinge creations. Writers accepted the idea of balanced CV - CR position suggested by (Tso and Myslimaj, 2003) to find the finest performance level of the frame. Depended on this investigation, they decided the finest position of CV - CR based on the damage indices and on the required performance level of the frame as shown in Table 2.2.

Table 2.2 Different locations of centers of strength, stiffness, mass and displacement for variant values of β (Moghadam and Aziminejad, 2005)

Sample NO	β	Position of C.M, C.V, C.D
1	*1*	Position of CV concide with CD, strength disribution takes same shape as yield displacement
2	*0-1*	Value of e_v decreases position of CV starts shifting from CD towards CM.
3	0	Position of CV coincides with CM and position of CR is shifted towards left of C.M at a distance equal to e_d
4	<1	CR and CV shift towards left of CM.

Ghobarah et al. (2005) studied the problem of non-structural components and critical installations or equipment attached to a mass eccentric primary system, by the use of a shake table (small-scale).

Shakib and Ghasemi (2007) calculated the influence of far fault and near fault excitations on earthquake reaction of variant kind of horizontal asymmetric frames with stiffness asymmetry. Depending (Tso and Myslimaj, 2003) who proposed balanced CV-CR position to reduce rotational buckles, the writers proposed a new tactic to reduce rotational buckles.

Pettinga et al. (2005) studied a one-storey frame underneath bi- and uni- directional seismic agitations. The purpose was to expand to 3D asymmetric plan structures the suggested performance based-design frame work that confirm the importance of evaluating and limiting residual distortions. The parametric analysis were conducted on the mass eccentric systems with the purpose of determine the major parameters effecting the residual distortions/displacements. Various plans in the plan, resulting to torsionally unrestrained or restrained systems, depending on Paulay's definition were studied. Opposing to anticipations, the results proposed that systems with little torsional restrain enhanced residual rotation behaviour.

Jarernprasert et al. (2008) investigated the torsional inelastic reaction of the one storey asymmetric plan frames with stiffness eccentricity calculated according to Mexico city code 2004 and the IBC 2006 code. The influence of seismic excitation on following factors was considered, (a) design target ductility, (b) ratio of uncoupled transitional to torsional frequencies, (c) normalized static eccentricity and elastic natural time period.

Trombetti and Conte (2005) advanced a technique (named the ALPHA method) to determining the extreme rotational response underneath forced and free vibrations of single storey elastic linear systems. The analytical capability of the procedure, originally designed for seismic asymmetric isolated frames, had confirmed consecutively if subjected to systems characteristic of the generic asymmetric plan frames (Pintucchi et al., 2005).

Ladinovic (2008) demonstrated seismic inelastic reaction of the asymmetric plan frames with strength and stiffness eccentricity in the mode of the BST surface. The reasons effecting BST surface were torsional and lateral capacities, strength eccentricity and the distribution of the strength along the plan.

The influences of the orthogonal components of earthquake was assess by the use of a linear single-storey bi-asymmetric system by (Heredia-Zavoni and Machicao-Barrionuevo, 2004). It was noted that this influences very variously with the translational natural period, counting on when the system was torsionally stiff or flexible, in addition to soil situations. Particularly, the implementation of a bi-directional inputs could significantly influence on the response of the stiff torsionally systems with high translational periods on the soft soils, whilst on the firm soils, reciprocally, it become significant for the flexible torsionally systems with the small translational periods. But, it would be noted that, although the suggestions of some prior investigations, the influences of earthquakes orthogonal component don't appear to be actually substantial, particularly when orthogonal parts are involved in the mode.

Aziminejad and Moghadam (2010) calculated the conformation of rigidity, mass and strength and the influences of the strength distribution on the earthquake reaction of single storey asymmetric plan structure submitted to far and near field ground movements. Various models amounts of yield displacement, stiffness and strength eccentricity were studied as displayed in Figure 2.12. Nonlinear dynamic technique were performed on the models and from the outcomes observed for the flexible torsionally frames, the strength distribution had slight influence for both far and near field excitations. However, earthquake reaction of stiff torsionally frame systems was hugely affected by the strength distribution. It was observed from the modal periods the modal periods along the X-axis had the peak amount when evaluated with two other modal periods and proportion of torsional to lateral frequency was observed to be larger in y direction. Furthermore, they concluded the stiff torsionally frame systems with balanced CV-CR position implemented better than the structure models in the situation of far and near field agitations.

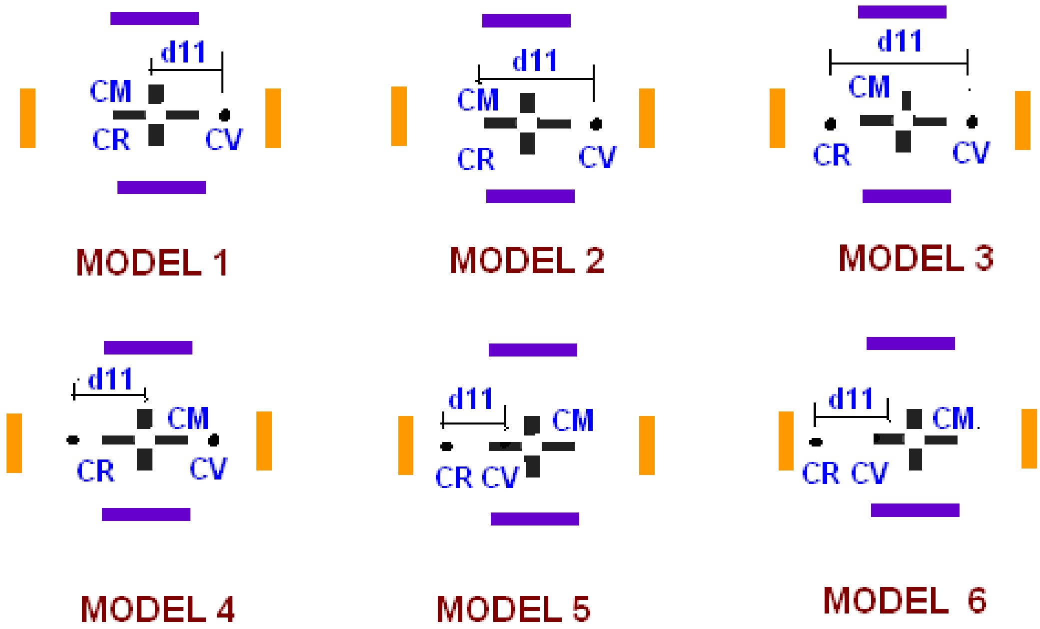
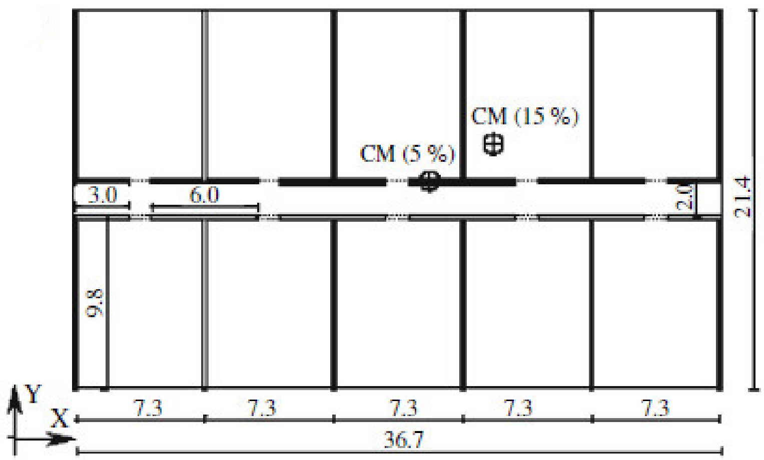


Figure 2.12 Models considered by (Aziminejad and Moghadam, 2010)

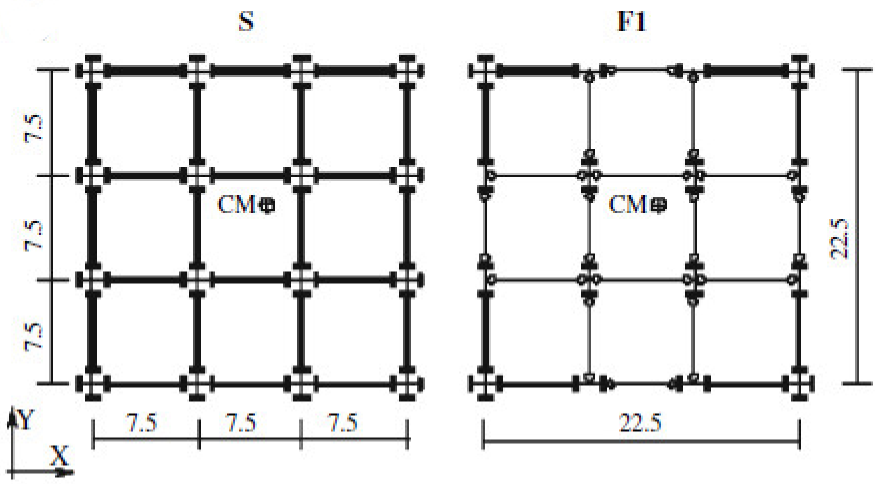
2.6 Multi-story plan asymmetric structures

In the studies of horizontal irregular buildings, one storey models were used because of their capability to describe the influence of variant earthquake reaction parameters and their simplicity. Major of the design standards were determined on the basis of results gained in one-storey models (Varadharajan, 2014). One-storey models characterize the best magnification of horizontal irregular structures, they had commonly used in the past because of their ability of explaining the effect of the governing factors and determine real design criteria. Furthermore, in recent time multistory frames widely used for two purposes: (1) inadequacies of single-storey frames in expecting the torsional behaviour of actual buildings had approved by some researchers, like (Stathopoulos and Anagnostopoulos, 2002; 2003), whose analytically researched the effectiveness of this frames; (2) advancement of software instruments is making modeling and analysis of three dimension multistory structure models more simpler (De Stefano and Pintucchi, 2008).

Multistory structure models provide a realistic expectation of torsional response. Most studies applied on multistory structures were directed to extending pushover analysis to plan asymmetric structures. Even though investigations on horizontal irregular structures established in 1990's (Killar and Fajfar, 2002; Moghadam, 1998) but Fajfar et al., (2002) was the important investigator who suggested a new technique. It was an extension of N2 technique. The suggested technique was usable to the realistic three dimensional structures by applying a height-wise distribution of the lateral force to the floor center of mass. The technique, originally designed for planner tow dimensional frames, involves a non-linear tactic that creates utilize of pushover analyses, inelastic response spectrum and equivalent SDOF system. The efficiency of the extended technique was confirmed by considering multistory frame steel structures and multistory reinforce concrete structures with structural walls (see Figure 2.13).



(a)



(b)

Figure 2.13 (a) Floor plan of eight storey reinforced concrete wall structure and (b) plans of floor of the analyzed five storey steel moment-frame structures (Fajfar et al., 2002)

The eight storey RC wall building and five storey steel moment-frame structures (S and F1) bi-axial asymmetry presented by supposing mass eccentricity equivalent to 15% plan dimensions (De Stefano and Pintucchi, 2008). The mass eccentricity determined in the RC wall structure by moving center of mass in each horizontal directions by 5% and 15%, respectively**.** The outcomes were evaluated with the non-linear time history analyses. The capability of suggested technique to anticipate the seismic behavior of torsionally stiff building was satisfied. Furthermore, the technique didn't cover the influences of the torsional lateral coupling. It was noted to be un-conservative once compared to N2 technique (Varadharajan, 2014).

De-la-Colina (2003), evaluated some code suggested techniques concerning analyses of methods for multi-storey buildings with irregularity of stiffness and mass submitted to (EI Centro earthquake) bi - directional seismic excitation. For analytical studies, some 5 storey structures having stiffness and mass eccentricity were carried out. To represent resisting parts, shear beam models were assumed by authors. Depended on the code designed techniques, seven static design procedures were evaluated, the researcher found the optimum amounts of the eccentricity in each story.

Chopra and Goel (2004) suggested a new technique depended on extension of their previous technique. They sought to extend the modal pushover analysis. The technique suggested the torsional expansion of the building was determined by implementation of the torsional moments in combination with the lateral loads at every floor of the building. The torsional moments and lateral loads were gained from the modal analyses of the building. Evaluation between the outcomes of expectations from the suggested technique and precise values calculated by non-linear time history analyses was made for 4 buildings with variant values of the proportion of periods of uncoupled torsional to lateral vibration. The efficiency of suggested method for symmetric buildings was confirmed. Furthermore, the accuracy of suggested method declines with the rise in quantity of torsional coupling because of the use of complete quadratic combination rule for modal combination.

Fajfar et al., (2005) suggested a new technique depend on N2 technique. New technique suggested combination responses of modal got from nonlinear static analyses of three dimension structural models with the outcomes got from dynamic linear analyses. The buckles and displacements distributions along the height of the building organized by N2 procedure and the amount of torsional expansion of lateral displacement was determined by the linear dynamic analysis. The procedure of dynamic linear analysis has satisfied by the supposition that, on the flexible edge, the elastic envelope of lateral displacements was conservative regarding the inelastic envelope (Peruš and Fajfar, 2005).

Stathopoulos and Anagnostopoulos (2005) tried to estimate torsional reaction of realistic three dimensional buildings by nonlinear analyses. They studied the inelastic seismic response of eccentric multistory structures. The researchers performed analytical studies on realistic three storeyed and five storeyed framed structures models with stiff and flexible edges submitted to bi - directional excitations. It was found from the outcomes gained from multistory buildings that inelastic displacement was smaller at stiff side when compared to flexible side. Furthermore, the outcomes attained in one storey frames were differing to the outcomes gained in multistory buildings with mass irregularity below the action of bi - directional seismic agitations. It was noted that the torsionally flexible structures undergo more plastic deformation when compared to the torsionally stiff structures.

Penelis and Kappos (2005) studied the torsional inelastic response of structures by using pushover static nonlinear analysis. They suggested a technique to estimate the plan asymmetric (inelastic torsional reaction) of single and multi-storey frames. The frames used for investigations were SDOF systems and combined the influences of torsional and translational modes. Suggested technique was confirmed for several cases: two multistory and two one storey mono-symmetric structures. Spectral force vectors in the suggested technique were gained from elastic spectral analyses and those force vectors were enforced on the frame to perform three dimension pushover procedure. The outcomes of the suggested technique were evaluated with outcome of dynamic nonlinear analyses. It has been determined the inelastic earthquake response gained by both techniques differ by 20% in multi-storey case and by 10% in one storey case.

The inelastic and elastic response of mass-asymmetric of five multistory steel buildings structures with mass eccentricity were considered by (Marusic and Fajfar, 2005). The eccentricities had been determined as 5%, 10% and 15% of the plan dimensions. Three model kinds were assumed for this study as shown in Table 2.3. To modeling the structure the height of first storey was 4 m and heights of other storey equal to 3.5 m. The multi-storey building was exposed to bi - directional ground motions. The outcomes gained at flexible edges were virtually comparable to (Perus and Fajfar, 2005).

More emphasis on impossibility of directly expanding outcomes from single-storey to multistory models coming from Stefano et al. (2006), they calculated the variance of inelastic earthquake reaction of one story and multi-storey plan asymmetric frames. One storey and a 6 storey steel structures with mass enforced at 0.15 b of the geometric frame producing mass eccentricity formed in the structure. Influence of over strength of the resisting parts was assessed. The effect of over-strength was described on the demand ductility of the structures and this effect described difference for one story and multi-storey. Lastly it was determined this earthquake response gained from one storey was different from that gained from multi-storey models.

Table 2.3 Models description used by (Marusic and Fajfar, 2005)

Model Name	Description
S	Torsionally stiff building model with moment resistant beam column connections (All beam-column connections).
F_1	Building Model with torsional stiffness equal to Model S with moment resistant beam column connections (Corner beams only)
F_2	Building Model with torsional stiffness less than Model S and F1.

De la Llera and Chopra (1996) considered the seismic inelastic performance and design of multi-storey asymmetric frames confirming the using of storey torque and shear histories. Six structural properties and their impact on the torsional response of structures were studied: strength asymmetry, stiffness asymmetry, strength of resisting orthogonal planes, number of resisting planes, plan wise distribution of strength, and concentration of component of ground motion in orthogonal direction. The outcome of the studies, some procedures and conceptual rules were established to rectify the plan wise unbalance in the demands deformation of asymmetric buildings. The most two significant guidelines were to rise the torsional magnitude of the system by producing resisting planes in orthogonal direction, and to justify the distribution of strength and stiffness to determine yielding in specified resisting planes.

Sommer and Bachmann (2005) studied the earthquake response of asymmetric plan multi-storey structures, stiffened by ductile reinforced concrete structural walls. They concentrated on the important subjects that would be related to: (1) a realistic plan wise distribution of the strength depending on real reinforced concrete wall characteristics, in expression of reinforcement ratio, always certainly not fulfils a criterion for the optimal position of the strength center with regard to the mass and stiffness centers. (2) The stiffness of element isn't independent of strength (Figure 2.14). So, a new distribution of stiffness and strength criterion was assumed with the purpose of getting steady distribution of demands ductility with the accurate reinforcement magnitudes in resisting parts.

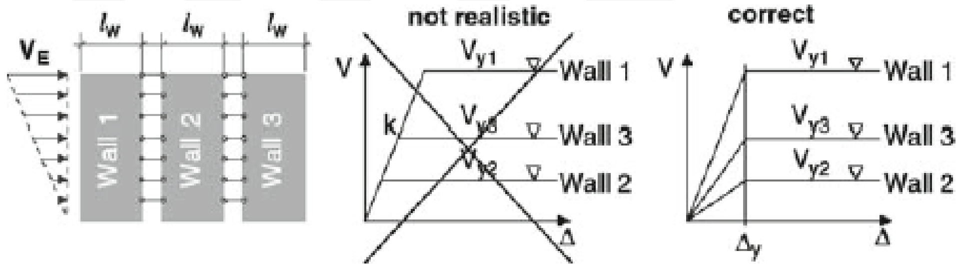


Figure 2.14 Relationship between displacement-force of reinforced concrete walls used by (Sommer and Bachmann, 2005)

Ghersi and Rossi (2006) assessed the analyses of modal effectiveness by calculating inelastic seismic reaction of multi-storey plan asymmetric frame. Six storey asymmetric steel structure, asymmetry defined by differentiate of providing force at 0.15L to the geometric center producing mass eccentricity. Outcomes were compared with that of Chandler method and static analysis procedure to check accuracy of the latter. The suggested method yielded good seismic performance when compared to other methods.

Fernandez-Davila and Cruz (2006) considered the effect of some factors on earthquake reaction using 3D five storey frames, that could be investigate direct extensions of single-storey frames. The influences of the following system factors were considered: degree of torsional coupling, amount of the resistant planes parallel to the earthquake motion, static normalized eccentricity, proportion of lateral uncoupled frequencies, proportion of torsional stiffnesses, fundamental uncoupled vibration period, and overall the design ductility.

