Seismic Behavior of Reinforced Concrete Frame Structures with and without Masonry Infill Walls

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ABSTRACT

Brick walls are often used as an infill element serving as partitions or as cladding in structure frames. In structural frame design method, infill walls are usually considered to be inert “nonstructural” elements and known for affecting on strength, stiffness and post peak behavior of the structure. The structure is assumed to carry the transverse loads by the frame elements resisting primarily in flexure. Often action of infill wall in frame analysis is ignored in the seismic area which is not on safe side and creates a major hazard during earthquake. RC frames having brick walls are a universal practice in countries like Turkey, where the region is prone to seismic activity. The structures in high seismic areas are greatly vulnerable to severe damages. Apart from the gravity load structure has to withstand to lateral load which may develop high stresses. Nowadays reinforced concrete frames are most common in building construction use around the world.

In this study, all the case studies are design under Turkish Building Codes TS 500 and Turkish Earthquake Codes TEC2007. An extensive analysis of typical RC building configurations, including brick masonry infill walls arranged either regularly or irregularly (creating soft-storeys) has been carried out. Pushover analysis method was carried out in SeismoStruct. Each case was compared to find out the performance of brick wall on RC frame.

Keywords: Pushover analysis, Infill panel, RC frame, Earthquake, SeismoStruct, Brick wall, Soft-story.

ÖZ

Tuğla duvarlar betonarme çerçeve sistemlerinde bölme duvar olarak sıklıkla kullanılmaktadır. Bu elemanların yapıların dinamik özellikleri üzerinde olumlu veya olumsuz etkileri olabilmektedir. Ancak, yapısal analizlerde dolgu duvarların sadece ölü yükleri hesaba katılarak bu etkiler gözardı edilmektedir. Bu da kimi zaman deprem etkisinde tehlike yaratabilmektedir. Betonarme çerçeve dünyada sıklıkla kullanılan yapı sistemlerindendir. Türkiye gibi deprem riski olan ülkelerde dolgu duvarların olumsuz etkilerinin yapısal analizlerde ihmal edilmesi, deprem yüklerinin etkisini artırabilecektir.

Bu çalışmada seçilen örneklemelerde TS500 ve 2007 Türk Deprem Şartnameleri kullanılmıştır. Örneklemelerde dolgu duvarların olumlu etkileri için düzenli, olumsuz etkileri için ise düzensiz olarak (yumuşak kat oluşumu da gözetilerek) yerleştirildiği durumlar ele alınmıştır. Düzenli ve düzensiz yerleştirilmiş dolgu duvarlara sahip yapılar, dolgu duvarların olmadığı sistemlerle de karşılaştırılmıştır. Bu maksatla SeismoStruct programı ile yapılan statik itme analizi sonunda elde edilen kapasite eğrileri kullanılmıştır.

Anahtar Kelimeler: Statik itme analizi, dolgu duvar, betonarme çerçeve, deprem, SeismoStruct, tuğla duvar, yumuşak kat.

Dedication

To the shining souls

for affirming the ideals of nonstop struggle,

which somehow made their kids educated and thus harmonized the humanity,

my

parents

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I wish my heartfelt gratitude to the praiseworthy supervisor, Asst. Prof. Dr. Giray Özay**,** for the apprehension with which he guide me and for his caring and sympathetic attitude, productive and stimulating criticism and valuable suggestions for the completion of this thesis. His zealous interest, and constant encouragement and intellectual support accomplished my task.

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Roman Bin Karim.

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LIST OF SYMBOLS

b_w	Diagonal strut thickness
d_w	Strut length
z	Contact length between frame and wall
h_z	Contact distance
λ	Relative stiffness between wall and reinforced concrete frame.
E_m	Masonry modulus of elasticity
E_c	Elastic modulus
I_c	Moment of inertia of concrete columns
Θ	Diagonal strut angle along beams
A_0	Effective ground acceleration coefficient,
I	Building importance factor,
$S(T)$	Spectrum coefficient,
g	Gravitational acceleration
$S(T)$	Spectrum Coefficient
T	Building natural period
$f'_{m\theta}$	Strength of masonry after transversely load is applied at θ
A_{ms}	Area of the equivalent strut
K_s	Stiffness of the shear spring

Chapter 1

INTRODUCTION

1.1 General

Masonry walls are often used as an infill element serving as partitions or as cladding in structure frames. In structural frame design method, infill walls are usually treated as a “nonstructural” element. The structure is assumed to carry the transverse loads by the frame elements resisting primarily in flexure.

From geometrical considerations it is evident that a rational firmly strong wall having finite stiffness will delay deformations compatible with frame under earthquake action. The frame having infill wall is usually firm and rigid than frame without infill wall. Ignoring the bond between frame and infill wall is equivalent in effect to neglect a very important structural contribution. Also, critical regions in the frame-wall composite may not be the same as those in the frame alone and designer may have some risk on brittle links of the frame-wall composite. So that way there is a noted view of having greater strength and stiffness in infill wall frame then frames with infill wall among many researchers. The lateral stiffness is also increase in the presence of infill wall. Due to the change in the mass and stiffness it will also automatically change the dynamic aspect of the structure. By knowing importance and negative behavior of infill frame structures and having a satisfactory method of analysis will help us to have more safer and economical solutions. In the past earthquake show us that infill wall had a vital reflex on the stiffness and resistance of buildings to withstand.

The behavior of the infill frame under seismic loading is very complicated and puzzling. Since the behavior is nonlinear and closely related to the link among frames and infill, it is very complex to find out it by analytical methods unless by using the experimental data for analytical procedure. Due to the complicated behavior of such composite structures, analytical as well as experimental research is of great importance to determine the stiffness, strength, and dynamic characteristics at each step of loading.

1.2 Scope and Aim of the Study

This research is about building structures with reinforced concrete frames having masonry infill under dynamic base excitation as Pushover analysis under seismic response is conducted. An important literature review is conducted with the purpose to summarizing results from previous research works as it worth nothing that, due to practical limitations, the different factors affecting the structural response of infilled frames cannot be investigated in a single research programme. Therefore, general conclusions should be obtained by complementing results from different sources. The main aim of this study is summarized as follow:

To observe the effect of brick infilled wall structure on the RC frame structure.

- To know about the behavior of masonry materials and performance of brick wall subjected under the shear and compressive loading.
- To find seismic, failure mode and main principle factors affecting the response.
- To observe positive and negative effect of infill wall on RC frame.

- To form an easy and compatible procedure of the evaluation of shear and compressive strength of masonry, including those parameters strongly affecting the response of infilled frames.
- To study the advantages and disadvantages of different analysis of frame structure with brick walls.
- To develop a macro-model to be used by designers with representing the main characteristics of these types of structure and simple equations.

1.3 Methodology of Thesis Work

The proposed methodology consists of the following steps:

- Different cases will be briefly discussed and then designed according to TEC2007 in design software known as Idecad.
- Reinforced concrete frame with bare frame, soft-storey and with fully infill wall be modeled in SeismoStruct using pushover analysis.
- The bricks wall will be model according as equivalent strut element.
- Dead load of wall on beam and diagonal strut will be considered only as active member having zero weight.
- Earthquake load will be applied as incrementally in order to monitor the formation of plastic hinges, stiffness degradation and plastic rotation.

1.4 Organization of the Thesis

The investigation conducted to get the goal of this study is presented in several chapters which are organized in a way to understand the research work step by step. This thesis contains seven chapters. The basic contents of chapters are detailed as follow:

- Chapter 1: Introduction is given to state the general idea of the objectives of the thesis and also its methodology, aim and scope.

- Chapter 2: Provides a brief review of past work on this research study
- Chapter 3: Earthquake analysis and performance analysis various parameters used in pushover analysis are discuss in detail.
- Chapter 4: The design and analysis procedure and different structural parameters used in SeismoStruct are discussed in detail.
- Chapter5: In this chapter the methodology and applied procedures on case studies using Idecad and SeismoStruct are given. Also different Structural parameters are discussed.
- Chapter 6: The outcomes of the applied procedures on case studies are given..
- Chapter 7: The summary of this study with drawbacks and recommendations for the future work are presented.

Chapter 2

LITERATURE REVIEW

2.1 Introduction

Walls are generally built in buildings by infill panel part of the frame such as brick, concrete blocks, etc. The structural interactions between frame and infill panels are often avoided in the design which is not good for seismic design point of view. This interaction has a major effect on the overall seismic response of the frame and also on the response of the individual member of a building. Many details of earthquake damage to both have been filed by Stratta and Feldman (1971). The previous works on infilled framed will be studied in this chapter.

2.2 Infill Panel

It has been observed that effects of infilled wall were not taken in the design and analysis of the building, due to lack of research and experimental work. The main concept of neglecting infilled wall throughout the analysis is due to its non-linear behavior. The uttermost primary cause of non-linear action of infilled frames is arises from material's non-linearity, which requires very complicated computing method for designing building. According to new researched, reinforced concrete structures having infilled wall can considerably add to the strength, firmness and energy dissipation individuality of frame of a structure.

In sort for learning more about the behavior of infilled wall and its failure mode's, numerous analytical models are suggested by researchers. These models are defined into two main groups, which are named as micro-model and macro-models.

2.2.1 Micro-Models

Micro-model is mostly defined by means of Finite Element method. In this method unlike elements are used for modeling such as plane element for representing infill wall, beam element for adjoining frame, and integrate element for wall and frame contact. In this model brick and plaster constrains are to be define separately. The importance to use finite element method to get all feasible failure modes; somehow it has limited use because of complex computational effort and the long period taken to analysis and model. Among many study on micro-models, the publications are Mallick and Severn 1967, Stafford 1962, Gooman 1968, KIng and Pandey 1978, Dhanasekar 1985 [1] [2] [3] [4] [5].

2.2.2 Macro-Models

Due to the computational difficulties requirement using micro-models, the researchers come through with a simple method to model an infill panel within a single element. Macro-modeling has shown inclusive effects of infilled panel on structure under tangential loads.

Ever since the very first attempt by Polyakov (1956) [6], experimental and analytical test has revealed that diagonal strut within the correct mechanical property can give an answer to the problem. Many researchers changed single strut properties to multiple strut configurations to know outcome of micro cracking's at end of infill which is by higher shear strength and tensile stresses of the infilled wall in a frame. Paulay and Priestley experimental works represented below. [7]

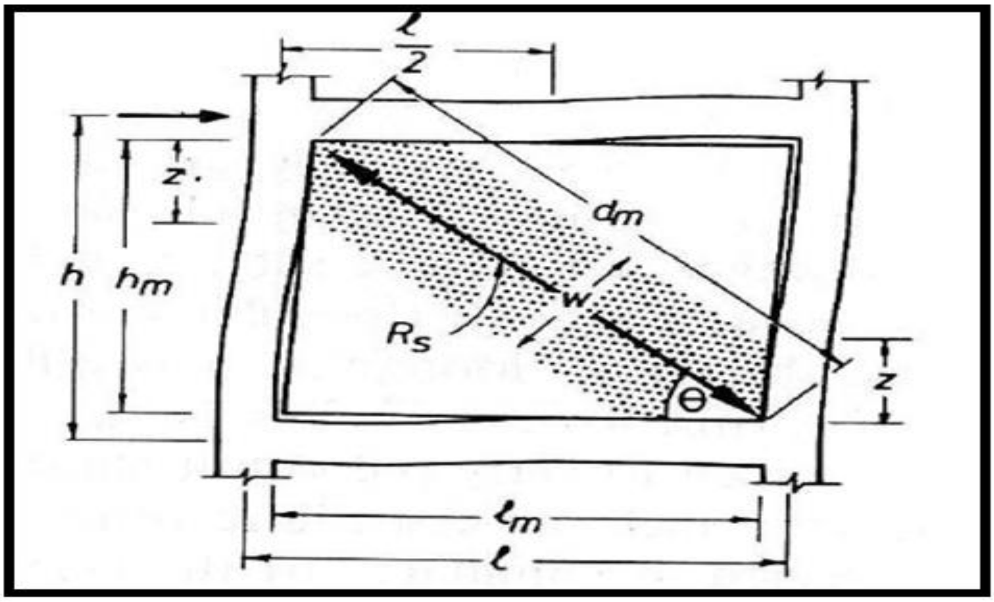

Figure 2.1: Frame bending under shear load [7]

Holmes (1961) recommended for replacing infilled panel by equivalent pin jointed diagonal strut as diagonal strut with geometry and material as same as infilled panel. Diagonal strut thickness 'b_w' is equal to $\frac{1}{3}$ of the strut length 'd_w' is used as shown below i.e.

$$b_w = \frac{d_w}{3} \tag{2.1}$$

Stafford (1966) performed different tests on square steel infilled frames. According to his observation the length between frame and wall is related to the strut's width. He proposed a relation from experimental result to find the contact length between frame and wall. [8]

$$z = \frac{\pi}{2\lambda} \tag{2.2}$$

$$\lambda = \sqrt[4]{\frac{E_m t_w \sin(2\theta)}{4 E_c I_c h_c}} \tag{2.3}$$

Where,

λ: relative stiffness between wall and reinforced concrete frame.

E_m : Masonry modulus of elasticity

E_c : Elastic modulus

I_c : Moment of inertia of concrete columns

θ: diagonal strut angle along beams

In 1971 researcher named Mainstone conducted a test on small size specimen with h= 406 mm which was transversely loaded in compression and proposed an expression shown below: [9]

$$b_w = 0.16\, {\lambda_h}^{-3} d_w \qquad (2.4)$$

Berter and Klingner in 1978 base on the scale test made by Mainstone (1971) suggested the following equation;

$$\frac{b_w}{d_w} = 0.175(\lambda * h)^{-0.4} d_w \qquad (2.5)$$

Liauw and Kwan in 1984 expressed the relation from past experimental data as:

$$b_w = \frac{0.95 h_m \cos\theta}{\sqrt{\lambda \,.h}} \qquad (2.6)$$

In the above equation θ was assumed to be 25° and 50°.

Crisafulli compare the difference of factor λ_h with the ratio $\frac{b_w}{d_w}$ and Figured that $\frac{b_w}{d_w}$ decreases as λ_h increases. This difference is presented in the Figure 2.2.

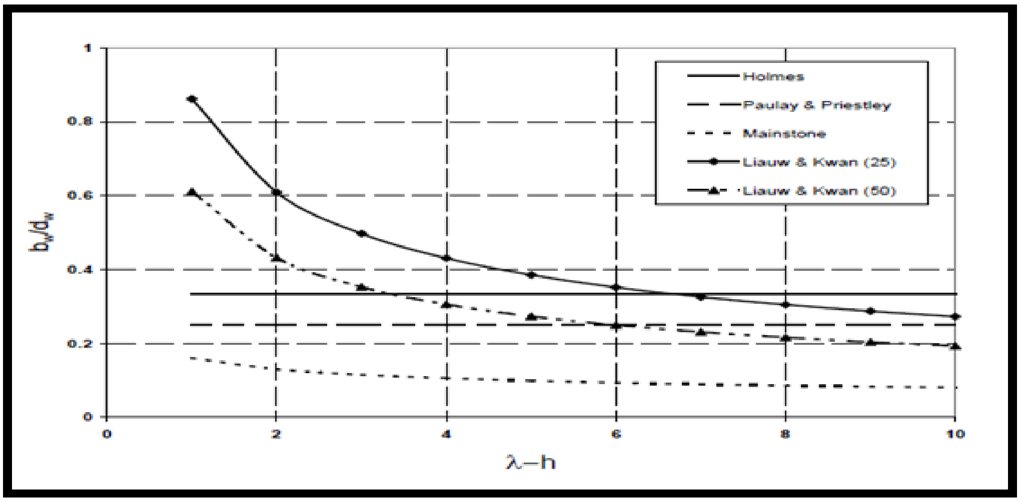

Figure 2.2: The deviation of $\frac{b_w}{d_w}$ for infilled panel as a function of λ.h [10]

In 1987 Decanini and Fantin consider cracked and uncracked effect of masonry and propose an equations base on the outcome from tested masonry framed by tangential force. The variations are shown in Figure 2.3;

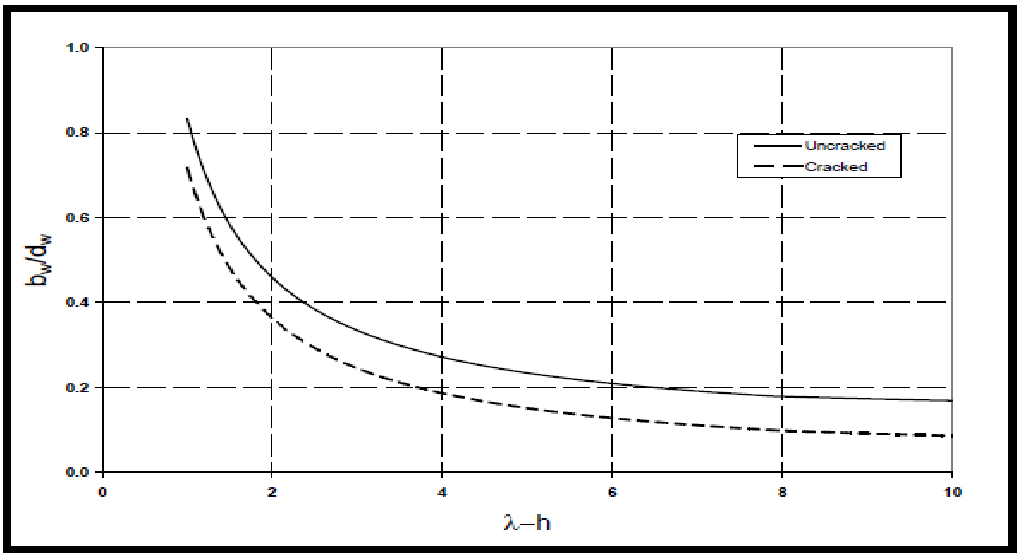

Figure 2.3: The ratio of $\frac{b_w}{d_w}$ as a function of λ.h [10]

2.3 Model Proposed for the Analysis of Infilled Wall Frames

Crisafulli (1997) adopt method of equivalent diagonal strut as previously discussed for macro-modeling of infilled frame structure by considering multi-strut as given in

Figure 2.4. This study determined out the limitation and influence differences between multi-strut and single-strut model on response structure [10].

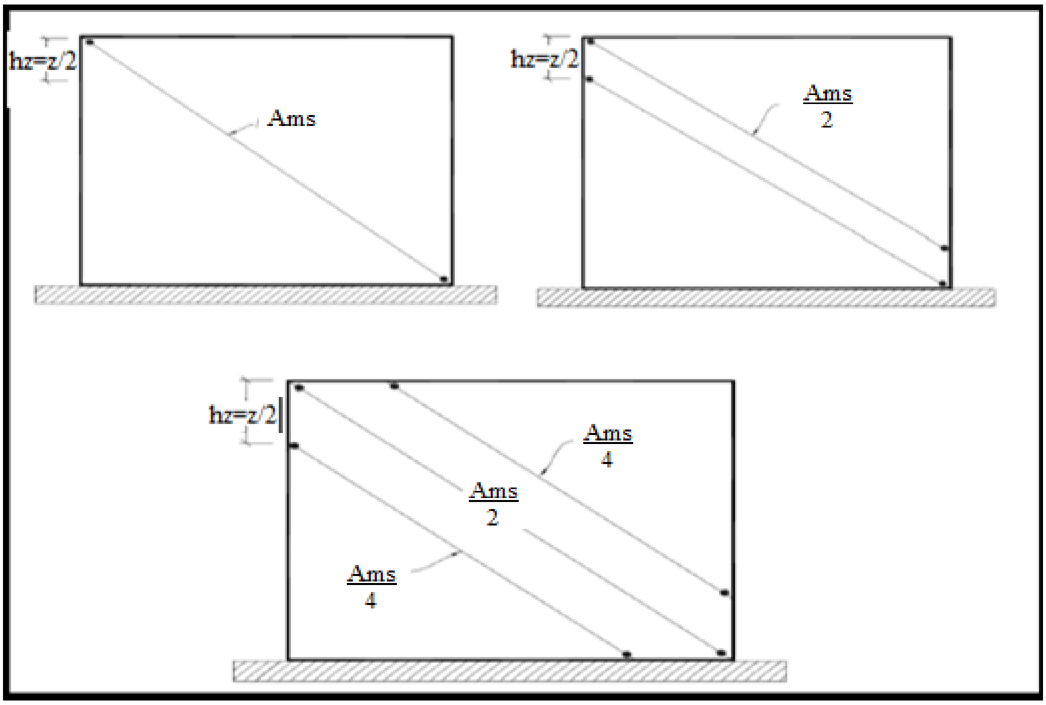

Figure 2.4: Strut models modified [10]

Micro-model formulation is compared with the result comes from three strut model. For finite element model nonlinear effects were considered to represent the panel frame interface. The area of equivalent strut is kept constant.

Stiffness is similar in all cases of infilled frame from the test results of different strut models. It decrease slightly for two and three strut models, however three strut model shows significant change in stiffness which depends on contact distance h_z, which is function of contact length z. It was also observed from the results that single strut model under-estimated the bending moment, two strut model showed much larger values while three strut model constituted better approximation with the finite element model.

2.4 Cyclic Behavior of Infill Wall Panel

Crisafulli (1997) proposed a hysteric model mentioning behavior of brick wall towards cyclic loading. This model was compared with non-liner response of masonry. It allows variation of strut's cross section as a function of the axial deformation by element, considering the stiffness loss between frame and masonry panel due to short contact length. Stress and strain relationships for this model is shown below; [10]

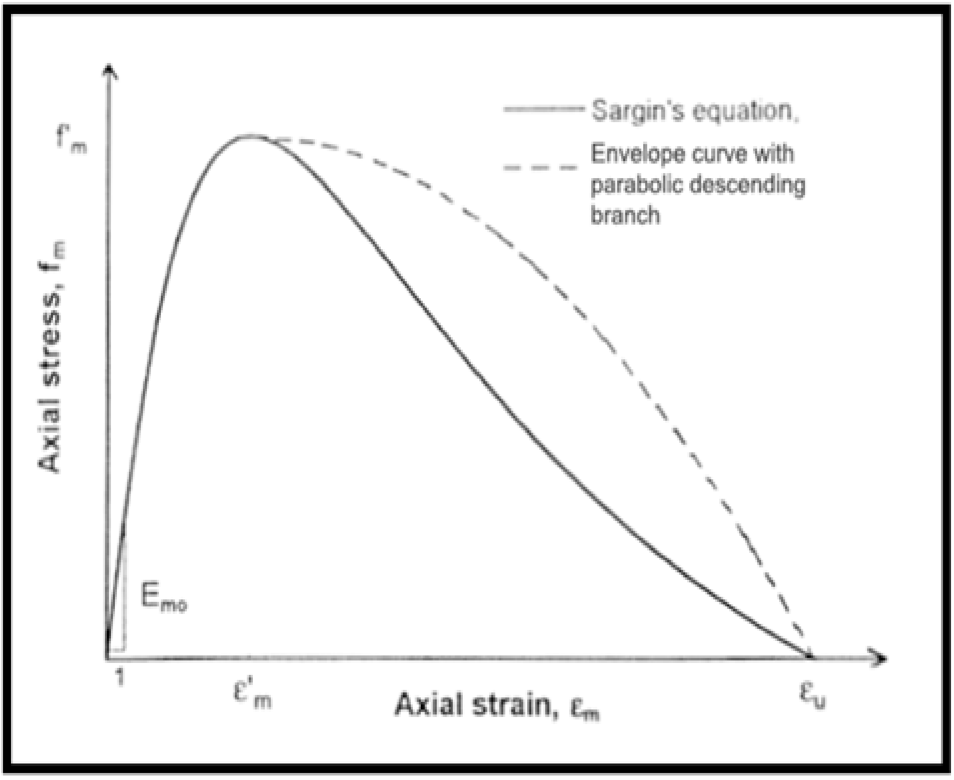

Figure 2.5: Strain stress curve [10]

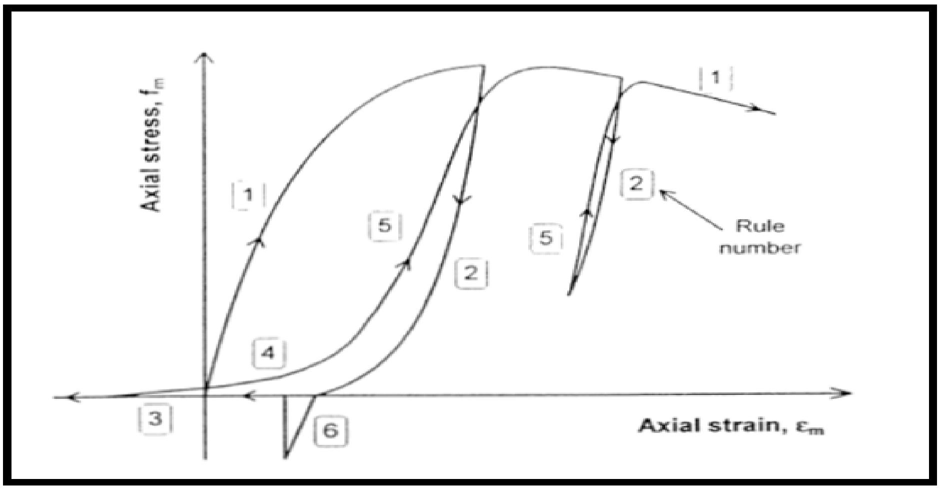

Figure 2.6: General cyclic behavior of masonry [10]

2.5 Soft- storey

A soft-storey (weak storey) is that storey of a building in which the resistance or stiffness is substantially less compared to the stories below or above it. In other words a soft storey has poor shear resistance and energy absorption capacity (poor ductility) to with hold the seismic-induced building stresses. Generally a soft-storey is at ground floor of the structure. It is because to have an open access to the public in the building. Thus it may contain open large areas between columns without poor shear resistance. Due to soft-storey, the first floor is subjected to large amount of stress, which causes the poor resistance to earthquake motion of a soft storey at the ground floor.

2.5.1 Defining of a Soft-Storey by Turkish Earthquake Code TEC 2007

The case in which stiffness irregularity factor (η_{ki}) in two orthogonal earthquake directions is more than 2, hence considered as soft storey. The relation is shown below by Equation [11] [12].

$$\eta_{ki} = \frac{\left(\frac{\Delta_i}{h_i}\right)_{ave}}{\left(\frac{\Delta_{i-1}}{h_{i-1}}\right)_{ave}} > 2.0 \qquad (2.10)$$

On the other side, according to TEC 2007, a storey is considered to be a soft storey if the effective shearing area of any storey to the next upper one is less than 0.8. The relation is given as [11]

$$\eta_{ci} = \frac{(\Sigma A_e)_i}{(\Sigma A_e)_{i+1}} < 0.8 \tag{2.11}$$

The following are the two examples of buildings having a soft story on the ground floor;

1. Chi-chi earthquake in Taiwan (September 21, 1999)

In Taiwan, it was a common practice to have an open first floor area by using columns to support under the floor. In many cases, the area between the columns is filled with the help of plate glass windows in order to create shops at the ground floor. This type of construction and the resulting damage caused by the Chi-chi earthquake is given in Figure 2.7;

Figure 2.7: Damage due to a soft story at the ground floor during Chi-chi earthquake in Taiwan (September 21, 1999) [13]

2. **Izmit earthquake in Turkey (August 17, 1999)**

According to Bruneau (1999), a general RC frame structures in Turkey consists simple symmetric floor plan, having rectangular or square columns and connecting beams. Ground stories (soft-stories) are commonly use as shops and business purpose, mostly in central part of cities. These areas are infilled with glass windows, and occasionally with single masonry infill as shown in Figure 2.8;

Figure 2.8: Damage due to a soft story at the ground floor during Izmit earthquake in Turkey (1999) [14]

2.5.2 Seismic Behavior of Infill Frame with Soft Storey

The soft storey has functional and technical advantages over the regular traditional construction. First, is the devaluation in base shear and spectral acceleration as in base isolated structures due to the increase of natural period of vibration, nonetheless, these decreases in force help in increasing in spectral inter storey draft and displacement, lead to a significant P-Δ effect, which is a great threat for the stability of the building (Figure 2.9) [15].

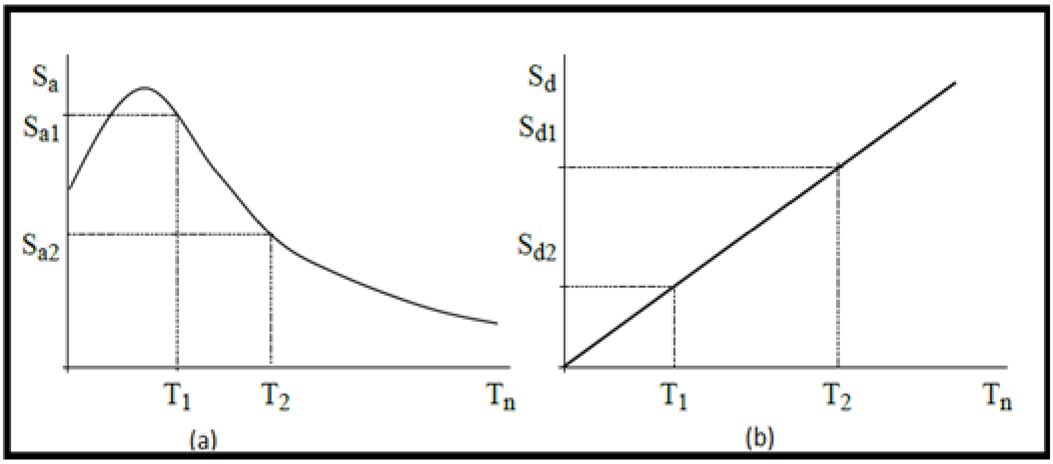

Figure 2.9: (a) Design earthquake spectral acceleration (Sa) versus time period (Tn); (b) Design earthquake spectral displacement (Sd) versus time period (Tn) [15]

Secondly, a taller soft storey in some cases is used for purpose of parking the vehicles or retail shopping, large space area for meeting room or banking hall as shown in Figure 2.10 [16]. Due to this, soft storey has less stiffness in columns as compared to the columns stiffness in upper floor frames, which are typically constructed with masonry infill walls [13].

