MAY 2017

M.Sc. in Civil Engineering

ALAA QASIM RAHMAN

UNIVERSITY OF GAZİANTEP

GRADUATE SCHOOL OF

NATURAL & APPLIED SCIENCES

EFFECT OF INFILL WALL CONFIGURATIONS ON THE BEHAVIOUR OF REGULAR AND IRREGULAR RC FRAMES IN ELEVATION

M. Sc. THESIS

IN

CIVIL ENGINEERING

BY

ALAA QASIM RAHMAN

MAY 2017

Effect of Infill Wall Configurations on the Behaviour of Regular and Irregular RC Frames in Elevation

M.Sc. Thesis

in

Civil Engineering

University of Gaziantep

Supervisor

Assoc. Prof. Dr. Esra METE GÜNEYİSİ

by

Alaa Qasim RAHMAN

May 2017

REPUBLIC OF TURKEY
UNIVERSITY OF GAZİANTEP
GRADUATE SCHOOL OF
NATURAL & APPLIED SCIENCES
CIVIL ENGINEERING DEPARTMENT

Name of the thesis: Effect of Infill Wall Configurations on the Behaviour of Regular and Irregular RC Frames in Elevation

Name of the student: Alaa Qasim RAHMAN

Exam date: 18.05.2017

Approval of the Graduate School of Natural and Applied Sciences

Prof. Dr. A. Necmeddin YAZICI
Director

I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science.

Prof. Dr. Abdulkadir ÇEVİK
Head of Department

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

Assoc. Prof. Dr. Esra METE GÜNEYİSİ
Supervisor

Examining Committee Members:	Signature
Assoc. Prof. Dr. Kasım MERMERDAŞ	…………………
Assoc. Prof. Dr. Ali Fırat ÇABALAR	…………………
Assoc. Prof. Dr. Esra METE GÜNEYİSİ	…………………

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Alaa Qasim RAHMAN

ABSTRACT

EFFECT OF INFILL WALL CONFIGURATIONS ON THE BEHAVIOUR OF REGULAR AND IRREGULAR RC FRAMES IN ELEVATION

RAHMAN, ALAA QASIM
M.Sc. in Civil Engineering
Supervisor: Assoc. Prof. Dr. Esra METE GÜNEYİSİ
May 2017
117 pages

In this study, regular and irregular in elevation reinforced concrete frame structures with different masonry infill wall configurations were investigated. The case study structure with seven storeys and four bays was designed as regular frame model (without any setbacks) while the other four structures had the same configuration in the first storey and larger setbacks in upper ones. Thus, one regular and four irregular frame models were evaluated with and without three infill configurations, namely, partially infill and fully infill. For the partially infill, two cases were adopted as double and quadruple bay infill. In both cases, the infill was omitted at ground floor. The diagonal strut approach was assumed for modeling the infill walls. FEMA 306 guidelines were utilized to determine the width of equivalent strut. Nonlinear pushover and time history analysis, in which nonlinear behavior is assumed to observe within the frame elements at concentrated plastic hinges, was performed to determine the effects of masonry infill walls on the seismic response of different frames. The finding of the study indicated that the presence and arrangement of infill walls remarkably altered the strength and stiffness as well as the collapse mechanisms of both regular and especially irregular case study structures.

Keywords: Equivalent diagonal strut, Masonry infill wall, Pushover analysis, Time history analysis, Irregular frame.

ÖZET

YÜKSEKLİK BOYUNCA DÜZENLİ VE DÜZENSİZ BETONARME ÇERÇEVELERİN DAVRANIŞINA DOLGU DUVAR KONFİGÜRASYONLARININ ETKİSİ

RAHMAN, ALAA QASIM
Yüksek Lisans Tezi, İnşaat Mühendisliği
Danışman: Doç. Dr. Esra METE GÜNEYİSİ
May 2017
117 sayfa