Mario et al. (2006) studied influences of the over strength in the cross sections of the elements on the seismic conduct of multistory asymmetric structures. They displayed in real structures this feature, that is occasionally much variable along the height and in plan of the frame could directed to distributions of the ductility demands variant from those anticipated depending on the outcomes of one storey frames. Therefore, torsional requirements, that their purpose at decreasing the ductility demands of the asymmetric one-storey frames to those of matching balanced torsionally systems, must be re-tested according to the behavior of realistic multistory frames.

Aziminejad and Moghadam (2009) studied performance of 8 buildings five storey asymmetric plan frames with variant strength distributions under earthquake action. The 8 building variant in position of location of center of strength and rigidity (Table 2.4) were studied. These models were analyzed using dynamic nonlinear analysis using OPENSEES program. Results were calculated that structure systems with strength eccentricity equivalent to (0.25) of displacement between locations of stiffness and strength achieved better on drift and rotation criteria.

Table 2.4 Different model configurations considered by Aziminejad and Moghadam (2009)

Sample No	Model Name	Ratio of stiffness to yield displacement eccentricity
1	Symmetric	0
2	Stiffness Symmetric	1
3	Balance (0.75 CV – CR)	0.75
4	Balance (0.5 CV – CR)	0.5
5	Balance (0.25 CV – CR)	0.25
6	6 Strength Symmetric 0	0
7	De-Stefano (0.25 CM-CR)	-0.33
8	De-Stefano (0.5 CM-CR)	-1

Stathopoulos and Anangnopoulos (2010) assessed the effectiveness of the accidental eccentricity requirements. They created four kinds of building models. The first model was single storey shear beam with stiffness eccentricity and second model was single storey frame model with mass eccentricity. Third sample and fourth sample were three and five storey frame kind building with mass and stiffness combination asymmetry along plan.

Anagnostopoulos et al. (2010) calculated torsional inelastic response of one storey and multistory structure samples with stiffness and mass eccentricity. The structure samples were considered in accordance with EC8 and IBC codes.

2.7 Nonlinear analysis

Although buildings are generally designed for earthquake resistance by means of elastic analysis, they will practice major inelastic distortions under huge seismic motion. Present behaviour-based design approaches need techniques to calculate the accurate performance of buildings under those circumstances. Allowed by improvements in computer programs and existing experiment data, nonlinear analysis supply the ability to calculate structural response behind the elastic range, counting strength and stiffness decline related with inelastic material conduct and high drift. Like this, nonlinear analysis can have a significant part in the design of new and existing structure (Deierlein et al., 2010).

2.7.1 Nonlinear static analysis

The effective inelastic procedures of analysis are anticipated to be obtainable to confirm the selected intervention of the structural rehabilitation. The linear method used for the design of the new buildings isn't appropriate for this purpose since the plastic collapse mechanism of the existing buildings, and therefore the behaviour factor, isn't known a priori. Due to this, this technique of analysis doesn't give a reliable prediction of the response of the existing structures. The nonlinear static procedure represents a fair compromise between the nonlinear dynamic analysis and the elastic method of analysis. Particularly, it means the use of the response spectra and needs that modelling efforts be concentrated only on the monotonic nonlinear behaviour of the structural members (Bosco et al., 2015).

The pushover static technique has no exact theoretical base. It is generally depend on the presumption that the frame response is controlled by the first few modes of vibration, or by the first mode of vibration with mode shape, and this shape remains constant through the inelastic and elastic response of the structure. This gives the basis for the transforming a dynamic problem to a static problem that is theoretically flawed. Moreover, the response of a MDOF model is connected to the response of an equivalent SDOF model, ESDOF. This concept is showed in Figure 2.15 (Themelis, 2008).

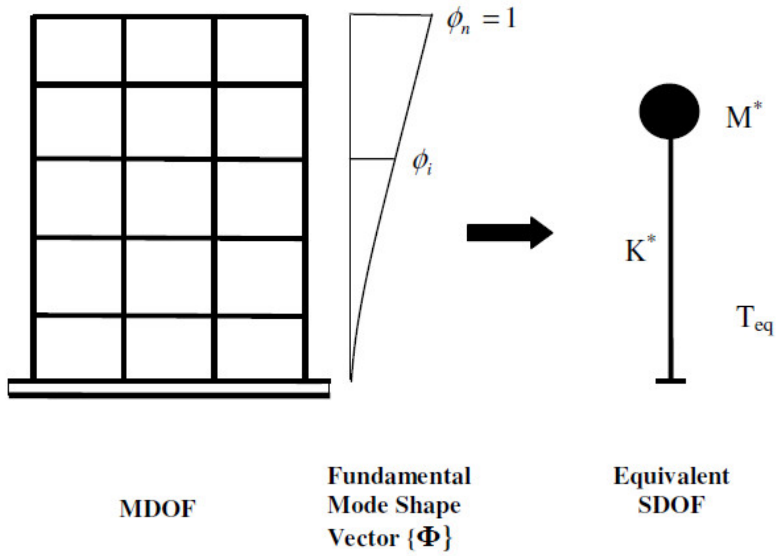


Figure 2.15 Conceptual diagram for transformation of MDOF to SDOF system (Themelis, 2008)

The earthquake prompted motion of an inelastic or elastic MDOF can be defined from its governing differential equation (Themelis, 2008):

$$[M]\{\ddot{U}\}+[C]\{\dot{U}\}+\{F\}=-[M]\{1\}\,\ddot{u}_g \qquad (2.1)$$

Where [M] and [C] represent mass and damping matrixes, {F} denotes the storey force vector, {1} is an influence vector characterizing the displacements of the masses once a unit ground displacement is statically applied, and $\ddot{u}g$ is the ground acceleration.

By supposing a single shape vector, {Φ}, which isn't a function of time and describing a relative displacement vector, U, of the MDOF as $U = \{\Phi\}u_t$, where u_t

represents the roof/top displacement, the differential equation of the MDOF will be converted to (Themelis, 2008):

$$[M]\{\Phi\}\ddot{u}_t + [C]\{\Phi\}\acute{u}_t + \{F\} = -[M]\{1\}\ddot{u}_g \tag{2.2}$$

It is usually accepted that nonlinear earthquake analysis provide more precise results than response spectrum analysis. Nonlinear static procedure is depend on a static nonlinear analysis named pushover analysis. In this technique, a monotonic force (displacements or forces) charactering the equivalent seismic action, with a stable or adaptable pattern, is gradually experienced in the structure. This analysis must be contain the gravity loads. The result of the pushover analysis is named capacity curve (pushover curve), which explains the differing of the base shear impedance (V) with regard to the highest point(roof) drift (displacement) (D) in a chosen controlled node, as shown in Figure 2.16. This curve provides significant information about the total strength and buckling capacity of the structure under analysis (Bhatt, 2012).

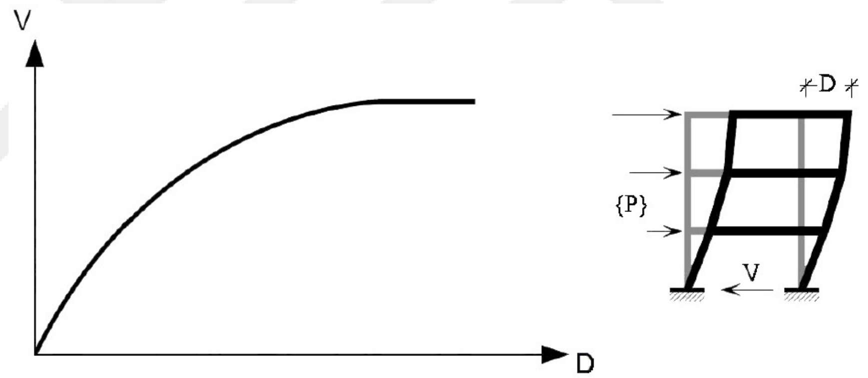


Figure 2.16 Pushover curve of the MDOF model (Bhatt, 2012)

Afterwards, the capacity curve of the (MDOF) is transformed into a capacity curve of an equivalent (SDOF), Figure 2.17 (Bhatt, 2012).

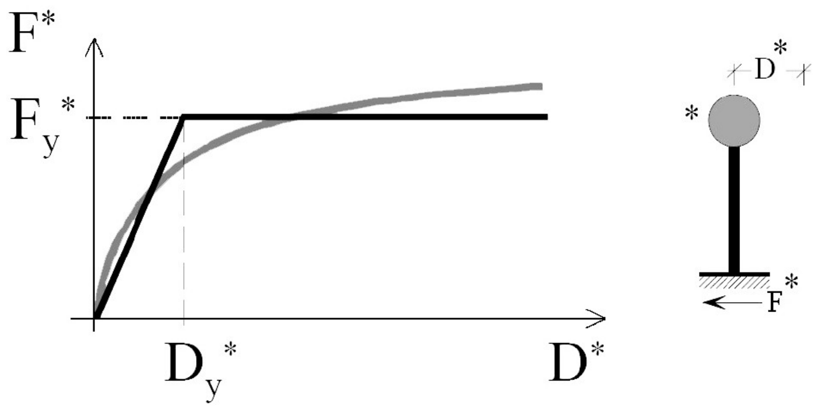


Figure 2.17 Pushover curve of the equivalent SDOF system (Bhatt, 2012)

The target displacement (Dt*) or inelastic displacement of the equivalent SDOF is determined by interconnecting its adaptive capacity curve to the elastic response spectrum reduced (in the format of acceleration-displacement) matching to the seismic action examined, Figure 2.18. The intersection point is named the performance point, and it matches to the inelastic acceleration and to the target displacement of the equivalent SDOF (Bhatt and Bento, 2014).

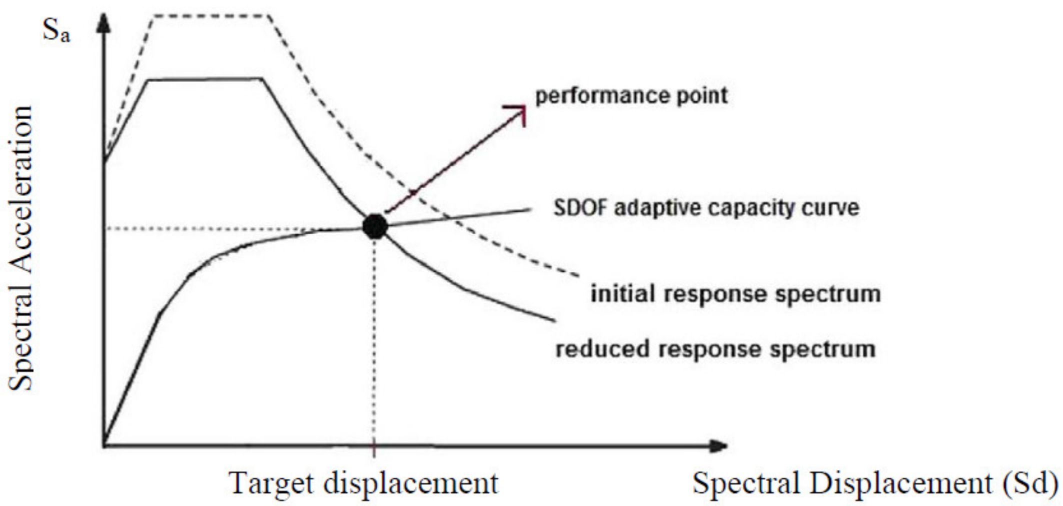


Figure 2.18 Calculation of the SDOF target displacement (Bhatt and Binto, 2014)

Inelastic displacement of the controlled node (*Dt*) is gained by creating the correspondence to the target displacement of the SDOF system to the MDOF. In order to gain the maximum inelastic deformations of the individual structural elements, like interstorey drifts or chord rotations, one must go back to the MDOF capacity curve step matching to the controlled node inelastic displacement previously determined, and pick the results in the required elements (Figure 2.19). The certain features of every of the previously mentioned steps based on the method used (Bhatt, 2012).

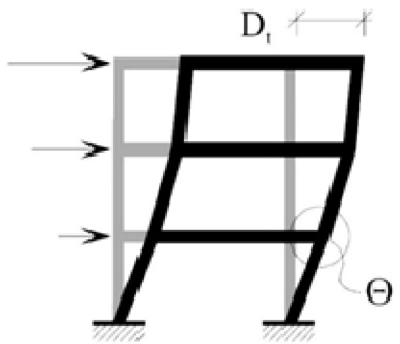


Figure 2.19 MDOF results corresponding to the SDOF target displacement (Bhatt, 2012)

The procedures of the nonlinear static can be categorized as displacement-based estimation methods for the valuation and restoration of existing frame structures. Furthermore, these methods can be performed simultaneously with displacement-based design methods for the design of new buildings. Actually, to carry out a pushover analyses it is necessary to improve a nonlinear model of the building, which contains the nonlinear formulation of the material relationships. The reinforcement in the elements should be correctly defined in the case of RC buildings. Therefore, in new buildings one would perform an initial design by the use of displacement-based design methods, and then check the suitability criteria by applying a nonlinear static analysis. If these standard specifications aren't verified, a new design must be implemented and a new confirmation should be done. This repeated process ends, when every desired criteria are checked. The process defined in this passage corresponds to the ideal seismic design procedure. Despite the encouraging results gained in some scientific studies, one would be aware that the NSPs have an intuitive basis instead of a pure mathematical basis (Bhatt, 2012).

2.7.2 Purpose of pushover analysis

It is anticipated that pushover give information on several response characteristics can't be attained from an elastic dynamic or static analyses. The following are the examples of this response properties (Govind et al., 2014).

- Assessments of inter story drifts and its distribution along the height of the structure.
- Estimation of load demands on brittle parts, for example moment demands on beam-column connections, axial force demands on columns.
- The capacity of the structure as characterized by the roof- displacement vs. base shear graph.
- Estimation of deformation demands for the ductile members.
- Maximum rotation and ductility of the critical members.
- Determination of position of the weak points in the building.
- Penalties of strength weakening of the individual parts on the behavior of the structural system.
- Confirmation of the adequacy and completeness of the load path.
- Determination of strength discontinuities in elevation or plan which would lead to changes in the dynamic properties in the inelastic range.
- To evaluate the structural performance of the existing or retrofitted structures.

2.7.3 Nonlinear time history analysis

The nonlinear dynamic technique considers obviously the inelastic buckle of the structural parts and can expect effectively the earthquake response of the existing buildings. But, the difficulties of modelling of both the cyclic nonlinear behavior of the structural parts and the seismic ground motion create this method not advised for everyday use (Bosco et al., 2015).

It is a technique to assess the dynamic reaction of the building at every rise of time, if its base is endangered a definite seismic motion time history. The dynamic nonlinear time-history analyses is mostly recognized as being the most correct method for the seismic evaluation/design of structures. This technique overcomes all the problems related with the response spectrum analyses (RSA). Each structural element properties are properly modelled, containing nonlinearities of the materials, with the analyses solution being calculated through a mathematical step by step integration of the equilibrium equation, (Bhatt, 2012).

$$M\{ar(t)\}+C\{vr(t)\}+K\{dr(t)\}=-M\{ag(t)\} \tag{2.3}$$

Where *M, C and K* characterize the mass, damping and stiffness matrixes, respectively. The terms $\{(t)\}$, $\{(t)\}$ and $\{d(t)\}$ are the relative acceleration, velocity and displacement vectors, respectively, and $\{ag(t)\}$ is the ground acceleration. Consequently, it permits the estimation of the dynamic response of the building over time, involving local and global responses. This fact prevents the use of behavior factors and their fallacious influences, then they may not account in a right way for the structural ductility (Bhatt, 2012).

2.7.4 Nonlinear Static vs. Nonlinear Dynamic Analysis

Dynamic nonlinear analysis procedures normally give further accuracy to models of structural response in the case of strong earthquakes and, thus, offer further consistent calculation of ground motion effects than static nonlinear analysis. Static nonlinear analyses is restricted in its capability to capture transient dynamic behaviour with repeated loading. However, the static nonlinear process give a suitable and also objectively dependable technique for buildings whose dynamic response is ruled by first-mode sway movements. This can be check by associating the distorted geometry from a pushover analyses to the elastic first-mode vibration form. Generally, the static nonlinear method performs fine for low-rise structures (less than about five stories) with symmetrical regular configurations. NIST (2010), FEMA 440 and FEMA 440A provided additional information on the simplifying rules and limits on nonlinear static analysis. Conversely, even if the static nonlinear method isn't suitable for a complete performance assessment, static nonlinear analysis can be an active design procedure to consider features of the model analyses and the nonlinear response that are difficult to done by nonlinear dynamic analysis. For instance, nonlinear static analysis can be useful to (Deierlein et al., 2010):

(1) Examining and correcting the nonlinear analyses model.

(2) Expand the knowledge of the yielding process and buckling demand.

(3) Examine different design limitations and how differences in the component properties may affect response.

CHAPTER 3

METHODOLOGY

3.1 Analytical models of structures

In present work, twenty four reinforced concrete (RC) structure of eight stories were compared to control structure of square dimension, with same stories that used for studied structure. The studies structure are divided in to six groups, the groups divided dependent on their spans layout, in the first group look like a cross section. C, O, H, ⊥, L and Z shapes. The column footings were considered as fixed for all models. These frames were designed as six bays on each direction and total length was equal to 36 m in each direction. The height of each storey is 3 m except the first storey which is 5 m which means the total height of the buildings equal to 26 m. Typical floor plan, three dimensional view, and planes of the case study regular and irregular RC buildings are shown in Figures 3.1 to 3.9.