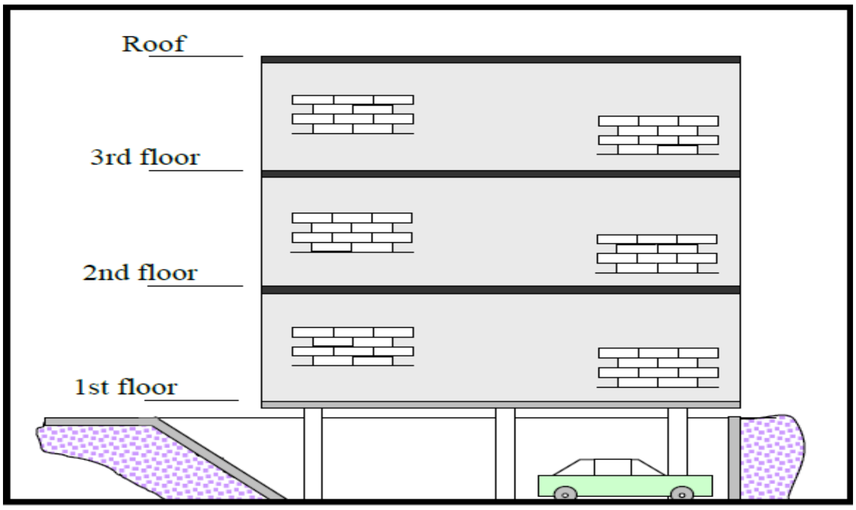

Figure 2.10: Construction of Soft storey types [16]

Soft storey failure is due to a combination of different objectionable reasons, such as P-Δ effects, torsion effect, enormous mass in upper floor, and inadequacy of ductility in ground storey. The P-Δ effect refers to the abrupt changes in ground shear, overturning moment, and/or the axial force distribution at the base of a sufficiently tall structure or structural component when it is subject to a critical lateral displacement.

The walls in upper stories make it stiffer than open ground storey. Therefore, higher stories move nearly equally acting as single block. In other words such structures swing back and forth during earthquake motion and the columns at open ground storey are objected to severe stresses as shown in Figure 2.11.

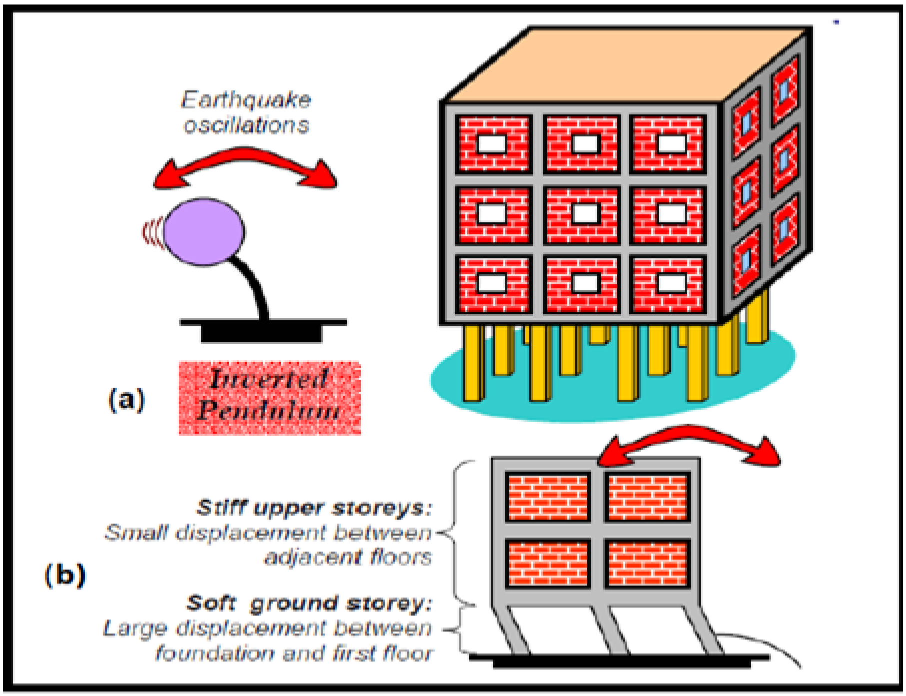

Figure 2.11: Upper stories of soft storey buildings move together as a single block [16]

Many researchers have studied about the behaviour of soft-storey reinforced concrete frames under seismic loading among them are Vasseva (1994), Arlekar, et al. (1997), Elnashai (2001), Dolsek and Fajfar (2002) [17] [18].

2.6 Failure Modes of RC Frames with Masonry Infill

Failure modes of masonry infilled frame show variations according to different properties of frame and infill wall. During the computation of lateral stiffness as well as strength of frame with masonry infill wall, it's essential to estimate various serious modes of failure. The common modes of frame failure are due to tension failure of nearby elements of a column or shear failure of the beams or columns as shown in Figure 2.12 [19].

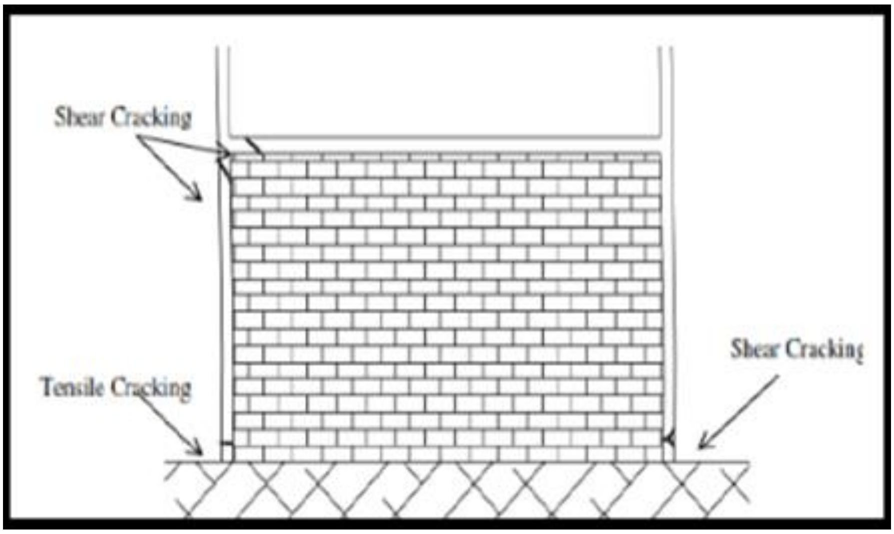

Figure 2.12: Failure modes of RC frame [19]

The failure of infill wall occurs due to the effect of the following modes; (a) Shear cracks occurs between the mortar and bricks along the interface between them (b) Cracking through the mortar joints and masonry due to the tension (c) Local crushing of mortar or masonry in compression corner of the wall as shown in Figure 2.13.

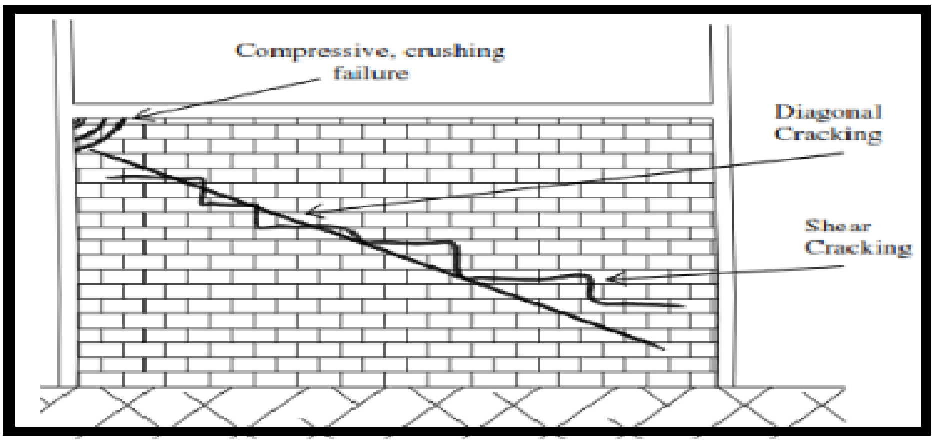

Figure 2.13: Infill failure mode [19]

Infill masonry wall shear failure is directly associated with horizontal shear caused in infill panel by load applied. Apart from the three modes of failure, another mode of failure which is known as sliding shear failure. If this failure occurs, the diagonally braced pin-jointed frame changes to knee braced frame, which results in shear failure of columns surrounding detail in Figure 2.14 [7].

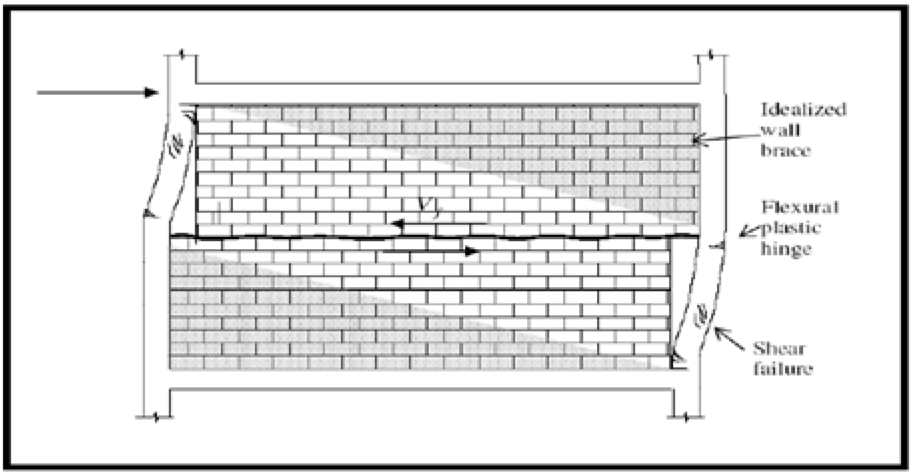

Figure 2.14: Infill with Sliding Shear Failure [7]

According to the research of Marzhan (1998) [20] , the walls with in-plane action may collapse in three main failure modes which are sliding, flexural and shear as given in Figure 2.15;

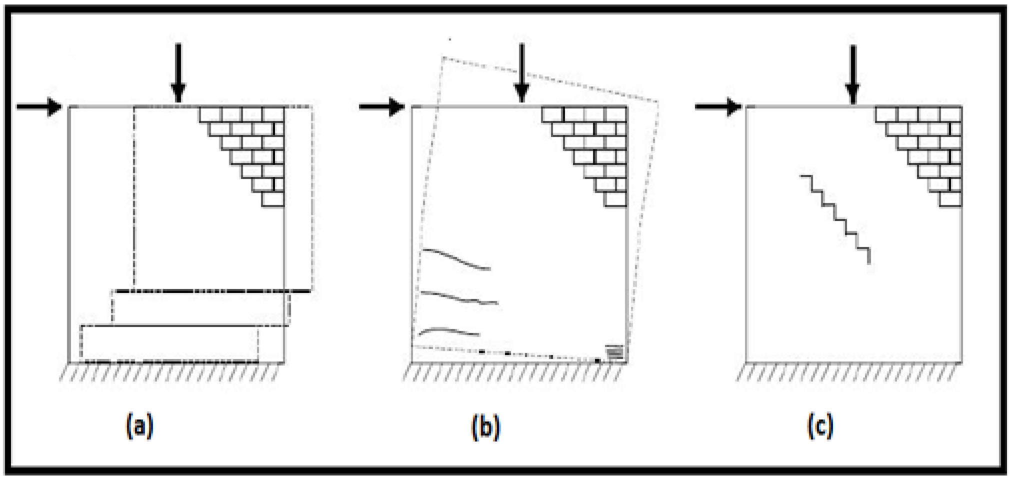

Figure 2.15: (a) Sliding Failure, (b) Flexural Failure, (c) Shear Failure [20]

2.7 Interaction of Frames and Infill Panel

The outcome of infill masonry walls on the response of reinforced concrete frames encountered to seismic action is commonly recognized and is subject of various investigations. The possible effect of interaction of infill panels on frame are as following;

- The existence of infill walls does not affect on structural response. In this case, infill walls are very flexible and lighter in weight, or completely isolated from the reinforced concrete frame.
- The infill walls are determined to have some denoting affect on structural response, and expected to be in elastic range.
- The infill walls are determined to have a denoting affect on the structural response, and estimated to undergo considerable damage during earthquake. In such cases the large probability of formation of a soft storey should be known and taken in calculation.

2.8 Effect of Infill Panels on Overall Seismic Response:

The main effects of infill wall on overall seismic response of structure are as follow:

- To increment stiffness, this tends to increase base shear response in the majority seismic action.
- To increase overall ductility of the structure.
- To develop the shear distribution all the way through the structure.

Chapter 3

SEISMIC DESIGN AND PERFORMANCE OF STRUCTURES

3.1 Introduction

In this chapter seismic analysis methods and performance analysis methods according to TEC2007 and Euro Code 8 will be summarized in this chapter.

3.2 Seismic Analysis According to TEC 2007

The Turkish Earthquake Code 2007 (TEC2007) requirement for design a structure in seismic zones was prepared under the direction of Prof. Dr .M.N Aydioglu. It is used for Turkey and Turkish Republic of Northern Cyprus, [11]. After the 1999 Marmara earthquake, which was the most dangerous earthquake of Turkey in the previous century, the requirements have been added to the Turkish Earthquake Code. 1998 disaster regulation was revised in 2007 in which the new regulation was called Specifications for Buildings to be built in Earthquake Areas.

3.2.1 Building Importance Factor

The basic principle of earthquake resistant design is to preventing structural and non-structural elements of buildings from damage. It limits the damage in the buildings to repairable levels in medium-intensity earthquakes, and prevents the comprehensive or partial collapse in high intensity earthquake to avoid losing life.

The Building Importance Factor used to design structure under earthquake action according to TEC2007 are described in the below table;

Table 3.1: Building Importance Factor [11]

Purpose of Occupancy or Type of Building	*Importance Factor* (I)
1. Buildings to be utilised after the earthquake and buildings containing hazardous materials **a)** Buildings required to be utilised immediately after the earthquake (Hospitals, dispensaries, health wards, fire fighting buildings and facilities, PTT and other telecommunication facilities, transportation stations and terminals, power generation and distribution facilities; governorate, county and municipality administration buildings, first aid and emergency planning stations) **b)** Buildings containing or storing toxic, explosive and flammable materials, etc.	**1.5**
2. Intensively and long-term occupied buildings and buildings preserving valuable goods **a)** Schools, other educational buildings and facilities, dormitories and hostels, military barracks, prisons, etc. **b)** Museums	**1.4**
3. Intensively but short-term occupied buildings Sport facilities, cinema, theatre and concert halls, etc.	**1.2**
4. Other buildings Buildings other than above defined buildings. (Residential and office buildings, hotels, building-like industrial structures, etc.)	**1.0**

3.2.2 Seismic Design

The spectral acceleration coefficient $A(T)$ given in equation (3.1) shall be used as foundation for determination of seismic loads. The elastic spectral acceleration S_{ae} (T), defined as the ordinate of elastic acceleration spectrum for 5% damped rate, and elastic acceleration spectrum is equal to spectrum acceleration coefficient times the acceleration of gravity ‘g’ as given in equation (3.2)

$$A(T) = A_0\, I\, S(T) \tag{3.1}$$

$$S_{ae}(T) = A(T)\text{g} \tag{3.2}$$

Here:

A_0 : Effective ground acceleration coefficient,

I : Building importance factor,

S(T) : Spectrum coefficient,

g : Gravitational acceleration (9.81 m/s^2),

The effective ground acceleration coefficient (A_o), is detailed in Table given below.

Table 3.2: Effective Ground Acceleration Coefficient [11]

Seismic Zone	A_0
1	0.40
2	0.30
3	0.20
4	0.10

The Spectrum Coefficient $S(T)$, given in Eq. (3.2) shall be determined by the following equations depending on local site conditions and the building natural period, *T* shown in Figure

$$S(T) = 1 + 1.5\frac{T}{T_A} \qquad (0 \leq T \leq T_A) \tag{3.3}$$

$$S(T) = 2.5 \qquad (T_A \leq T \leq T_B) \tag{3.4}$$

$$S(T) = 2.5\left[\frac{T_B}{T}\right]^{0.8} \qquad (T_B < T) \tag{3.5}$$

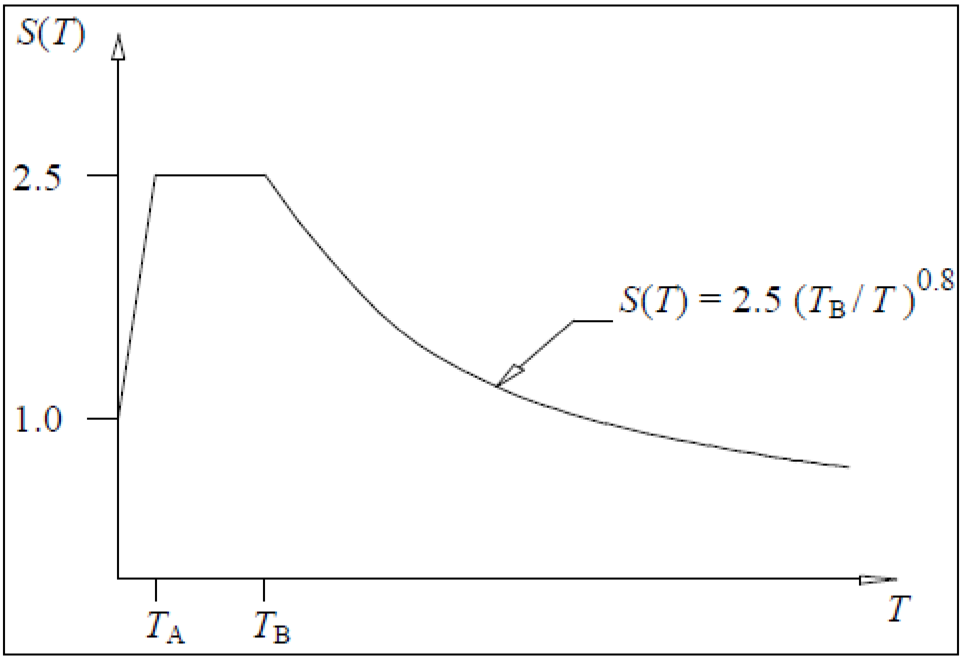

Figure 3.1: Design Acceleration Spectrums [11]

The spectrum characteristic periods, *T*A and *T*B, are specified in Table 3.3.

Table 3.3: Spectrum Characteristic Period [11]

Local Site Class	T_A (second)	T_B (second)
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

In order to consider the specific nonlinear behavior of the structural system during earthquake, seismic load reduction factor should be calculated according to equations (2.6) or (2.7) in terms of structural system behavior factor '*R*' detailed in Table 2.6 and defined for various structural systems and natural vibration period *T*.

$$Ra(T) = 1.5 + (R - 1.5)\frac{T}{T_A} \qquad (\,0 \leq T \leq T_A\,) \qquad (3.6)$$

$$Ra(T) = R \qquad (\,T_A < T\,) \qquad (2.7)$$

Table 3.4: Structural System Behavior Factors [11]

BUILDING STRUCTURAL SYSTEM	Systems of Nominal Ductility Level	Systems of High Ductility Level
1. CAST-IN-SITE REINFORCED CONCRETE BUILDINGS		
1.1. Buildings in which seismic loads are fully resisted by frames………	4	8
1.2. Buildings in which seismic loads are fully resisted by coupled structural walls …………	4	7
1.3. Buildings in which seismic loads are fully resisted by solid structural walls …………	4	6
1.4. Buildings in which seismic loads are jointly resisted by frames and solid and / or coupled structural walls …………	4	7
2. PREFABRICATED REINFORCED CONCRETE BUILDINGS		
2.1. Buildings in which seismic loads are fully resisted by frames with connections capable of cyclic moment transfer…………	3	7
2.2. Single-storey buildings in which seismic loads are fully resisted by columns with hinged upper connections …………	-	3
2.3. Prefabricated buildings with hinged frame connections in which seismic loads are fully resisted by prefabricated or cast – in – situ solid structural walls and / or coupled structural walls…………	-	5
2.4. Buildings in which seismic loads are jointly resisted by frames with connections capable of cyclic moment transfer and cast-in-situ solid and / or coupled structural walls …………	3	6
3. STRUCTURAL STEEL BUILDINGS		
3.1. Buildings in which seismic loads are fully resisted by frames ……..	5	8
3.2. Single – storey buildings in which seismic loads are fully resisted by columns with connections hinged at the top …………	-	4
3.3. Buildings in which seismic loads are fully resisted by braced frames or cast-in-situ reinforced concrete structural walls		
a- Centrically braced frames …………	4	5
b- Eccentrically braced frames …………	-	7
c- Reinforced concrete structural walls…………	4	6
3.4. Buildings in which seismic loads are jointly resisted by structural steel braced frames or cast-in-situ reinforced concrete structural walls		
a- Centrically braced frames…………	5	6
b- Eccentrically braced frames…………	-	8
c- Reinforced concrete structural walls…………	4	7

3.2.3 Equivalent Seismic Load Method

Equation 2.13 using to determine the total equivalent seismic load (base shear), Vt, acting on the whole building in the direction of earthquake TEC2007 [11].

$$V_t = \frac{\mathrm{WA(T1)}}{\mathrm{Ra(T1)}} \geq 0.10\ A_0 IW \tag{3.8}$$

Where:

Vt: Total equivalent seismic load acting on the building,

$T1$: First natural vibration period of the building,

W: Total building weight,

A: Spectral Acceleration Coefficient,

Ra: Seismic Load Reduction Factor,

Ao: Effective Ground Acceleration Coefficient,

I: Building Importance Factor,

Total building weight (W) is used in Equation 3.8.

Total equivalent seismic load determined by Equation 3.8 is expressed by Equation 3.9:

$$V_t = \Delta FN + \sum_{i=1}^{N} Fi \tag{3.9}$$

Additional equivalent seismic load, ΔFN, acting at the N'th storey (top) must be calculated by using Equation 3.10 [11].

$$\Delta FN = 0.0075\ NVt \tag{3.10}$$

Excluding ΔFN, remaining part of the total equivalent seismic load must be distributed to stories by Equation 3.11 [11].

$$Fi = (Vt - \Delta FN)\frac{wiHi}{\sum_{j=1}^{N} wjHj} \tag{3.11}$$

Where:

Fi: Design seismic load acting at i'th storey,

W: Weight of i'th storey,

Hi: Height of i'th storey,

3.2.4 Selection of Ground Motions

The most common local soil conditions Table 3.5. details the soil types in TEC-2007 that represent. Table 3.6. details the local site classes that shall be considered as the bases of determination of local soil conditions.

Table 3.5: Local Site Classes [11]

Local Site Class	*Soil Group according to Table 6.1 and Topmost Soil Layer Thickness* (h_1)
Z1	Group (**A**) soils Group (**B**) soils with $h_1 \leq 15$ m
Z2	Group (**B**) soils with $h_1 > 15$ m Group (**C**) soils with $h_1 \leq 15$ m
Z3	Group (**C**) soils with 15 m $< h_1 \leq 50$ m Group (**D**) soils with $h_1 \leq 10$ m
Z4	Group (**C**) soils with $h_1 > 50$ m Group (**D**) soils with $h_1 > 10$ m

Table 3.6: Soil Groups [11]

Soil Group	*Description of Soil Group*	*Standard Penetration* (N/30)	*Relative Density* (%)	*Unconfined. Compressive Strength* (kPa)	*Drift Wave Velocity* (m / s)
(A)	**1.** Massive volcanic rocks, unweathered sound metamorphic rocks, stiff cemented sedimentary rocks **2.** Very dense sand, gravel... **3.** Hard clay and silty clay…	— > 50 > 32	— 85 – 100 —	> 1000 — > 400	> 1000 > 700 > 700
(B)	**1.** Soft volcanic rocks such as tuff and agglomerate, weathered cemented sedimentary rocks with planes of discontinuity…… **2.** Dense sand, gravel......... **3.** Very stiff clay, silty clay…	— 30 – 50 16 – 32	— 65 – 85 —	500 – 1000 — 200 – 400	700 – 1000 400 – 700 300 – 700
(C)	**1.** Highly weathered soft metamorphic rocks and cemented sedimentary rocks with planes of discontinuity **2.** Medium dense sand and gravel.......................... **3.** Stiff clay and silty clay.....	— 10 – 30 8 – 16	— 35 – 65 —	< 500 — 100 – 200	400 – 700 200 – 400 200 – 300
(D)	**1.** Soft, deep alluvial layers with high ground water level **2.** Loose sand...................... **3.** Soft clay and silty clay…..	— < 10 < 8	— < 35 —	— — < 100	< 200 < 200 < 200

3.3 Irregular Bearing of Structures

The general vertical and horizontal shape of structure is important factor in seismic performance and damage of a building. Buildings with simple, regular, and symmetric configurations, exhibit the best performance to seismic action. Irregular buildings design and construction should be avoided because of their unfavorable seismic behavior, types of irregularities in plan and in elevation according to TEC2007 are shown in the below tables;

Table 3.7: Irregularities in Plan [11]

A – *IRREGULARITIES IN PLAN*	*Related Items*
A1 – Torsional Irregularity : The case where *Torsional Irregularity Factor* η_{bi}, which is defined for any of the two orthogonal earthquake directions as the ratio of the maximum storey drift at any storey to the average storey drift at the same storey in the same direction, is greater than 1.2 (**Fig. 2.1**). $[\eta_{bi} = (\Delta_i)_{max} / (\Delta_i)_{ort} > 1.2]$ *Storey drifts shall be calculated in accordance with* **2.7**, *by Considering the effects of* $\pm$ *%5 additional eccentricities.*	**2.3.2.1**
A2 – Floor Discontinuities : In any floor (**Fig. 2.2**): **I** - The case where the total area of the openings including those of stairs and elevator shafts exceeds 1/3 of the gross floor area, **II** – The cases where local floor openings make it difficult the safe transfer of seismic loads to vertical structural elements, **III** – The cases of abrupt reductions in the in-plane stiffness and strength of floors.	**2.3.2.2**
A3 – Projections in Plan : The cases where projections beyond the re-entrant corners in both of the two principal directions in plan exceed the total plan dimensions of the building in the respective directions by more than 20%. (**Fig. 2.3**).	**2.3.2.2**

Table 3.8: Irregularities in Elevation [11]

B – *IRREGULARITIES IN ELEVATION*	*Related Items*
B1 – Interstorey Strength Irregularity (*Weak Storey*) : In reinforced concrete buildings, the case where in each of the orthogonal earthquake directions, *Strength Irregularity Factor* η_{ci}, which is defined as the ratio of the *effective shear area* of any storey to the *effective shear area* of the storey immediately above, is less than 0.80. $[\eta_{ci} = (\sum A_e)_i / (\sum A_e)_{i+1} < 0.80]$ *Definition of effective shear area in any storey :* $\sum A_e = \sum A_w + \sum A_g + 0.15 \sum A_k$ (*See* **3.0** *for notations*)	**2.3.2.2**
B2 – Interstorey Stiffness Irregularity (*Soft Storey*) : The case where in each of the two orthogonal earthquake directions, *Stiffness Irregularity Factor* η_{ki}, which is defined as the ratio of the average storey drift at any storey to the average storey drift at the storey immediately above or below, is greater than 2.0. $[\eta_{ki} = (\Delta_i/h_i)_{ort} / (\Delta_{i+1}/h_{i+1})_{ort} > 2.0$ or $\eta_{ki} = (\Delta_i/h_i)_{ort} / (\Delta_{i-1}/h_{i-1})_{ort} > 2.0]$ *Storey drifts shall be calculated in accordance with* **2.7**, *by considering the effects of* $\pm$ *%5 additional eccentricities.*	**2.3.2.1**
B3 - Discontinuity of Vertical Structural Elements : The cases where vertical structural elements (columns or structural walls) are removed at some stories and supported by beams or gusseted columns underneath, or the structural walls of upper stories are supported by columns or beams underneath (**Fig. 2.4**).	**2.3.2.4**

3.4 Eurocode 8

The European Standard Eurocode8 has started in 1975 by the European Committee for Standardization or Committee European de Normalization (CEN). It is a non-profit association whose mission is to develop the European economy in global trading, the benefit of European people and the environment by provide an efficient infrastructure to interest parties for the development, repairs and division of logical sets of standards and specifications. European earthquake regulation is "Eurocode8" called "Design of Structures for Earthquake Resistance".

3.4.1 Definitions of Performance Level According to Eurocode8

Limitations on the maximum damage sustained during a ground motion are described as performance levels. Eurocode8 presents three main structural performance levels, Damage Limitation (DL), Significant Damage (SD) and Near Collapse (NP) [24].

1. **Damage Limitation (DL)**

- Very light damage,
- Structural elements retain their strength and stiffness,
- No permanent drifts,
- No significant cracking of infill walls,
- Damage could be economically repaired.

2. **Significant Damage (SD)**

- Significant damage to the structural system however retention of some lateral strength and stiffness,
- Vertical elements capable of sustaining vertical loads,
- Infill walls severally damaged,
- Moderate permanent drifts exist,
- The structure can sustain moderate aftershocks,
- The cost of repair may be high. The cost of reconstruction should be examined as an alternative solution.

3. **Near Collapse (NP)**

- Structure heavily damaged with low lateral strength and stiffness,

- Vertical elements capable of sustaining vertical loads,
- Most non-structural components have collapsed,
- Large permanent drifts,
- Structure is near collapse and possibly cannot survive a moderate aftershock,
- Uneconomical to repair. Reconstruction the most probable solution.