Bu çalışmada, farklı dolgu duvar konfigürasyonlarına sahip düzenli ve yükseklik boyunca düzensiz betonarme çerçeve yapılar incelenmiştir. Yedi katlı dört açıklıklı vaka incelemesi yapılan yapı düzenli çerçeve modeli olarak (herhangi bir geri çekme düzensizliği olmaksızın) tasarlanırken, diğer dört yapı birinci katta aynı geometriye üst katlarda ise daha büyük geri çekme düzensizliğine sahiptir. Böylece, bir düzenli ve dört düzensiz çerçeve modeli üç dolgu duvar konfigürasyonu dikkate alınarak, kısmen dolgulu ve tamamen dolgulu olmak üzere değerlendirilmiştir. Kısmen dolgulu duvarlı durum için iki ve dört açıklık dolgulu durum düşünülmüştür. Her iki durumda da, dolgu duvarlar zemin katta bulunmamaktadır. Dolgu duvarlarının modellenmesi için diyagonal çubuk yaklaşımı varsayılmıştır. Eşdeğer çubuğun genişliğinin hesaplanmasında FEMA 306 kullanılmıştır. Plastik mafsallaşmanın çerçeve elemanlarda gözlemlendiğinin varsayıldığı doğrusal olmayan itme ve zaman tanım alanındaki analizler, dolgu duvarlarının farklı çerçevelerin sismik tepkisi üzerindeki etkilerini belirlemek için gerçekleştirilmiştir. Çalışmanın sonuçları dolgu duvarlarının varlığı ve yapı içerisindeki düzeni hem düzenli hem de özellikle düzensiz yapılarının dayanım ve rijitliği ile göçme mekanizmalarını önemli ölçüde değiştirdiğini ortaya koymuştur.

Anahtar Kelimeler: Eşdeğer diyagonal çubuk, Dolgu duvar, İtme analizi, Zaman tanım alanı analizi, Düzensiz çerçeve.

To My Parents

ACKNOWLEDGEMENT

Firstly, praises to Allah, the Almighty, on whom ultimately we depend for sustenance and guidance. I am ever grateful to Lord, the Creator and the Guardian, and to whom I owe my existence.

I would like to express my sincere thanks to my supervisor, Assoc. Prof. Dr. Esra METE GÜNEYİSİ, for all of her help, patience, valuable advices and her kind guidance. I'm very grateful and honored to work under her academic guidance.

I would like to take a chance to thank all of my friends here in Gaziantep and those who are back home at Erbil, for their continuous support, motivation and Help, they were right by my side whenever I needed them at every stage of my thesis.

Finally, I am very grateful to my parents, my brothers and my sisters, for their continues love, support and dedication, and to my wife Bala Raouf, for her Love, patience and endless support for finishing my work.

TABLE OF CONTENTS

Page

LIST OF FIGURES

Page

LIST OF TABLES

Page

LIST OF SYMBOLS/ABBREVIATIONS

ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
ATC	Applied Technology Council
CP	Collapse Prevention
FEMA	Federal Emergency Management Agency
g	Acceleration of Gravity (cm/s^2)
IO	Immediate Occupancy
IS	Indian Standard
LFEM	Introduction to Finite Element Methods
LS	Life Safety
PBEE	Performance Based Earthquake Engineering
P-M-M	Axial Force-Biaxial Moment Hinges
RC	Reinforced Concrete
UBC	Uniform Building Code
LATBSDC	Los Angeles Tall Building Structural Design Council
K_1	Initial shear stiffness of the un-cracked panel (N/mm)
K_2	Axial stiffness of the equivalent strut (N/mm)
G_W	Shear modulus of the wall (Mpa)

E_W	Young's modulus of infill (Mpa)
Fy	Yielding force corresponding to the first cracking of the panel (N)
Fm	Maximum (ultimate) force of infill (N)
Fr	Collapse(residual) force of infill (Mpa)
ftp	Tensile strength of the panel (Mpa)
Sy	Displacement at the yielding force (mm)
Sm	Displacement corresponding to the maximum force (mm)
Sr	Displacement at collapse force of infill (mm)
D	Diagonal length of infill panel (m)
bw or w	Width of diagonal strut (m)
tw	Thickness of infill (m)
Hw	Height of infill (m)
H	Height of infill (m)
Lw	Length of infill (m)
Ec	Young's modulus of the column (Mpa)
Ic	Second moment of area of the column
It	Second moment of area of the beam
θ	Slope of the infill diagonal to the horizontal

CHAPTER 1

INTRODUCTION

In frame buildings, masonry infill is used in interior and exterior walls, usually designers assume that infill walls are nonstructural and they ignore the interaction of masonry infill with around structural frame. When infilled frames are under lateral loading, the infill wall acts together with the surrounding structural frame and the impact of this action results in an increment in the strength and rigidity of the construction (Agrawal et al., 2013).

Many frame structures in the urban areas in the more of countries are reinforced concrete (RC) structures with masonry infill walls, so it is very important for construction engineer to have a knowledge about the behavior of infill wall in terms of stiffness and strength, or other characters that influences the behavior of the structures, among them, resistance of RC structures against earthquake activity must be invested.