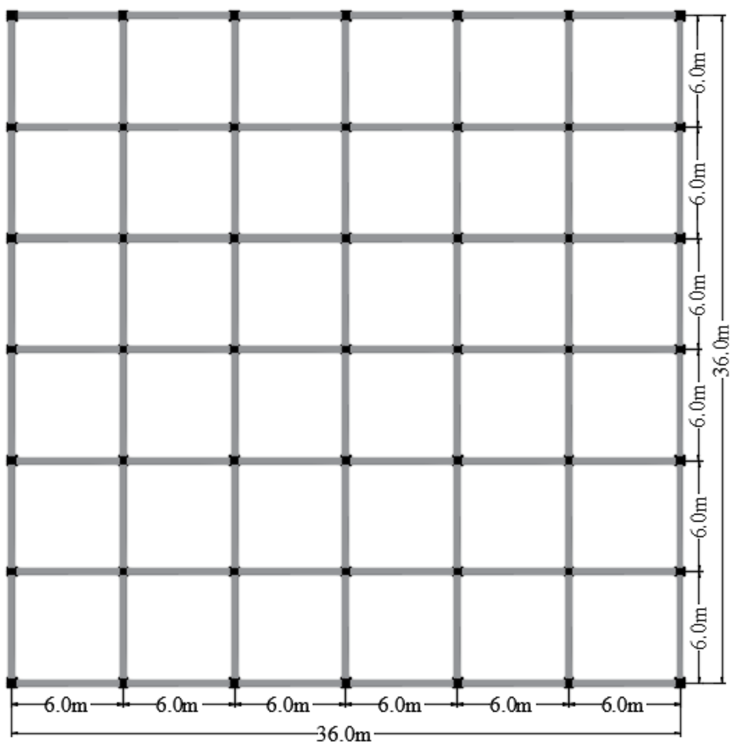


Figure 3.1 Plan view of a regular model eight story reinforced concrete building

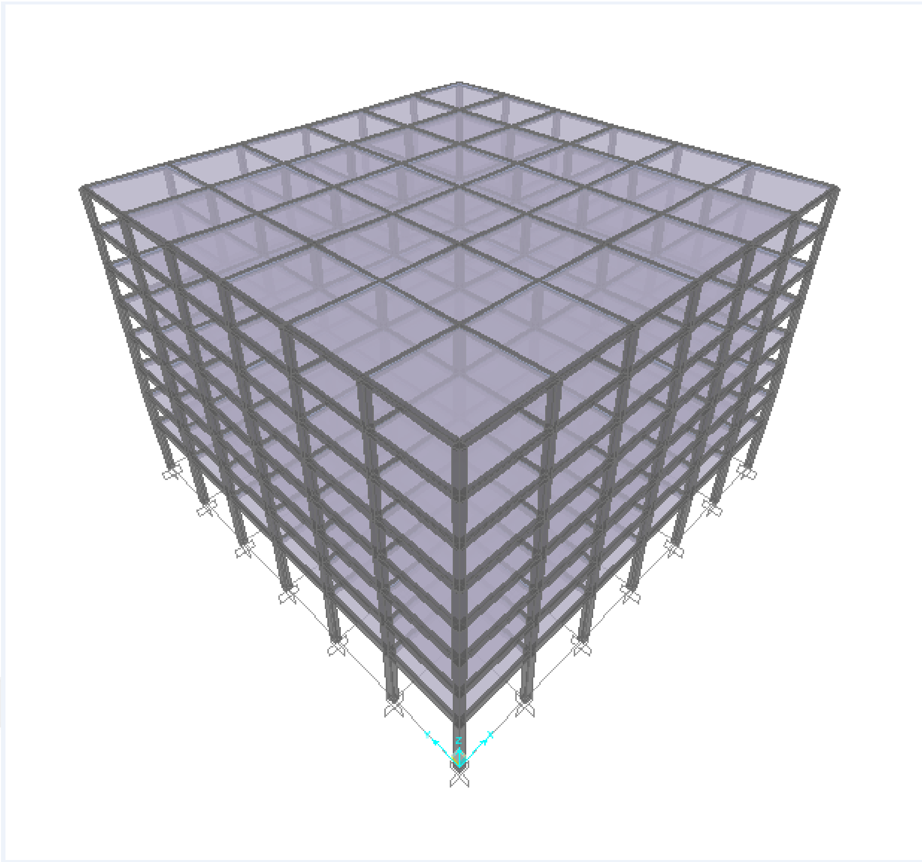


Figure 3.2 3D view of a regular model - eight story reinforced concrete building

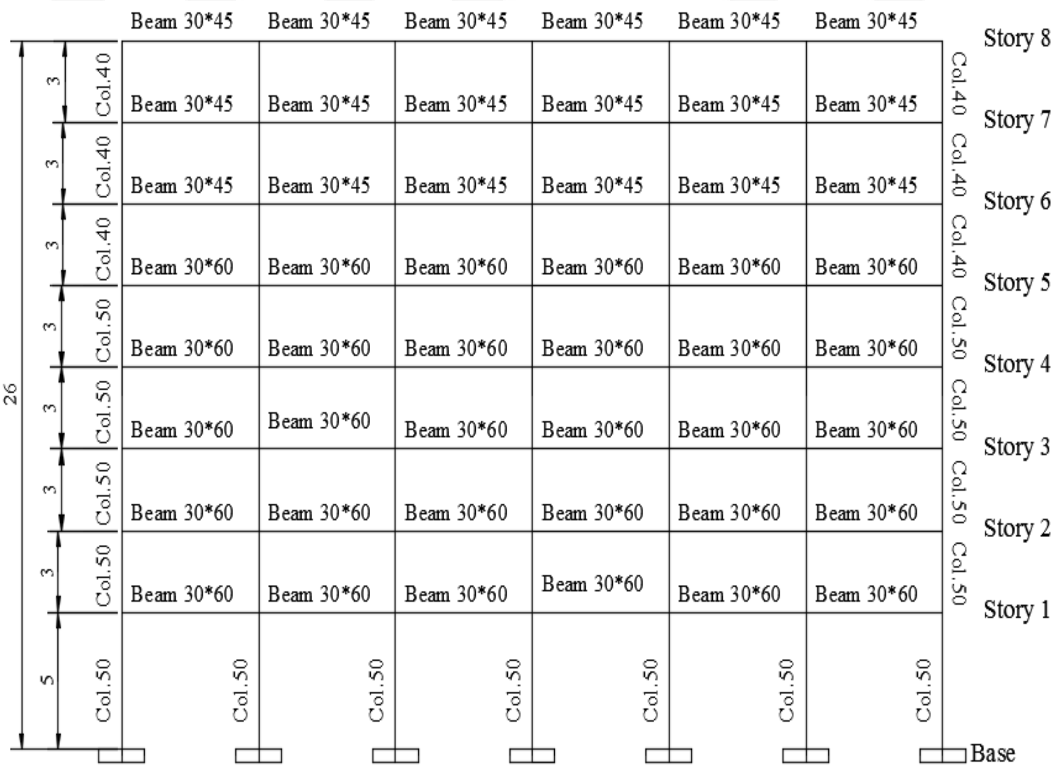


Figure 3.3 Frame elevation of a regular reinforced concrete building

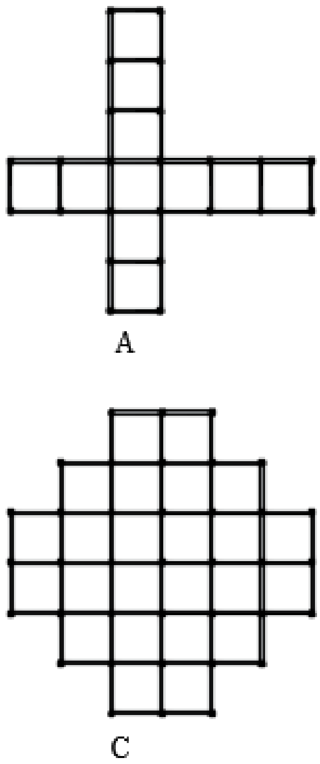


Figure 3.4 Plan view of + section irregular models

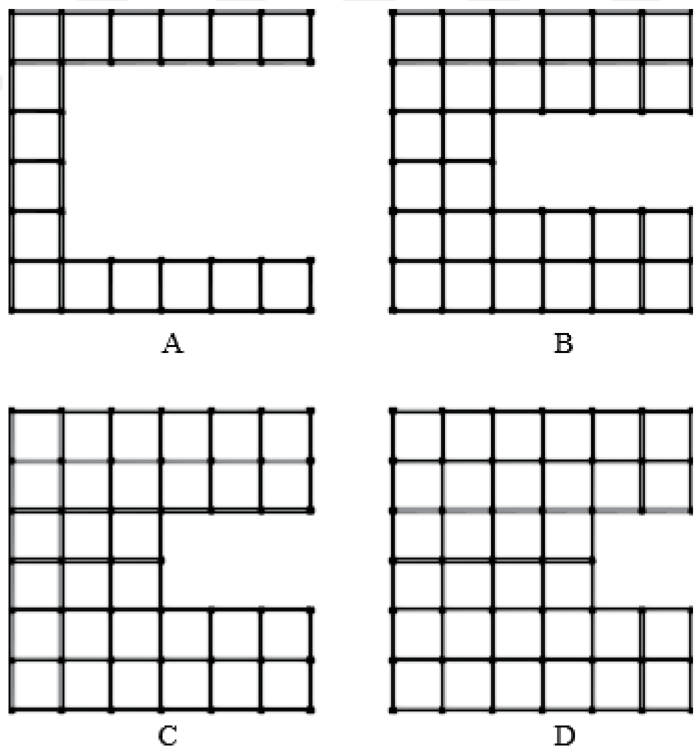


Figure 3.5 Plan view of C section irregular models

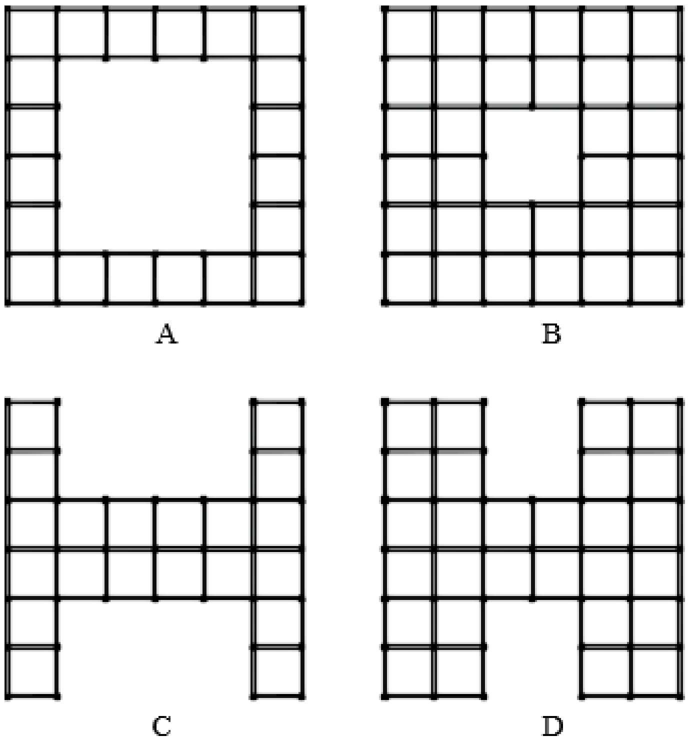


Figure 3.6 Plan view of O and H section irregular models

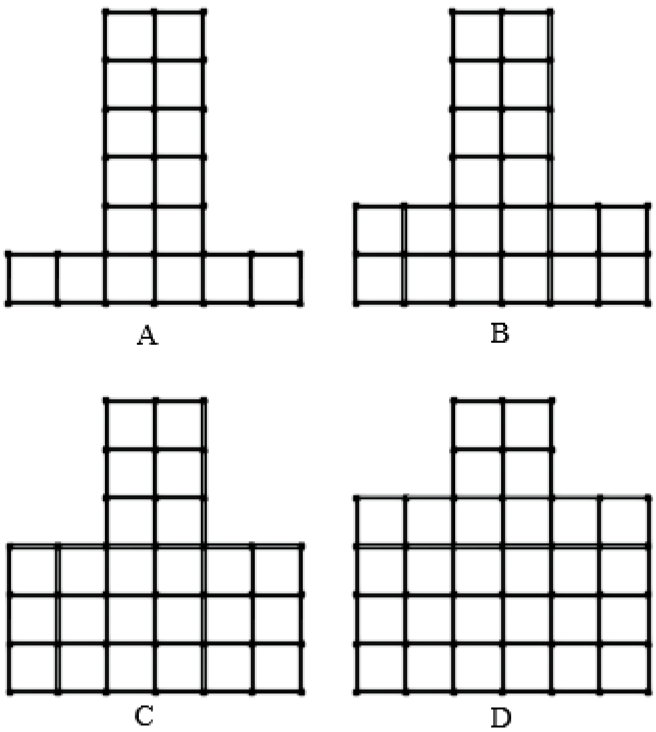


Figure 3.7 Plan view of ⊥ section irregular models

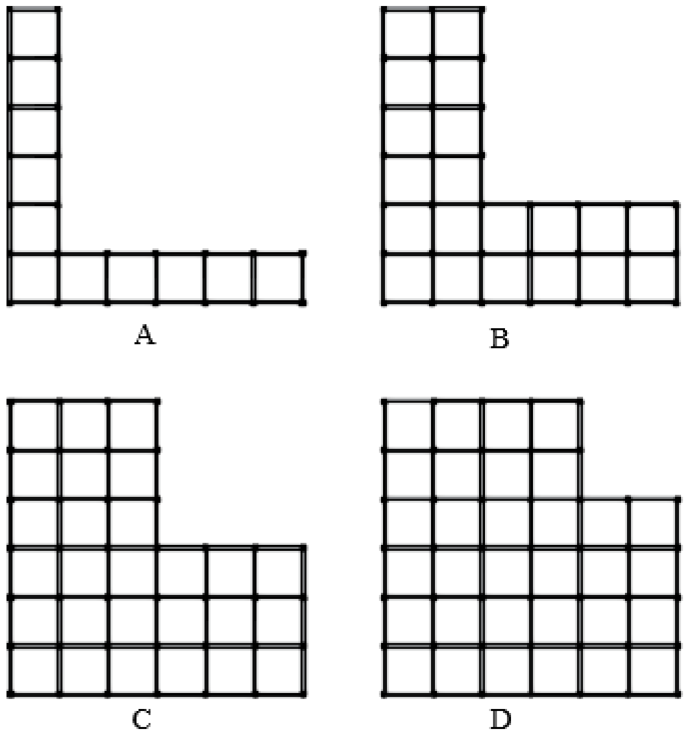


Figure 3.8 Plan view of L section irregular models

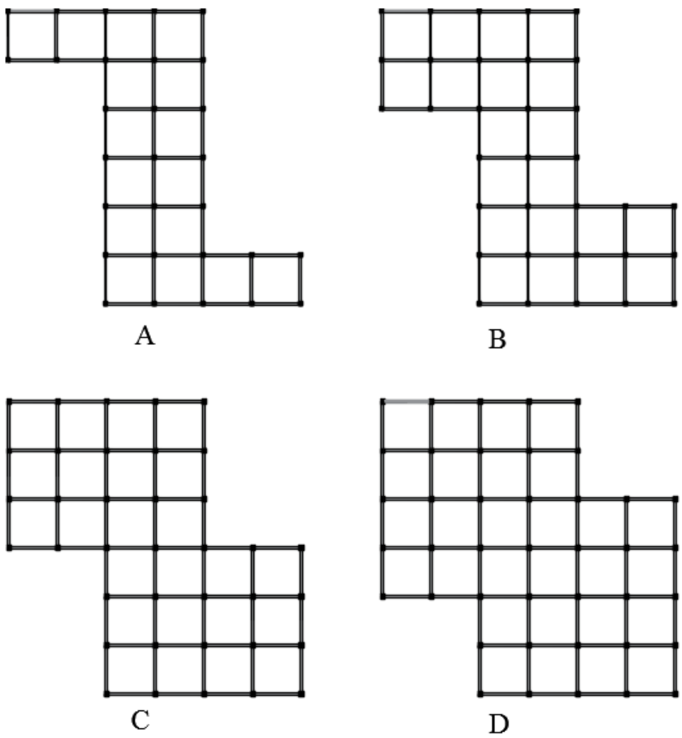


Figure 3.9 Plan view of Z section irregular models

As seen from previous figure, the first frame was regular in plane while the rest frames had some irregularities in plan according to UBC (1997) and IS 1983 (part

1):2002. Each storey had a height of 3 m except the first storey which is 5 m and all slab thicknesses were 15 cm. The exterior and interior frames of the buildings comprised six bays in each direction. The dimensions of beams for first five storey were 30 cm in width and 60 cm in height, and the dimension of beams for last three storey were 30 cm in width and 45 cm in height. Square columns were used for all models; the dimensions of the columns were varied from 50 cm for first five storey to 40 cm for last three storey as shown in Table 3.1.

Table 3.1 Dimension of the columns and beams in all buildings

Storey No.	Dimensions of the columns (cm)	Dimensions of the beams (cm)
1	50X50	60X30
2	50X50	60X30
3	50X50	60X30
4	50X50	60X30
5	50X50	60X30
6	40X40	45X30
7	40X40	45X30
8	40X40	45X30

The design live load was 2.5 kN/m^2 and dead load was 3 kN/m^2 including 1.5 kN/m^2 floor cover and 1.5 kN/m^2 partition load which assumed to be uniform distributed load over slab. The compressive strength of concrete (fc') was 27.6 N/mm^2, minimum yield stress of rebar steel (Fy) was 413 MPa, minimum tensile stress of rebars (Fu) was 620 MPa. Poison ratio was 0.2. The modulus of elasticity of concrete and steel were 24,855 N/mm^2 and 200,000 N/mm^2, respectively. Moreover, the column to base connections was assumed fully restrained and beams were considered to be rigidly connected to the columns in all models.

3.2 Nonlinear pushover and time history analysis

Pushover analyses is a nonlinear static technique in which the quantity of the lateral load is gradually raised, keeping the predefined distribution pattern along the heights of the structure. With the rise in the quantity of the forces, failure modes and weak links of the structure are established. It can estimate the demeanor of a building, containing the maximum inelastic deflection and the ultimate load (Govind et al., 2014).

It is the most widely used method to determine the nonlinear behavior of the buildings and it is an approximate analyses procedure. Nonlinear characteristics were modelled and the buildings are pushed until a collapse mechanism produced. The roof displacement and the base shear could be plotted to produce the capacity curve. It submitted an indication of the extreme capacity that building was able to resisting at the time of ground motions. It provide a rough concept about the global stiffness of the regular structure (Qadersheen, 2015).

In this method, the structures are monotonically loaded with increasing lateral loads with a steady height-wise distribution until the coveted displacement is gotten. It includes a chain of serial elastic analyses, overlaid to estimate a displacement - force curve of the entire structure. The equivalent static lateral forces approximately represent earthquake induced forces (Gültekin, 2014).

The ATC-40 and FEMA-356 documents provided acceptance criteria, analysis methods and modeling techniques of pushover analyses. This method efficiently determine the anticipated performance level of the structural system by the capacity curve of the structure. Depend on this capacity curve, target displacement is evaluated which is anticipated to be established during the ground motions (Alashker et al., 2015). Analysis also enables to evaluate the ductility capacity and collapse load. These documents characterize deformation-force criteria for hinges used in pushover procedure characterized the acceptance criteria depending on the plastic hinge rotations by considering different performance levels. Every plastic hinge is designed as a segregate point hinge. All plastic deformation is rotation or displacement develops within the point hinge (Qadersheen, 2015).

Two parts of a performance based design technique are capacity and demand. Demand is an impersonation of the ground motion. Capacity is an impersonation of the structures capability to resist the seismic demand. The performance is based on the way that the capacity is capable to handle the demand. The building must have the ability to resist the demands of the ground motions which means the performance of the building is compatible with the purposes of the design (Suryawanshi et al., 2014).

The frame model used in the nonlinear static method is depend on the procedures of the material, introducing deformation–force criteria for the hinges. Figure 2.10 characterize the relation between force-deformation suggested by ATC-40 and FEMA-356. Five points categorized A, B, C, D and E are used to determine the load deflection behaviour of the hinge and these points labelled A to B – Elastic state, B to IO- below the immediate occupancy, IO to LS – between immediate occupancy and life safety, LS to CP- between life safety to collapse prevention, CP to C – between collapse prevention and ultimate capacity, C to D- between C and residual strength, D to E- between D and E-collapse (Govind et al., 2014).