Hazard (return period of the design spectrum)	Required performance
T_R=2475 years (2% in 50 years)	Near Collapse (NC) (heavily damaged, very low residual strength & stiffness, large permanent drift but still standing)
T_R=475 years (10% in 50 years)	Significant damage (SD) (significantly damaged, some residual strength & stiffness, non-strutural comp. damaged, uneconomic to repair)
T_R=225 years (20% in 50 years)	Limited damage (LD) (only lightly damaged, damage to non-structural components economically repairable)
T_R values above same as for new buildings. National authorities may select lower values, and require compliance with only two limit-states	

Figure 3.2: The discrete Structural Performance Levels according to Eurocode8 [24]

3.5 Seismic Design Philosophies

The seismic evaluations of structures are mainly based on Force-Based design methodology, where the structural elements are evaluated in terms of stresses caused by the earthquake-related forces. The general aim of this design concept is to give strength to the structure rather than capacity of displacement. Some procedures have been proposed in the past, such as Response Spectrum Analysis (RSA) [25] procedure was mainly used. This procedure was implemented in earthquake codes all over the world and is still commonly used by majority of structural engineers. In RSA, the structures were considered to have elastic behavior and the periods, modes of vibration and the response of structure is calculated through a response spectrum application. The forces in the elements are divided by a comportment factor in order to take into account the non-linearities of the materials [26].

More recently on this method, Priestley [27] published a critical review on the drawbacks of this method. Other authors have additionally been scrutinizing more drawbacks in the RSA procedure. Gutierrez and Alpizar [28] added in there publication that, this procedure does not give any idea about global ductility, failure mode and corresponding inelastic deformation of structural elements.

The structural engineering society has been engendering an incipient generation of design and analysis procedures predicated on an incipient philosophy of performance-predicated engineering concepts. It has been accepted widely to consider damage circumscription as an explicit design consideration. In fact, the damage and behavior of the structures during an earthquake is mainly by the inelastic deformation capacity of ductile members. Therefore, seismic evaluations of structure should be predicated on the deformations caused by the earthquake, in lieu of the element stresses induced by the computed equipollent seismic forces, as transpires in the Force-base philosophy [29].

In recent years, several endeavors have been made to introduce Displacement-base methodologies in seismic engineering. These methodologies are divided into two main groups: Displacement-Base design methods for the design of incipient structures, and Displacement-Base evaluation method for seismic performance assessment of subsisting structures or pre-designed [30].

A performance-based procedure is based on two key elements which are capacity and demand. The demand represents the effect of the earthquake ground motion which is defined by means of response spectrum. The capacity of a structure represents its resistance towards the seismic demand. The performance depends, how its ability of handling the demand. The structure should be able to resist earthquake demands such that its performance is compatible with the design goals.

Within this context, earthquake-related analyses of structures are prodigiously paramount in order to correctly assess their earthquake-related performance, as given in Figure 3.1 [31].

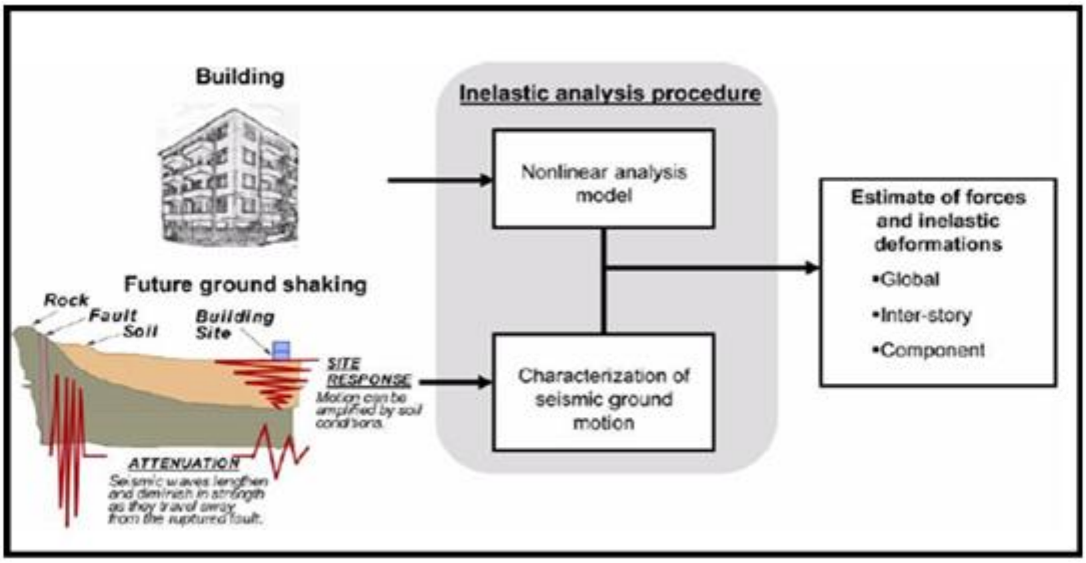

Figure 3.3: Inelastic analysis procedures [31]

3.6 Nonlinearity Concept

Non-linear structural behavior can be study under the material or geometric nonlinearities. Geometric nonlinearities are directly depending on structural deformation. It plays a fundamental role in the global response of the structure when the occurrences of large deformation in the structural elements induce displacements not more proportional to the loads effectively applied. Involving both local and global aspects, three are the most important sources of geometric nonlinearities: the beam column effects, the large displacement/rotation effects and the P-Δ effects. These geometric nonlinear effects are typically distinguished between P-δ effects, associated with deformations along the members, measured relative to the member chord, and P-Δeffects, measured between member ends and commonly associated with story drifts in buildings. In buildings subjected to earthquakes, P-Δeffects are much more of a concern than P-δ effects, and provided that members conform to the slenderness limits for special systems in high seismic regions. The P-δ effects do not generally need to be modeled in nonlinear seismic analysis. On the other hand, *P*-Δ effects must be modeled as they can ultimately lead to loss of lateral resistance, ratcheting (a gradual build up of residual deformations under cyclic loading), and dynamic instability. Nonlinearities in geometry suggested by Li (1996) is shown the below Figure 3.6;

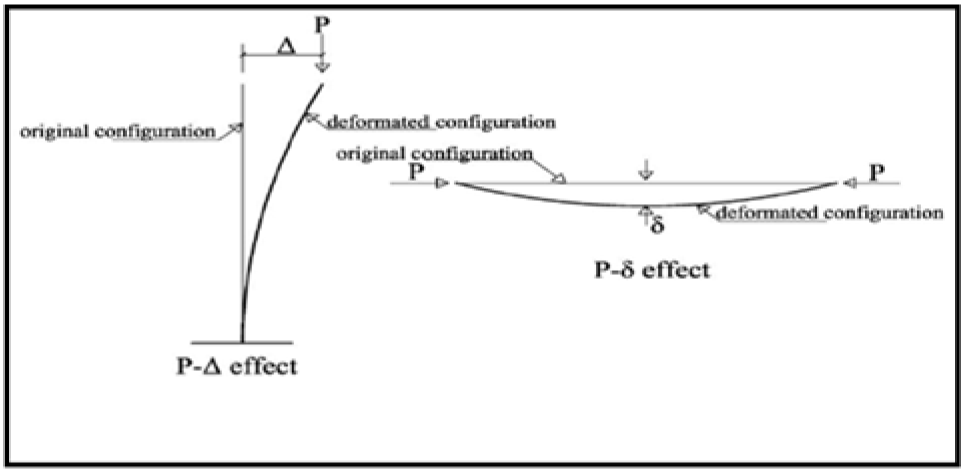

Figure 3.4: The effect of P-$\boldsymbol{\delta}$ and P-Δ [32]

The P-Δ directly effect on lateral or flexural stiffness of the structure. These effects are caused by side swaying of the system. P-Δ effect creates the additional overturning moments to the structure and this effect reduces the flexural stiffness of elements and system. The P-Δ effect should be considered in the analysis as it is mostly related to compression member and play an essential role in overall firmness of structures.

It was well known that, the relationship of stress-strain of a material is normally having non-linear behavior. According to material's stress-strain relation, its nonlinearity is subjected to nonlinear behavior of members which is given in the Figure 3.7. Inelastic behavior of member is considered under loading and unloading path [33].

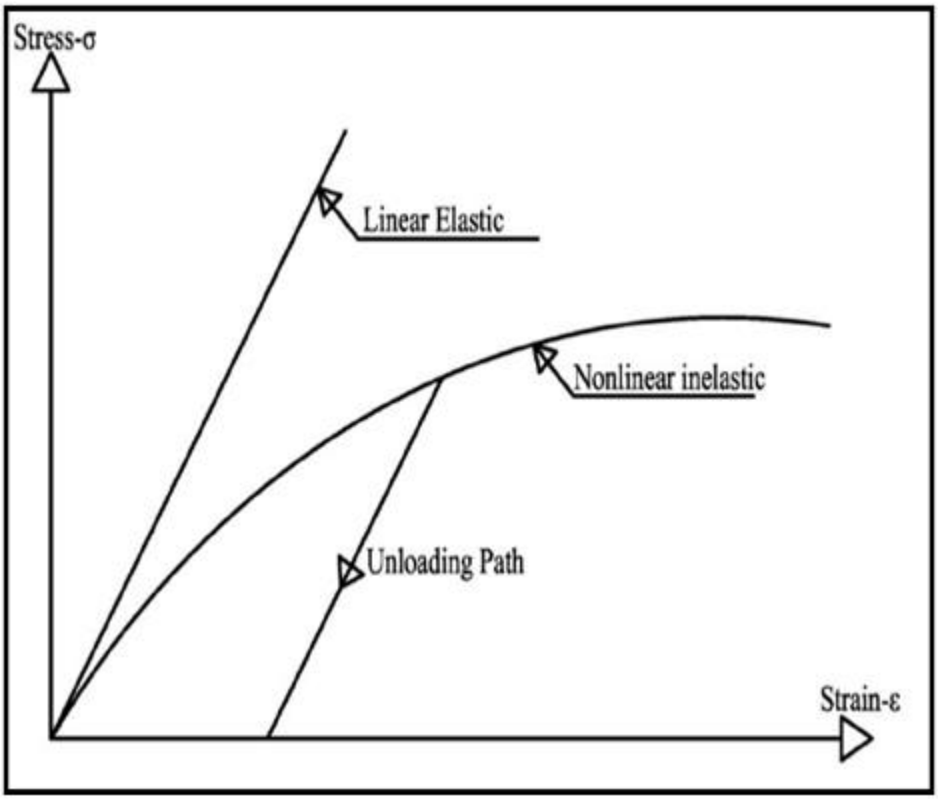

Figure 3.5: Elastic and inelastic behavior of material [33]

Before starting non-linear analyses, non-linear behavior of structural element is briefly study and describe with loading and unloading path.

3.7 Pushover Analysis

Pushover analysis is a performance based method requires a reasonable estimate of inelastic deformation or damage in structures [33]. Pushover analysis is widely used process to get an earthquake performance of structure.

Pushover analysis consists in a static non-linear analysis of the structure under monotonically increased horizontal loads, representing the effect of a horizontal seismic component. The main objectives of the analysis are the estimation of the sequence and the final pattern of plastic hinge formation, the estimation of the redistribution of internal forces following the formation of plastic hinges, and the assessment of the force-displacement curve of the structure ("capacity curve") and of the deformation demands of the plastic hinges up to the ultimate constitutive materials strain limits. In the basic approach described in EC8-2 informative annex H, horizontal forces are distributes according to the initial elastic fundamental mode shape, and the displacement demand evaluation of the reference point (chosen at the centre of mass of the deck) is based on the code elastic response spectrum for five percent damping.

Main criticisms that can be addressed on this basic pushover analysis approach consist in the facts that it does not take into account some dynamic or non-linear behavior aspects of prime importance such as higher modes effects, structural softening, modification of the vibration modes and damping increase with post-yield plastic deformations and damage.

Non-linear static pushover analysis may be reasonable in providing location estimates of inelastic behavior, but alone it's not capable of providing maximum deformation estimate. The basic issue in this analysis is how far to push? Like such as Capacity Spectrum Approach is used in concert with Non-linear response history analysis to determine how far to push. The minimum needed thing about methods of analyses, including pushover, is that it should be good enough to design [34].

It consists of two components. Firstly, the pushover is induced through incremental static load application to inelastic model of a building. Secondly, this curve is used with other "Demand" tool to find target displacement.

3.7.1 Development of Capacity Curve

The main features of this method of describe below as;

- It helps in developing analytical models of structures which includes gravity loads, P-Δ effects and sources of inelastic behavior,
- It also calculates model different properties such as period and mode shape, model participation factor,
- In this method it considers lateral inertial force distribution,
- It gives pushover curve as shown in the Figure 3.5 [36].

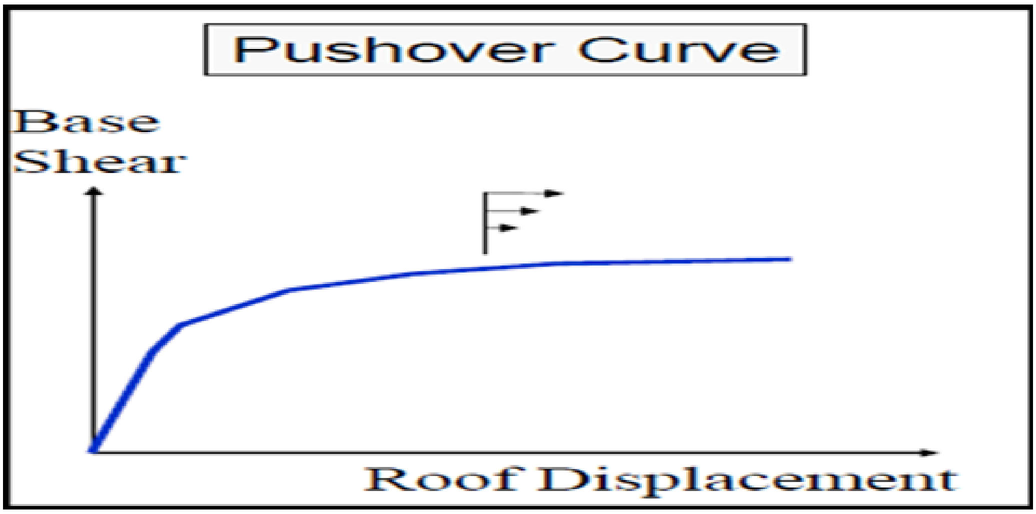

Figure 3.6: Pushover curve [36]

In pushover curve, the above symbol on the curve shows that lateral load pattern for this curve is in upper triangular. Further load patterns, like proportional or uniform to first mode shape will construct different curves.

3.7.2 Event-to-Event steps in Pushover Analysis

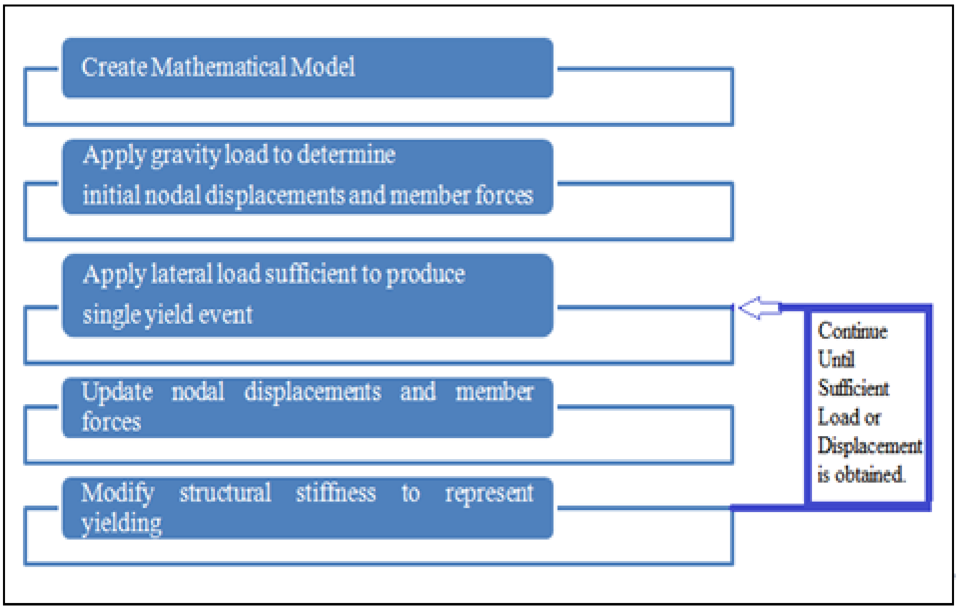

Figure 3.7: Event-to-Event steps in Pushover Analysis [36]

This is the general flowchart for event-to-event steps in pushover analysis. Each step is explained in detail in later topics. The analysis is performed under displacement or control force. And also it should be noted that no yielding occurs under gravity load in this sequence as assumed otherwise if it does, the structure should be redesigned [36].

3.8 Target Displacement

Target displacement is calculated to represent the ultimate displacement resulting during the design earthquake. For all capacity curves an increase in base-shear occur which causes a displacement up to a certain point where slope of the line is changed.

At this point decrease starts in the strength and stiffness of the buildings and at this point structure is yield. It is very important to compute target displacement point in the seismic performance assessment of structures. SeismoStruct 2016 [37] gives a very accurate result for target displacement point of a structure. The allowable drift in most codes is about (H/400 to H/600) and reduction factor is ranged between 3-8. Hence the max inelastic drift is about (H/200 to H/50). Consider material factor of 1.5 suggest target displacement of (H/20 to H/30), where H is the height of structure.

Chapter 4

MODELING IN SEISMOSTRUCT

4.1 Introduction

In this chapter different parameters involved in designing and analysis of the buildings using SeismoStruct are discuss in detail.

4.2 SeismoStruct

It is based on finite element package which has an ability to predict a great displacement behavior of a frame under dynamic or static loads, in taking account of material's inelasticity and geometric nonlinearities. SeismoStruct have the capability to perform under Eigenvalue, non-linear static time-history analysis, non-linear static pushover both adaptive and conventional, non-linear dynamic and incremental dynamic analysis [37].

Geometric non-linearities have a central role in structure global response in the occurrence of big deformation in elements leads to displacement not further corresponding to applied effective load. By involving together global and local aspect, there are three most fundamental sources of geometric non-linearity: the large displacement/rotation effect, the beam-column effect and the P-Δ effect.

To model geometric non-linearities, both local and global geometric non-linearities are taken into account in SeismoStruct 2016. Axial strain shape function is use to adopt cubic formulation, it results in non-linear response of mainly small member.

Due to large displacement and rotation, a local system is introduced to every finite element known as chord system, followed by element movements both rotation and translation. Stiffness matrix and internal forces are both obtain in local chord system as shown in Figure 4.1;

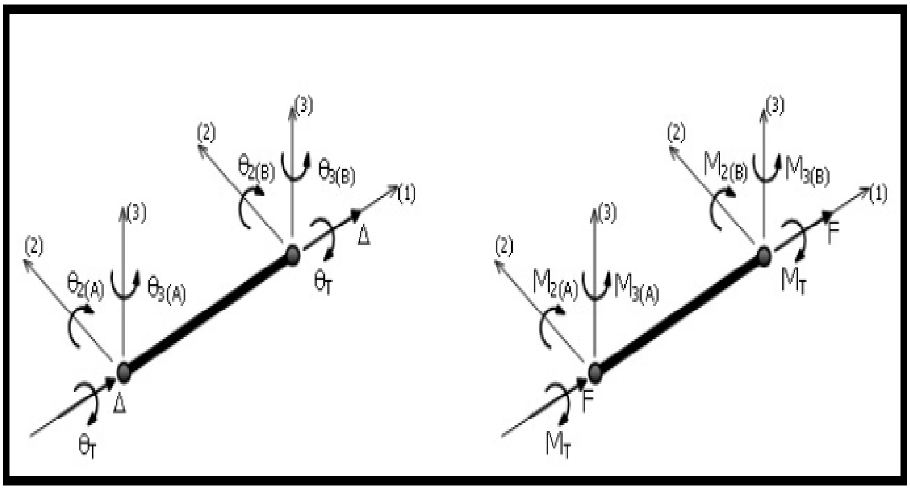

Figure 4.1: Local Cord System [37]

In SeismoStruct software, material inelasticity of the elements is made of so called fiber modeling approach in which the element has been subdivided into many segments. The section is discretized in sufficient quantity of fibres and the response of sections are obtained through the integration single fiber's response of individual fibres (typically 100-150) [37].

To find an accurate estimate of structure damage distribution, the increase inelasticity of the material along with length and cross section area of the member is clearly represented by an employment of fiber modeling method as shown in Figure 5.2. If an appropriate number of elements are used for example 4-5 per structural member, then spread of inelasticity alongside length of the member can be computed precisely [37].

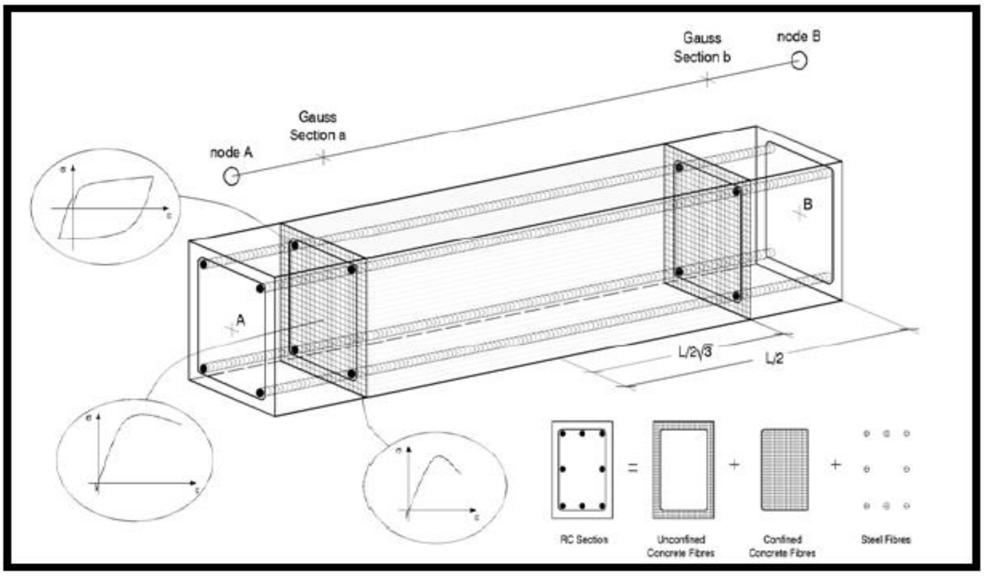

Figure 4.2: Fibre element model [37]

4.3 SeismoStruct Modeling

By exploiting through the tools available in SeismoStruct version 7.0.4 2016 [37], several models are developed so as to simulate different in the existence of the pinned connections. In SeismoStruct all wall are connected to the beams and column by pin-end systems. The beams are subjected to the gravity load, which are laded in transverse direction are used as simple supports at each end, these beams induce no seismic action both in the RC columns and infill walls. On the other side, in the longitudinal direction, if even no moment is transmitted into the flange wall, its seismic shear increases the moments at the centre line of walls and decreases in base moments at weak axis direction. Hence it has deep influence on the seismic interaction between wall and frame system, the true modeling of pinned connections assumes a rule of principal value. For this purpose, two different modeling tools (*nodal constraint* tools and *link element* tools) are studied and compared.

4.3.1 Consideration for Modeling

Starting according to the common most facts describing the 3D structural modeling is proposed next. The main features of 3D structural modeling are described in detail, which includes material features, global mass direction, 3D layout scheme, correct mass distribution, floor modeling, and simulation of pinned connection.

4.3.2 Material

All elements are define as three dimensional inelastic column beam elements, having an ability of capture the material and geometric non-linearities considering 150 and 200 section fibres of each element [37].

The materials used in modeling such as concrete, steel and infill are selected accordingly to fulfill the requirement. The following are the properties of different material in used in these models.

4.3.2.1 Concrete Model

The following physical properties are defined for concrete in SeismoStruct for all the models;

a) Compressive strength = 28000 *kPa*

b) Mean tensile strength = 2200 *kPa*

c) Modulus of elasticity = 2.4870E+007 *kPa*

d) Strain at peak sress = 0.005 *m/m*

e) Specific weight = 24 kN/m^3

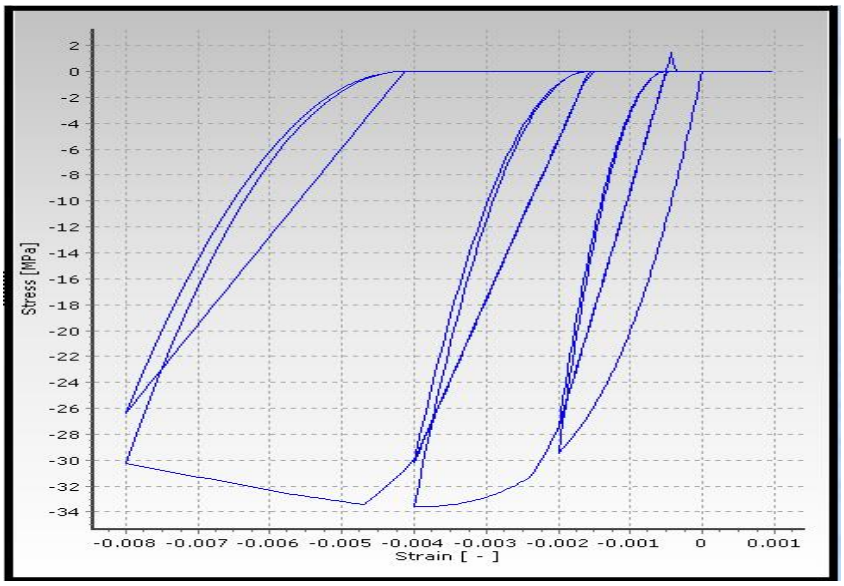

Figure 4.3: Non-linear constant concrete model [37]

4.3.2.2 Steel Model (Menegotto-pinto stl-mp)

It depends on stress-strain relationship suggested by Menegotto and Pinto [37]. This model is selected to model both the reinforcing and structural steel as shown in Figure 5.4. The following properties are defined for reinforcement in SeismoStruct for all the models are;

a) Yield strength = 450000 kPa

b) Modulus of elasticity = 2.0000E+008 kPa

c) Strain hardening parameter (u) = 0.005

d) Specific weight = 78 kN/m^3

e) Fracture/buckling strain (-) = 0.1

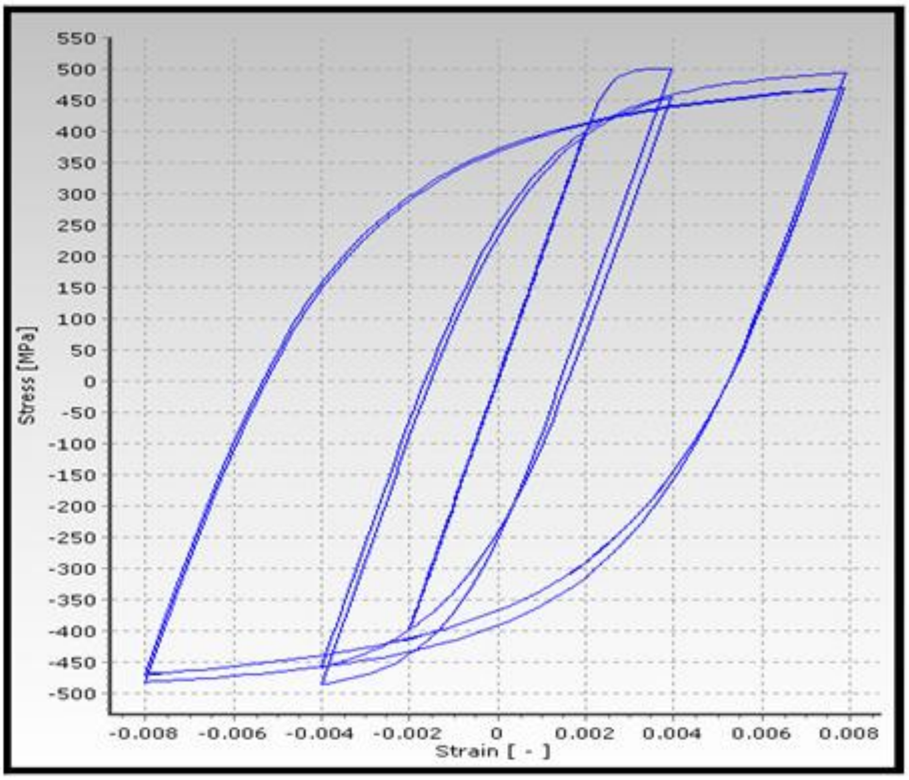

Figure 4.4: Menegotto-pinto steel model [37]

1. Infill wall

The following physical properties are defined for infill wall in SeismoStruct for all the models are;

a) Young modulus-Em = 1600000 *kPa*

b) Compressive strength-fm = 1000 *kPa*

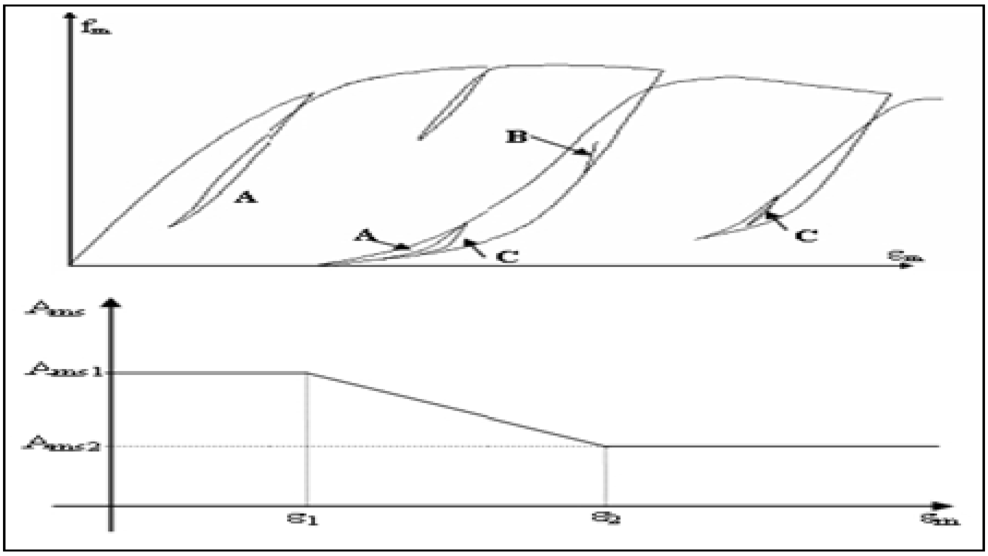

Figure 4.5: Infill brick wall compressive strength curve [37]

a) Shear bond strength = 300 kPa

b) Friction coefficient = 0.7

c) Maximum resistance = 600 kPa

d) Reduction shear factor 1.5

e) Specific weight = 5 kN/m^3

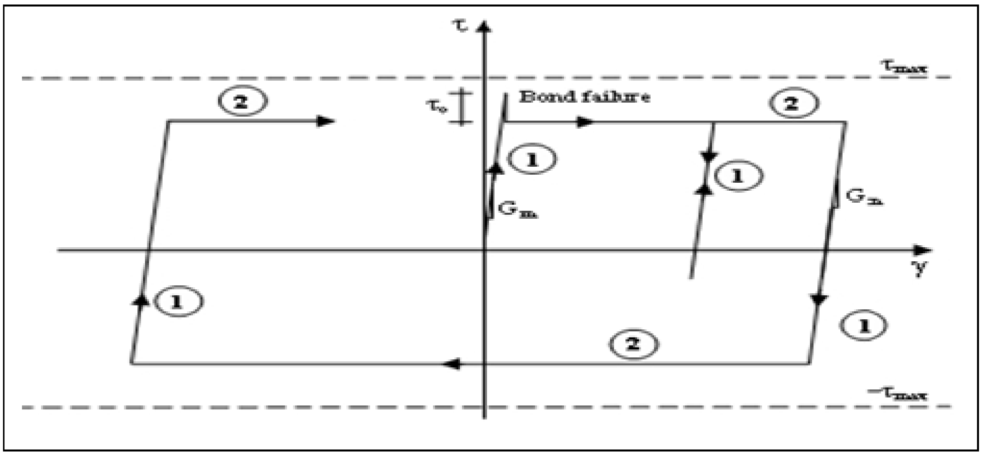

Figure 4.6: Bond Failure in infill brick wall [37]

4.3.3 Formulation of an Element

The elements are base on stiffness- or displacement based, or flexibility or force-based interpolation function. For controlling distribution of the inelastic strains consideration of the element type is important.