In recent years, there have been considerable researches in the field of infill panel in the works. The seismic response of RC frame structures with infill panel has been one of the main topics to investigate among the researchers of structural engineering in attempts to develop a logical approach to design of such frame systems (Duan, 2012).

When infilled frames are subjected to lateral loads, the frame and the masonry infill operate initially similar to a monolithically system. The infills are compressed in the one diagonal direction and are stretched in the other diagonal direction. Then, they get separated from the frame in tensile corner due to the tensile forces and cracks, these cracks are named generally as boundary crack that identifies the situation of boundary linking frame and infill. Different failure types of masonry infilled frames can be observed including: crushing at the corners, sliding shear, diagonal cracking, diagonal compression, and frame failure (Gopalarathnam and Kumar, 2013). Additionally, experimental and analytical researches have shown that infill panels decrease the story drift and increase the resistance of story lateral force, while their involvement is significant to the energy dissipation capacity, always keeping in mind that if they are efficiently confined by the surrounding frame (Warashina et al., 2008).

The contact of masonry infill panel with the building frame system is frequently neglected in the non-linear analysis of structure frame because of the complexity to estimate. Such a statement may lead to a significant lack of accuracy in anticipating the structures response particularly when subjected to strong lateral loads, such as earthquakes (Perera, 2005).

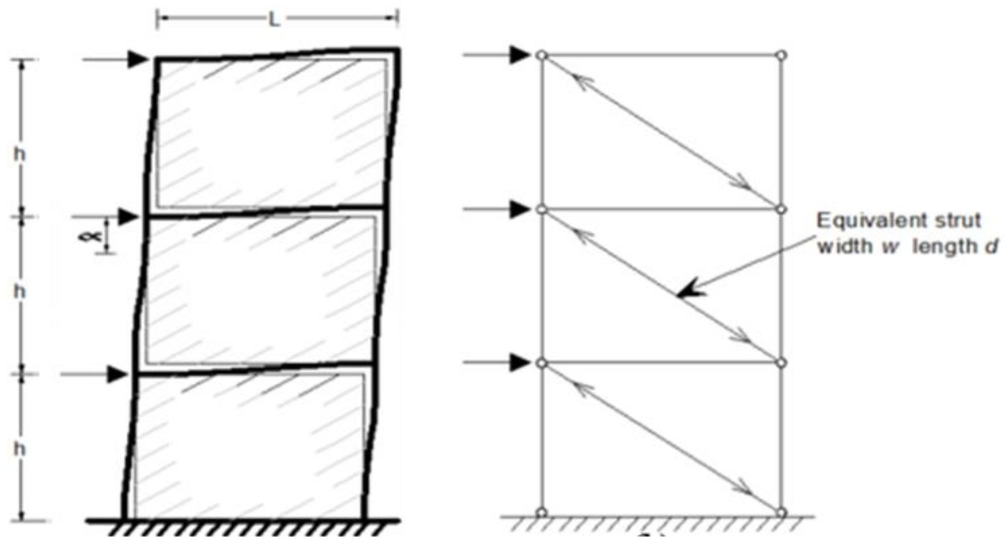
The recent seismic codes used in developing countries is disregarded the input of the infill to the lateral strength and stiffness of a structure, due to the wide variation of opening sizes and material characteristics of the infill walls and due to insufficient research information on the complex response of infill building frame structures to seismic these codes do not have adequate guidance (Dorji and Thambiratnam, 2009).

The contact with the surrounding frame is frequently ignored in the analysis of structure frame, but the lateral load capacity of the frame structure mainly increases when infill walls are considered to interact with their bounding frames system. Infill walls involvement in the transfer of loads and role of them to change the behavior of moment resisting frames has been established by decades of research (Wakchaure and Ped, 2012).

Composite structure formed by the combination of masonry infill walls and a moment resisting plane frame, the term infill frame is used, depending on the connectivity of the infill to the frame, the infill may be integral or non-integral, when integral connection is assumed in the case of buildings under consideration, the composite behavior of an infill building frame imparts lateral stiffness and strength to the building (Davis et al., 2004).

Structural frame and masonry infill separate over a large part of the length of each side when a non-integral infilled frame is subjected to a racking load, and only the end-to-end to the corners at the ends of the compression diagonal remain as regions of contact, as shown in Figure 1.1a. So, as uneven estimation, it may be assumed that the infill behaviors as a diagonal bracing strut, as shown in Figure l.1b.