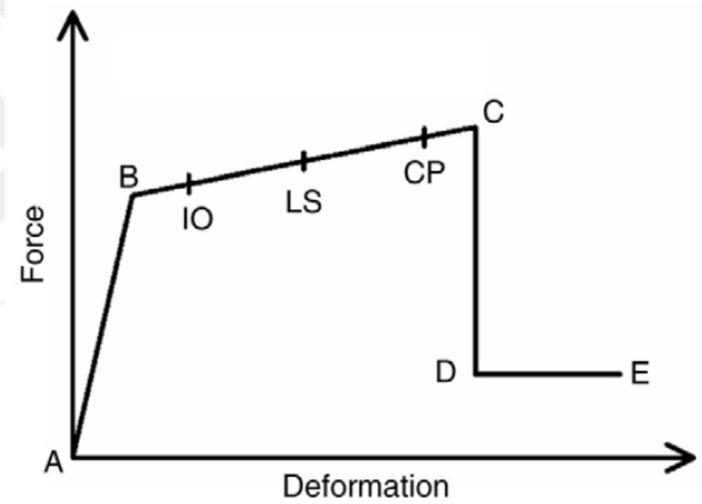


Figure 3.10 Force-deformation for pushover analysis (Govind et al., 2014)

The structural performance levels which written on the capacity curve i.e. IO , LS and CP. Depending on FEMA-356, IO implies the structural damage has appeared very limited in post-earthquake damage case. The life-threatening hurt as an outcome of structural damage is much low. Small repairs may be desired but not before the reoccupancy. LS implies significant damage to the structure has happened in the post-earthquake damage case but not resulting to partial or total crumbling of structure. The overall life-threatening hurt as an outcome of structure damage is low. It should be prudent to making repairs before the reoccupancy. CP implies a great amount of damage to the structure has happened containing strength and stiffness degradation of the lateral force resisting system and huge permanent deformation of the structure. The building is on the edge of the experiencing partial or total crumbling. The structure couldn't be technically practical to repair and isn't safe for reoccupancy.

In this study, the steps in implementing the nonlinear static pushover analysis of twenty five three dimensional building model in SAP 2000 as tool for implementing the pushover analysis procedure as follow:

- The properties of twenty five building were made and assigned in SAP 2000, and then acceptance criteria and properties for the hinges were defined. The program contain some build-in default hinge characteristics that were depend on average values from ATC-40, FEMA-356, and ASCE 41 (ASCE, 2007) for concrete and steel member.
- The pushover hinges on the model was localized by designing two hinges to every columns and beams with deformation properties based on an assumed hinge length.
- The pushover load case introduced using gravity force and then subsequent lateral pushover force cases identified to start from the final conditions of the gravity pushover. SAP 2000 allowed the distribution of lateral load used in the pushover to be depend on a uniform acceleration in an identified direction, an identified mode shape, or a user-defined static load case.

To estimate the actual nonlinear behavior of buildings, besides implementing pushover analysis, time history nonlinear analysis was performed on the same analytic models with the same hinges properties. In this technique, the buildings were submitted to real ground motion record. Subsequently, inertial forces were estimated from the earthquake ground motions and the response of the structure either in forces or in deformations were determined as a function of time.

In nonlinear direct integration time-history and nonlinear static analyses, the post-yield behaviour by appointing concentrated plastic hinges to tendon and frame objects was simulated. Elastic behaviour appeared over the length of member, and then deformation after the elastic limit occurred entirely within hinges, which were modeled in separate positions. Inelastic behaviour was gained by integration of the plastic curvature and plastic strain that appeared within a specified hinge length, usually on the order of member depth FEMA 356 (2000). To catch plasticity distributed along the length of member, a series of hinges were modeled. Multiple hinges were coincide at the same position. Plasticity were related with displacement - force behaviours (shear and axial) or moment-rotation (bending and torsion). The nonlinearity was taken into account by adopting plastic hinges with hysteretic relationships depend on FEMA 356 (2000) at every end of the beam and column members. Both fiber hinges and P-M2-M3 hinges are obtainable to capture coupled biaxial-bending and axial behaviour. The fiber hinge is good for hysteretic dynamics, while the P-M2-M3 hinge is best suited for nonlinear static pushover. For the column members, axial force and biaxial moment hinges (P-M2-M3) and for the beams, flexural moment hinges (M3) were considered.

In the dynamic analysis, Altadena-1 was used as a ground motion. The properties of the selected earthquake acceleration record which used in this study are given in Figure 3.11 which shows the acceleration time plot of the earthquake.

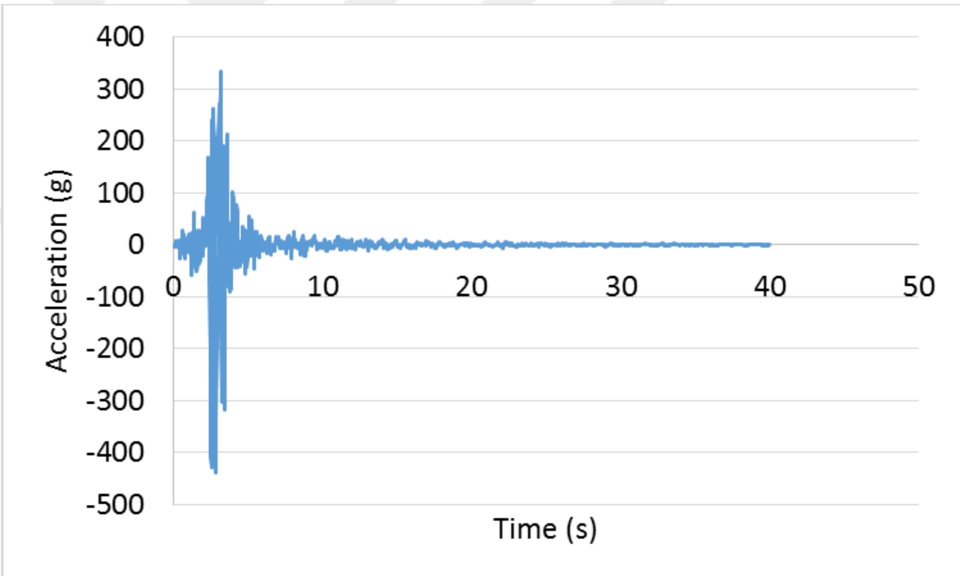


Figure 3.11 Acceleration time plot of the earthquake ground motion (Altadena-1)

CHAPTER 4

RESULTS AND DISCUSSION

In this chapter, the results based on nonlinear static and nonlinear dynamic time history analyses were given for the regular building and twenty four irregular buildings with different irregularities in plan that include +, C, O, H, ⊥, L and Z shapes. As a ground motion record, Altadena-1earthquake was used. The structural response of the different frame systems were examined by mean of pushover curves, roof displacement time history, and base shear time history.

4.1 Pushover Curves

When the pushover curves (capacity curves) in Figure 4.1 was assessed for all building types, an irregularity causes decreasing the capacity of the seismic performance of the building. Figure 4.1 demonstrates the comparison of the pushover curves of regular building with irregular buildings 4 +, 3 +, 2 + and 1 +. The maximum base shear of regular and irregular buildings 4 +, 3 +, 2 + and 1 + was observed as 138524, 127067, 102894, 85247 and 56051 kN, respectively. Variation of the capacity of irregular buildings 4 +, 3 +, 2 + and 1 + with respect to regular building was computed as 92%, 74%, 62% and 41%, respectively as clear in Figure 4.2. Thus, it was observed that 4+ section model gave the closet result in terms of the capacity to that of the regular section model. However, 1+ section model yielded far away result due to the negative effect of the irregularity in its plan.

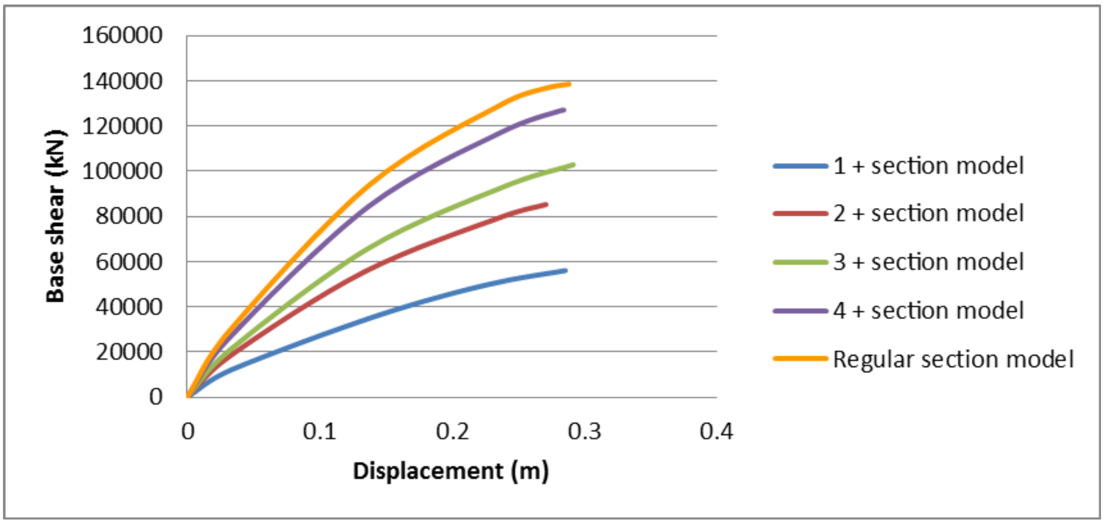


Figure 4.1 Pushover curves of regular and irregular + section models

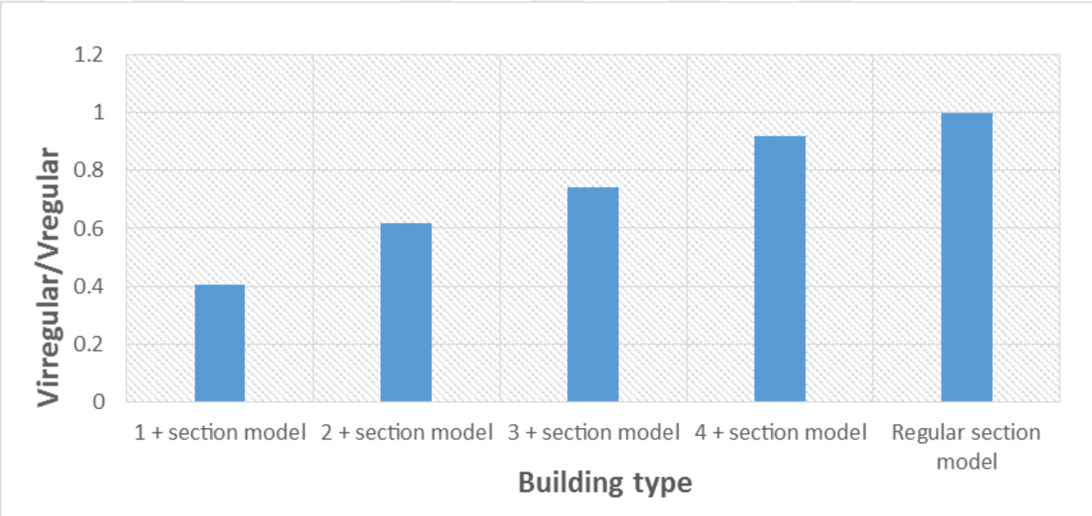


Figure 4.2 Base shear comparison between regular and irregular + section models

Figure 4.3 demonstrates the comparison of the pushover curves of the regular building with the irregular buildings 4C, 3C, 2C and 1C. In accordance with the result of the analysis, the seismic performance of the building increased with decreasing the irregularity. The maximum base shear of the regular building was 138524 kN, it decreased to 128309, 123732, 119166 and 79530 kN for the irregular buildings 4C, 3C, 2C and 1C, respectively. By comparing the result of the regular building with irregular building 4C, 3C, 2C and 1C, the best performance capacity obtained for building 4C was 93% of the capacity of regular building. On the other hand, the worst performance capacity is obtained for building 1C is 57% of the regular building capacity. Moreover, irregular buildings 2C and 3C provided moderate results and fell between the 1C and 4C irregular cases. These findings are indicated in Figure 4.4.

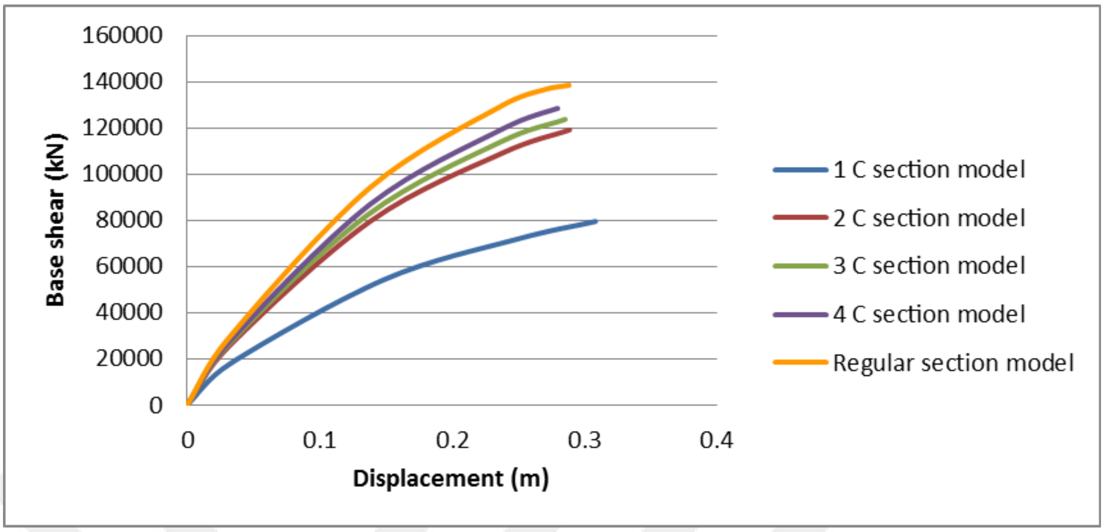


Figure 4.3 Pushover curves of regular and irregular C section models

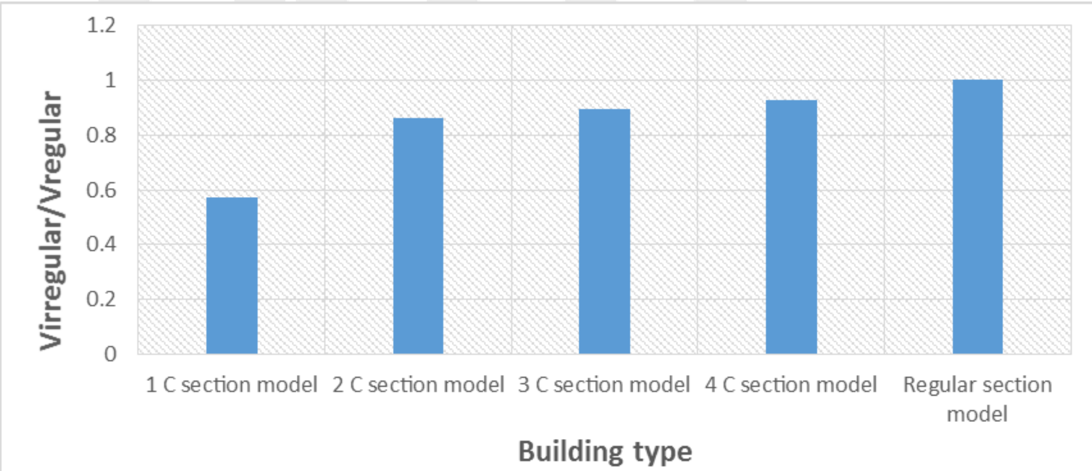


Figure 4.4 Base shear comparison between regular and irregular C section models

Figure 4.5 presents the comparison between the pushover curves of the regular building and the irregular buildings 2O, 2H, 1O and 1H. According to the analysis results, the highest value of the base shear of the regular building is 138524 kN, it decreased to 134798, 117196, 95424 and 88765 kN for the irregular buildings 2O, 2H, 1O and 1H, respectively. By comparing the result of the regular building with irregular building 2O, 2H, 1O and 1H, the best capacity obtained for building 2O was 94% of the capacity of the regular building. On the other hand, the worst capacity was obtained for building 1H is 69% of the capacity of the regular building as shown in Figure 4.6.

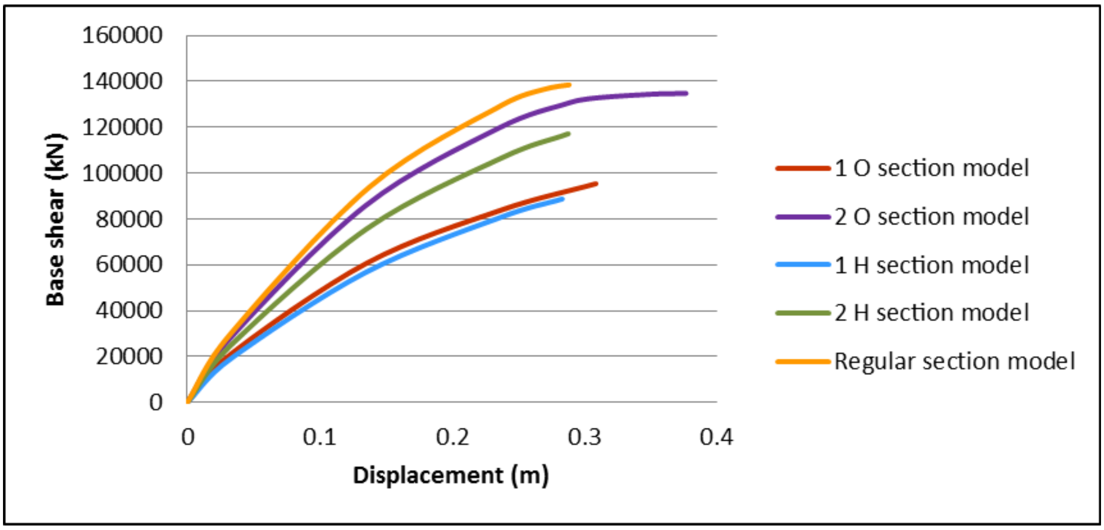


Figure 4.5 Pushover curves of regular and irregular O and H section models

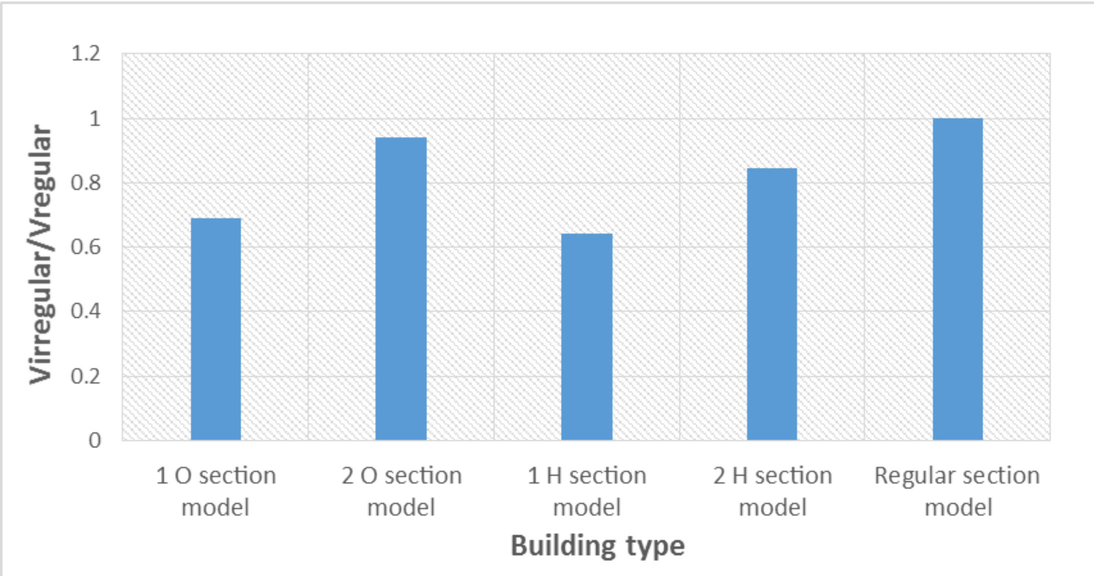


Figure 4.6 Base shear comparison between regular and irregular O and H section models

Figure 4.7 illustrates the comparison of the pushover curves of regular building with irregular buildings 4┴, 3┴, 2┴ and 1┴. The peak value of the base shear of regular building and irregular buildings 4┴, 3┴, 2┴ and 1┴ were about 138524, 114514, 99293, 81794 and 73018 kN, respectively. The comparison between regular building and irregular buildings indicated a reduction of base shear of 17.3%, 28.3, 41.0% and 47.3% for 4┴, 3┴, 2┴ and 1┴, respectively as cleared in Figure 4.8.