In SeismoStruct 2016 the assigned inelastic frame elements are carried out with the formulation of displacement based finite elements. For this purpose, cubic Hermitical polynomial is use as displacement shape function along the entire length of the element's linear variation of curvature. As the curvature field could be extremely non-linear at the time of inelastic analysis such as pushover or inelastic dynamic time-history, an advanced meshing of the structural element is required with displacement based formulation where typically 4- 5 elements per structural member.

4.3.4 Scheme of 3D Layout

It defines structural skeleton of a building completely as modeling of the walls, beams and columns essentially present in sample structure by mean of displacement based finite element. Columns and beams are modeled as reinforced concrete element having I-shape profile. All frame elements are modeled by centre-line node to centre-line node, without any other specific elements used to represent between column and beam joint. This modeling method also used in advantage structural analysis. By taking into account the presence of stiff wall elements acting parallel to the frames, this failure appears also minor in the prototype structures numerical models as shown in Figure 5.7 [38];

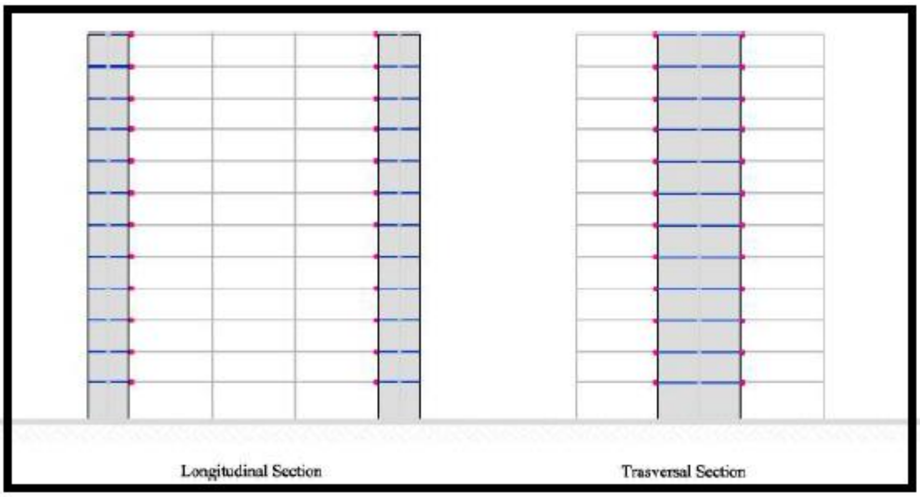

Figure 4.7: Longitudinal and Transverse Section of Structure's Numerical models [38]

Already in the past, non-planar walls have been common structural elements providing lateral stiffness and strength to RC buildings. Since even within elastic systems the force distribution between the different components (webs and flanges) of non-planar walls can be quite complex, the development of simple, computational inexpensive analysis models for such structures was a research objective from the early beginnings of computational structural analysis. One of the modeling approaches that found broad application was the "wide-column analogy" (known also as the "equivalent frame method"). It was originally developed for planar wall structures such as structural walls with openings and structural walls coupled by beams or slabs and were later extended to non-planar structures. In WCMs of non-planar walls the web and flange sections are represented by vertical column elements located at the centroid of the web and flange sections. These vertical elements are then connected by horizontal links running along the weak axis of the sections having common nodes at the corners [39].

In this method vertical element (representing web and flange) are connected to by horizontal weak axis of the sections with same nodes at the end as detailed in Figure

4.8. The wide-column analogies require the sub-division of U-shaped section into three rectangular sections of wall as web and flanges.

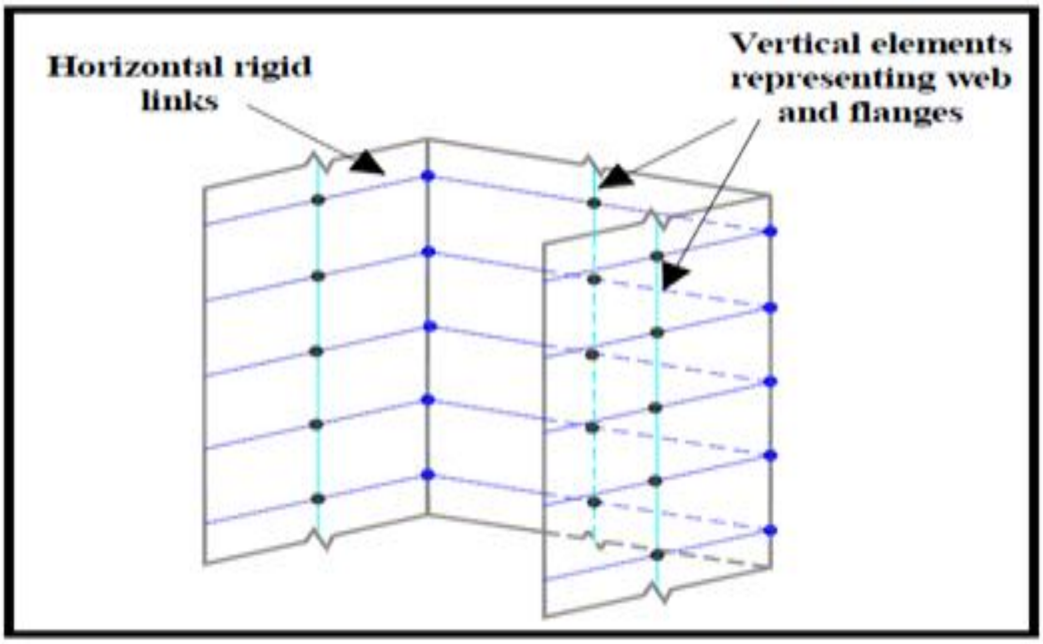

Figure 4.8: U-shape wall system [39]

To develop a simple computational analyses model is a main importance of this study where inelastic analysis is characterized by different acceleration or displacement field applied to the 3D model. For this propose the number of element between following node are reduced to one unit. In SeismoStruct 2016 the column, beam, and wall in the model is assembly as shown in Figure 4.9.

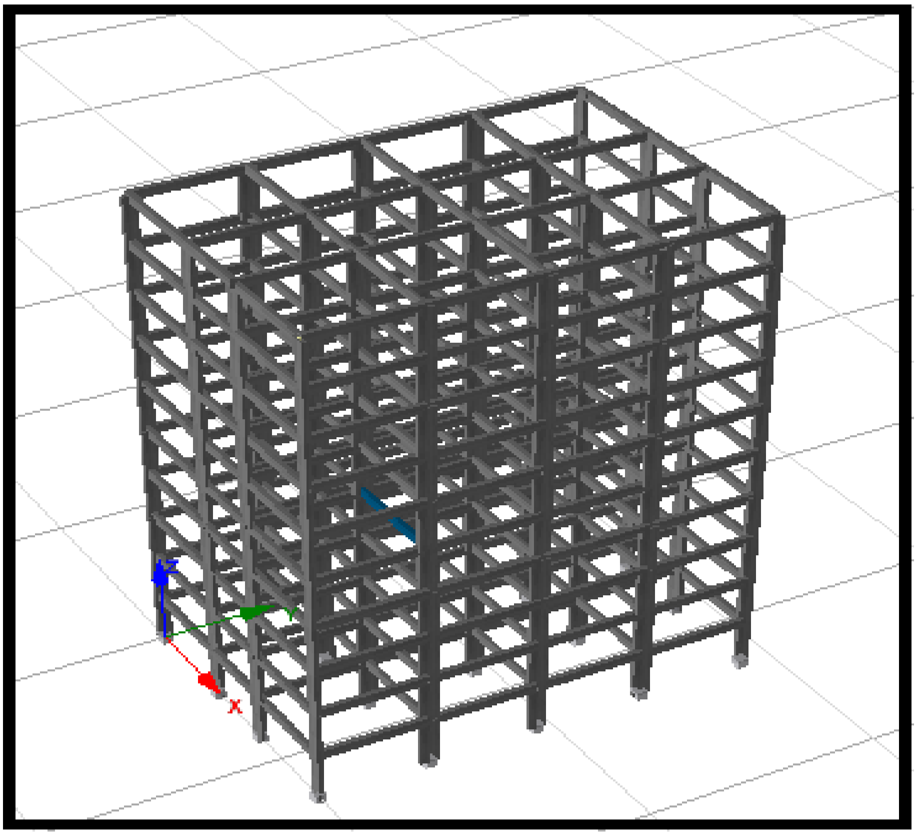

Figure 4.9: 3D view of SeismoStruct model [37]

4.3.5 Modeling of a Floor

In SeismoStruct the rigid floor state is realize by imposing rigid diaphragm constraint at every level of structure. All joints at the same story level are connected to each other through a special connection work as rigid link in the story plane and also allowing out-of-plane deformation (z-direction). Displacement in x-y parallel plane is not allowed, but remains completely endorsed in flexibility of the floor as apparently establish the rigid diaphragm behavior.

In SeismoStruct 2016 the rigid diaphragm tools a master node is selected which defines constraint's net in area of slab [37]. All joints are directly linked to it which becomes the storey orientation point for the software elaborations. (Figure 4.10)

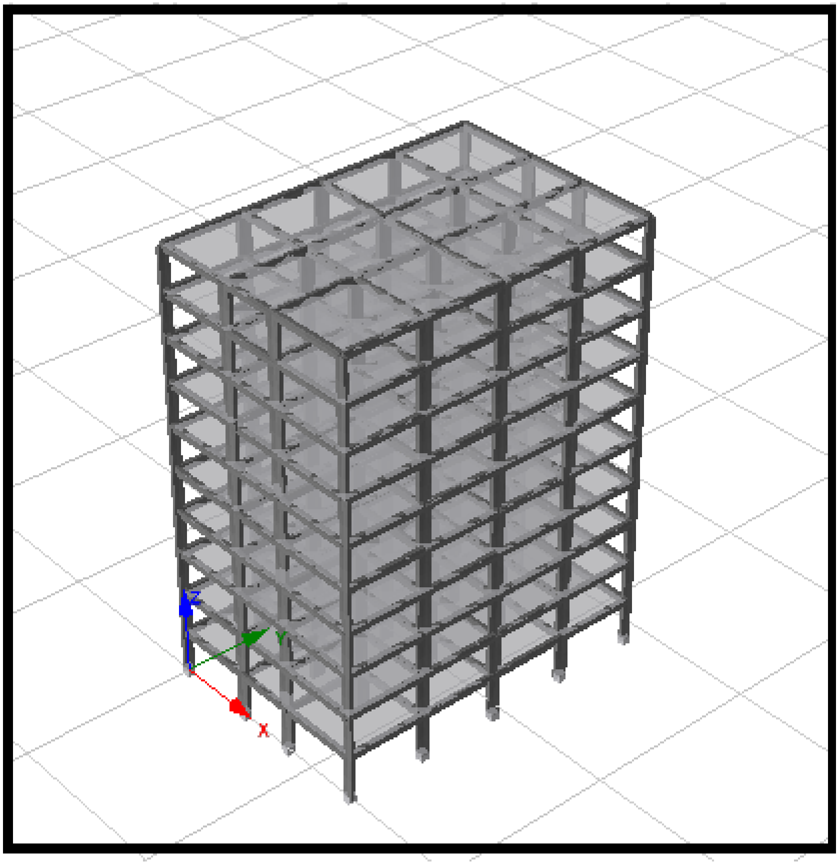

Figure 4.10: Modeling of floor rigid diaphragm constraints [37]

4.4 Description Summary of Proposed Double Strut Model

New model was proposed by Crisafulli which represent shear failure of masonry. The model accounts separately compressive and shear behavior of masonry by using a double truss mechanism and shear spring in each direction. In this model struts is parallel and alienated through vertical space same as h_z [10].

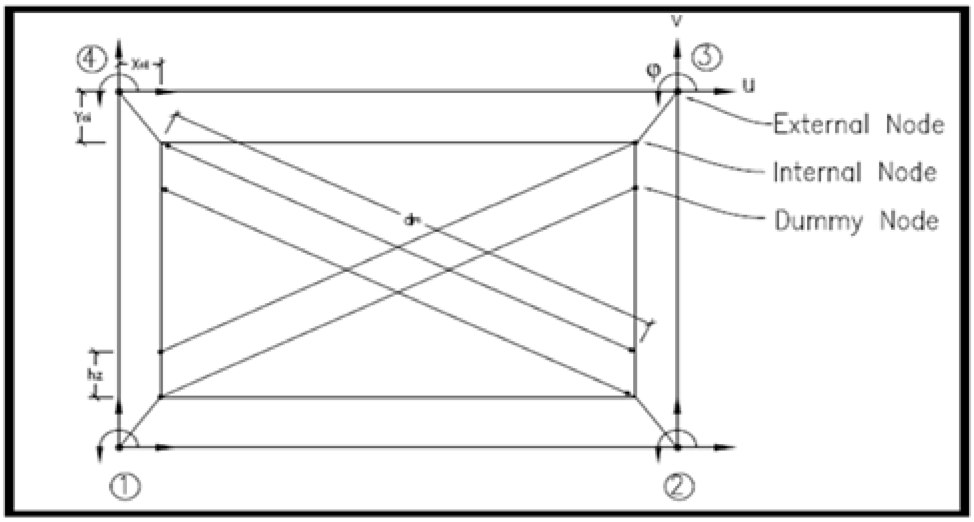

Figure 4.11: Infilled panel element configurations [10]

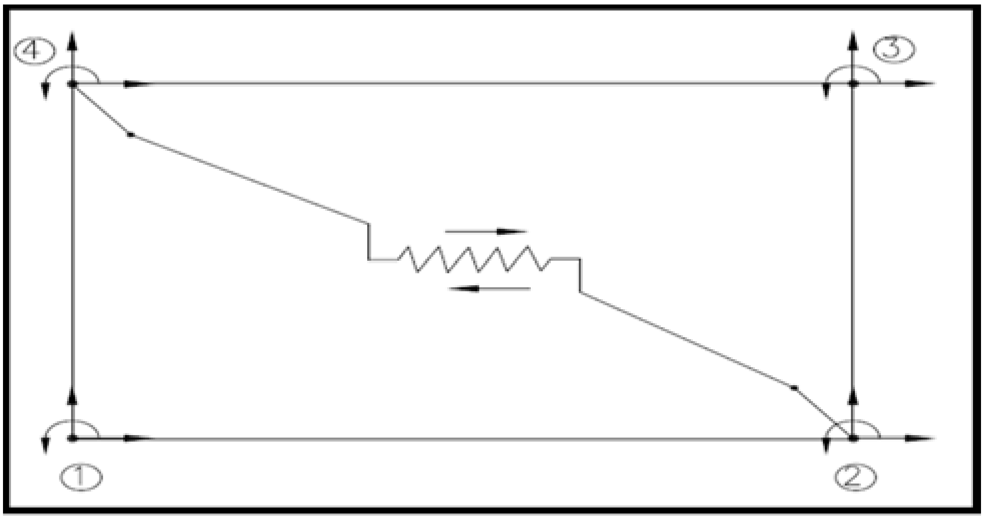
Figure 4.12: Shear Configurations [10]

In this proposed model, three different set of nodes namely External, Internal and Dummy nodes were used for development of the infill panel. External node is directly linked to frame panel and internal nodes are define by vertical and horizontal offset x_{oi} and y_{oi}. Four dummy nodes define one end of strut members, and they are not connected to corner of the panel shown in Figures 4.12.

4.5 Input Parameters for Infill Walls

Crisafulli propose a width range of input parameters for some several mechanical and geometrical parameters values from his experiments. These parameters need to be defined in order to fully characterize the response curve [10]. Lists of those parameters are described as following.

4.5.1Compressive Strength (f_n)

Decanini and Fantin (1987) proposed an expression for the compressive strength of diagonal strut which can be estimated by the following expression [40]:

$$R_c = f'_{m\theta} A_{ms} \tag{4.1}$$

Where, $f'_{m\theta}$: strength of masonry after transversely load is applied at θ , and A_{ms} : area of the equivalent strut i.e. $A_{ms} = b_w \times$ t,

Crisafulli adapted the hypothesis from Mann and Muller development theory of a failure of unreinforced masonry subjected to compressive stress as well as shear stress base on equilibrium considerations by the following expression [10]:

$$f_n = f_1 \sin^2\theta \tag{4.2}$$

$$\tau = f_1 \sin\theta \cos\theta \tag{4.3}$$

Where, f_1 : positive. These equations are derived from principal stresses f_2 occur at masonry wall which are not considerable.

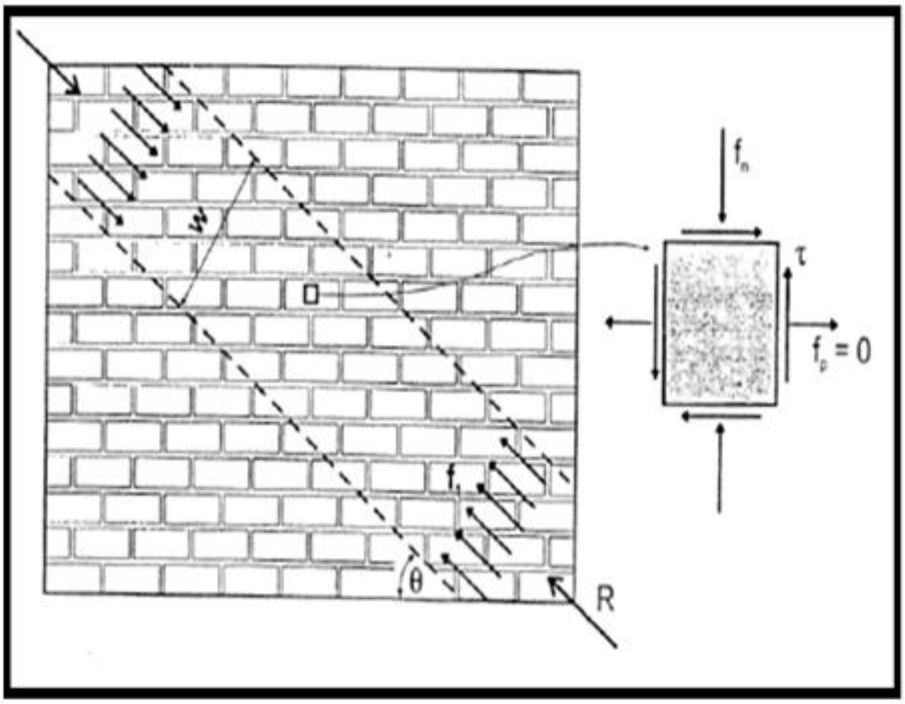

Figure 4.13: Masonry Stresses state [10]

5.3.2 Stiffness of Element

Element's stiffness is dispersed in a fraction of strut and shear spring. Stiffness of the shear spring K_s is calculated by fraction γ_s of the total stiffness of masonry strut. Each struts area is assumed to be same, so the combination of two masonry struts and shear spring resulting in total stiffness. Strut, and Shear stiffness's are solved by;

$$K_S = \gamma_s \frac{A_{ms}E_m}{d_m} \cos^2 \theta \qquad (4.4)$$

$$K_A = (1\text{-}\gamma_S) \frac{A_{ms}E_t}{2d_m} \qquad (4.5)$$

5.3.3 Tensile Strength (f'_t)

Usually it is much lower than compressive strength from experimental and analytical result point of view; therefore tensile strength can be taken as zero during analysis. Somehow, it can be been introduce 10% of compressive strength to gain generality.

5.3.4 Strain at Maximum Stress (ε'_m)

It influence by means of alteration of the secant stiffness of increasing in stress strain curve. Its value ranges from 0.002 to 0.005.

5.3.5 Closing Strain (ε_{cl})

It represents the limiting strains where cracks are closed partially and compressive stresses are resisted. Its value range varies between 0 and 0.003, in analysis very large value is not considered such as $\varepsilon_{cl} = \varepsilon_u$.

5.3.6 Ultimate Strain (ε_u)

Decreasing branch of the stress-strain curve is control by this parameter. For greater value such ε_u= $20\varepsilon'_m$, where the reduction in compressive stress is obtained.

5.3.7 Elastic Modulus (E_{mo})

It is the initial slope of the stress-strain curve. Masonry being made up composite material results in a large variation in its value having different property. According to Crisafulli (1997), initial stiffness of the infill frames undervalue these values and assume the following expression [10]:

$$E_{mo} \geq \frac{2f'_{m\theta}}{\varepsilon'_m}$$

5.3.8 Empirical Parameters

The masonry infill strut model requires nine empirical curve calibrating factors to be defined [10]:

- **Unloading Stiffness Factor (γ_{un})**

Slope of unloaded branch is control by it and ranged is 1.5 to 2.5, and also $\gamma_{un} \geq 0$.

- **Strain Inflection Factor (α_{ch})**

Strain inflection factor predicts inflection point at reloading curve of the strain, value vary from 0.1 to 0.7. It also control fatness of hysteresis loops.

- **Complete Unloading Strain Factors (β_a)**

It defines the plastic deformation point after compete unloading. Typically its value ranges between 1.5 and 2.0.

- **Reloading Strain Factor (α_{re})**

This parameter points out the strength envelope, in which the reloading curve reaches to the strength envelope. Generally it's varying between 0.2 and 0.4 somehow the value of 1.5 for non-linear infilled frames analysis was used by Crisafulli (1997).

- **Reloading Stiffness Factors (γ_{plr})**

In this parameter reloading stiffness modulus is defined, after taking place of complete loading. Its value ranges between 1.1 and 1.5, (γ_{plr}> 1).

- **Stress Infection Factor (β_{ch})**

In this parameter defines stress point. Value is between 0.5 and 0.9.

- **Zero Stress Stiffness Factor (γ_{plu})**

It defines zero stress at hysteric curve. Its value varies between 0 and 1.

- **Plastic Unloading Stiffness Factor (e_{x1})**

It controls degradation of stiffness and ranges from 1.5 and 2.0.

- **Repeat Cycle Strain Factor (e_{x2})**

It is between 1.0 and 1.5. It increase the strain at which the envelope curve is reached.

Chapter 5

METHODOLOGY AND CASE STUDIES

5.1 Introduction

In this chapter different buildings are modeled according to Turkish Earthquake Code TEC2007. The reliability of the data collected to used in this research and the definition of parameters affecting on earthquake analysis of reinforce concrete buildings which represent the input of the data collected have been explained. This chapter also contains information about the geometric property and dynamic properties of the case buildings which are described briefly.

5.2 Main Methodology of Structures

Different reinforced concrete structures having different elevation according to each case are considered as a low-rise, medium-rise and high-rise reinforced concrete frame structure with different cases like bare frame, soft-storey, fully brick wall and partially brick wall used in RC frame buildings. These buildings are designed according to Turkish Earthquake Code (2007) [11]. The following methodology will be adopted in these case studies,

- All floors are of different height depending on the case study.
- In order to design structures, Equivalent static analysis defined by TEC2007 [11] response spectrum method is used.
- According to TEC2007 Structural Behavior Factor level is 8 for Systems of high Ductility as detail in Table 5.2 (TEC2007) [11].

- Earthquake analysis parameters according to TEC2007 used in this study are detailed in table 5.1.

Table 5.1: Earthquake Analysis Parameter

Parameter	First Case	Second Case	Third Case
Earthquake Code	2007	2007	2007
Earthquake Zone	0.2	0.2	0.2
Soil Type (Z)	Z3	Z3	Z3
Importance factor	1	1	1
Dead Load Factor	1.4	1.4	1.4
Live Load Factor	1.6	1.6	1.6

In this thesis for designing purpose Idecad Structural version7 is used and for performance analysis SeismoStruct is used. Different case studies are discussed further in detail.

5.2 Case Studies

The geometric and dynamic properties of the case studies are described below.

5.2.1 First Case Study

In this case study, 3-storey reinforced concrete frame with 9.6 m in elevation and 6-storey reinforce concrete frame with 19.2 m in elevation will be discuss in detail. Each of these two cases is further divided into three other cases for comprising which are bare frame, soft-storey, fully infilled RC frame and also partially infilled RC frame building. According to TEC2007 3-storey and 6-storey buildings are design by software known as Idecad. All the floors are having the same elevation of 3.2 m. The total area of this building is 412.3 m^2. The lateral load of 10 kN, 7.5 kN and 5 kN is applied for pushover method. The general plan and plan with some irregularity are shown in Figure 5.1. The section characteristics of the beams and columns are detail in table 5.2 and 5.3. The 3D layouts are showing in Figure 5.2.