To determine the diagonal stiffness and strength values of the infill, it would be appropriate to anticipate the lateral strength and stiffness of the frame infilled by assuming all the infill walls to be substitute by equivalent diagonal struts. The infill, which does not have to be an integral part with the frame is to be of an isotropic material with a brittle type of tensile failure, and the type of compressive plastic failure similar to medium strength concrete (Smith, 1967).



(a) (b)

Figure 1.1 Elevation views of (a) infill frame and (b) equivalent frame (Smith, 1967)

Masonry infill are positive aspects to the presence of higher strength and higher stiffness, in the face of this structural behavior is not desirable, and field analytical and empirical experience and research has shown that a useful influence of the infill walls to the overall seismic performance of the building especially when the latter exhibits limited engineering seismic resistance. Masonry infill panels decrease the story drift and increase the resistance story lateral force respectively through them in plane horizontal strength and stiffness. However, their contribution is significant to the global energy dissipation capacity, always under the assumption that they are effectively confined by the bounding frame (Wakchaure and Ped, 2012).

Nonlinear analyses is necessary to capture behavior of structures under earth quake, since inelastic behavior is intended in most structures subjected to infrequent earthquake loading. The employment of nonlinear static procedures (NSP) in the seismic evaluation of existing structures (or design verification of new ones) has gained considerable popularity in the recent years, backed by a large number of extensive verification studies that have demonstrated its relatively good accuracy in estimating the seismic response of buildings. Due to its simplicity, the structural engineering profession has been used NSP or pushover analysis. Modeling for such analysis requires the determination of the nonlinear properties of each component in the structure, quantified by strength and deformation capacities, which depend on the modeling assumptions. Pushover analysis is carried out for either user-defined nonlinear hinge properties or default-hinge properties, available in some programs based on the FEMA-356 and ATC-40 guidelines (Lodi and Mohammad, 2012). In the literature, several attempts have been made to model the nonlinear behavior of infill masonry wall buildings.

1.1 Objectives of the Thesis

The aim of this study is to compare the seismic behavior of the regular and irregular RC frames with and without different arrangement of infill walls. To this objective, seven storey and four bay regular frame model (without any setbacks) was designed while the other four structures had the same configuration in the first storey and larger setbacks in upper ones. Thus, one regular and four irregular frame models were evaluated by considering three infill configurations, namely, partially infill and fully infill. For the partially infill, two cases were used as double and quadruple bay infill. In both cases, the infill was omitted at ground floor. The diagonal strut approach was taken account for modeling the infill walls. The analytical models of the structures as well as the nonlinear static (pushover) and time history analysis were carried out by using SAP2000 structural analysis program. The analysis results for all frame structures were presented and discussed accordingly.

1.2 Outline of the Thesis

Chapter 1-Introduction: The brief review of the literature based on the scope of the study and the main objective of the investigation are provided in this chapter.

Chapter 2-Literature review: This chapter includes an overview of the literature about RC structures with irregularities, type and code provisions of irregularities.

The response of RC structures with masonry infill walls and some methods which have been used for modeling of the infill wall are also given in this chapter.

Chapter 3-Case study: The definition of the analytical models of the regular and irregular buildings, the method used in the analysis of the structures with infill masonry walls and the method used for modeling of infill masonry walls are presented in this chapter.

Chapter 4-Results and discussion: In this chapter, the results obtained from the nonlinear analysis of the structures with and without infill walls are mentioned and discussed.

Chapter 5-Conclusions: The conclusions built on the results or these comparative investigations were provided in this section.

CHAPTER 2
LITERATURE REVIEW

2.1 Reinforced concrete structures with irregularities

Reinforced concrete (RC) structures with irregularities are one of the major causes of damage amplification under the seismic actions. Past earthquakes, certainly, have shown that buildings with irregular configuration or asymmetrical distribution of structural properties are subjected to an increase in seismic demand causing greater damages (Athanassiadou, 2008).

Structural irregularities are normally found in structures and constructions; designer's demands are commonly the cause of such irregularities. Current earthquake codes define structural configuration as either regular or irregular in terms of size and shape of the building, arrangement of the structural and non-structural elements within the structure, distribution of mass in the building etc. A regular structure can be envisaged to have uniformly distributed (mass, stiffness, strength and structural form). When one or more of these properties is non-uniformly distributed, either individually or in combination with other properties in any direction, the structure is referred to as being irregular. Structural irregularity may occur for many reasons (Dinh, 2004).