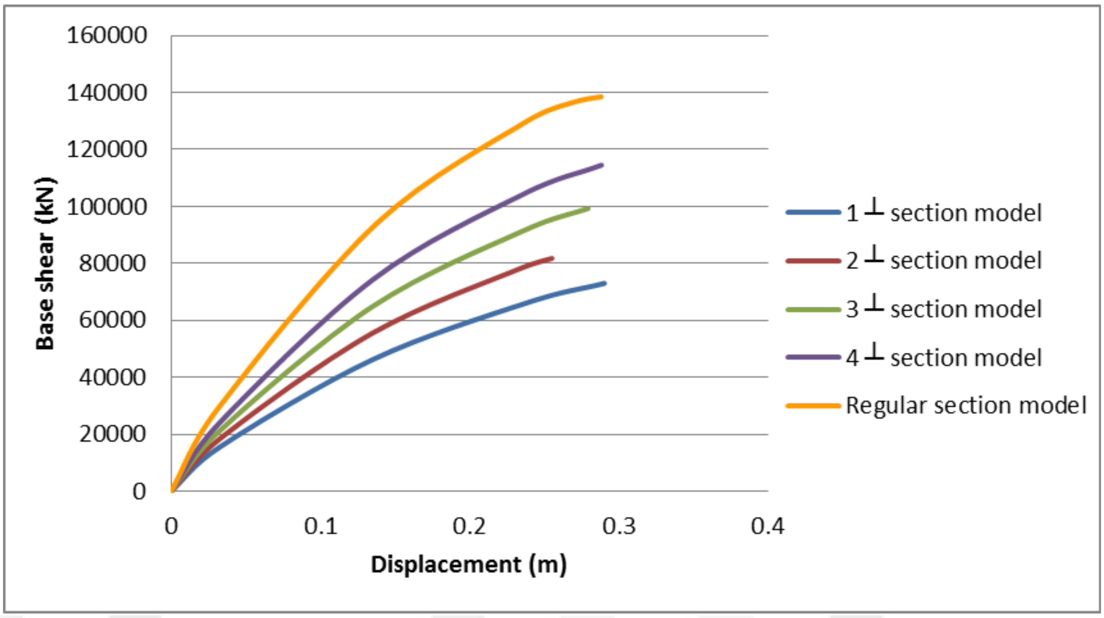


Figure 4.7 Pushover curves of regular and irregular ⊥ section models

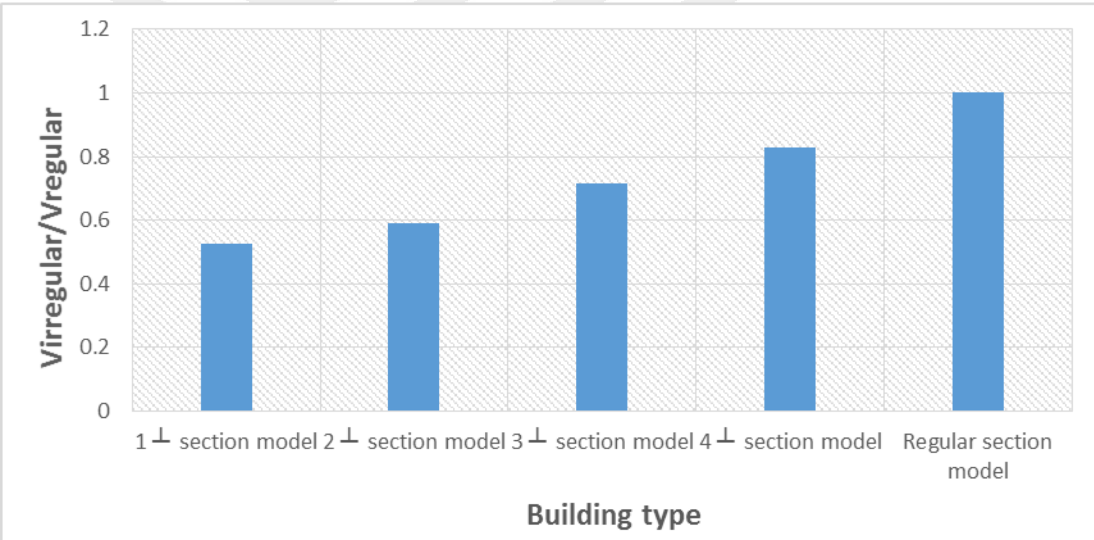


Figure 4.8 Base shear comparison between regular and irregular ⊥ section models

Figure 4.9 illustrates the comparison of the pushover curves of regular building with irregular buildings 0.33L, 0.5L, 0.67L and 0.83L. The maximum base shear of regular building and irregular buildings 0.33L, 0.5L, 0.67L and 0.83L were about 138524, 124412, 108732, 86097 and 52772 kN, respectively. The comparison between regular building and irregular buildings indicated 89.9%, 78.5%, 62.2% and 38.1% differences for 0.33L, 0.5L, 0.67L, and 0.83L, respectively as cleared in Figure 4.10. More differences in the maximum base shear value were occurred due to increasing in the irregularity value of the buildings.

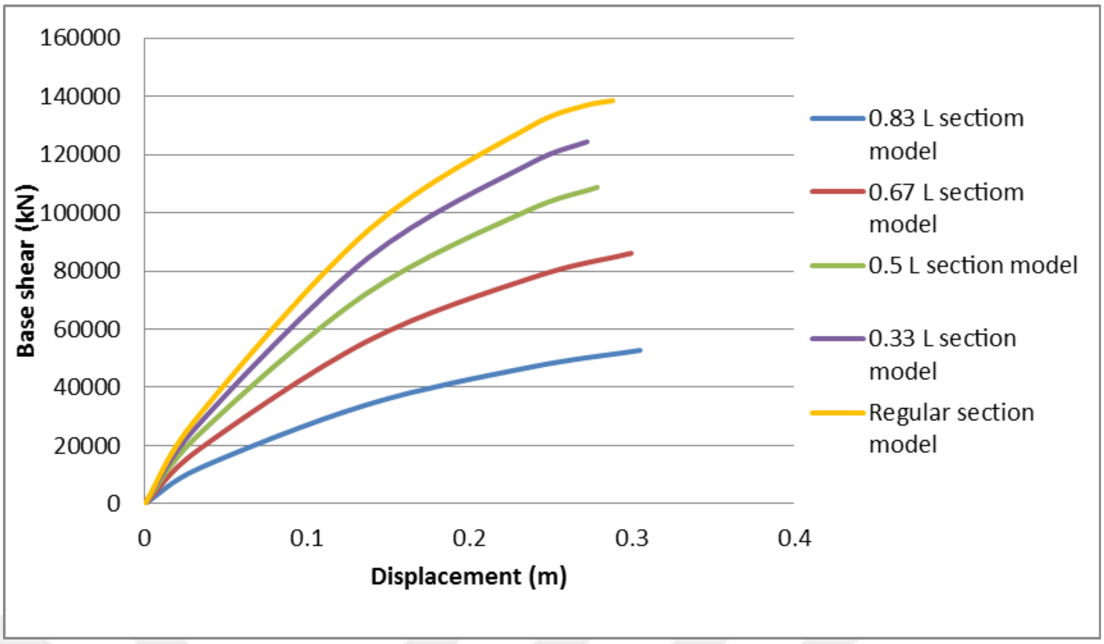


Figure 4.9 Pushover curves of regular and irregular L section models

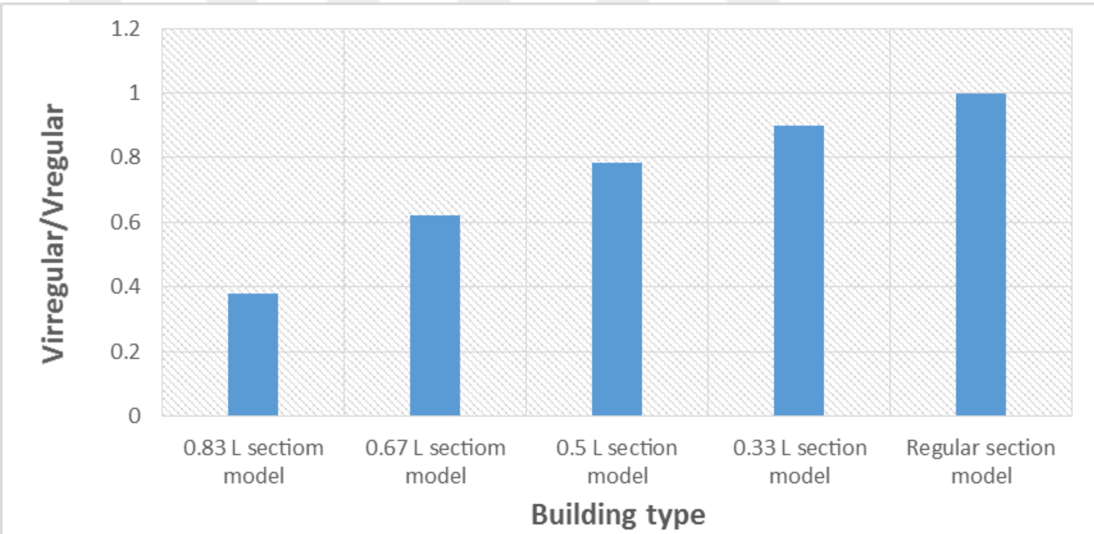


Figure 4.10 Base shear comparison between regular and irregular L section models

Figure 4.11 shows the comparison of the pushover curve of regular building and irregular buildings 4Z, 3Z, 2Z and 1Z. As seen from the figure, the regular building had considerably greater capacity in comparison to irregular buildings due to the fact that irregularity causes decreasing the capacity of the buildings. For regular building and irregular buildings 4Z, 3Z, 2Z and 1Z, the peak base shear were observed as 138524, 114902, 102715, 82084 and 73252 kN, respectively. Figure 4.12 reveals the maximum base shear ratio of the irregular building to regular building. It was evident from the figure that 4Z section model had about 18% less capacity as compared to the regular section model. This capacity reduction was more pronounced for the 4Z section model, indicating about 50% lower capacity.

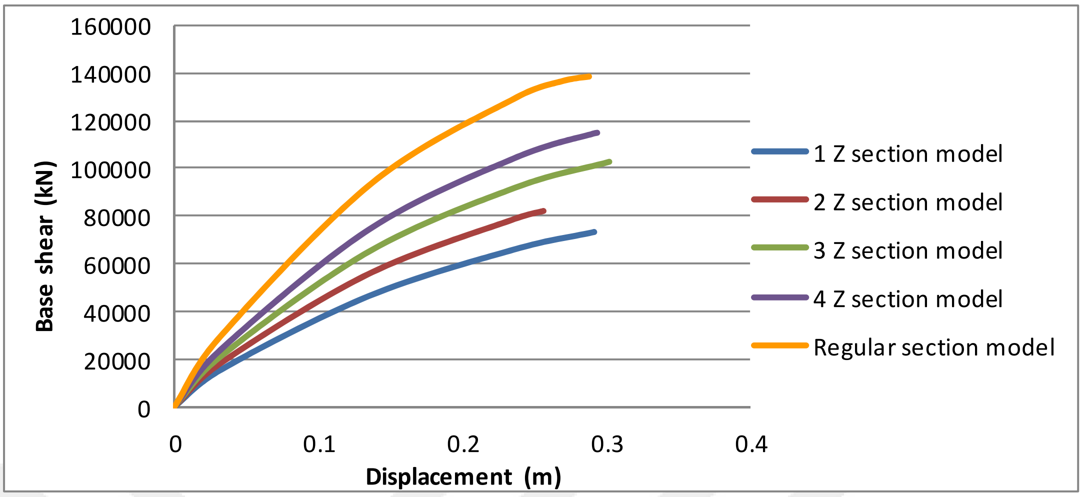


Figure 4.11 Pushover curves of regular and irregular Z section models

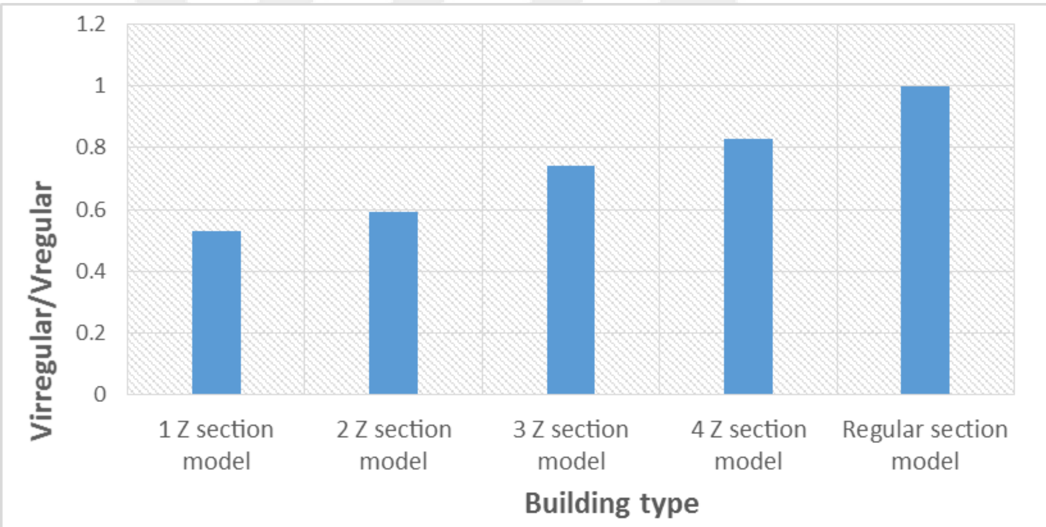


Figure 4.12 Base shear comparison between regular and irregular Z section models

4.2 Variation of Story Displacement

Figure 4.13 shows the time history storey displacement of the regular building and the irregular buildings (4+, 3+, 2+ and 1+) under the seismic action (Altadena-1 earthquake). The effect of plan irregularity can be noted clearly and caused increasing the value of the maximum displacement. This is more pronounced at the roof level of the buildings. As can be seen from the result gotten from the nonlinear time history analysis as shown in Figure 4.14, the maximum roof displacement of the regular building and the irregular buildings 4+, 3+, 2+ and 1+ were observed as 53.96, 54.09, 54.63, 55.28 and 57.78 cm, respectively. The difference between displacement values seemed to be closed each other. This may be the characteristic of the earthquake record used. In the case of more severe seismic action, the difference in the response of the irregular buildings could be expected to be more.

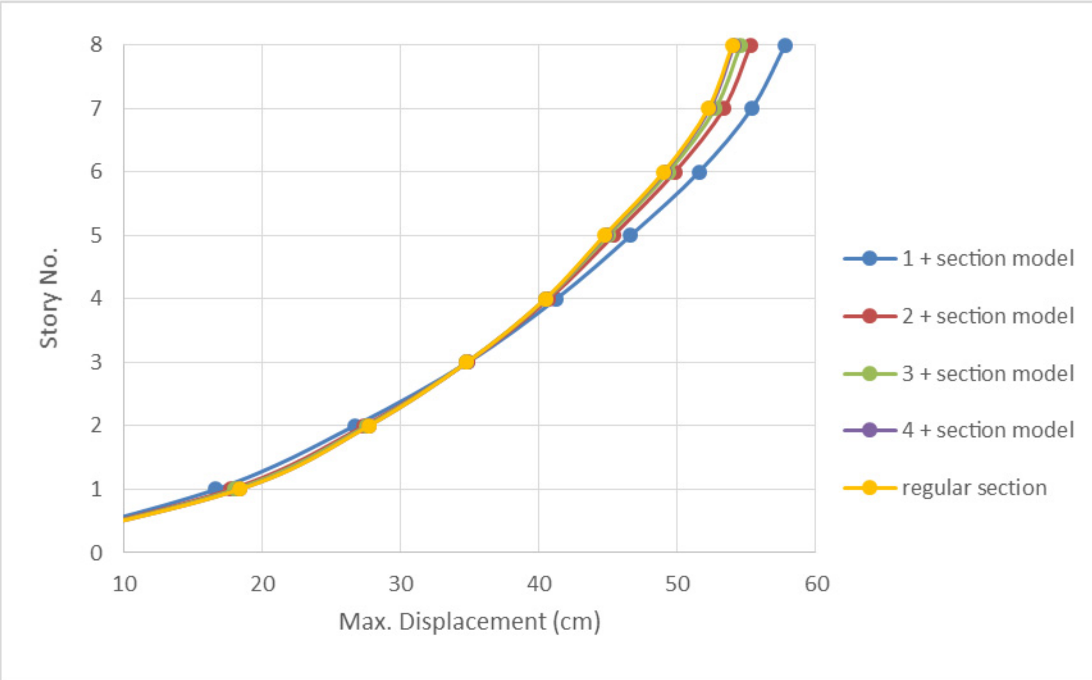


Figure 4.13 Variation of storey displacement of regular and irregular + section models

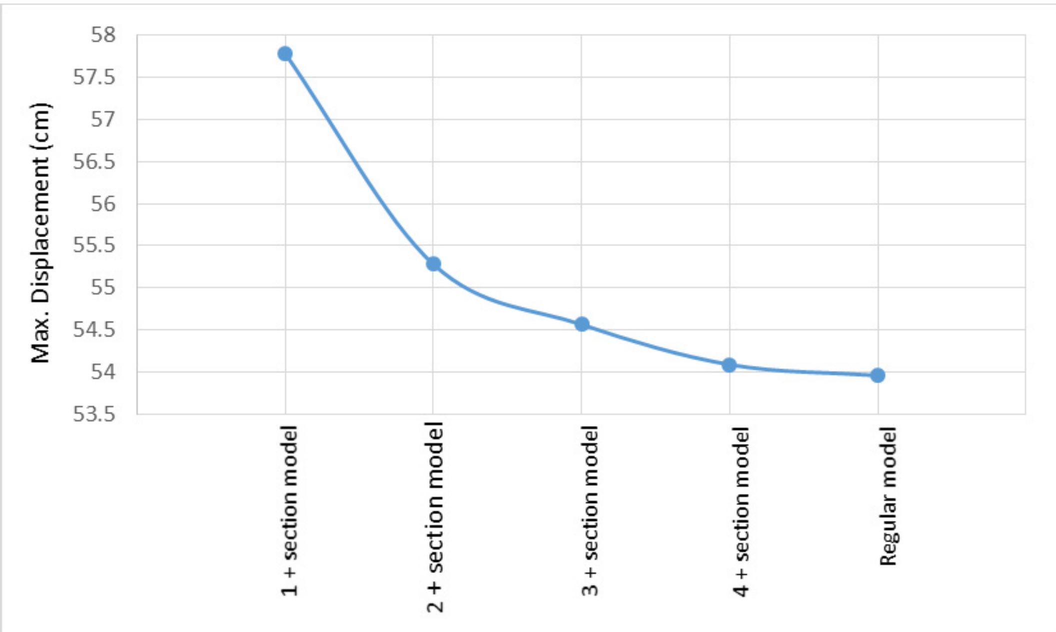


Figure 4.14 Maximum roof displacement of regular and irregular + section models

Figure 4.15 shows the story displacement for the regular building and the irregular buildings (4C, 3C, 2C and 1C) while Figure 4.16 demonstrates the maximum displacement at the top floor of the regular building and the irregular buildings (4C, 3C, 2C and 1C). The minimum top displacement appears in the case of regular building and 4C building nearly 54 cm, the moderate displacement was recorded for 2C building and 3C building nearly 55 cm, and the maximum top displacement was at the case of 1C building nearly 56 cm. However, the amount of displacement for the irregular C section models was close to each other.