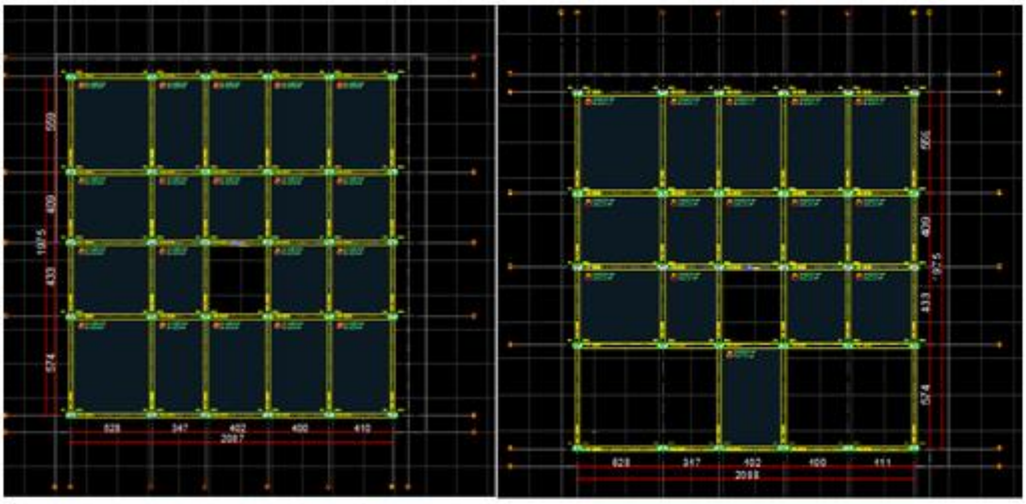

Figure 5.11: Plan details of first case study

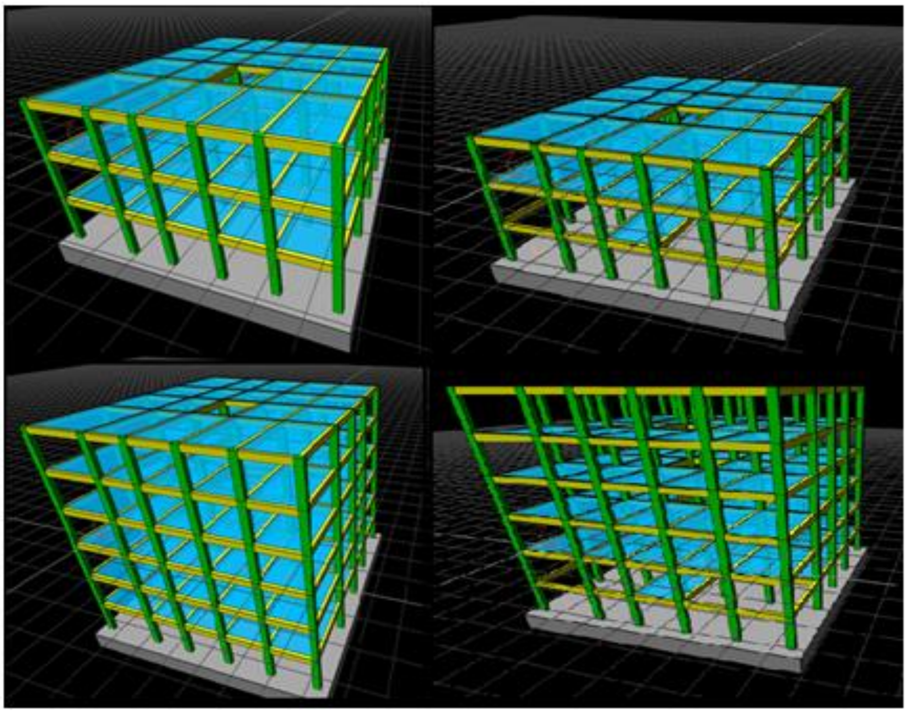

Figure 5.2: 3D layout of 3-storey and 6-storey building

Table 5.2: Reinforcement Detail of the beams First Case Study

Beam	Dimension (cm)	Top	Bottom	Stirrups
3-storey	50×25	3Ø14	3Ø14	Ø8/20/10
6-storey	50×25	3Ø14	3Ø14	Ø8/20/10

Table 5.3: Reinforcement detail of the columns for First Case Study

ID	B	H	Major	Minor	Lateral	Perc.	S.R.Percenta...
C01	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.35%
C02	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.34%
C03	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.29%
C04	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.29%
C05	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.3%
C06	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.48%
C07	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.18%
C08	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	-0.3%
C09	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.11%
C10	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.08%
C11	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	-0.26%
C12	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.21%
C13	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.18%
C14	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.11%
C15	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.18%
C16	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.11%
C17	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.12%
C18	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.28%
C19	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.19%
C20	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	-0.29%
C21	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.1%
C22	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	-0.28%
C23	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	-0.25%
C24	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.23%
C25	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.35%
C26	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.35%
C27	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.3%
C28	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.32%
C29	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.48%
C30	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.29%

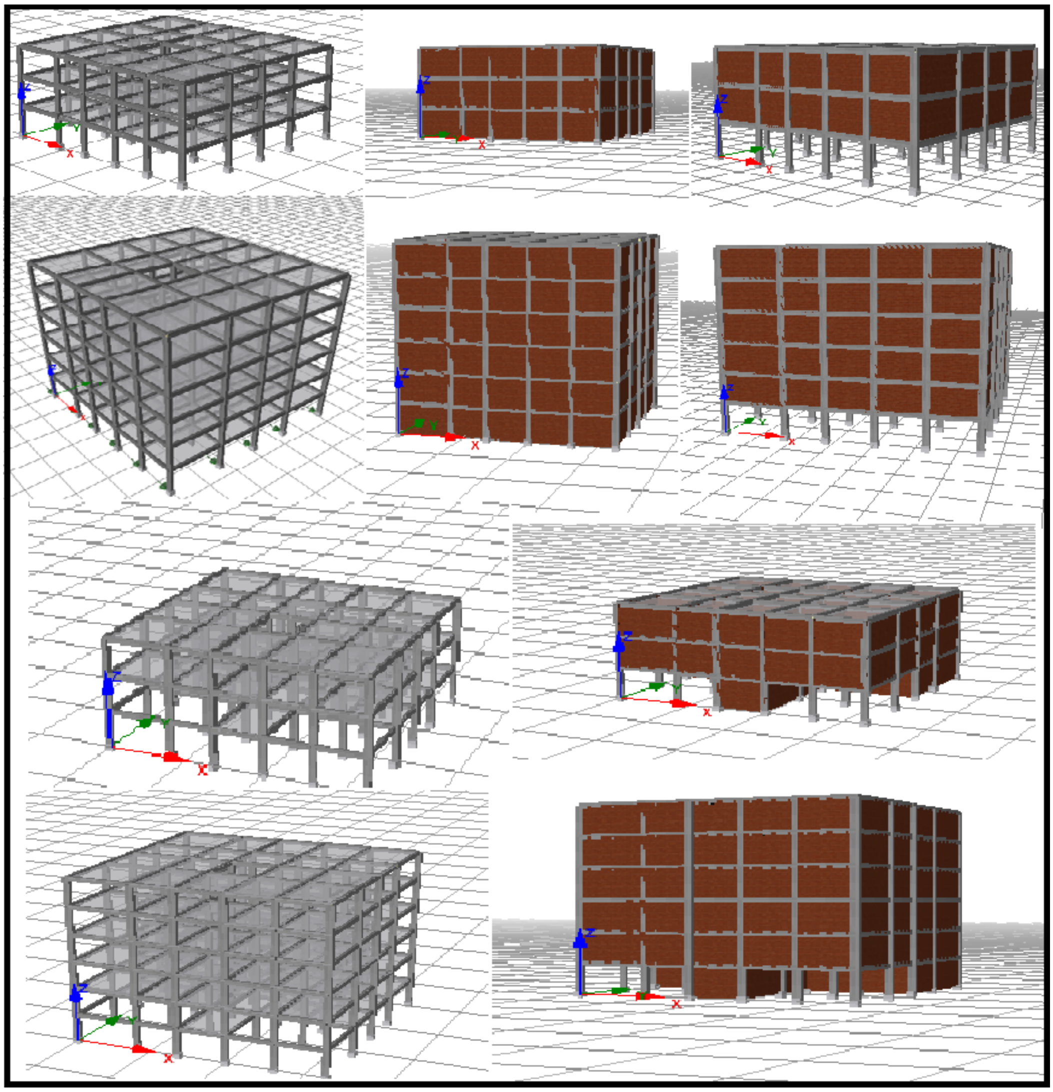

Figure 5.12: 3D Layout of First Study Case in SeismoStruct

5.2.2 Second Case Study

In this case study, 4-storey reinforce concrete frame with 12.8 m in elevation and 8-storey reinforce concrete frame with 25.6 m in elevation will be discuss in detail. Each of these two cases is further divided into three other cases for comprising which are bare frame shear, soft storey, and fully infilled RC frame with all cases having shear wall. First of all 4-storey and 8-storey building is designed with a software known as Idecad according to Tec2007. The lateral load of 10 kN, 7.5 kN and 5 kN is applied for pushover analysis. Some of the parameters the structure is given below. All the floors are having the same elevation of 3.2 m. The total area of this building is 298.480 m^2. The plan is shown in Figure 5.4.The 3D layouts are showing in Figure 5.5.The section characteristics of the beams and columns are detail in table 5.3 and 5.4.

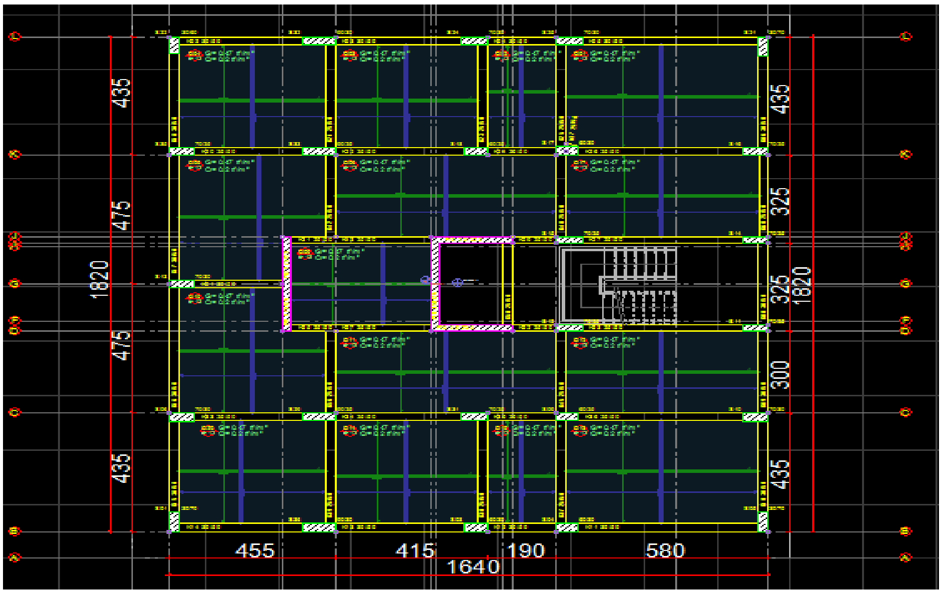

Figure 5.13: Plan details of Second Case Study

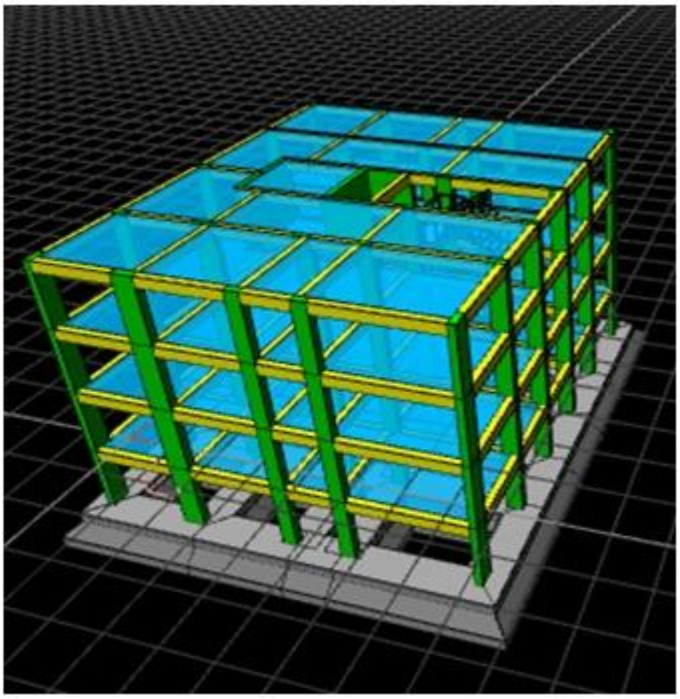
Figure 5.14: 3D layout of Second Case study in Idecad

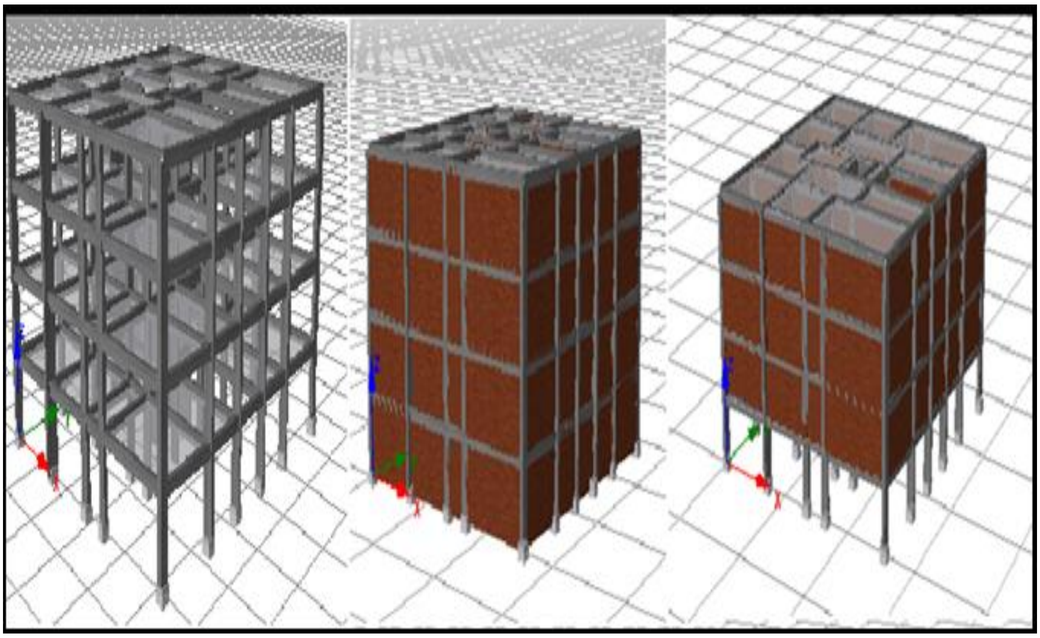

Figure 5.15: 3D Layout of Second Case study in SeismoStruct

Table 5.4: Reinforcement details of the beams for Second Case Study

Beam	Dimension (cm)	Top	Bottom	Stirrup
B1	50×25	3Ø14	3Ø14	Ø8/20/10
B2	50×25	3Ø14	3Ø14	Ø8/20/10

Table 5.5: Reinforcement details of columns for Second Case Study

ID	B	H	Major	Minor	Lateral	Perc.	S.R.Percenta...
S01	70 cm	30 cm	6 ø 14	8 ø 14	ø8/15/9/10	1.03%	0.05%
S03	30 cm	60 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.18%
S04	30 cm	60 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.01%
S05	30 cm	70 cm	6 ø 14	8 ø 14	ø8/15/9/10	1.03%	0.03%
S06	70 cm	30 cm	6 ø 14	8 ø 14	ø8/15/9/10	1.03%	0.16%
S09	60 cm	25 cm	6 ø 14	4 ø 14	ø8/12/7/10	1.03%	-0.01%
S10	70 cm	30 cm	6 ø 14	8 ø 14	ø8/15/9/10	1.03%	0.08%
S11	70 cm	25 cm	6 ø 14	6 ø 14	ø8/12/8/10	1.06%	0.06%
S12	70 cm	25 cm	6 ø 14	6 ø 14	ø8/12/8/10	1.06%	0.26%
S13	70 cm	30 cm	6 ø 14	8 ø 14	ø8/15/9/10	1.03%	0.17%
S14	70 cm	25 cm	6 ø 14	6 ø 14	ø8/12/8/10	1.06%	0.06%
S15	70 cm	25 cm	6 ø 14	6 ø 14	ø8/12/8/10	1.06%	0.22%
S16	70 cm	25 cm	6 ø 14	6 ø 14	ø8/12/8/10	1.06%	0.08%
S17	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.06%
S18	25 cm	60 cm	6 ø 14	4 ø 14	ø8/12/7/10	1.03%	0.25%
S21	70 cm	30 cm	6 ø 14	8 ø 14	ø8/15/9/10	1.03%	0.05%
S22	60 cm	30 cm	6 ø 14	6 ø 14	ø8/15/8/10	1.03%	0.09%
S24	70 cm	30 cm	6 ø 14	8 ø 14	ø8/15/9/10	1.03%	0.32%
S25	70 cm	30 cm	6 ø 14	8 ø 14	ø8/15/9/10	1.03%	0.08%
S29	90 cm	25 cm	6 ø 14	10 ø 14	ø8/12/7/10	1.09%	0.14%
S31	70 cm	25 cm	6 ø 14	6 ø 14	ø8/12/8/10	1.06%	0.17%
S32	90 cm	30 cm	6 ø 14	12 ø 14	ø8/15/9/10	1.03%	0.31%
S33	90 cm	25 cm	6 ø 14	10 ø 14	ø8/12/7/10	1.09%	0.19%
S35	70 cm	25 cm	6 ø 14	6 ø 14	ø8/12/8/10	1.06%	0.19%
S36	90 cm	30 cm	6 ø 14	12 ø 14	ø8/15/9/10	1.03%	0.16%

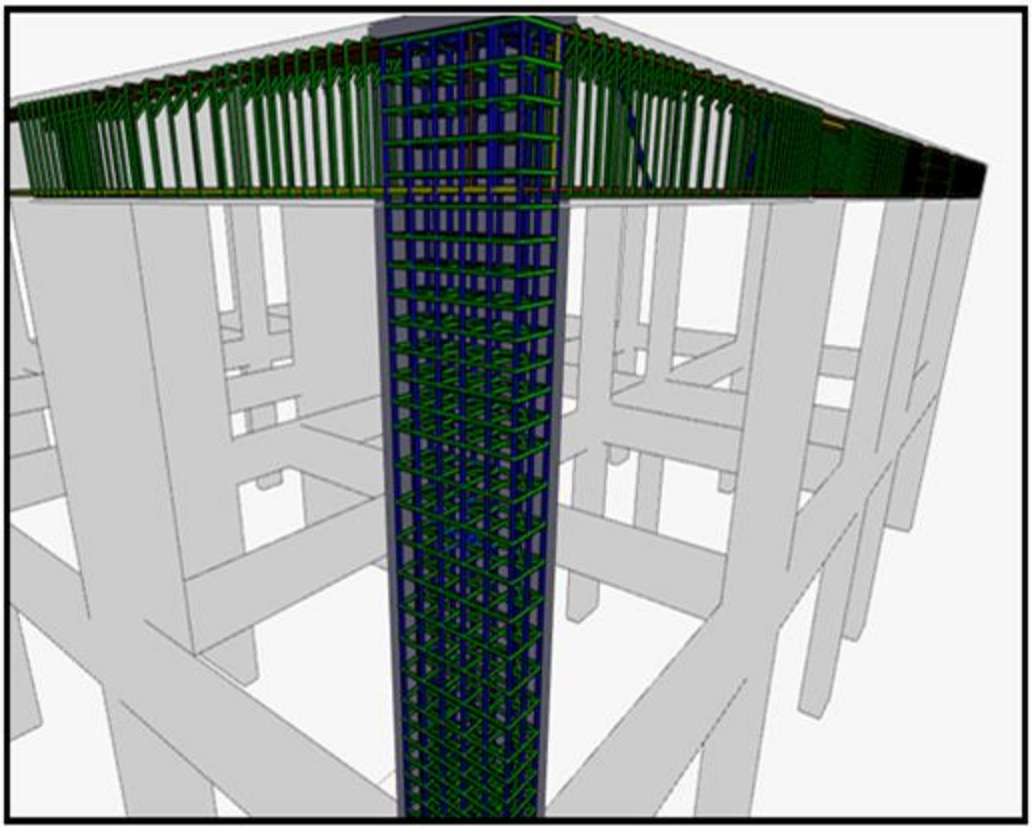

Figure 5.16: 3D View of beams and columns in Idecad

5.2.3 Third Case Study

In this case study, 4-storey reinforce concrete frame with 12.8m in elevation and 8-storey and 12-storey reinforce concrete frame having an elevation of 25.6 m and 38.4 m respectively will discuss in detail. Each of these three cases are further divided into three other cases for comprising which are bare frame, soft storey, and fully infilled RC frame all cases of two types with and without shear wall. First of all 4-storey building is design based on software known as Idecad according to Turkish Earthquake Code TEC2007. The lateral load of 10 kN, 7.5 kN and 5 kN is applied for pushover method. Some of the parameters the structure is given below. All the floors are having the same elevation of 3.2 m. The total area of this building is 1035.125 m^2. The plan is shown in Figure 5.8 and 5.9. The 3D layouts are given in

Figure 5.10 and Figure 5.11. The section characteristics of the beams and columns are detail in table 5.5 and 5.6.

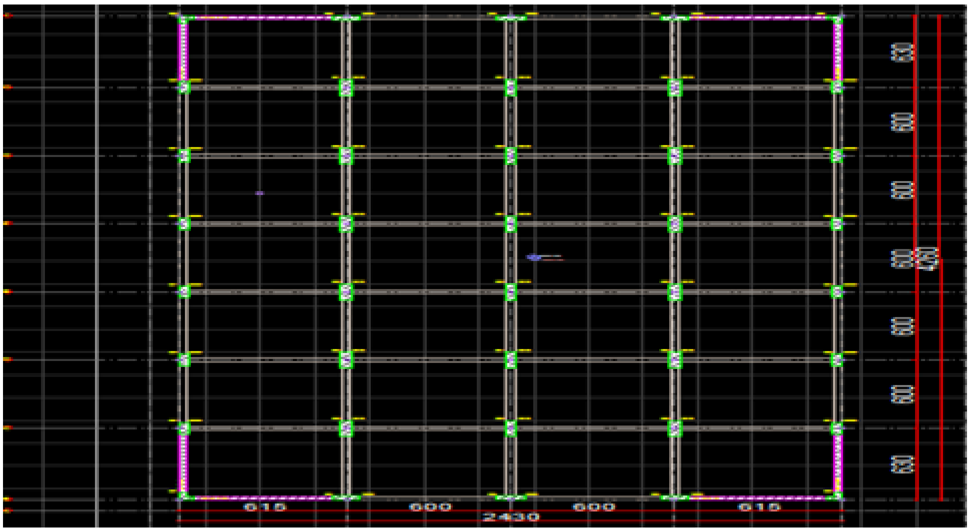

Figure 5.17: Plan details of Second Case Study with Shear wall

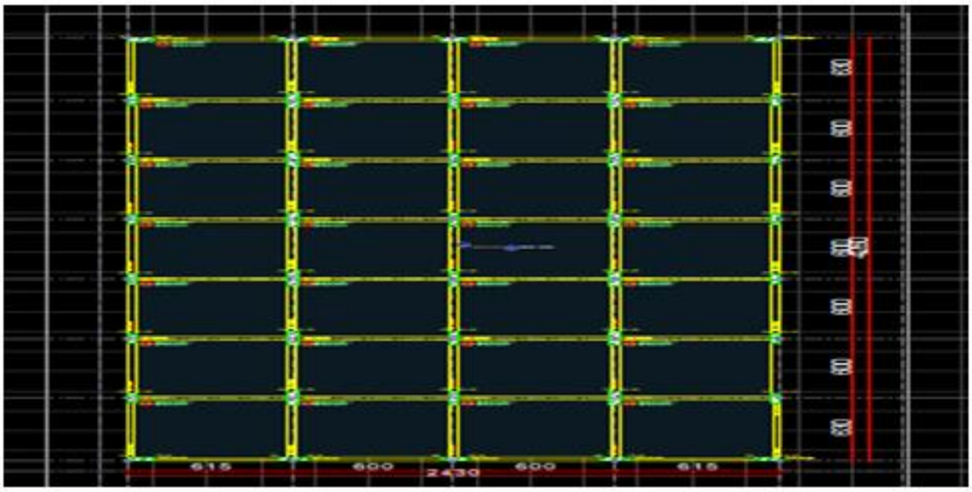

Figure 5.18: Details of third Case Study without shear wall

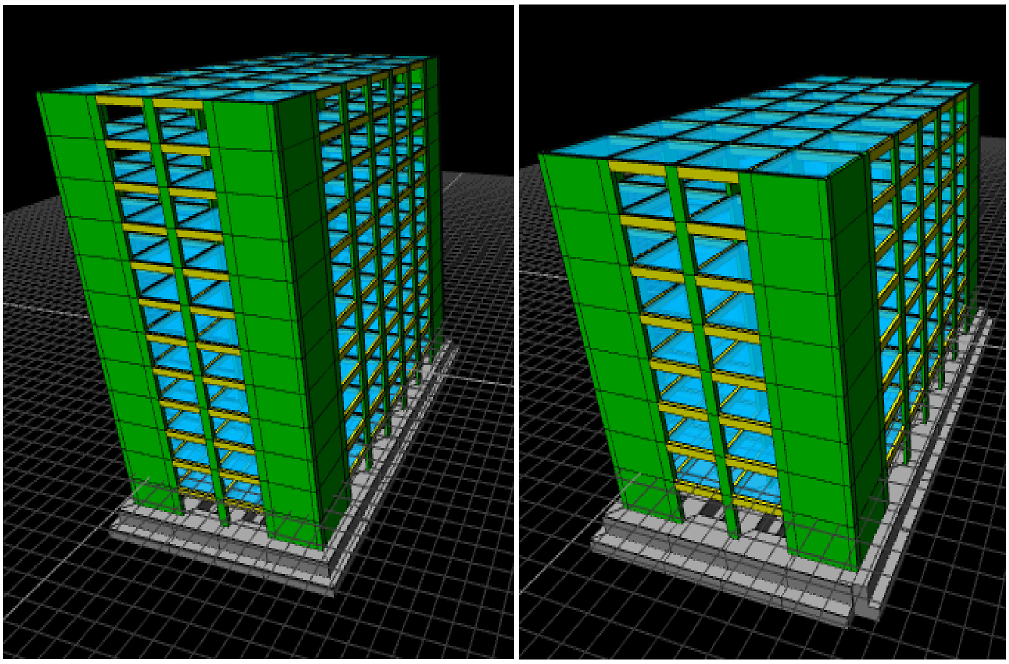

Figure 5.19: 3D layout of, 8-storey building and 12-storey building with shear wall Idecad

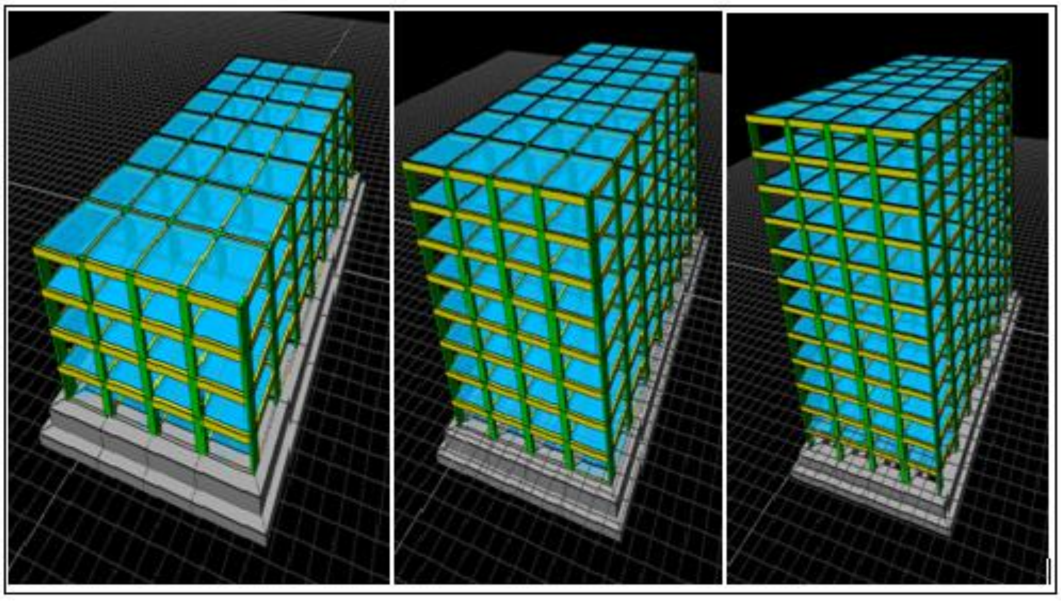

Figure 5.11: 3D layout of 4-storey, 8-storey building and 12-storey building without shear wall

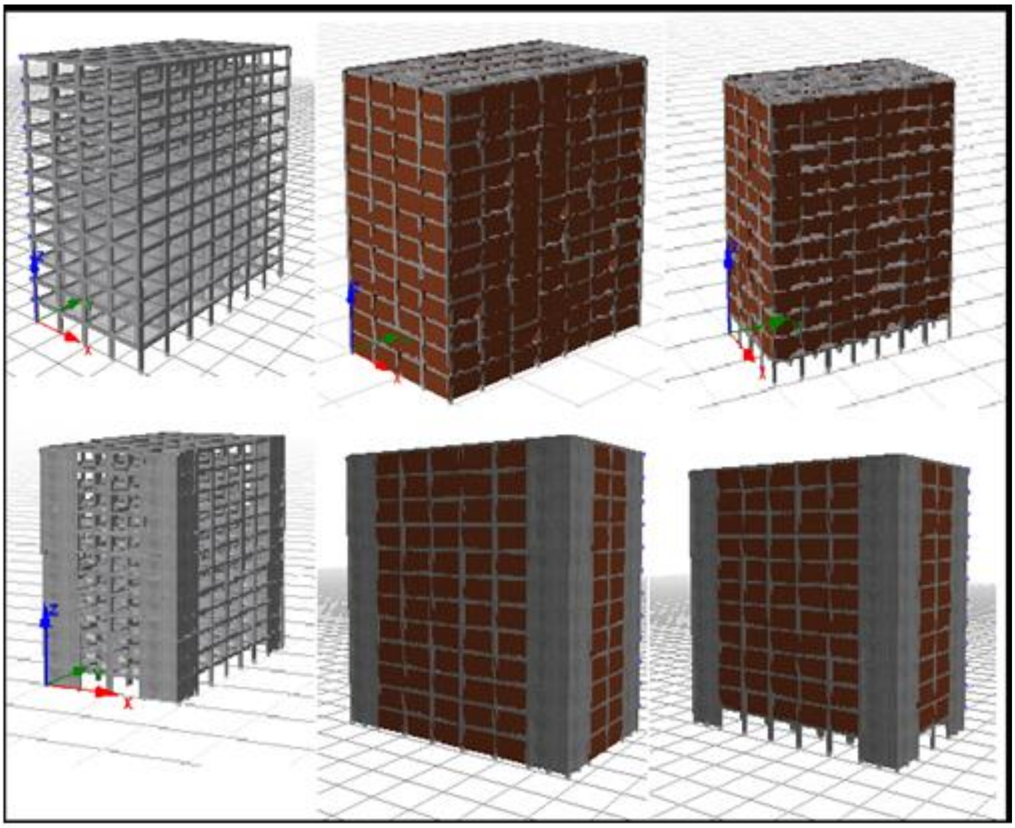

Figure 5.20: 3D layout of 12-storey in third study case in Seismostruct

Table 5.6: Reinforcement details of the beams for Third Case Study

Beam	Dimension (cm)	Top	Bottom	Stirrups
B1	35×60	4Ø14	4Ø14	Ø8/20/10
B2	40×60	4Ø14	4Ø14	Ø8/20/10

Table 5.7: Reinforcement details of columns for Third Case Study

ID	B	H	Major	Minor	Lateral	Perc.	S.R.Percenta...
S01	35 cm	100 cm	8 ø 14	16 ø 14	ø8/10/10/10	1.06%	0.12%
S02	35 cm	100 cm	8 ø 14	16 ø 14	ø8/16/10/10	1.06%	-0.73%
S03	35 cm	100 cm	8 ø 14	16 ø 14	ø8/10/10/10	1.06%	0.12%
S04	35 cm	100 cm	8 ø 14	16 ø 14	ø8/16/10/10	1.06%	0.08%
S05	40 cm	140 cm	10 ø 14	28 ø 14	ø8/10/10	1.04%	-0.92%
S06	40 cm	140 cm	10 ø 14	28 ø 14	ø8/10/10	1.04%	-0.94%
S07	40 cm	140 cm	10 ø 14	28 ø 14	ø8/10/10	1.04%	-0.92%
S08	35 cm	100 cm	8 ø 14	16 ø 14	ø8/16/10/10	1.06%	0.09%
S09	35 cm	100 cm	8 ø 14	16 ø 14	ø8/16/10/10	1.06%	-0.83%
S10	40 cm	140 cm	10 ø 14	28 ø 14	ø8/10/10	1.04%	-0.95%
S11	40 cm	140 cm	10 ø 14	28 ø 14	ø8/10/10	1.04%	-0.95%
S12	40 cm	140 cm	10 ø 14	28 ø 14	ø8/10/10	1.04%	-0.95%
S13	35 cm	100 cm	8 ø 14	16 ø 14	ø8/16/10/10	1.06%	-0.82%
S14	35 cm	100 cm	8 ø 14	16 ø 14	ø8/16/10/10	1.06%	-0.93%
S15	40 cm	140 cm	10 ø 14	28 ø 14	ø8/10/10	1.04%	-0.96%
S16	40 cm	140 cm	10 ø 14	28 ø 14	ø8/10/10	1.04%	-0.96%
S17	40 cm	140 cm	10 ø 14	28 ø 14	ø8/10/10	1.04%	-0.95%
S18	35 cm	100 cm	8 ø 14	16 ø 14	ø8/16/10/10	1.06%	-0.93%
S19	35 cm	100 cm	8 ø 14	16 ø 14	ø8/16/10/10	1.06%	-0.93%
S20	40 cm	140 cm	10 ø 14	28 ø 14	ø8/10/10	1.04%	-0.95%
S21	40 cm	140 cm	10 ø 14	28 ø 14	ø8/10/10	1.04%	-0.96%
S22	40 cm	140 cm	10 ø 14	28 ø 14	ø8/10/10	1.04%	-0.96%
S23	35 cm	100 cm	8 ø 14	16 ø 14	ø8/16/10/10	1.06%	-0.93%
S24	35 cm	100 cm	8 ø 14	16 ø 14	ø8/16/10/10	1.06%	-0.82%
S25	40 cm	140 cm	10 ø 14	28 ø 14	ø8/10/10	1.04%	-0.95%
S26	40 cm	140 cm	10 ø 14	28 ø 14	ø8/10/10	1.04%	-0.95%
S27	40 cm	140 cm	10 ø 14	28 ø 14	ø8/10/10	1.04%	-0.95%
S28	35 cm	100 cm	8 ø 14	16 ø 14	ø8/16/10/10	1.06%	-0.82%
S29	35 cm	100 cm	8 ø 14	16 ø 14	ø8/16/10/10	1.06%	0.09%
S30	40 cm	140 cm	10 ø 14	28 ø 14	ø8/10/10	1.04%	-0.94%
S31	40 cm	140 cm	10 ø 14	28 ø 14	ø8/10/10	1.04%	-0.92%
S32	35 cm	100 cm	8 ø 14	16 ø 14	ø8/16/10/10	1.06%	0.08%
S34	35 cm	100 cm	8 ø 14	16 ø 14	ø8/16/10/10	1.06%	0.12%
S35	35 cm	100 cm	8 ø 14	16 ø 14	ø8/16/10/10	1.06%	-0.73%
S36	35 cm	100 cm	8 ø 14	16 ø 14	ø8/10/10/10	1.06%	0.12%
S38	40 cm	140 cm	10 ø 14	28 ø 14	ø8/10/10	1.04%	-0.92%
S39	35 cm	100 cm	8 ø 14	16 ø 14	ø8/13/10/10	1.06%	0.05%
S40	35 cm	100 cm	8 ø 14	16 ø 14	ø8/13/10/10	1.06%	0.06%
S41	35 cm	100 cm	8 ø 14	16 ø 14	ø8/13/10/10	1.06%	0.07%
S42	35 cm	100 cm	8 ø 14	16 ø 14	ø8/13/10/10	1.06%	0.07%

Chapter 6

RESULTS AND DISCUSSIONS

6.1 Introduction

The analyses results are summarize in this chapter. This chapter begins with the pushover analysis result, performance level due to target displacement. In SeismoStruct, pushover analysis give different results such as displacement-base shear curve, deformed shape of the structure, shear force and bending moment diagram of the elements, hysteretic curve.