The component of the building, which resists the seismic forces, is known as lateral force resisting system. The most common forms of these systems in a structure are special moment resisting frames, shear walls and frame-shear wall dual systems. The damage in a structure generally initiates at location of the structural weak planes present in the building systems. These weaknesses trigger further structural deterioration which leads to the structural collapse. These weaknesses often occur due to presence of the structural irregularities in stiffness, strength and mass in a building system. The structural irregularity can be broadly classified as plan and vertical irregularities (Alireza and Hamid, 2010).

2.1.1 Types of structural irregularities

The basis of irregularity in a building could be in different types and is usually called as in two main categories, namely; irregularities in plan and irregularities in elevation. The first type is due to asymmetrical stiffness, strength and mass distributions in plan, leading to a considerable increase of the torsional forces especially under seismic effects. The second type is due to variation of geometrical and/or structural characteristics of the structural members along the height of the building, commonly leading to an increase of the seismic demand in particular stories (Fragiadakis et al., 2005). Both these types of irregularities cause the development of brittle failures due to a local increase of the seismic demand in particular elements that are not always provided with sufficient strength and ductility (Penelis and Kappos, 2005).

A frame structure can be classified as vertically irregular if it contains irregular distribution of (mass, strength and stiffness) along the building height. As per IS 1893:2002, a storey in a building is said to contain mass irregularity if its mass exceeds 200% than that of the end-to-end storey. If stiffness of a storey is less than 60% of the end-to-end storey, then a storey is termed as weak storey. If stiffness of a storey is less than 70% or above as compared to the adjacent storey, then the storey is termed as soft storey (Al-Ali and Krawinkler, 1998).

In reality, many existing buildings contain irregularity, and some of them have been designed initially to be irregular to fulfill different functions e.g. basements for commercial purposes created by eliminating central columns. Also, reduction of size of beams and columns in the upper storeys to fulfill functional requirements and for other commercial purposes like storing heavy mechanical appliances etc. This difference in usage of a specific floor with respect to the adjacent floors results in irregular distributions of mass, stiffness and strength along the building height (Alba and Bento, 2005). In addition, many other buildings are accidentally rendered irregular due to variety of reasons like non-uniformity in construction practices and material used. The building can have irregular distributions of mass, strength and stiffness along plan also. In such a case it can be said that the building has a horizontal irregularity, the detailed classification of structural irregularity is presented in Figure 2.1 (Aranda, 1998).

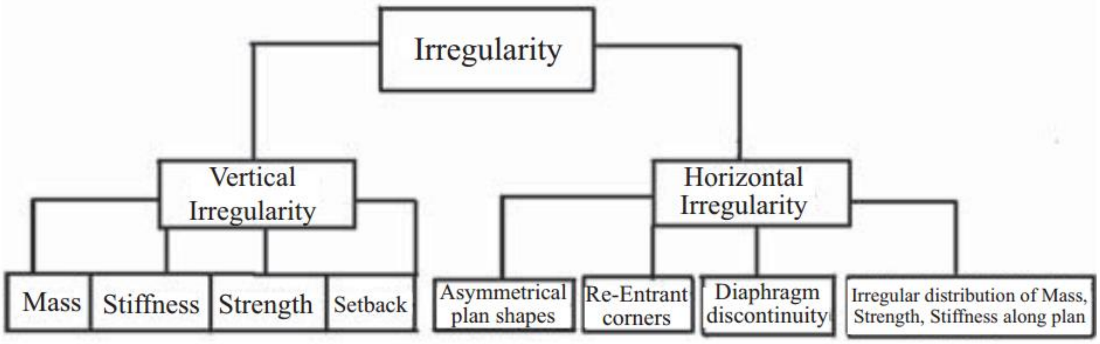


Figure 2.1 Classification of different types of structural irregularities (Aranda, 1998)

The sources of irregularity in a building configuration can be multiple and of different kinds and are usually classified in two major categories, irregularities in plan and in elevation. The first type is related to in plan asymmetrical mass, stiffness and strength distributions, causing a substantial increase of the torsional effects when the structure is subjected to lateral forces. The second one involves variation of geometrical and structural properties along the height of the building, generally leading to an increase of the seismic demand in specific storeys. Both these types of irregularity often entail the development of brittle collapse mechanisms due to a local increase of the seismic demand in specific elements that are not always provided with sufficient strength and ductility (Athanatopoulou and Avrimidis, 2008).