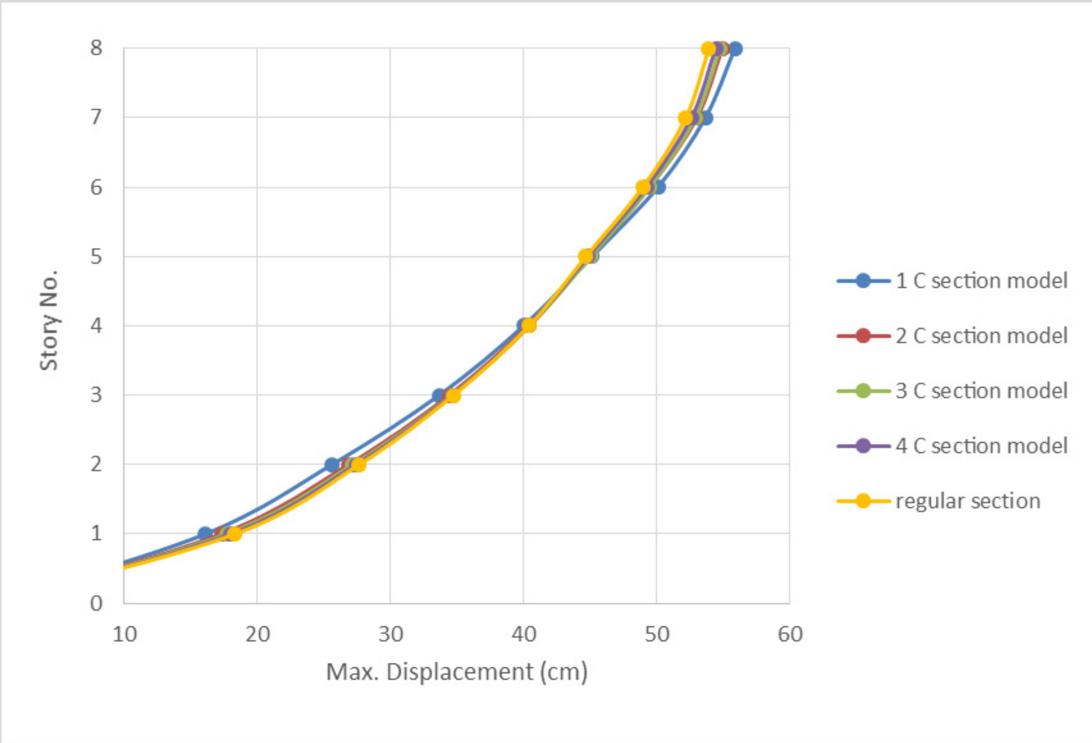


Figure 4.15 Variation of storey displacement of regular and irregular C section models

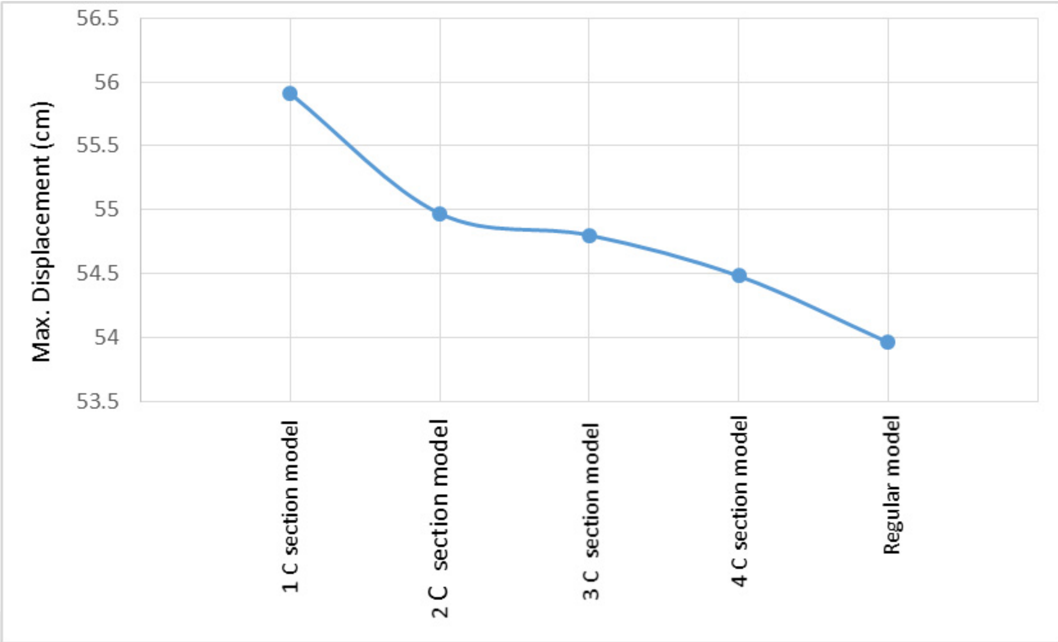


Figure 4.16 Maximum roof displacement of regular and irregular C section models

Figure 4.17 shows the distribution of displacement of the regular building and the irregular buildings 2H, 2O, 1H and 1O along the height of the buildings. According to the result of plots from Figure 4.18, it was indicated that the regular building had considerably smaller roof displacement as compared to the irregular buildings. Similar to the previous irregularities, with increasing the amount of irregularity in plan, more displacement at roof level were pointed out.

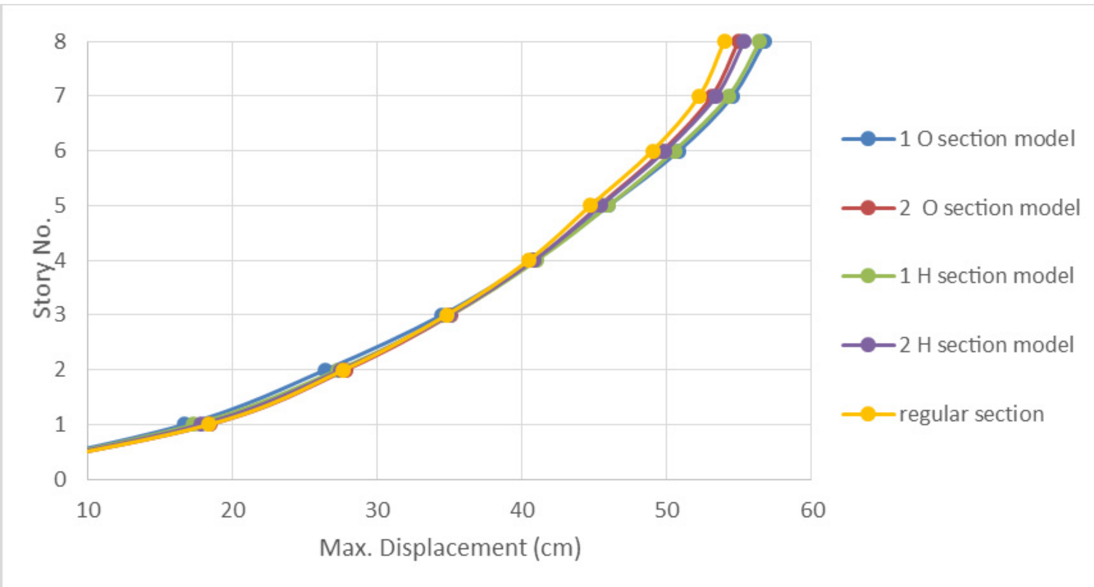


Figure 4.17 Variation of storey displacement of regular and irregular O and H section models

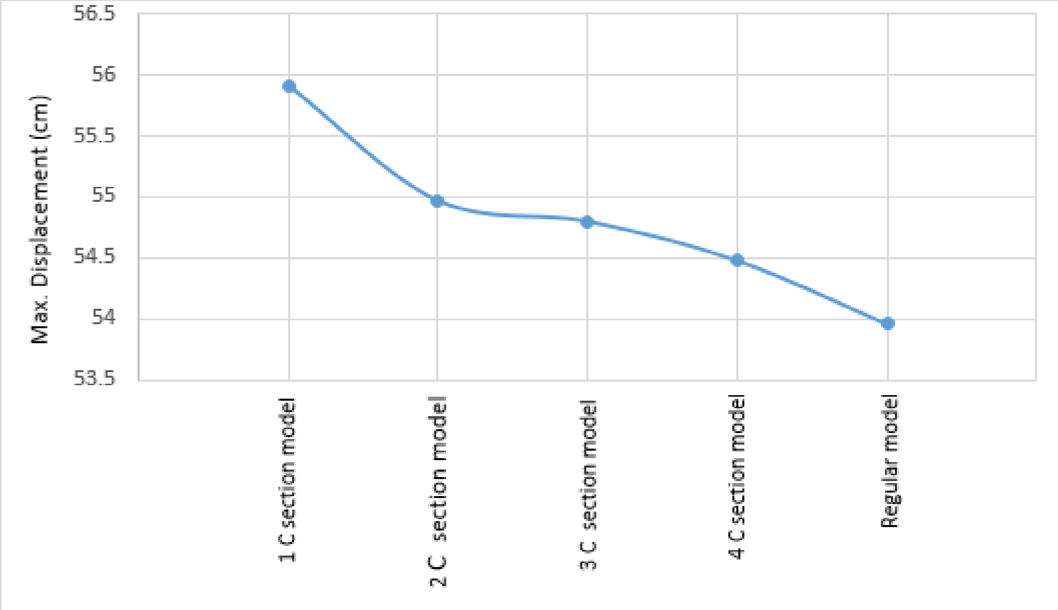


Figure 4.18 Maximum roof displacement of regular and irregular O and H section models

Figure 4.19 shows the storey displacements for the regular building and the irregular buildings 1⊥, 2⊥, 3⊥ and 4⊥. According to the findings obtained from the nonlinear time history analysis, the regular building had relatively smaller roof displacements than the irregular buildings under the earthquake ground motion. For example, as shown in Figure 4.20, the maximum roof displacement of the regular building was obtained as 53.96 cm and the peak roof displacement for the irregular buildings 4⊥, 3⊥, 2⊥ and 1⊥ were attained as 54.28, 54.89, 56.07 and 59.89 cm, respectively.

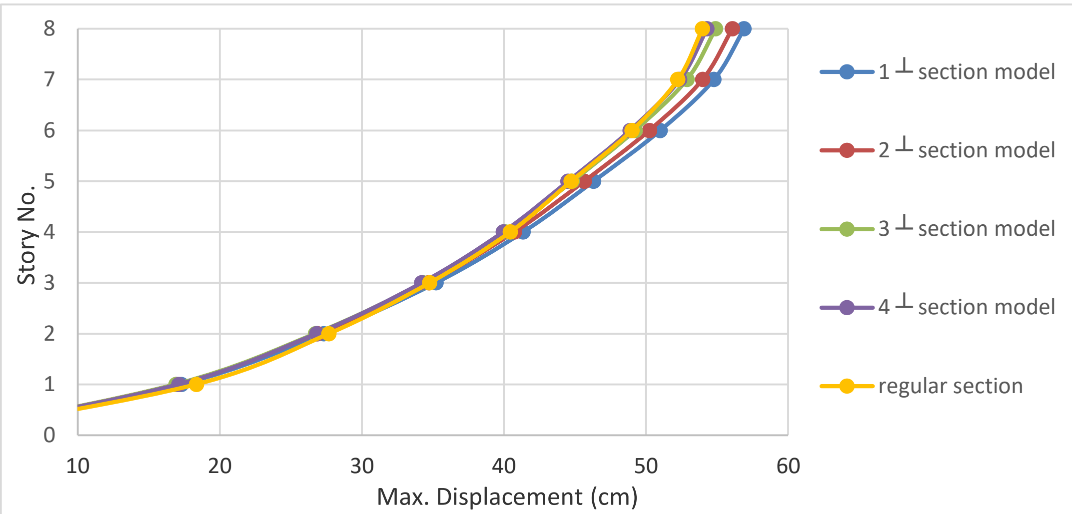


Figure 4.19 Variation of storey displacement of regular and irregular ⊥ section models

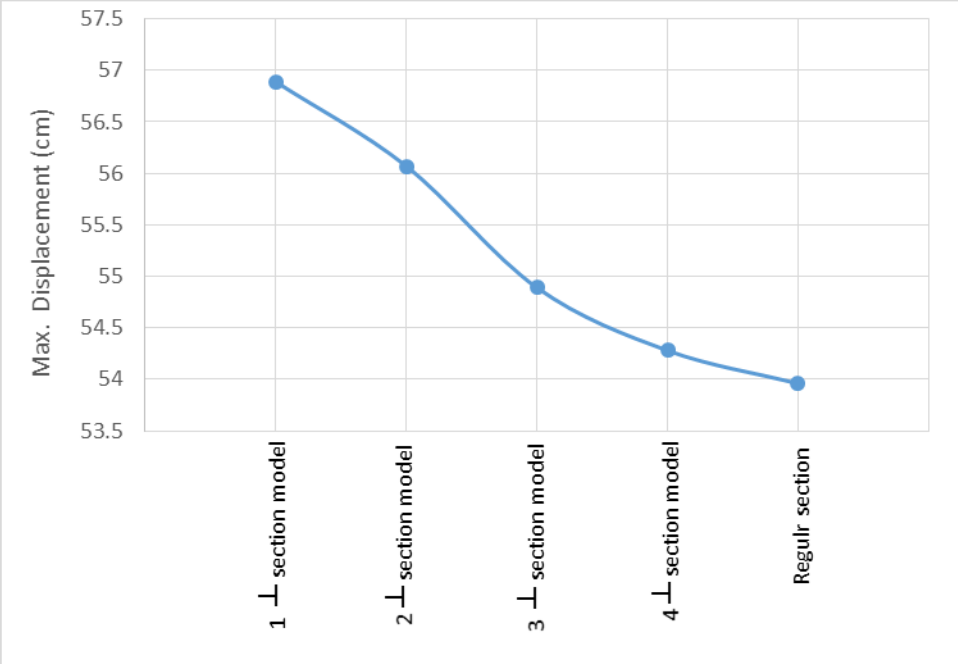


Figure 4.20 Maximum roof displacement of regular and irregular ⊥ section models

In Figure 4.21, the storey displacement was influenced by the number of stories and frame type. It was found that the displacement of regular building and irregular buildings increased through the height of the structures. The maximum displacement was obviously appeared in the top storey of the buildings. However, the amount of the displacement varied, depending on the amount of the irregularity considered. Figure 4.22 displays the peak roof displacement time history of the regular and irregular buildings, 0.33L, 0.5L, 0.67L and 0.83L. They were equal to 53.9, 54.1, 54.8, 56.2 and 60.0 cm, respectively.

The response of structure against seismic forces changed with plan irregularity. The behaviour of the buildings were observed that 0.83L shape building displaced more nearly 60 cm, these may be due to lesser weight and slender geometry as in comparison to other plans of irregular buildings. Considering all these above factors, complex shaped buildings gave large response in terms of the displacement under the given loading conditions. Hence, we may say that simple shape geometry of structure must be adopted to minimize the effects of seismic actions (Mohod, 2015).

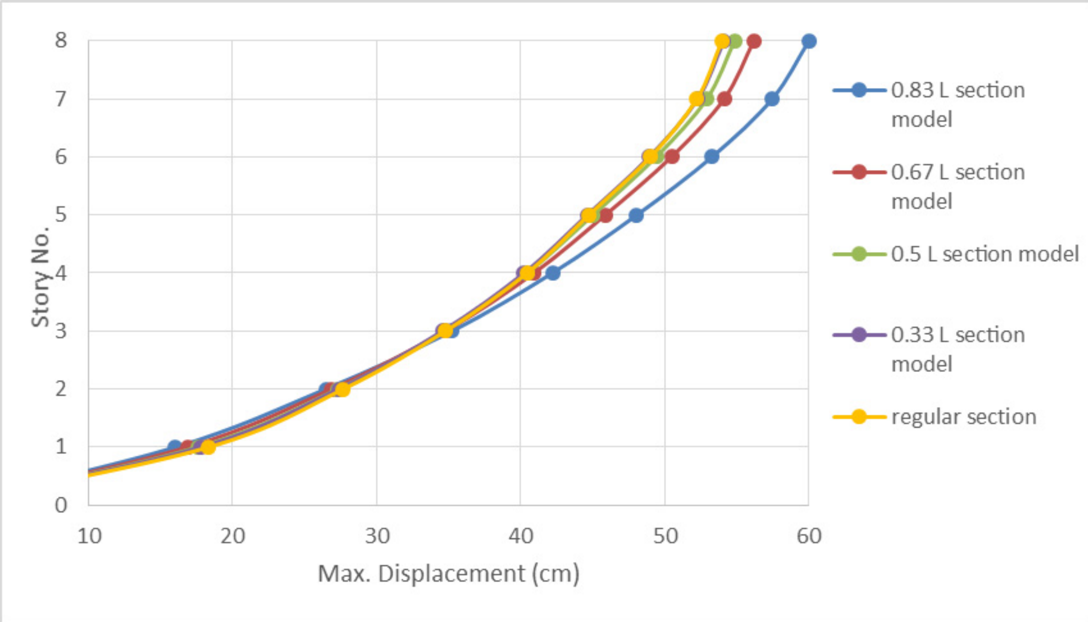


Figure 4.21 Variation of storey displacement of regular and irregular L section models

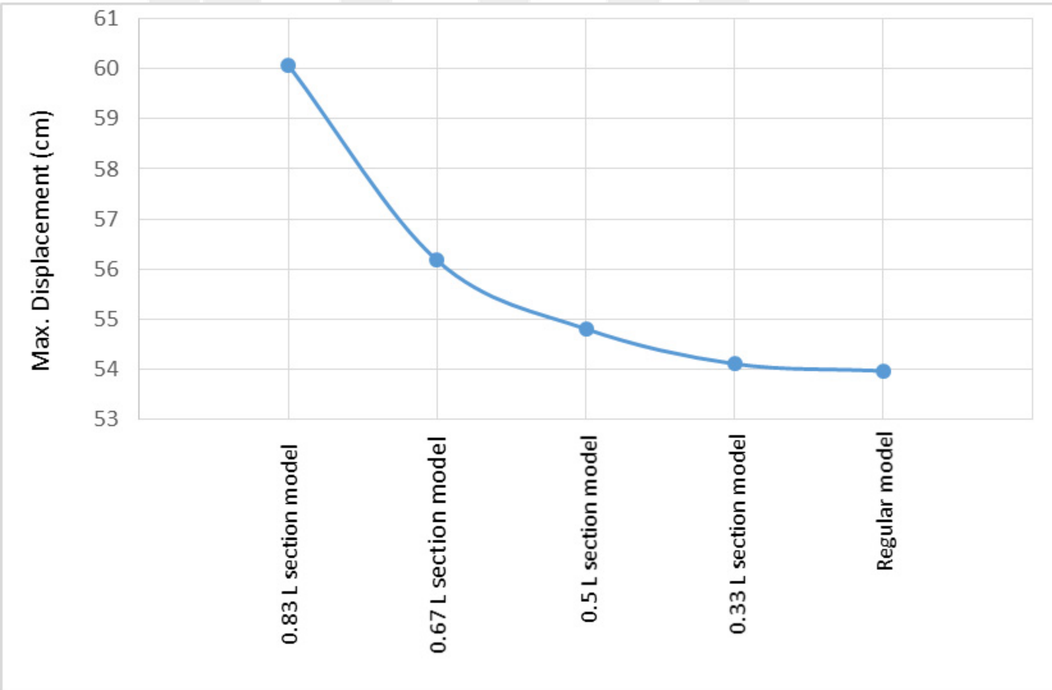


Figure 4.22 Maximum roof displacement of regular and irregular L section models

Figure 4.23 shows the comparison of storey height and displacement curves for regular building and irregular buildings 4Z, 3Z, 2Z, and 1Z while Figure 4.24 demonstrates the peak roof displacement of the regular and irregular buildings. The roof displacement values of about 53.9, 54.32, 54.66, 55.4 and 55.86 cm were determined for regular building and irregular buildings 4Z, 3Z, 2Z, and 1Z, respectively.