6.1 Displacement-Base Shear Curve (Capacity curve)

Pushover analysis result show an inflation in the firmness, strength and energy dissipation of a RC frame structure having infill wall as compared to bare RC frame. As discuss in the previous chapter for each case study different comparison are made to find the seismic behavior of infill wall in RC frame structure.

The pushover curves obtain for different case studies are shown in detail as following.

6.1.1 First case study:

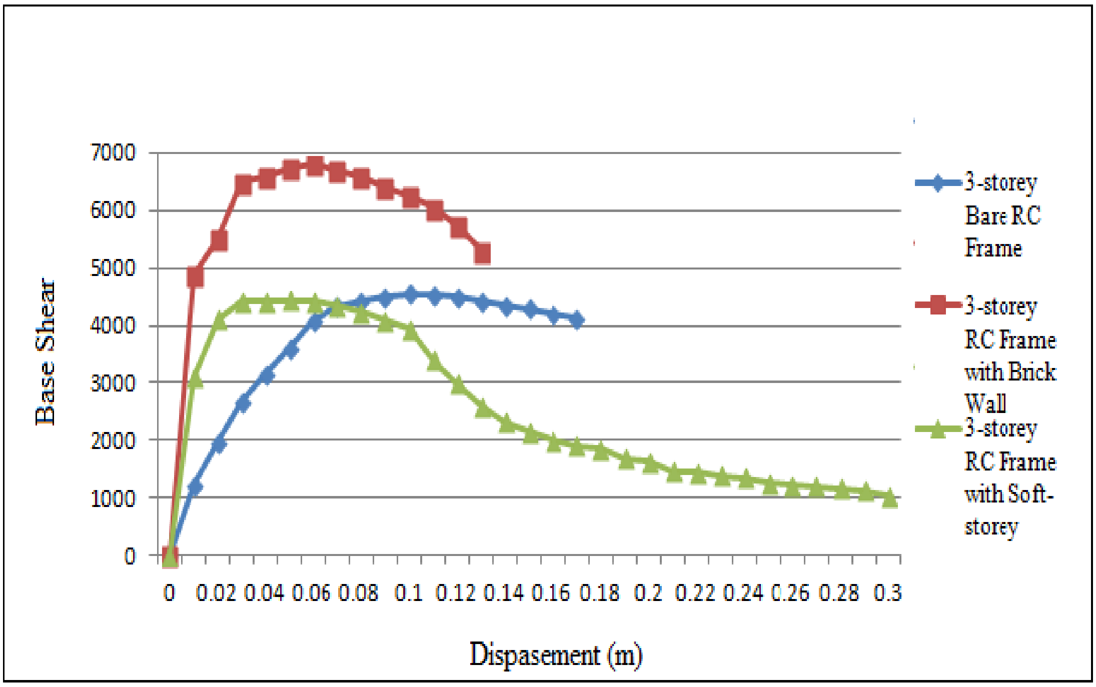

Figure 6.1: Comparison of Displacement-Base Shear Curve between 3-storey buildings

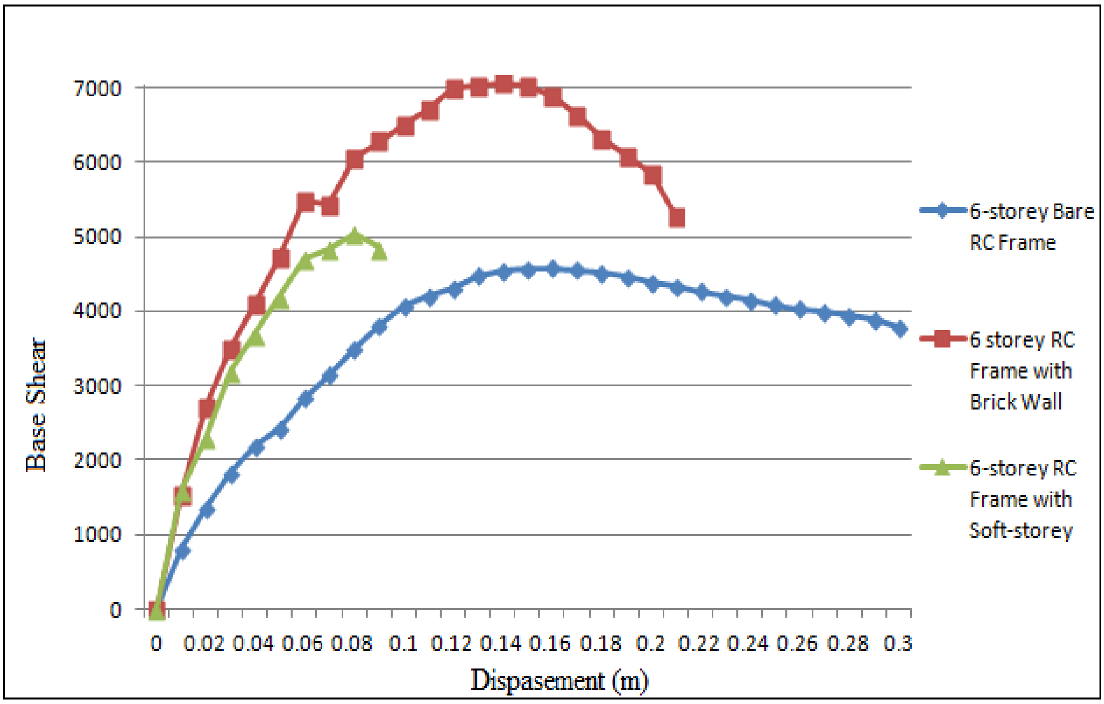

Figure 6.2: Comparison of Displacement-Base Shear Curve between 6-storey buildings.

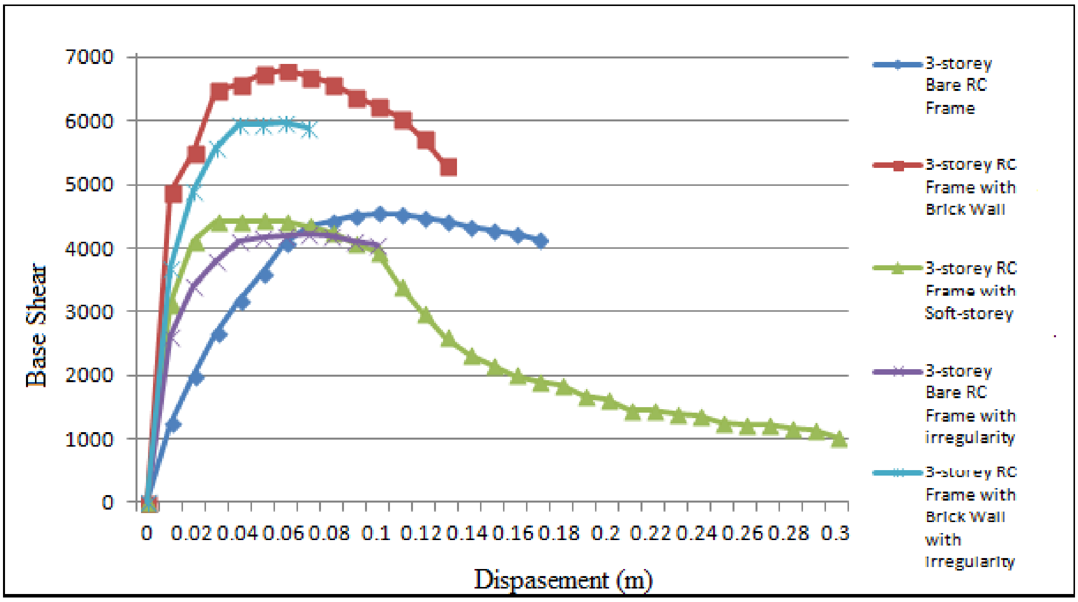

Figure 6.3: Comparison of Displacement-Base Shear 3-storey building irregularity case

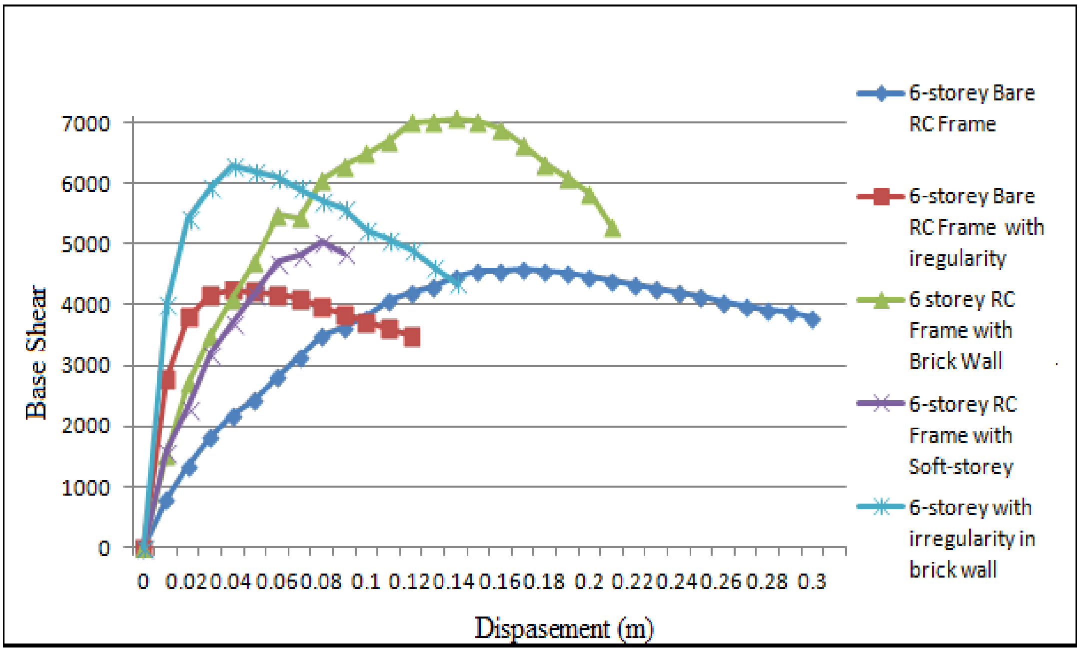

Figure 6.4: Comparison of Displacement-Base Shear 6-storey building irregularity case.

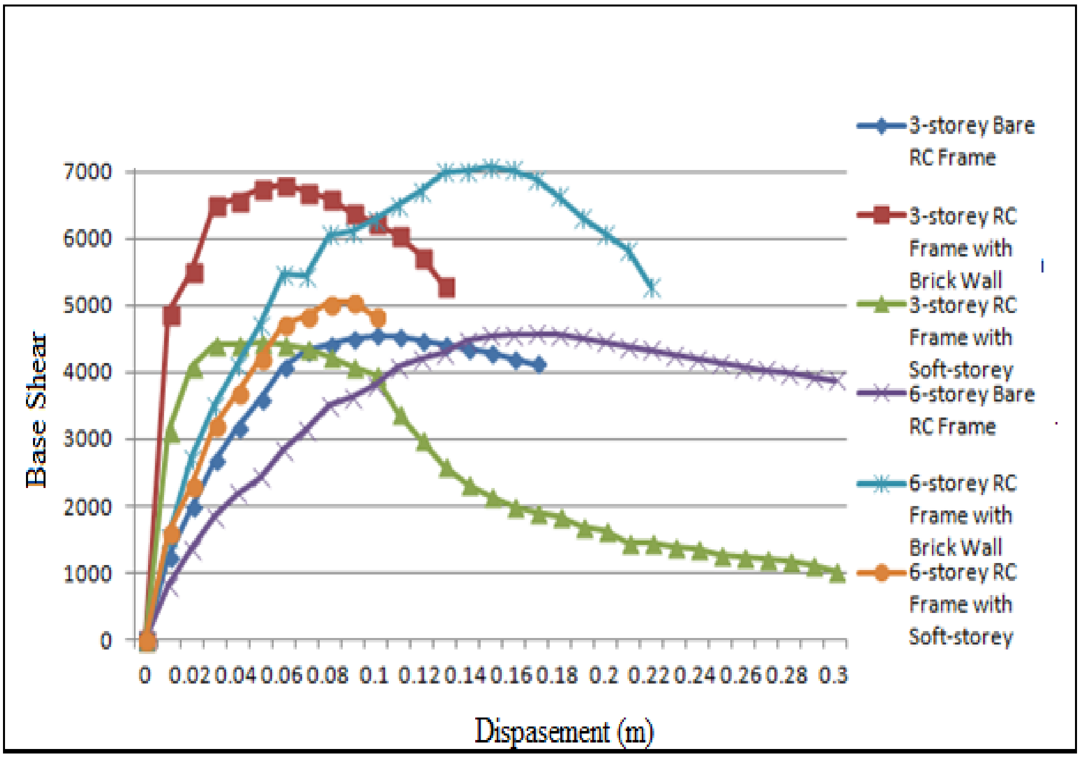

Figure 6.5: Comparison of Displacement-Base Shear Curve of between 3-storey and 6-storey building

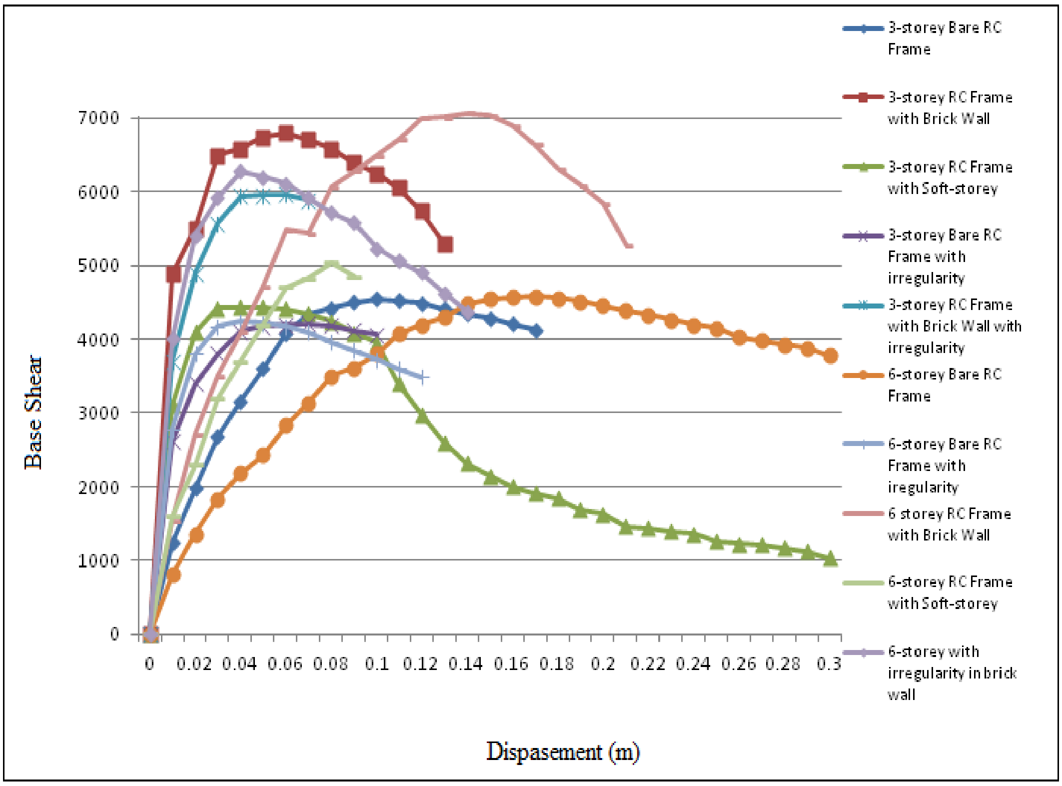

Figure 6.6: Comparison of Displacement-Base Shear Curve of between 3-storey and 6-storey building irregularity case

6.1.2 Second Case study:

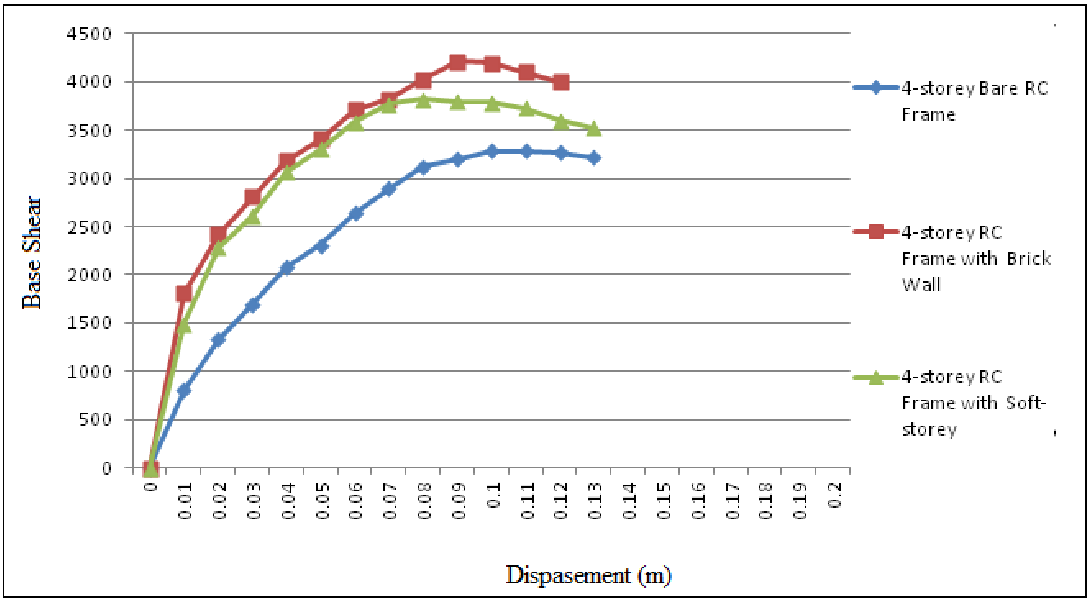

Figure 6.7: Comparison of Displacement-Base Shear Curve of 4-storey building

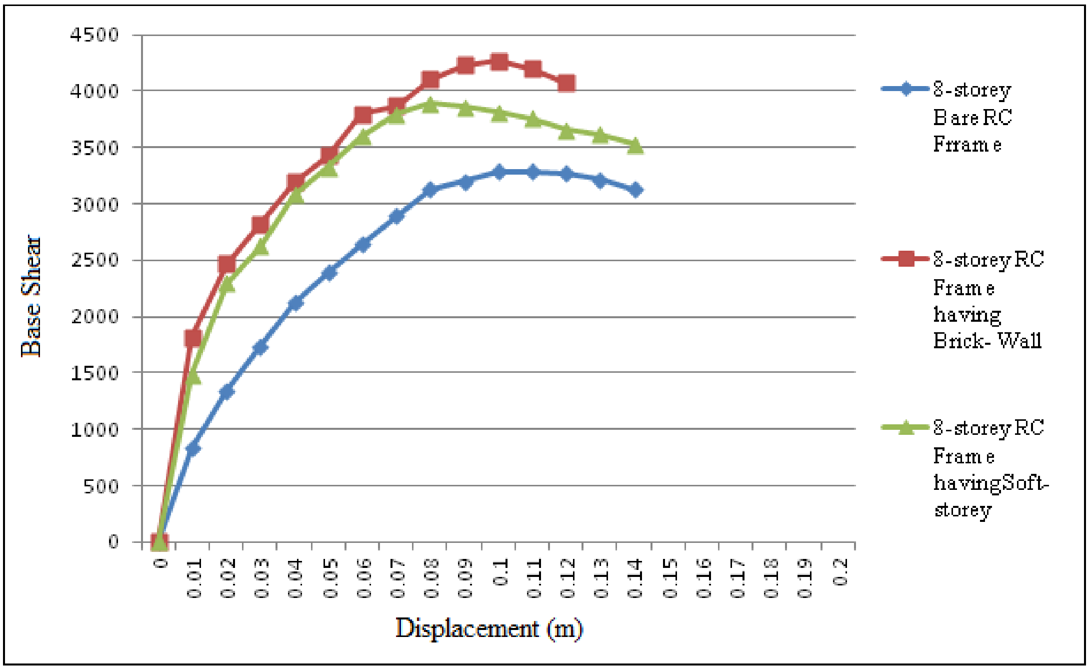

Figure 6.8: Comparison of Displacement-Base Shear Curve of 8-storey building

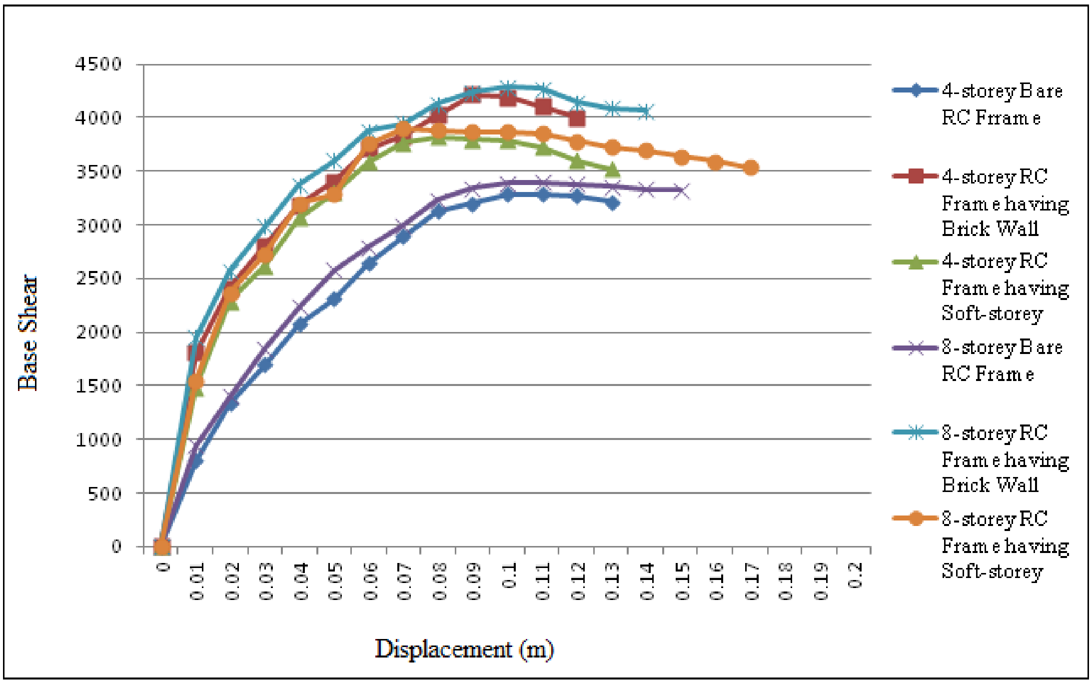

Figure 6.9: Comparison of Displacement-Base Shear Curve between 4-storey and 8-storey building