Most seismic codes, including EC8-1 (2004), provide empirical criteria for the classification of buildings into regular and irregular categories with reference to: mass and lateral stiffness variations in plan and in elevation (and related eccentricities), shape of the plan configuration, presence of set-backs, in plan stiffness of the floors (rigid diaphragm condition), continuity of the structural system from the foundations to the top of the building. This list is not comprehensive of all the possible causes of irregularity and there is no definition for the degree of irregularity of the overall three-dimensional system. Code definitions fail to capture some irregularities, especially those resulting from the combination of both plan and vertical irregularities. Moreover, system irregularity does not solely depend on geometrical and structural properties of the building, but can also be induced by the features of the earthquake excitation and increased by the progressive damage of the structure (Antoniou and Pinho, 2004).

2.1.2 Seismic code provisions and limitations for irregularities

The seismic is a quick release of energy in the Earth's crust that creates seismic waves. The seismic occurrence can neither be predicted nor prevented but structures can be built to resist these seismic forces using quality material and skilled workmanship. The damages caused due to the seismic forces depends on the type of materials used in the building, type of seismic waves, ground on which the structure is built. During seismic, damages to the structure start from the weakest structural member due to the presence of irregularity such as discontinuities in stiffness, diaphragm, strength or mass between adjacent stories. Considering horizontal irregularity and irregularity in elevation, main reason for the structure failure during earthquake is vertical irregularities (Guler et al., 2008).

In order to construct a seismic resistant structure, seismic codes are available that are usually revised, which includes proper design and detailing of structures against seismic. For the purpose of revising the seismic codes and also to improve the construction techniques in seismic prone region, damage patterns in reinforced concrete frames are studied thoroughly from the past seismic. To minimize the failure of structures in earthquake prone regions, the concerned authorities should develop an awareness programs related to earthquake, good construction practice methods, role of building configurations, use of good quality materials, need for skilled workmanship (Kumar et al., 2012).

A structure can be classified as irregular if the structure exceeds the limits as prescribed by different seismic design codes. The irregularity limits for both horizontal and vertical irregularities as have been discussed briefly in Table 2.1 and Table 2.2 (Varadharajan et al., 2012).

Table 2.1 Irregularity limits prescribed by IS 1893:2002, EC8:2004, UBC 97, NBCC 2005 adopted by Varadharajan et al. (2012)

Type of Irregularity	IS 1893:2002	EC8 2004	UBC 97	NBCC 2005
Horizontal				
a) Re-entrant corners	$R_i \leq 15\%$	$R_i \leq 15\%$	$R_i \leq 15\%$	-
b) Torsional irregularity	$d_{max} \leq 1.2\ d_{avg}$	$r_x > 3.33\ e_{ox}$ $r_y > 3.33\ e_{oy}$ r_x and $r_y > l_s$	$d_{max} \leq 1.2\ d_{avg}$	$d_{max} \leq 1.7\ d_{avg}$
c) Diaphragm Discontinuity	$O_a > 50\%$	$r_x^2 > l_s^2 + e_{ox}^2$	$O_d > 50\%$	-
	$S_d > 50\%$	$r_y^2 > l_s^2 + e_{oy}^2$	$S_d > 50\%$	
Elevation or Vertically				
a) Mass	$M_i < 2\ M_a$	Should not reduce abruptly	$M_i < 1.5\ M_a$	$M_i < 1.5\ M_a$
b) Stiffness	$S_i < 0.7S_i+1$ or $S_i < 0.8\ (S_i+1+S_i+2+S_i+3)$	$S_i < 0.7S_i+1$ or $S_i < 0.8(S_i+1 + S_i+2 + S_i+3)$	$S_i < 0.7S_i+1$ or $S_i < 0.8(S_i+1 + S_i+2 + S_i+3)$	$S_i < 0.7S_i+1$ or $S_i < 0.8(S_i+1 + S_i+2 + S_i+3)$
c) Soft Storey	$i < 0.7S_i+1$ or $S_i < 0.8\ (S_i+1 + S_i+2 + S_i+3)$	-	$S_i < 0.7S_i+1$ or $S_i < 0.8(S_i+1 + S_i+2 + S_i+3)$	$S_i < S_i+1$
d) Weak Storey	$S_i < 0.8S_i+1$	-	$S_i < 0.8S_i+1$	-
e) Setback irregularity	$SB_i < 1.5\ SB_a$	$Rd < 0.3T_w < 0.1T_w$ at anylevel	$SB_i < 1.3\ Sb_a$	$SB_i < 1.3\ Sb_a$

Table 2.2 Irregularity limits prescribed by IBC 2003, TEC 2007 and ASCE – 7.05 adopted by Varadharajan et al. (2012)