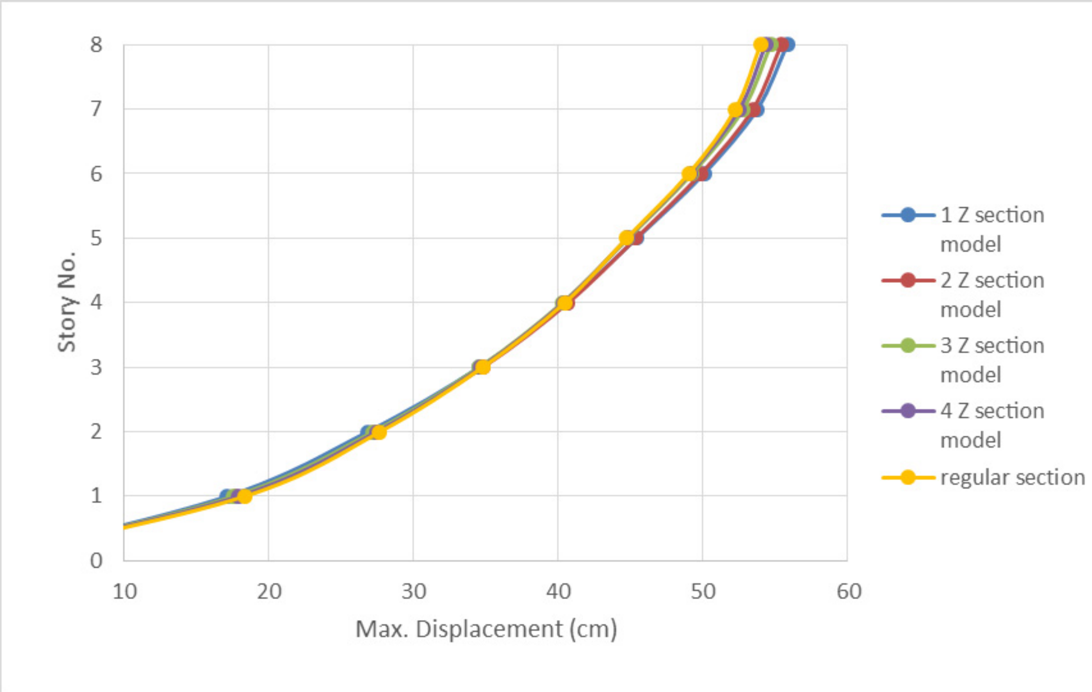


Figure 4.23 Variation of storey displacement of regular and irregular Z section models

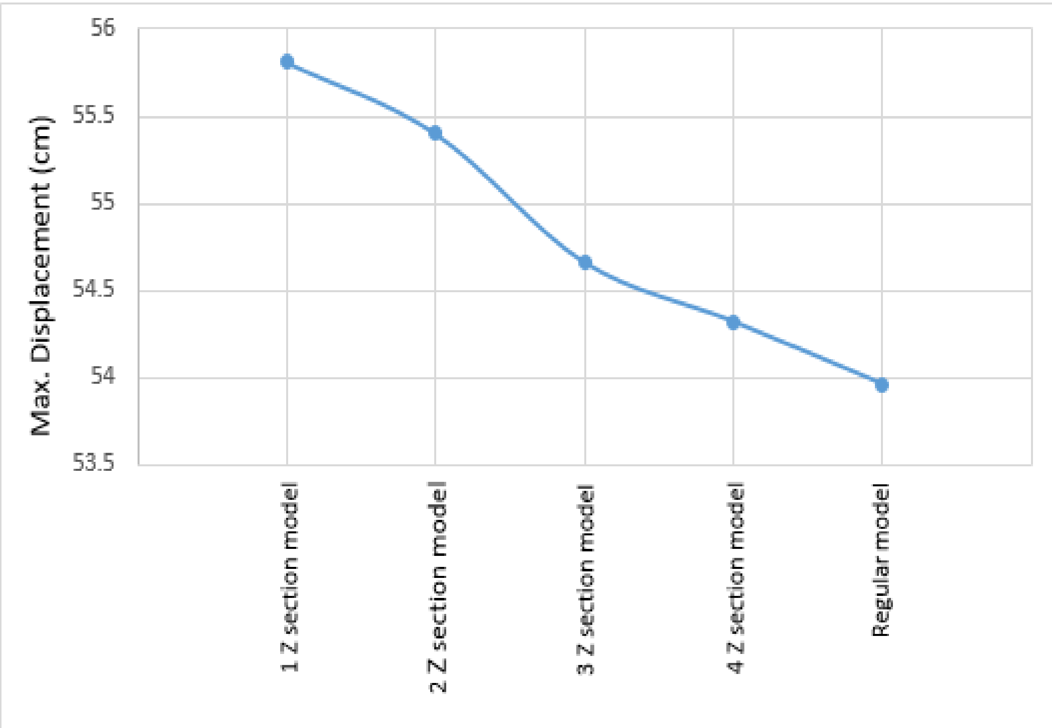


Figure 4.24 Maximum roof displacement of regular and irregular Z section models

The different irregularities generated in the floor plan of the building results in different dynamic behaviors because of the particular geometric of the structure. The irregular buildings subjected to earthquakes has shown that the condition determines an irregular distribution of the lateral force resisting elements proceeding considerable torsional effects and concentration of stress at the vertice of the reentrant corner. Although building damage cannot entirely be attributed to floor plan irregularities, this aspect has been acknowledged as one important factor on the response of buildings to earthquake effects. So that buildings with irregular floor plans appear to be more susceptible to larger deformations and damage when subjected to earthquake motions than those with regular floor plans (Abd-el-rahim and Farghaly, 2010).

4.3 Base shear time history

The results of the base shear versus time plots for different cases are given in Figures 4.25 and 4.26. It can be realized from the figures that the base shear value for the building was decreased due to irregularity. For the regular building and irregular buildings 4+, 3+, 2+ and 1+, the maximum base shear were calculated as 43700, 39160, 30880, 26940 and 17560 kN, respectively. The minimum value of the maximum base shear among all irregular building models was indicated for 1+ irregular building. However, the maximum one was noted for 4+ irregular building

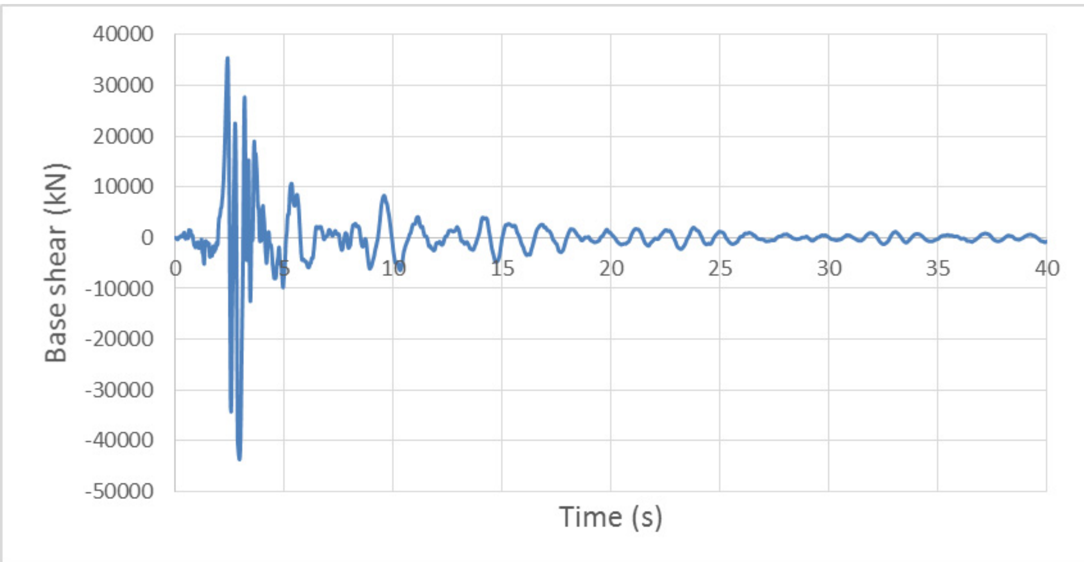
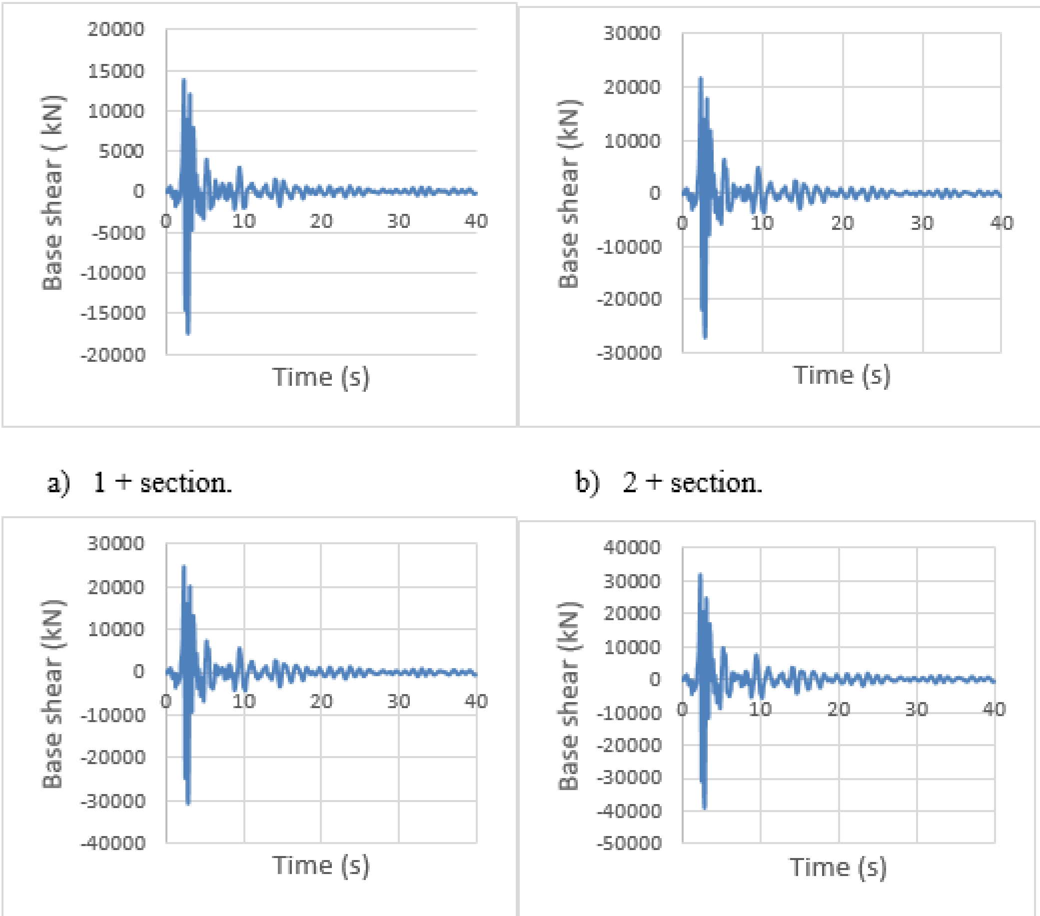


Figure 4.25 Base shear time history variation of regular building



a) 1 + section.

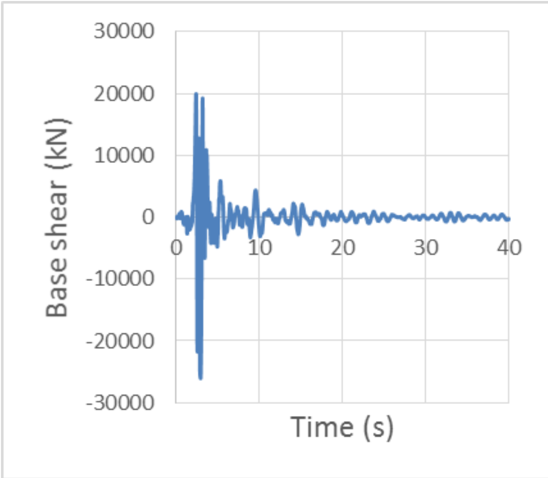
b) 2 + section.

c) 3 + section.

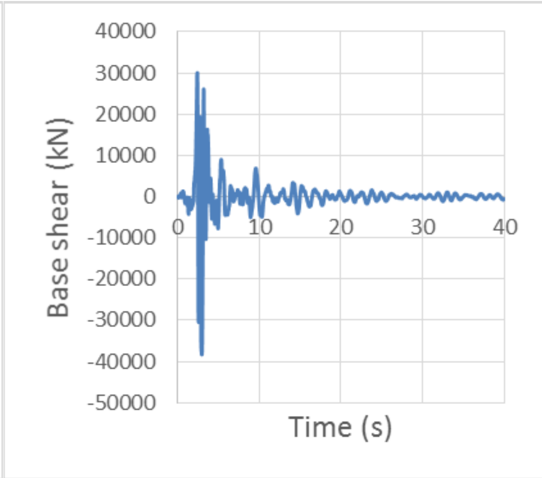
d) 4 + section.

Figure 4.26 Base shear time history variation of irregular + section models

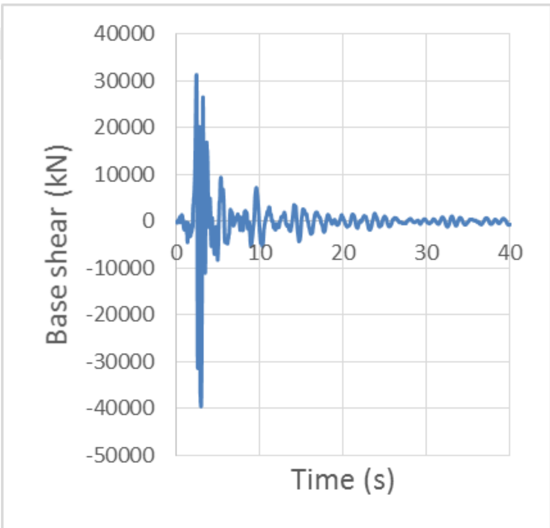
The base shear versus time relationship was also evaluated for irregular C section buildings. The comparison of the base shear time history is shown in Figures 4.25 and 4.27 for the regular building and irregular buildings (4C, 3C, 2C and 1C), respectively. As expected, the regular frame building had the maximum base shear of 43700 kN while the other irregular buildings had smaller value of the base shear due to their irregularities. The base shear values of 40430, 39580, 38310 and 26050 kN were computed for the irregular buildings 4C, 3C, 2C and 1C, respectively. It was clearly noted that C section irregularity resulted in a reduction of the base shear up to 60% with respect to the regular section model.



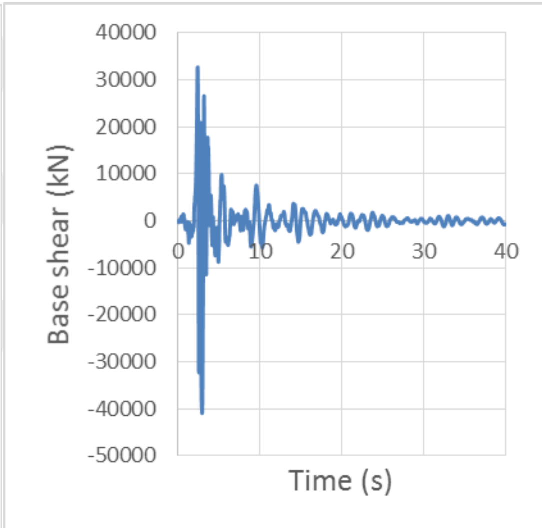
a) 1 C section.



b) 2 C section.



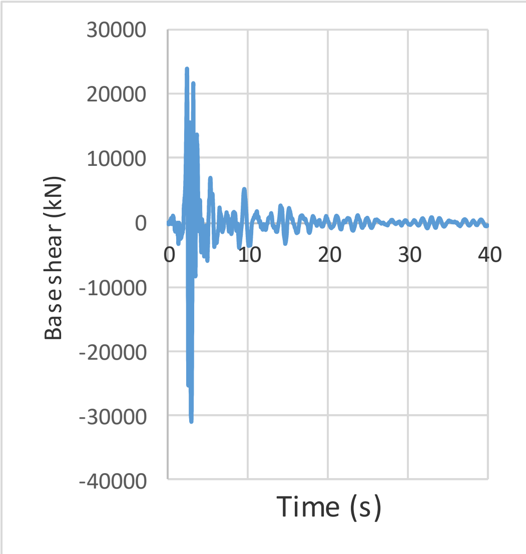
c) 3 C section.