6.1.3 Third Case study:

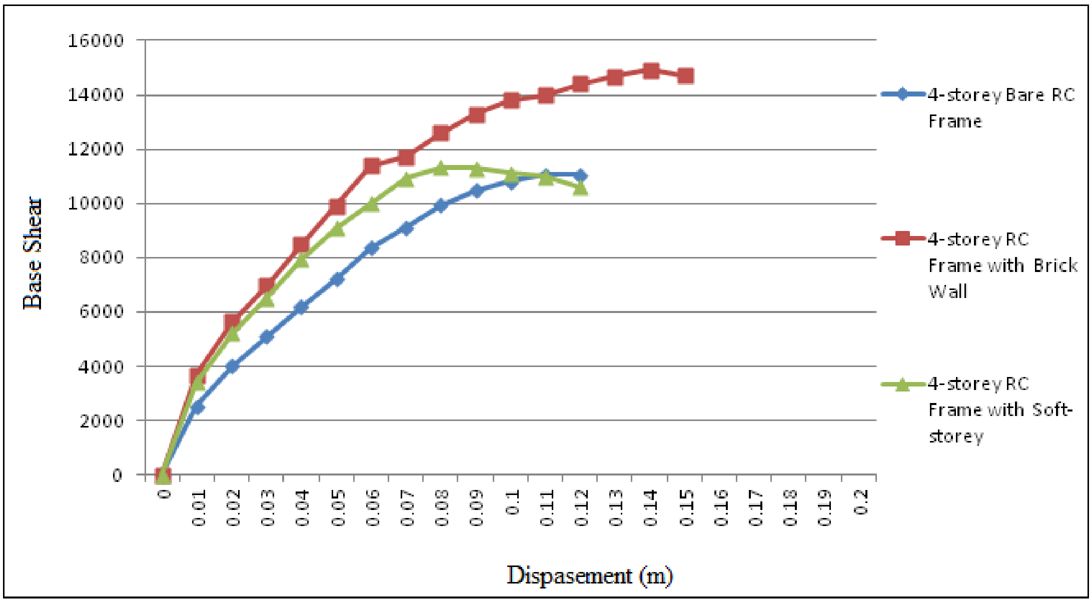

Figure 6.10: Comparison of Displacement-Base Shear Curve of 4-storey building

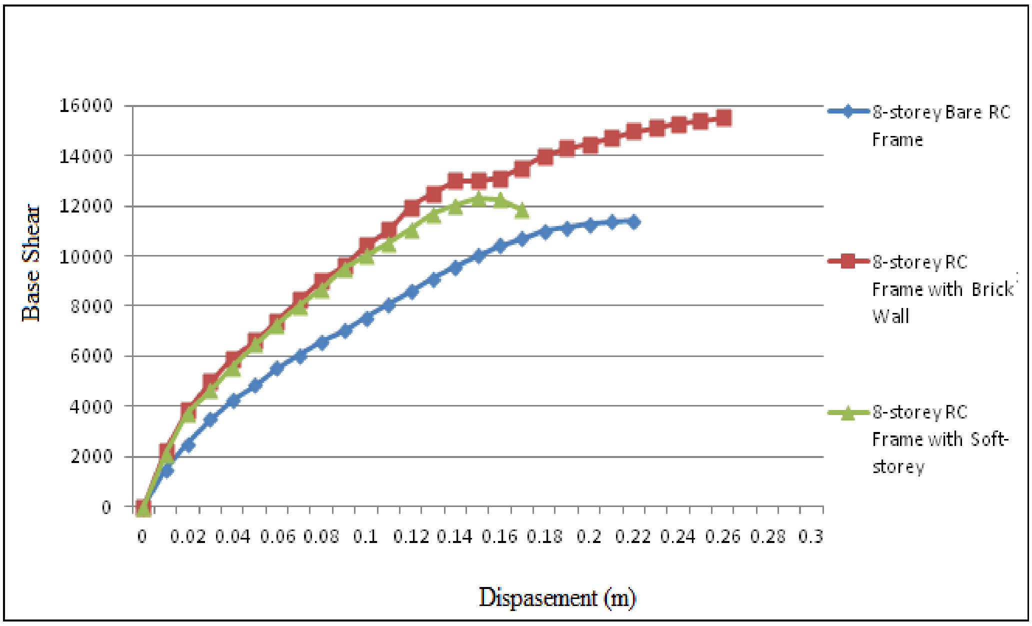

Figure 6.11: Comparison of Displacement-Base Shear Curve of 8-storey building

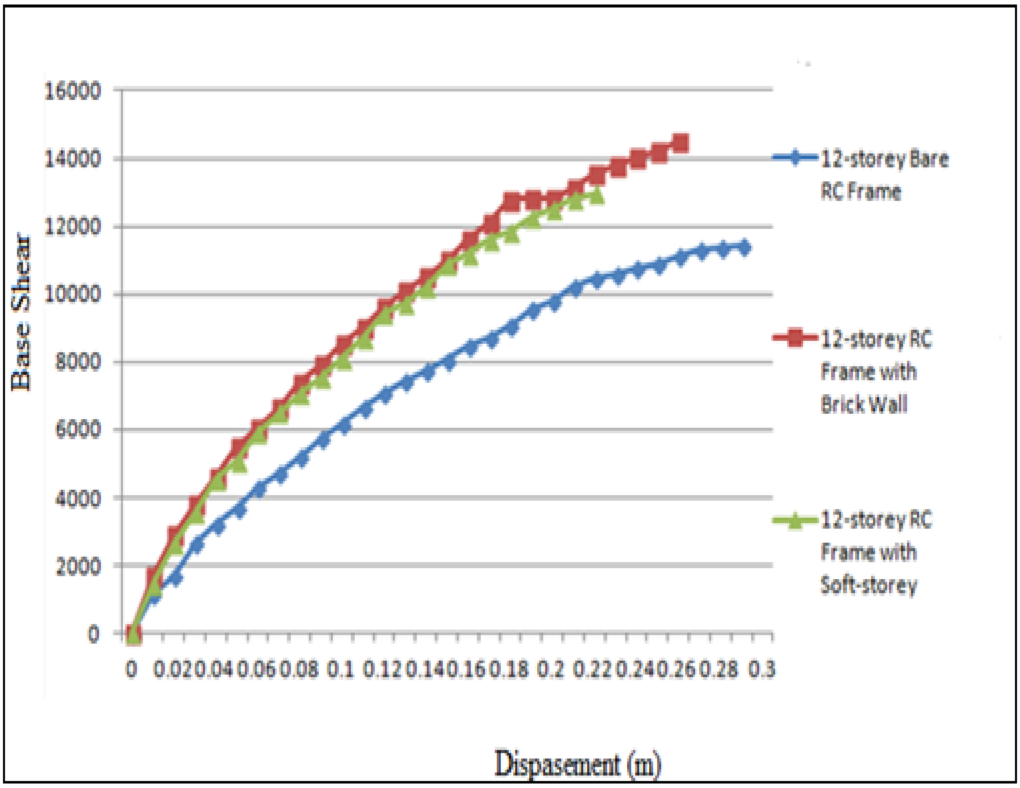

Figure 6.12: Comparison of Displacement-Base Shear Curve of 12-storey building

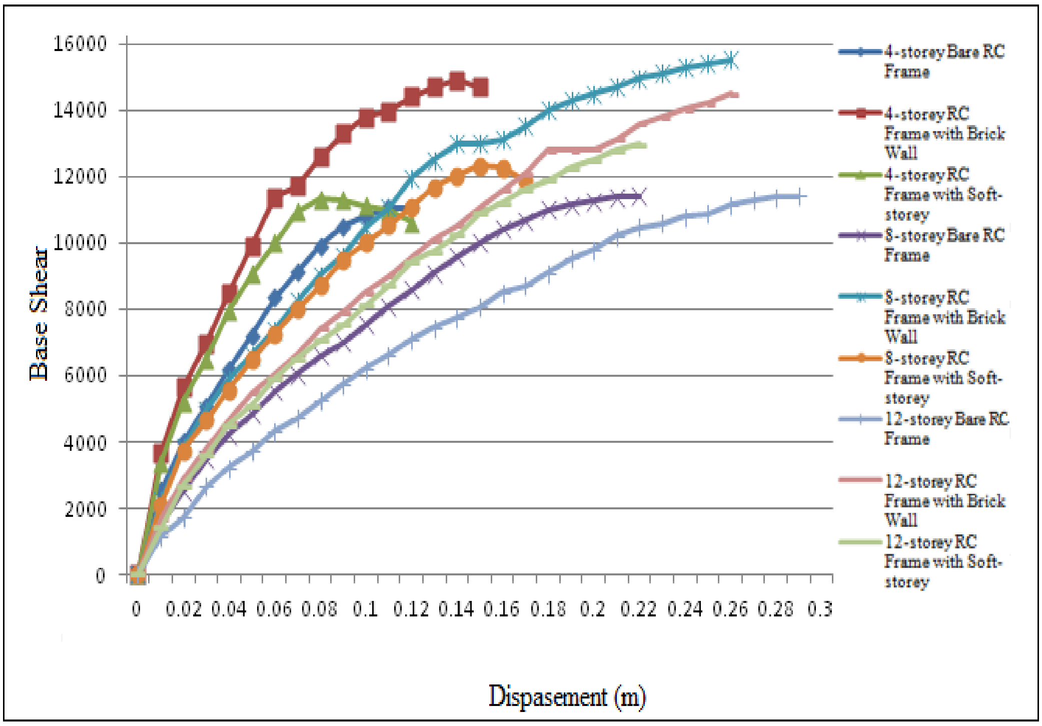

Figure 6.13: Comparison of Displacement-Base Shear Curve of between 4-storey, 8-storey and 12-storey buildings

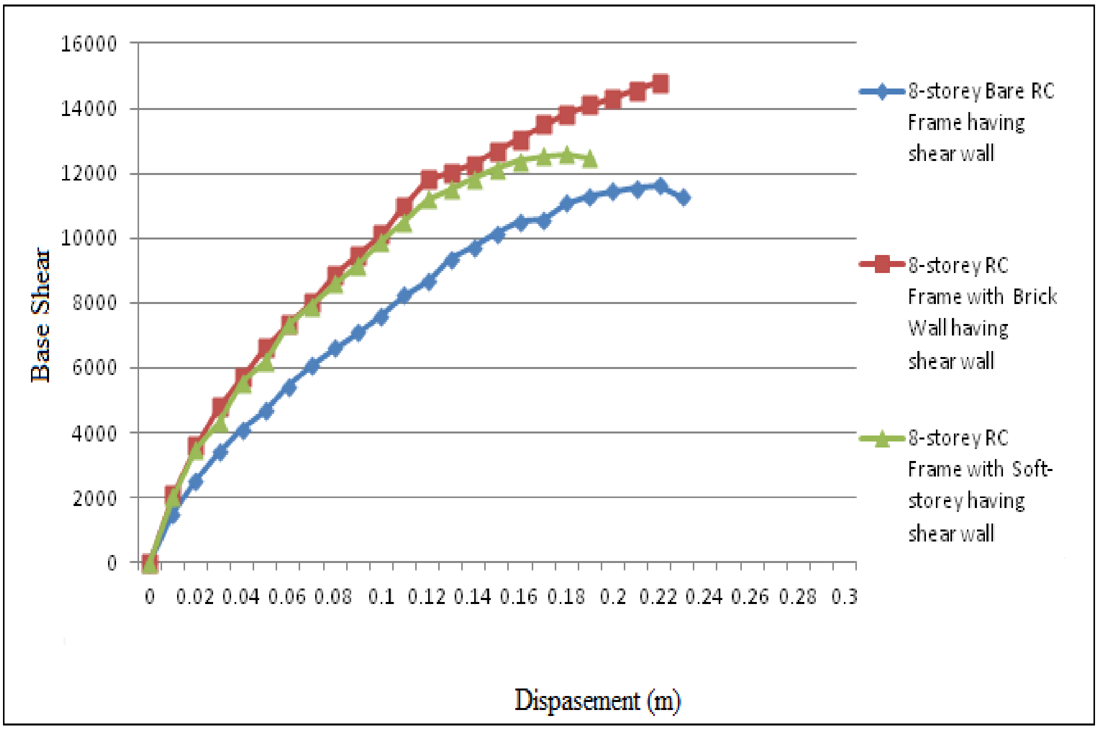

Figure 6.14: Comparison of Displacement-Base Shear Curve of 8-storey building having shear walls

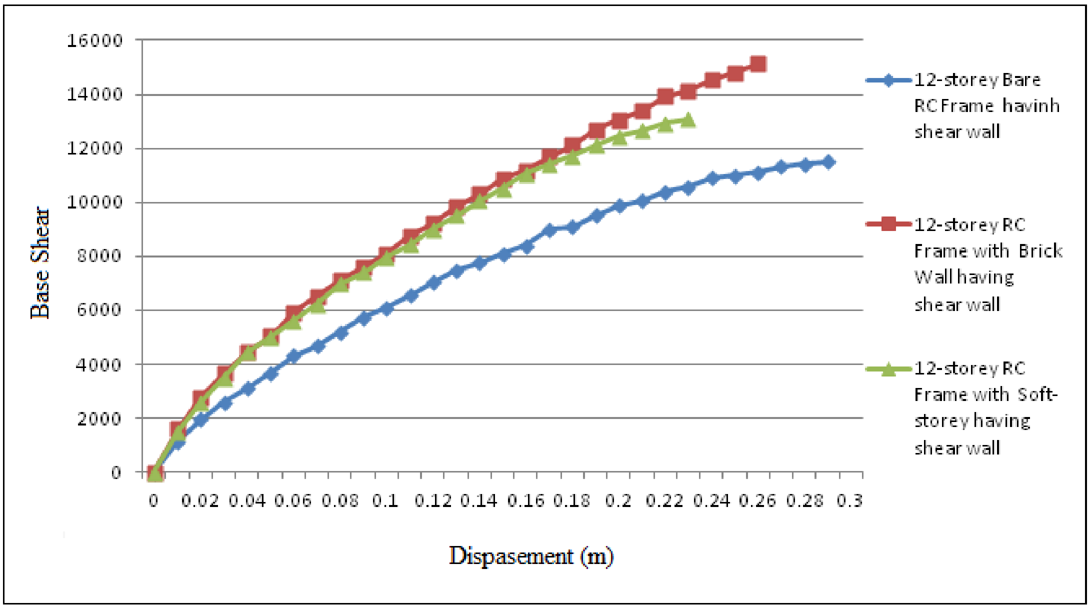

Figure 6.15: Comparison of Displacement-Base Shear Curve of 12-storey building having shear walls

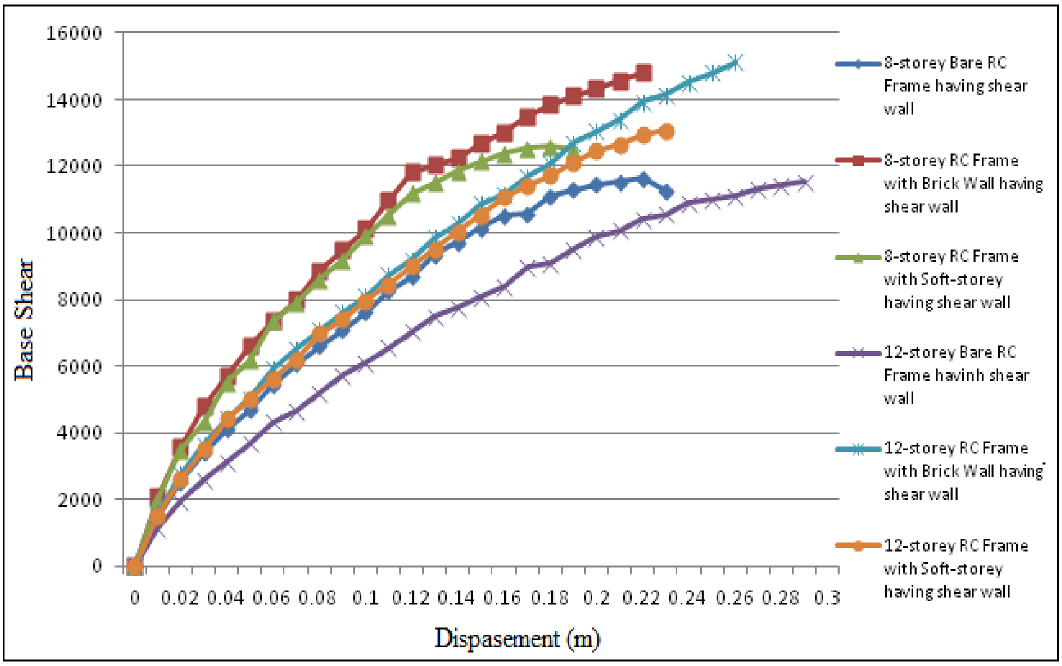

Figure 6.16: Comparison of Displacement-Base Shear Curve of 8 –storey and 12-storey building having shear walls

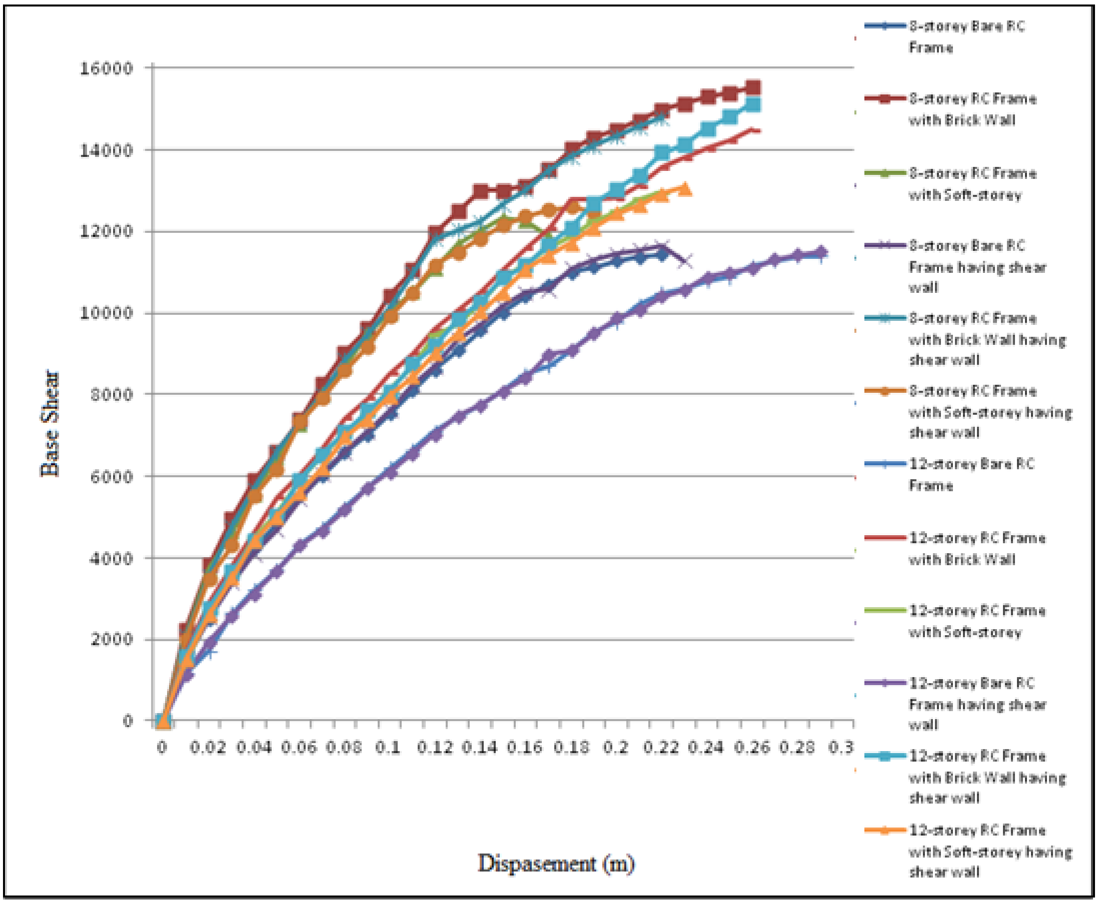

Figure 6.17: Comparison of Displacement-Base Shear Curve of 8 –storey and 12-storey building with or without shear walls

The results obtained from pushover analysis showed great response in the strength, stiffness and energy dissipation of the building when infill is added to the frame. Fully-infilled frame shows way greater peak strength and larger stiffness as compare to bare frame in all study cases. The Pushover responses of soft-storey frame structures are in between fully infill wall cases and bare frame cases. Soft-storey in all studies cases shows slight greater peak strength and larger stiffness then bare frames cases. Fully infill wall and soft-storey cases cause decrease in the deformation capacity of the building. As the building elevation rise it shows more stiffness and strength and less deformation. In first case the partially infilled wall give high base shear compare to soft-storey but give lower base shear then fully infilled wall. In addition of shear wall in third cases it has a slightly impact on response of stiffness of the structure. As from the result its show that, the brick infilled wall in RC frames have great base shear towards displacement compare to soft story and bare Frame.

6.2 Target Displacement

Target displacement is obtained to find out the structure's response for critical point where the structure's behavior is changing due to the increase of lateral forces. For all capacity curves an increase in base-shear occur which causes a displacement up to a certain point where slope of the line is changed. This is where the point decrease starts in the strength and stiffness of the buildings and at this point structure is yield.

In SeismoStruct 2016 target displacement point is directly pointed out on Base-shear displacement curve. The yielding of the structures can be observed from the capacity curves of all the study cases. The obtained results demonstrate that the 3-story bare RC frame starts to change their performance level to Operational Level (OL) at this level the structure has 0.017 m top displacement. According to Eurocode 8, Very light damage occurs, Structural elements retain their strength and stiffness, No permanent drifts, No significant cracking of infill walls, and slight damage could be economically repaired. In the next step, top displacement is reached to 0.0218 m and the performance level of frame raise and change to Significant Damage. In this level it shows significant damage to the structural system however retention of some lateral strength and stiffness, Vertical elements capable of sustaining vertical loads, Infill walls severally damaged, Moderate permanent drifts exist, The structure can sustain moderate aftershocks, The cost of repair may be high. The cost of reconstruction should be examined as alternative solution some, structural components and elements have suffered comprehensive damage and there is risk of injury to life. Structure collapse at step where top displacement is equal to 0.0397m.

For different cases studies performance level for the target displacement visualized in the form of in Tables and Figures are given as follow;

Table 6.1: Performance level for target displacement of First Study Case

Target Displacement (m)	3-storey Bare RC Frame	3-storey RC Frame with Brick Wall	3-storey Bare RC Frame with soft storey	3-storey Bare RC Frame with irregularity	3-storey RC frame with Brick irregularity	6-storey Bare RC Frame	6-storey Bare RC frame with irregularity	6-storey RC Frame with brick wall	6-storey RC frame with soft storey	6-storey RC frame with soft storey
Operational Level (OL)	0.0170	0.0050	0.0063	0.0198	0.0051	0.0375	0.0105	0.0326	0.0303	0.00916
Damage Limitation (DL)	0.0218	0.0064	0.0080	0.0254	0.0065	0.0482	0.0134	0.0418	0.0389	0.0117
Significant Damage (SD)	0.0379	0.0111	0.0140	0.0441	0.0144	0.0836	0.0272	0.0725	0.0675	0.0236

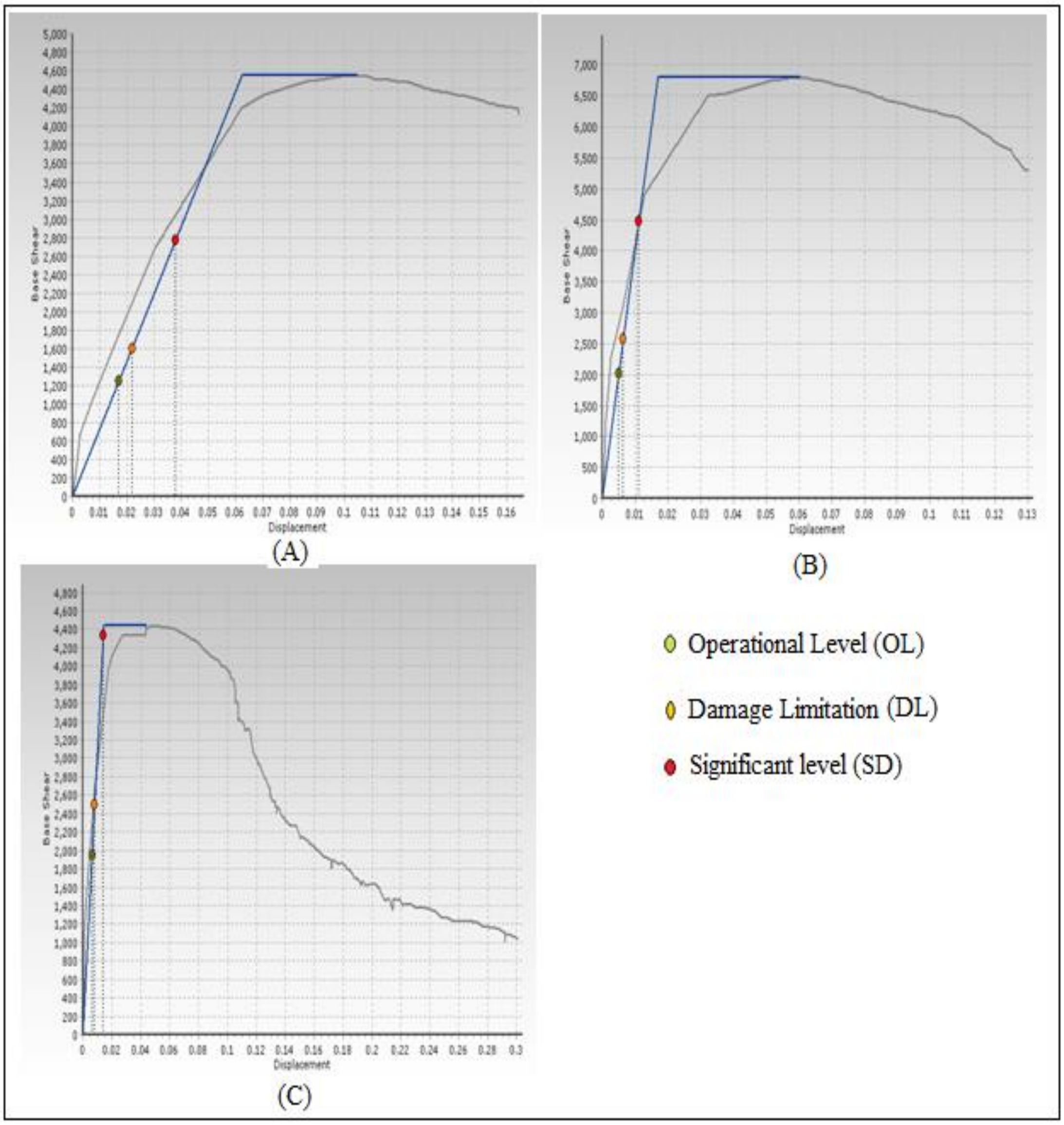

Figure 6.18: Target displacement performance level for (A) 3-Stories Bare RC Frame, (B) 3-stories RC Frame having Brick Wall and (C) 3-storey RC Frame having soft storey of first study case

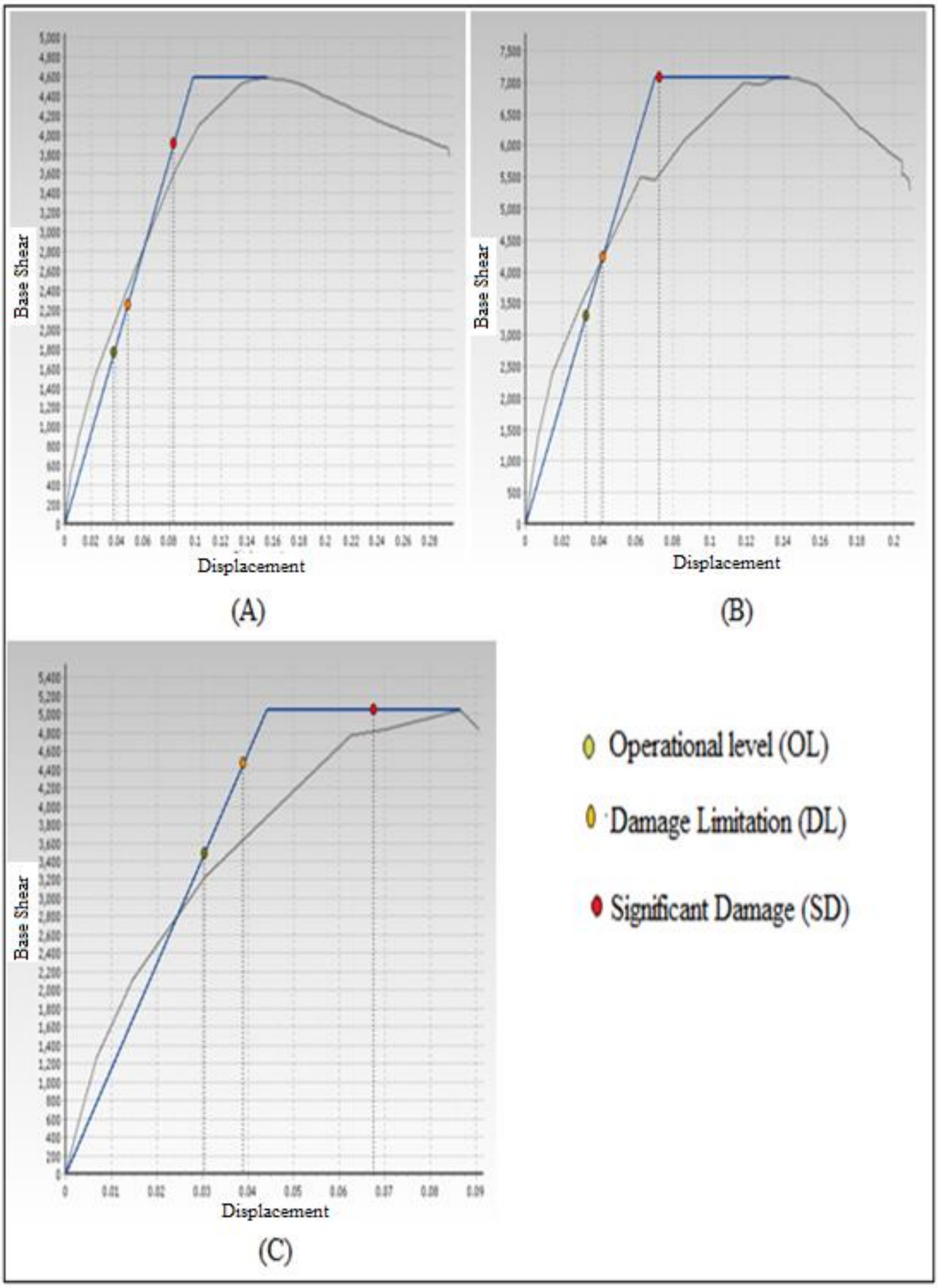

Figure 6.19: Target displacement performance level for (A) 6-Storey Bare RC Frame, (B) 6-storey RC Frame having Brick Wall and (C) 6-storey RC Frame having soft storey of First study case

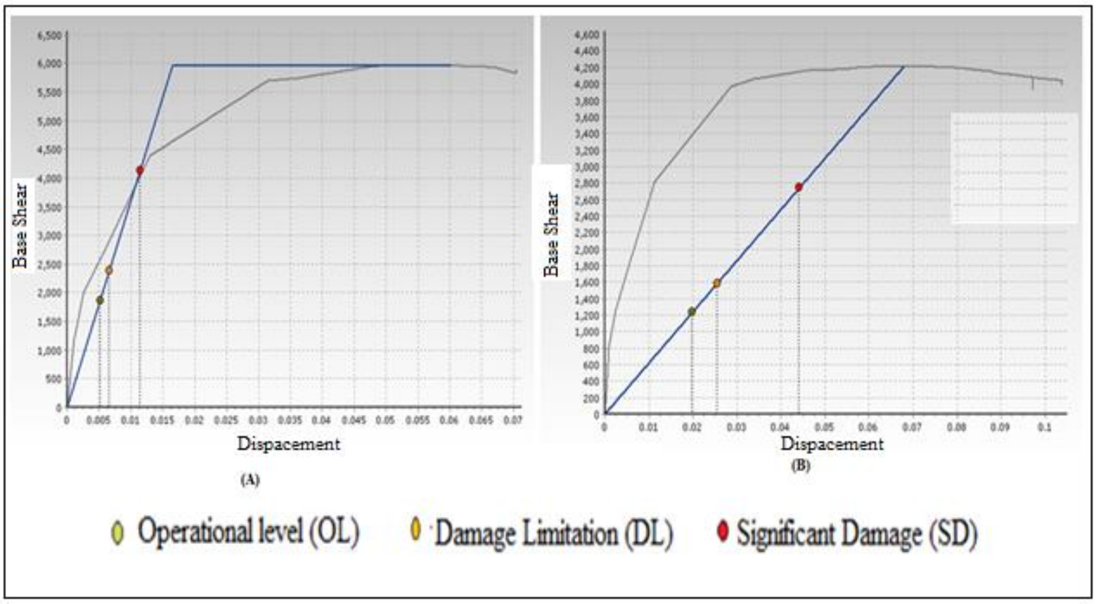

Figure 6.20: Target displacement performance level for (A) 3-Storeey Bare RC Frame, (B) 3-storeey RC Frame having Brick Wall irregularity case for first study case