Type of Irregularity	IBC 2003	TEC 2007	ASCE – 7.05
Horizontal			
a) Re-entrant corners	-	$R_i \leq 20\%$	$R_i \leq 15\%$
b) Torsional irregularity	-	$d_{max} \leq 1.2\ d_{avg}$	$d_{max} \leq 1.2\ d_{avg}$
c) Diaphragm Discontinuity	-	O_a>33%	$d_{max} \leq 1.2\ d_{avg}$ $d_{max} \leq 1.4\ d_{avg}$
Elevation or Vertically			
a) Mass	M_i < 1.5 M_a	-	M_i < 1.5 M_a
b) Stiffness	S_i < 0.7S_i+1 or S_i < 0.8 (Si+1 + S_i+2 + Si+3)	-	S_i < 0.7S_i+1 or S_i < 0.8 (S_i+1 + S_i+2 + S_i+3)
c) Soft Storey	S_i < 0.7S_i+1 or S_i < 0.8 (S_i+1 + S_i+2 + S_i+3)	[ηki = (Δi / hi) avr / (Δi+1 / hi +1) avr > 2.0	S_i < 0.7S_i+1 or S_i < 0.8 (S_i+1 + S_i+2 + S_i+3)
d) Weak Storey	Si < Si+1	[ηci = (Ae)i / < 0.80]	S_i < 0.6S_i+1 or S_i < 0.7 (S_i+1 + S_i+2 + S_i+3)
e) Setback irregularity	SB_i < 1.3 Sb_a	-	SB_i < 1.3 Sb_a

2.1.3 Research works on RC structure irregular in elevation

Fernandez (1983) calculated the seismic response of multistory mass and stiffness irregular building. Results of analytical study suggested poor seismic performance of irregular structures as compared with regular structures. Moelhe (1984) resolved that the seismic response depended not only on the extent of structural irregularities but also on the location of irregularities.

Ruiz and Diederich (1989) evaluated the seismic response of structure frame with irregular distribution of strength and stiffness. The irregularity in the frame model was introduced by creating a weak story at the 1st floor in the first case. The masonry infill walls at the top story were modeled as brittle and ductile in the 2nd and 3rd cases. The results of the analytical study it was observed that factors such as failure, yielding and formation of plastic hinges in the infill walls strongly depended upon the fundamental period.

Valmundsson and Nau (1997) showed analytical studies on structure frame models with vertical irregularities. The structural irregularities were created in the frame models by irregular distribution of mass, strength and stiffness. The results of the analytical study suggested that the effects of stiffness and strength irregularity on the seismic response were more dominant as compared with mass irregularity.

Al-Ali and Krawinkler (1998) evaluated the effect of stiffness, mass, strength and their combinations on the seismic response of a ten story structure frame. For the analytical study, elastic and inelastic dynamic analyses were used. On the basis of the results of the analytical study, it was observed that, when irregularities were considered separately, the strength irregularity had maximum impact on roof displacement and the mass irregularity had minimum impact on roof displacement. When combination of irregularities was considered, the combination of stiffness and strength irregularities had maximum impact on roof displacement.

Magliulo et al. (2002) evaluated the seismic response of multistory RC frames with mass, stiffness and strength irregularity. These frames were designed for low ductility class as per EC 8 provisions. From the analytical studies, it was found that mass irregularity does not affect the plastic demands. Also, the parameter of story strength as prescribed by EC 8 and international Building (IBC) codes was found to be ineffective in predicting strength irregularity.

Das and Nau (2003) determined the inelastic seismic response of a large number of multistory structures (5–20 storeys) with strength, stiffness and mass irregularity in elevation. The irregularities in the building models were introduced by variation of the mass, stiffness and strength ratio by considering the effect of masonry infill walls. These frames were designed as special moment-resisting frames (SMRF) in accordance with different codes of practice (ACI 1999 and UBC 97). From the results of the analytical study, it was found that the combination of irregularities had maximum impact on the seismic response, and an abrupt variation in the seismic response was observed near the presence of irregularities. Lastly, it was concluded that the mass irregularity had the least influence on the structural damage index and, for all the building models analyzed, it was found to be less than 0.40. Chintanapakdee and Chopra (2004) with their analytical studies concluded that the combination of irregularities had maximum impact on the seismic response. Moreover, the presence of irregularities in the bottom stories generated maximum variation in the seismic response.