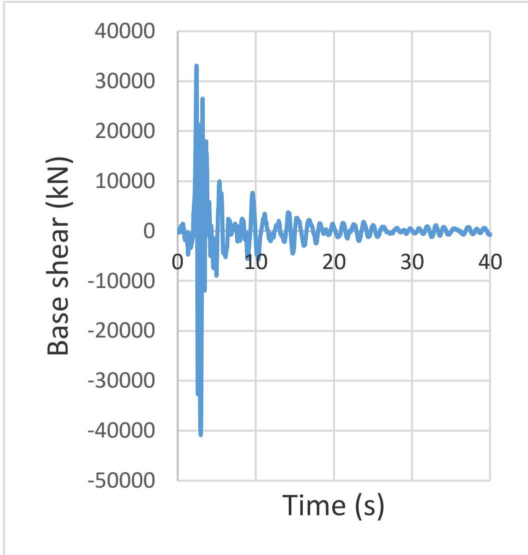
d) 4 C section.

Figure 4.27 Base shear time history variation of irregular C section models

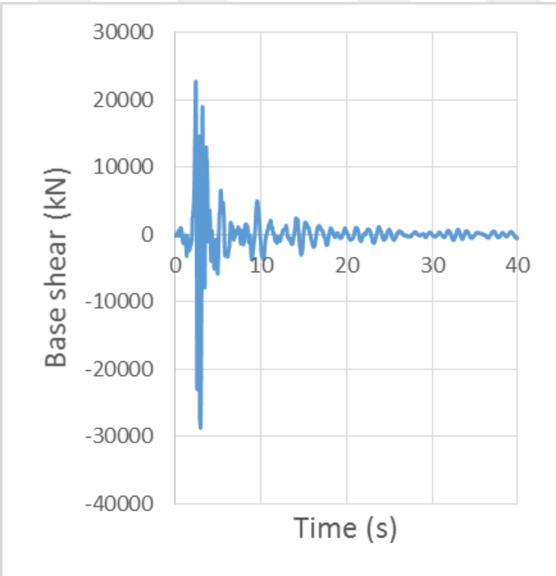
Figures 4.25 and 4.28 demonstrate the base shear time history of regular building and irregular buildings (1O, 1H, 2O, 2H). From the figures, the maximum base shear values were observed as 43700, 40840, 37120, 30980 and 28710 kN, respectively. In accordance with the result of the analysis, the base shear of the regular building was greater than the irregular buildings since the irregularity decreased the base shear.



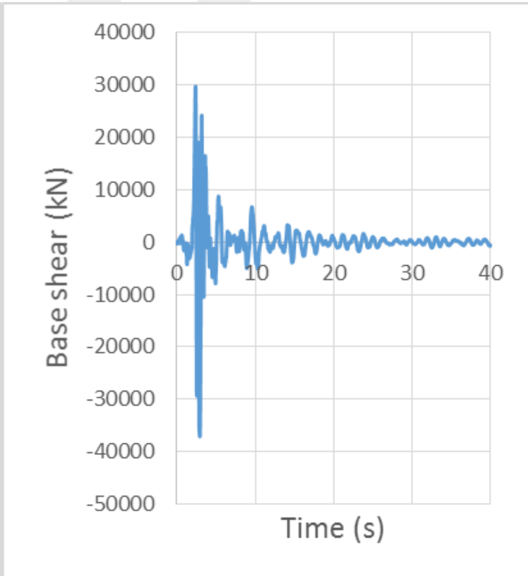
a) 1 O section.



b) 2 O section.



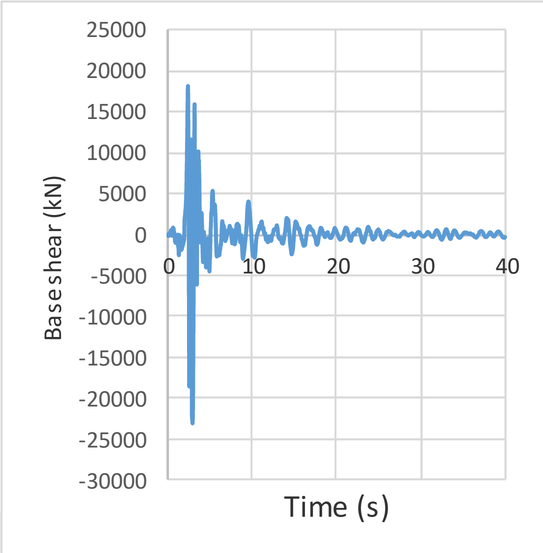
c) 1 H section.



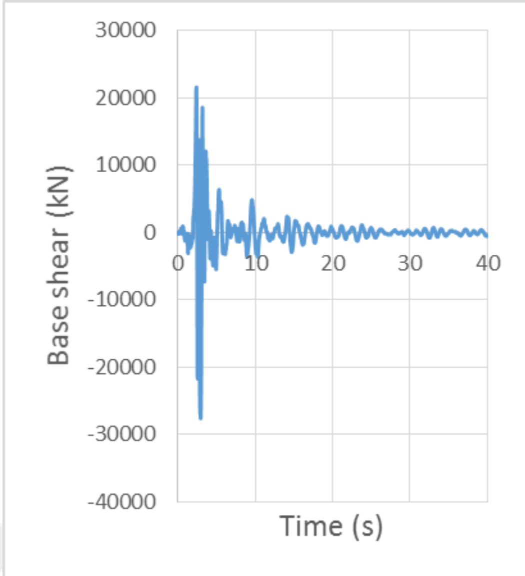
d) 1 H section.

Figure 4.28 Base shear time history variation of irregular O and H section models

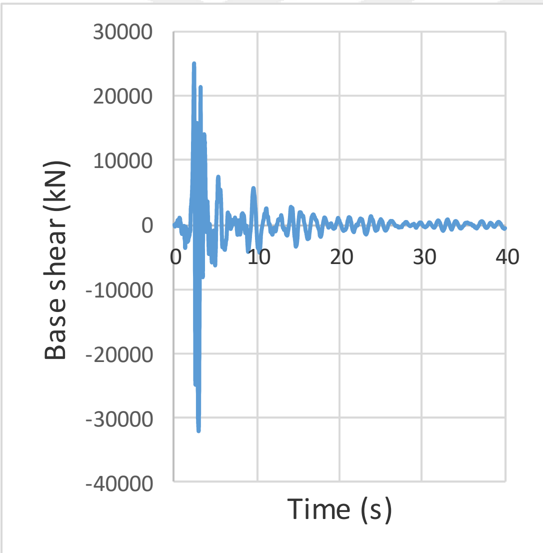
The base shear time history of the regular and irregular buildings under the earthquake acceleration are given in Figures 4.25 and 4.29. It was observed from the figures, the effect of plan irregularity remarkably decreased the value of the base shear. Under the Altadena-1 earthquake acceleration, as also previously mentioned, the maximum base shear of the regular building was obtained about 43700 kN. On the other hand, under the same earthquake acceleration, the maximum base shear of the irregular buildings 4⊥, 3⊥, 2⊥and 1⊥ were equal to approximately 36060, 32080, 27610 and 23070 kN, respectively.



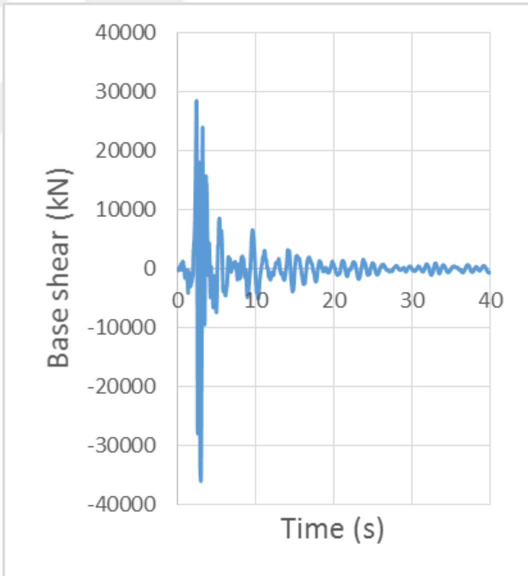
a) 1 ⊥ section.



b) 2 ⊥ section.



c) 3 ⊥ section.

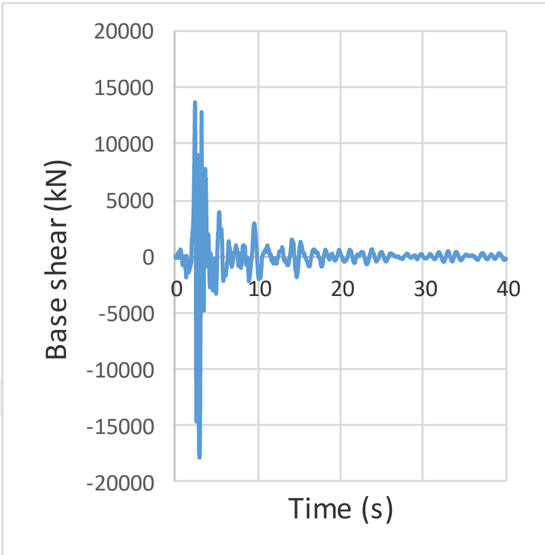


d) 4 ⊥ section.

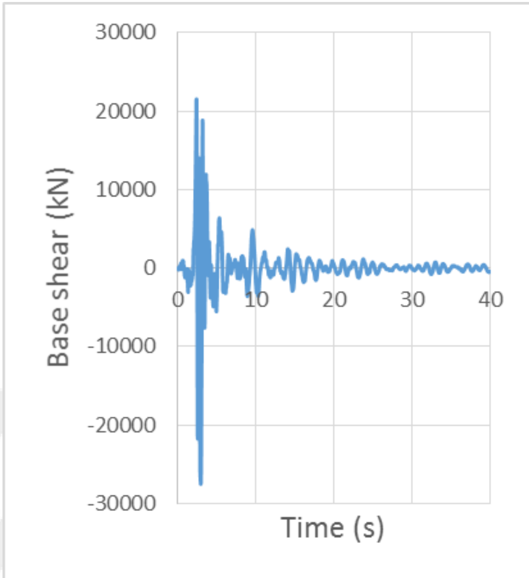
Figure 4.29 Base shear time history variation of irregular ⊥ section models

From the nonlinear dynamic analysis which gives valuable information about the strength and stiffness characteristics of the structures, the base shear were obtained for the regular building and irregular buildings 0.33L, 0.5L, 0.67L and 0.83L as 43700, 39830, 35070, 27530 and 17870 kN, respectively. Similar to the other irregularities, L shape floor plan irregularity caused less base shear value. Figures

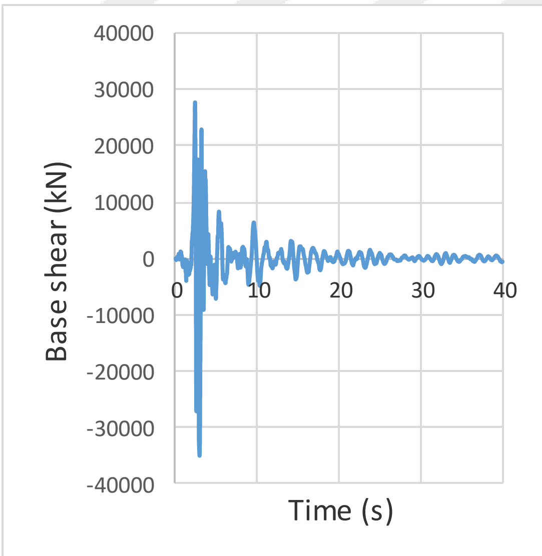
4.25 and 4.30 show the comparison of the base shear time history of different building models.



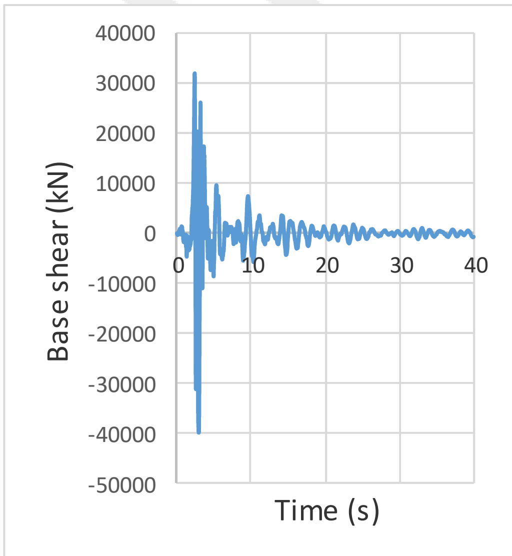
a) 0.83 L section.



b) 0.67 L section.



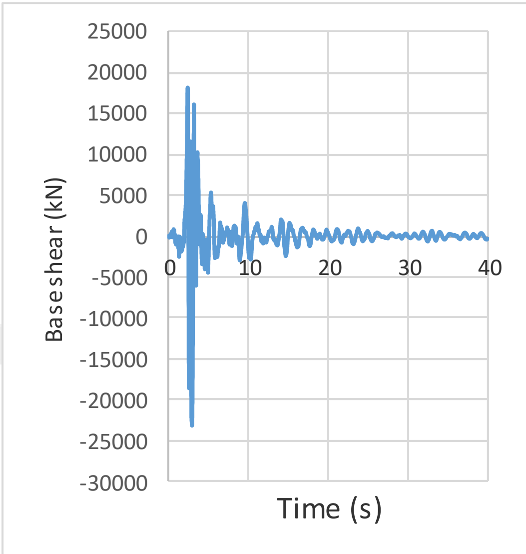
c) 0.5 L section.



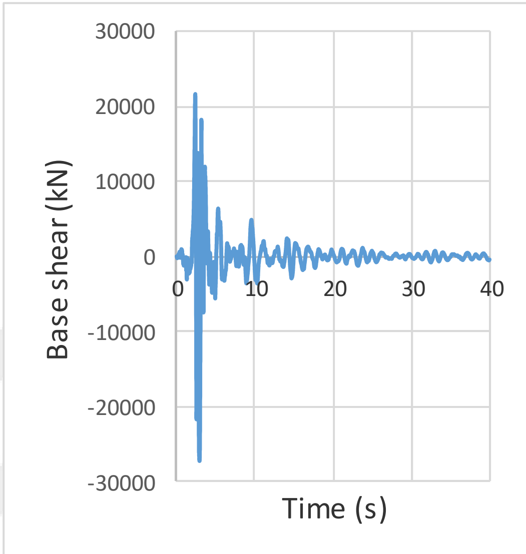
d) 0.33 L section.

Figure 4.30 Base shear time history variation of irregular L section models

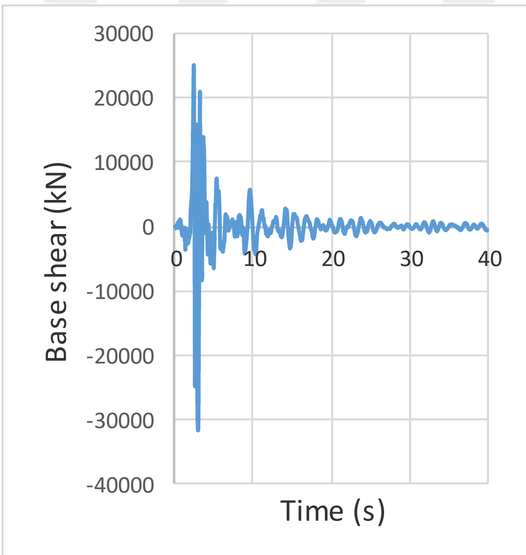
In Figures 4.25 and 4.31, the base shear time history of the regular building and irregular buildings 4Z, 3Z, 2Z and 1Z under Altadena-1 earthquake acceleration are compared. As it was observed from the figures, the irregularity remarkably decreased the value of the base shear. The peak base shear of regular building and irregular buildings (4Z, 3Z, 2Z and 1Z) were noted as 43700, 35610, 31680, 27240 and 23150 kN, respectively.



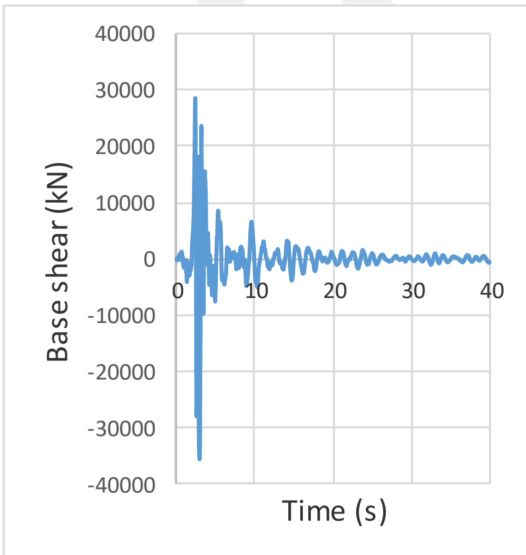
a) 1 Z section.



b) 2 Z section.



c) 3 Z section.



d) 4 Z section.

Figure 4.31 Base shear time history variation of irregular Z section models

CHAPTER 5

CONCLUSIONS

In this study, the nonlinear static pushover analysis and nonlinear time history analysis were carried out for regular building and twenty four irregular buildings of eight story. From these analyses, the capacity curve, story displacement time history and base shear time history were obtained for the regular and irregular buildings. Depend on the results observed in this study, the following conclusions can be drawn:

- There was an inverse relationship between the plan irregularity and the base shear of buildings. When the irregularity increased, the base shear of the buildings decreased. Consequently, the lateral load carrying capacity of the building declined directly.
- The regular building had considerably greater capacity in comparison to irregular buildings due to the fact that irregularity caused decreasing the capacity of the buildings. For example, the maximum base shear of regular building and irregular buildings of 0.33L, 0.5L, 0.67L and 0.83L were about 138524, 124412, 108732, 86097 and 52772 kN, respectively. The base shear comparison between regular building and irregular buildings indicated 10.1%, 21.5%, 37.8% and 61.9% differences for 0.33L, 0.5L, 0.67L, and 0.83L, respectively. More differences in the maximum base shear value were occurred due to increasing in the irregularity value of the buildings.
- Large displacements were observed in the irregular buildings with respect to the regular building. It indicated that the building with severe irregularity showed the maximum displacement.
- The effect of plan irregularity could be noted clearly and caused increasing the value of the maximum displacement. This is more pronounced at the roof level of the buildings.

- The storey displacement was influenced by the number of stories and frame type. It was found that the displacement of regular building and irregular buildings increased through the height of the structures. The maximum displacement was obviously appeared in the top storey of the buildings. However, the amount of the displacement varied, depending on the amount of the irregularity considered.
- The response of structure against seismic forces changed with plan irregularity. The behaviour of the buildings were observed that 0.83L shape building displaced more nearly 60 cm, these may be due to lesser weight and slender geometry as in comparison to other plans of irregular buildings. Considering all these above factors, complex shaped buildings gave large response in terms of the displacement under the given loading conditions. Hence, it may be said that simple shape geometry of structure could be adopted to minimize the effects of seismic actions.
- It was noted that the base shear time history variation of regular building was greater than that of the base shear of irregular buildings, which implied that the base shear increased with decreasing the irregularity. For example, the maximum base shear for the regular building and irregular buildings 4+, 3+, 2+ and 1+ were obtained as 43700, 39160, 30880, 26940 and 17560 kN, respectively. So, it was concluded that the structures were more vulnerable when they have more irregularities.

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