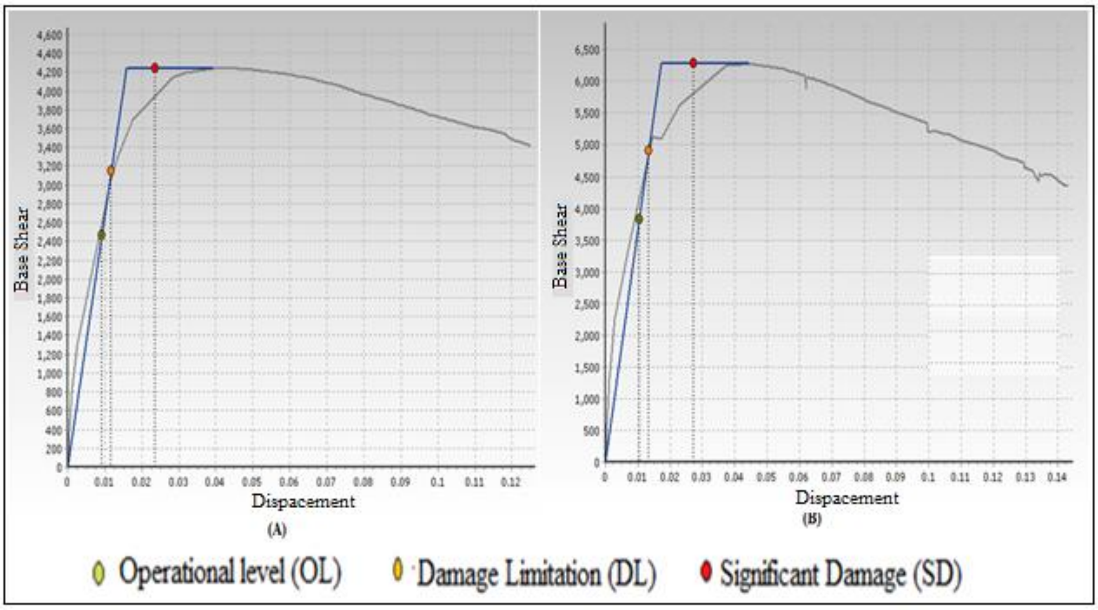

Figure 6.21: Target displacement performance level for (A) 6-Storey Bare RC Frame, (B) 6-storey RC Frame having Brick Wall irregularity case

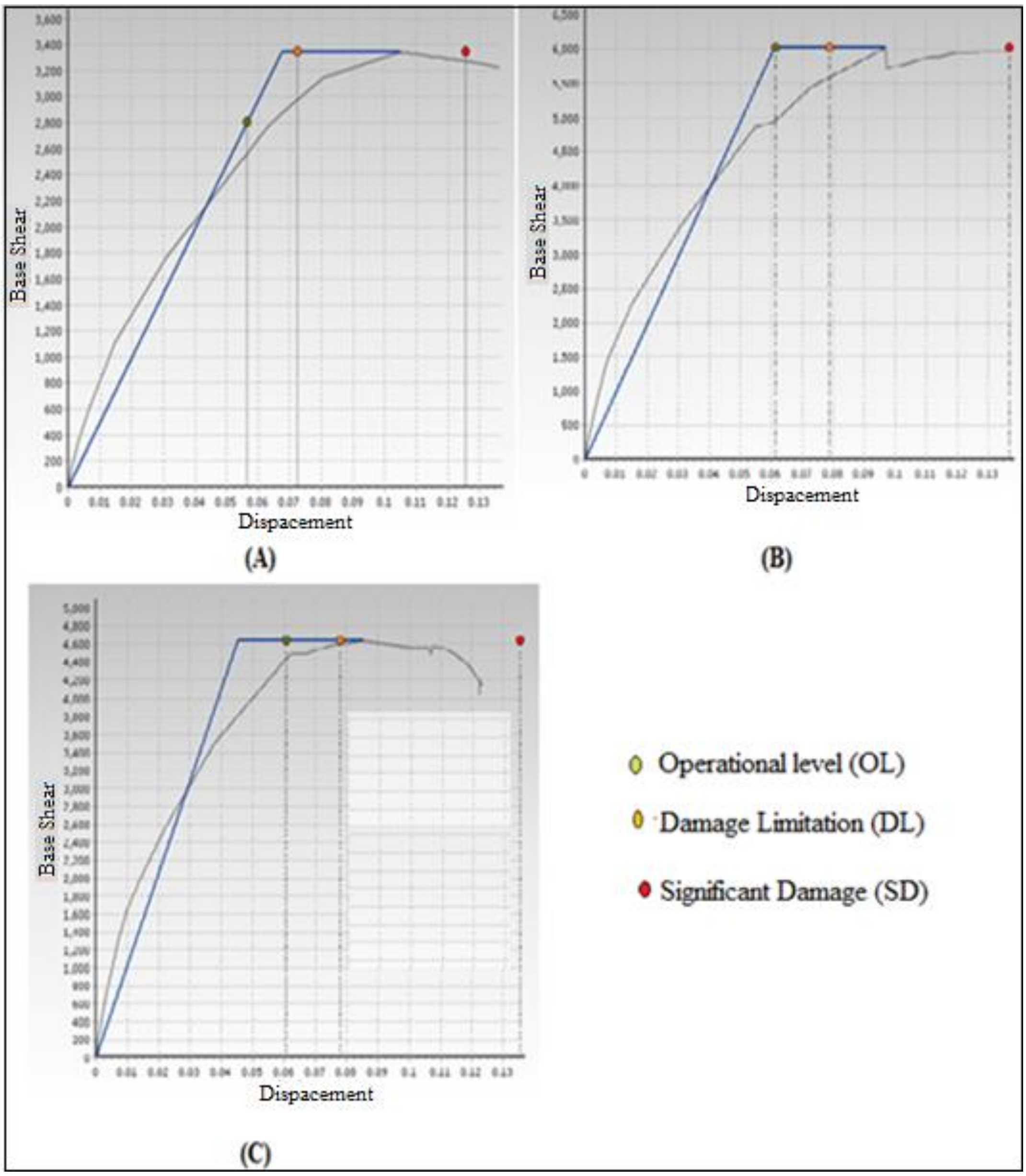

Figure 6.22: Target displacement performance level for (A) 4-Storey Bare RC Frame, (B) 4-storey RC Frame having Brick Wall and (C) 4-storey RC Frame soft storey for second case study

Table 6.2: Performance Level for Target Displacement 0f Third Case

Target displacement (m)	4-storey Bare RC Frame	4-storey RC Frame with brick wall	4-storey RC Frame with soft storey	8-storey Bare RC Frame	8-storey RC Frame with brick wall	8-storey RC Frame with soft storey	12-storey Bare RC Frame	12-storey RC Frame with brick wall	12-storey RC Frame with soft storey
Operational Level (OL)	0.0290	0.0278	0.0268	0.0578	0.0539	0.0546	0.0866	0.0751	0.0706
Damage Limitation (DL)	0.0372	0.0357	0.0344	0.0741	0.0691	0.0701	0.1111	0.0964	0.0906
Significant Damage (SD)	0.0646	0.0619	0.0597	0.1235	0.1199	0.1215	0.1926	0.1671	0.1572

Table 6.3: Performance Level for Target Displacement 0f Third Case with Shear Wall

Target displacement (m)	8-storey Bare RC Frame	8-storey RC Frame with brick wall	8-storey RC Frame with soft storey	12-storey Bare RC Frame	12-storey RC Frame with brick wall	12-storey RC Frame with soft storey
Operational level (OL)	0.0598	0.0545	0.0347	0.0835	0.0783	0.07614
Damage Limitation (DL)	0.0767	0.0700	0.0446	0.1071	0.1005	0.0976
Significant Damage (SD)	0.1331	0.1213	0.0773	0.1857	0.1742	0.1693

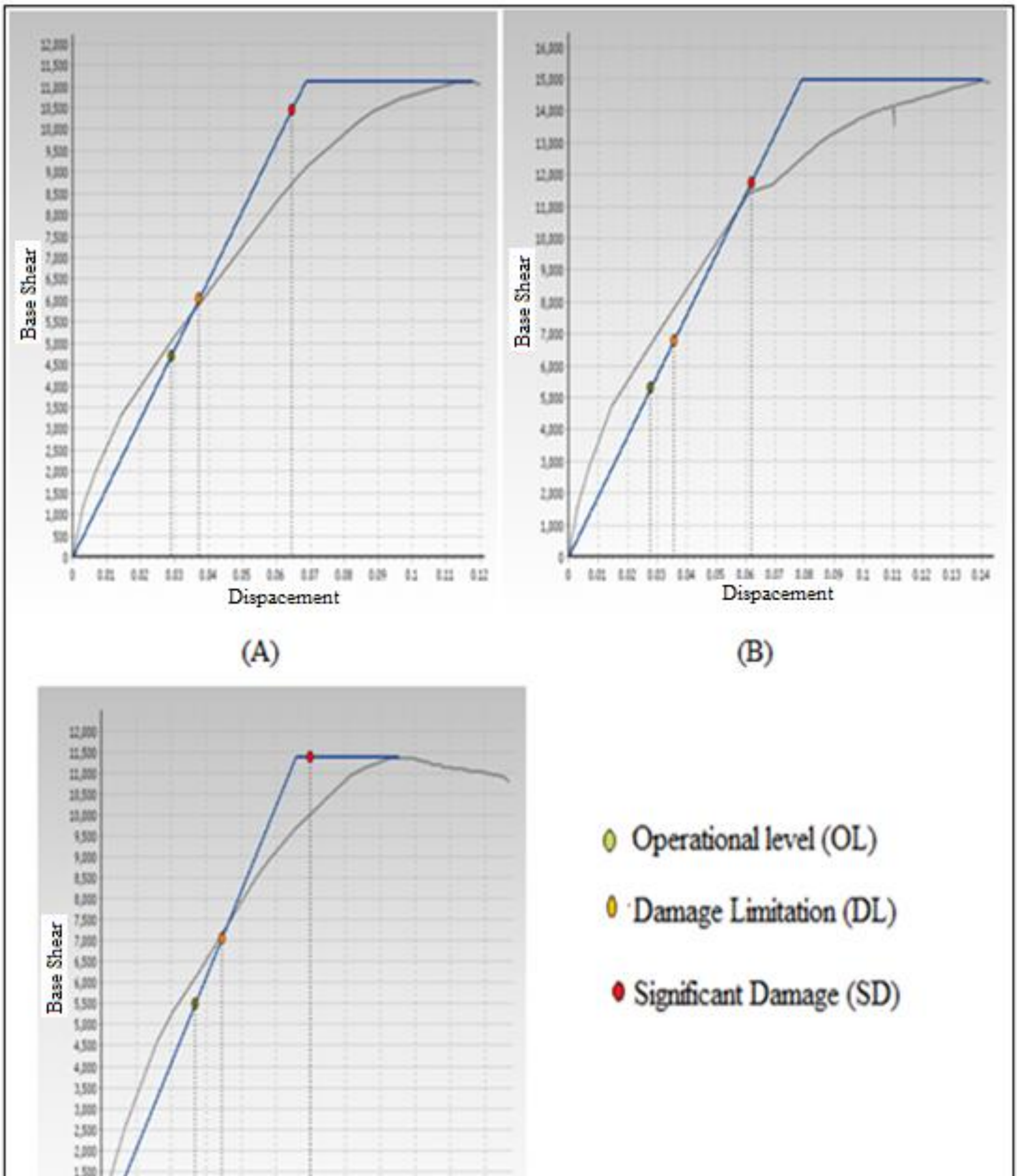

Figure 6.23: Target displacement performance level for (A) 4-Storey Bare RC Frame, (B) 4 storey RC Frame having Brick Wall and (C) 4-storey RC Frame soft storey

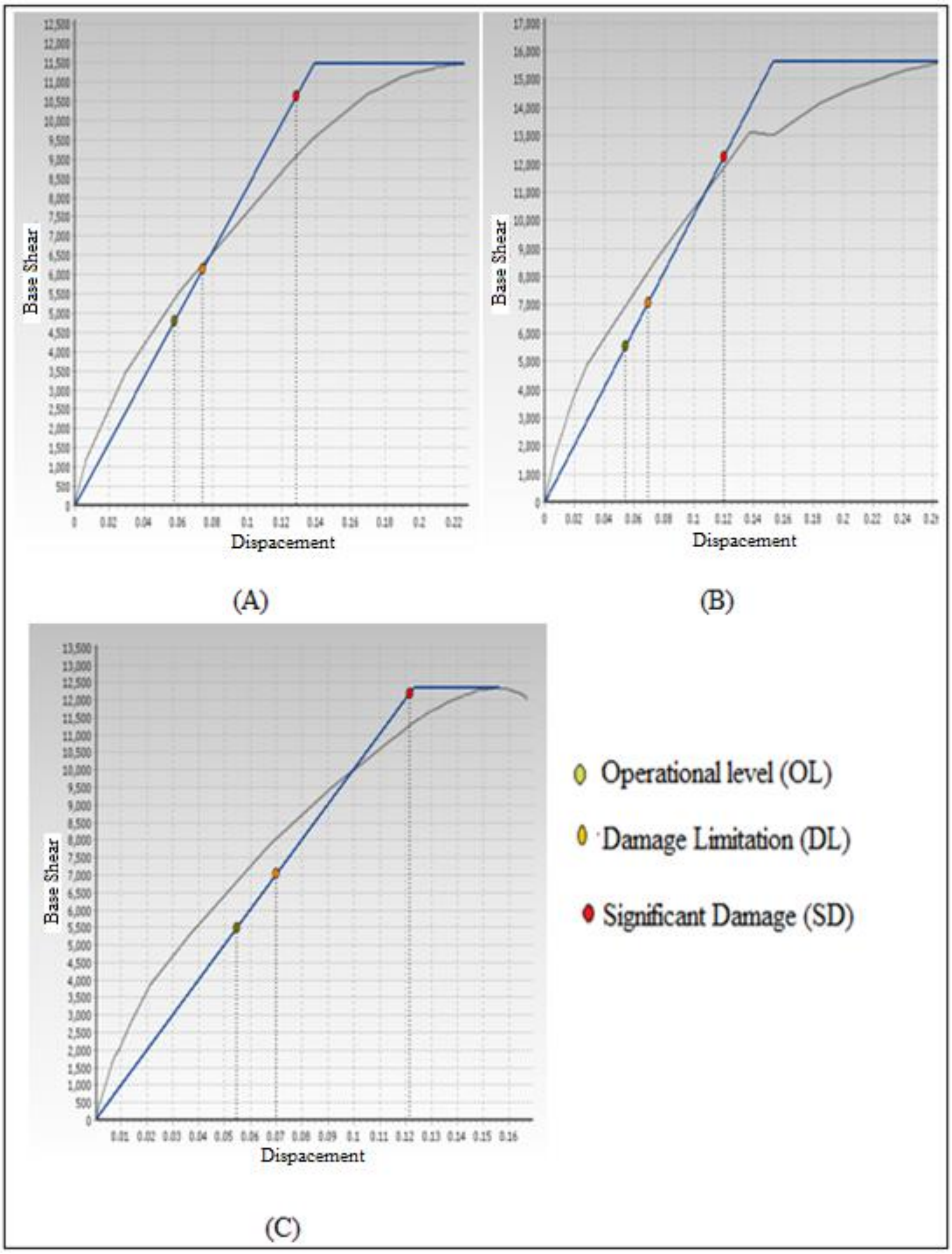

Figure 6.24: Target displacement performance level for (A) 8-Storeey Bare RC Frame, (B) 8 storey RC Frame having Brick Wall and (C) 8-storey RC Frame having soft storey

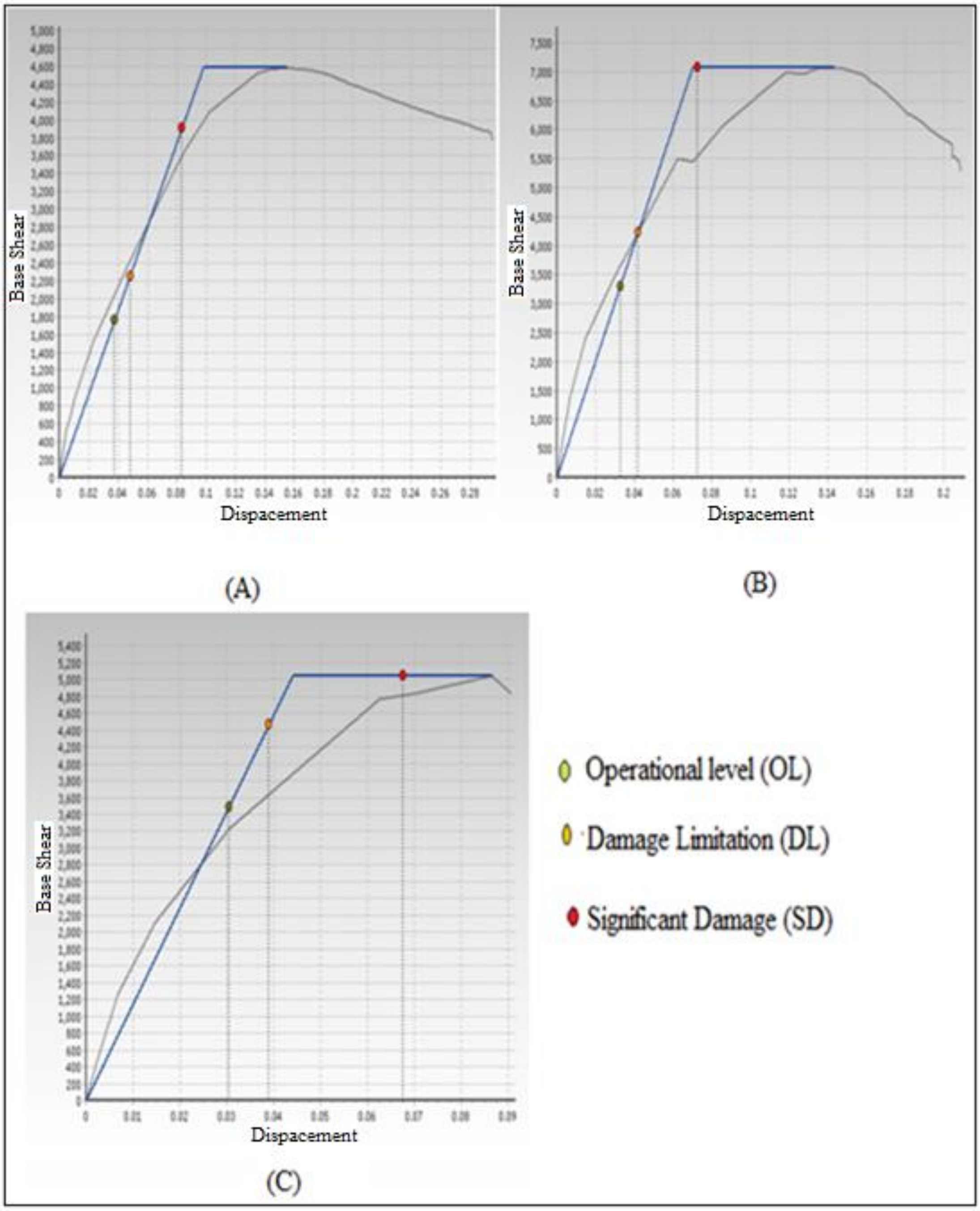

Figure 6.25: Target displacement performance level for (A) 12 Storey Bare RC Frame, (B) 12 storey RC Frame having Brick Wall and (C) 12 storey RC Frame having soft storey

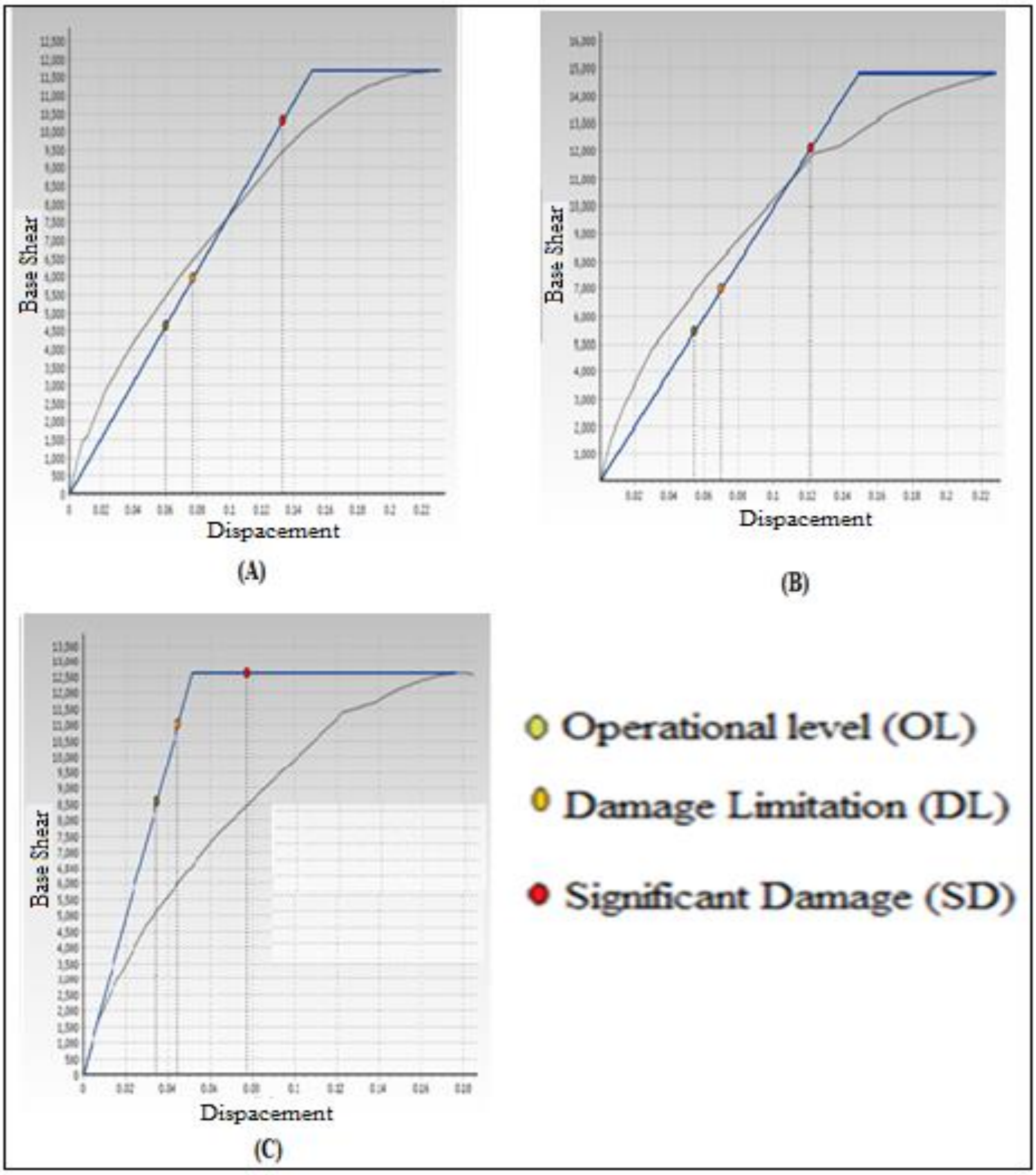

Figure 6:26: Target displacement performance level for (A) 8-Storey Bare RC Frame, (B) 8-storey RC Frame having Brick Wall and (C) 8-storey RC Frame having soft storey all having shear wall

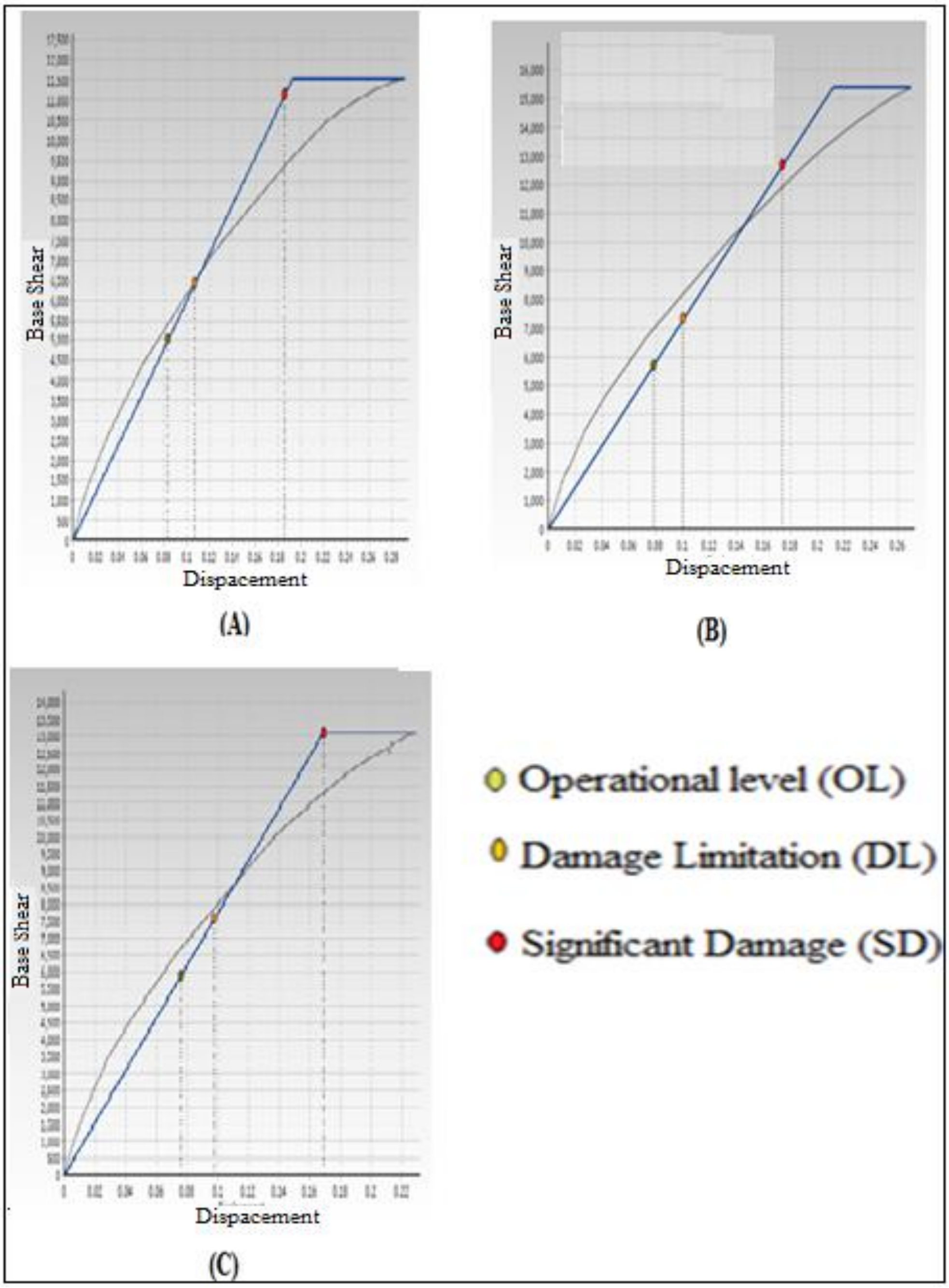

Figure 6.27: Target displacement performance level for (A) 12-Storey Bare RC Frame, (B) 12-storey RC Frame having Brick Wall and (C) 12-storey RC Frame having soft storey all having shear wall

Chapter 7

CONCLUSION AND RECOMMENDATION

7.1 Conclusions

Brick wall influence is been generally avoided at non-linear analyses of RC structures having infill panel, basically because of the multifaceted nature of infill walls analysis involved in the procedure. However, in recent investigation has shown that the consideration of brick wall in non-linear analysis shows significantly impact the, stiffness, strength and also on energy dispersal mechanism of a structure.

Comprehensive computational investigations have been on different RC frame structures with having infill brick wall to identify the behavior of brick wall on RC frame buildings. All the case studies are design in Idecad according to Turkish building codes and seismic code 2007.

In this thesis seismic analysis of RC frame models has been studied that includes bare frame, infilled frame, and soft-storey frame. From the seismic analysis of RC frames following conclusions are drawn,

1. The seismic analysis of RC frames should be done by considering the infill walls in the analysis. For modeling the infill wall the equivalent diagonal strut method can be effectively used.
2. Pushover analysis result show greater base shear for the RC frame structure having infill wall as compared to bare RC frame, even though the infill wall

having brittle failure mode. Similar results were observed in 3,4,6,8 and 12 storey building.

3. Infilled frames should be preferred in seismic regions than the open first storey frame (soft-storey), because the displacement in first storey is very large than the upper stories, this may probably cause the collapse of structure.
4. The presence of infill wall can affect the seismic behavior of frame structure to large extent, and the infill wall increases the strength and stiffness of the structure.
5. The seismic analysis of RC (Bare frame) structure leads to under estimation of base shear. The underestimation of base shear may lead to the collapse of structure during earthquake shaking. Therefore it is important to consider the infill walls in the seismic analysis of structure.

The study carried out would help understand the infilled wall behavior and identify the governing factors of the complex failure process. Effects of loading condition, material characteristics and construction practice on the response have been discussed using numerical analysis results by one set of example for each case.

The brick wall models used may be more adequate to characterize infill walls of buildings in areas with higher seismic hazard, which include seismic resistant design. Therefore the obtained results may be biased. Anyway, the results clearly indicate the importance of considering such brick walls in the dynamic analyses and in the seismic risk evaluation. They clearly modify the seismic behavior and, under the assumptions here adopted, they improve the seismic strength of the structures.

7.2 Recommendation for Future

There are still many problems which are needed to investigate in future studies. Some of the recommendation is listed below:

- . In seismic analysis of RC frame with brick wall more work is needed to refine the models and to improve the results.
- In this research the effect of opening in walls are not computed which is very important and not workable in Seismostruct.
- In this study only three kinds of frames are investigated, as a soft story at any floor of the building can be investigated.
- In this study only brick wall effects is studied, however detail analysis can be carry out by using reinforced masonry wall under considering the effects of plastering.
- RC frames with infill wall having different orientation in upper floors can be study.
- The arrangement of infill walls works should clearly place otherwise it will easily collapse.

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