Tremblay and Poncet (2005) conducted analytical studies on building models with irregularity in elevation. These models were designed according to the NBCC provisions and analyzed by both static and dynamic methods of analysis. The results of the analytical study suggested the ineffectiveness of both methods on predicting the effects of mass irregularity.

Choi (2004) determined the seismic response of multistory building frames with irregularity in elevation. Three cases were studied with mass irregularities placed at the bottom position in the first case and at the middle and top story in the second and third cases, respectively. The seismic demands were higher for the third case when mass irregularity was placed in the top position, and hysteric energy demands were observed to be greater than drift demands.

Fragiadakis et al. (2005) with their analytical studies concluded that the location of irregularity and the intensity of an earthquake had maximum influence on the seismic response. Lignos and Gantes (2005) found modal pushover analysis procedure ineffective in predicting the collapse and failure mechanism of irregular building models.

Aydin (2007) determined the seismic behavior of mass irregular buildings by ELF (Equivalent Lateral Force procedure) procedure prescribed by the Turkish code of practice, the ELF procedure was compared with time history analysis. Results of the analytical studies indicated that the ELF procedure overestimates the seismic response as compared with the time history method of analysis.

Sadasiva et al. (2008) studied the variation in seismic response of mass irregular structures due to the change in location of mass irregularity. A nine-story regular and irregular (with irregularity in elevation) frame was analyzed and designed as per New Zealand code of practice in two ways. First, it was designed to have maximum inter-story drift at all levels (represented as CISDR). Second, it was designed to have a constant stiffness (represented as CS) at all levels. To make clear the distinction between regular and irregular structure, a special notation was used by the authors of the form NS-M-L-(A), where N is the number of stories, S is shear beam, M is type of model (i.e. S (shear beam) or SFB (shear flexure beam), (A) is mass ratio). The deformation is represented in form of graphs. For making the study, Los Angeles earthquake records had been used, and authors carried out inelastic time history analysis of the structure using Ruamoko software. With this analysis, it was concluded that in the case of both the CS and CISDR model, the inter-story drift produced is maximum when mass irregularity is present at the topmost story and irregularity increases the inter-story drift of the structure. However, this magnitude varies for both CS and CISDR type of models.

Van and Thuat (2011) determined the story strength demands of irregular buildings under strong earthquakes; the strength irregularity in the building models was introduced in terms of story strength factor, which represents the relative reserve strength of the story against failure. The author conducted a large number of analyses of building models ranging from 7 to 19 stories. The analyses results indicate the variation in seismic demands due to the introduction of irregularity.

Kim and Hong (2011) determined the collapse-resisting capacity of building models with stiffness and strength irregularity. The irregularity in the building models was created by the removal of a column in the intermediate story. However, analysis results suggested minor variation in the collapse potentials of regular and irregular structures.

2.2 RC structures with masonry infill walls

2.2.1 Motivation

Masonry infill walls are usually used as infill in reinforced concrete (RC) structures. While masonry significantly improves both the stiffness and strength of the frame, its contribution is often not considered in calculation due to poverty of knowledge of the composite conduct of the frame and the infill wall (Dhanasekar and Page, 1986).

The reasoning behind ignoring infill walls in the design process is partially the result of insufficient knowledge of the behavior of quasi-brittle materials such as unreinforced masonry (URM) and the absence of conclusive experimental and analytical results to prove a reliable design procedure for this type of structure frame. On the other hand, since of the large number of interacting parameters, if the infill wall is to be contributed in the analysis and design stages a modeling problem appeared because of the many possible failure modes that need to be evaluated with a high degree of uncertainty. This is why it is not surprising that no agreement has appeared leading to an integrated approach for the design of infill frame structures (Asteris et al., 2011).

It limited the possibility of using infills to brace tall steel or concrete framed structures until a short time ago because of insufficient information about the composite strength and stiffness of such systems. Since 1948 this topic has been the subject of some researchers that they have several separate investigations published at various institutions throughout the world (Smith and Carter, 1969). The positive effect of masonry infill panels in framed structures has been well real in research publications in the last four decades (Seah, 1999). It has been realized that it was the last for some time past that the stiffness and strength of a steel frame is extremely enhanced when it acts compositely with a masonry wall panel (Holmes and Malcolm, 1961).

Commonly, the availability of masonry infill panels enhances adequate stiffness to otherwise flexible frame systems and in an interactional manner, the enclose frames containing the brittle masonry panels thereby providing necessary ductility. After cracking, a masonry infill panel is capable of sustaining displacements and load much higher than that as compared to the masonry panel without the frame structure (Dawe and Seah, 1989).

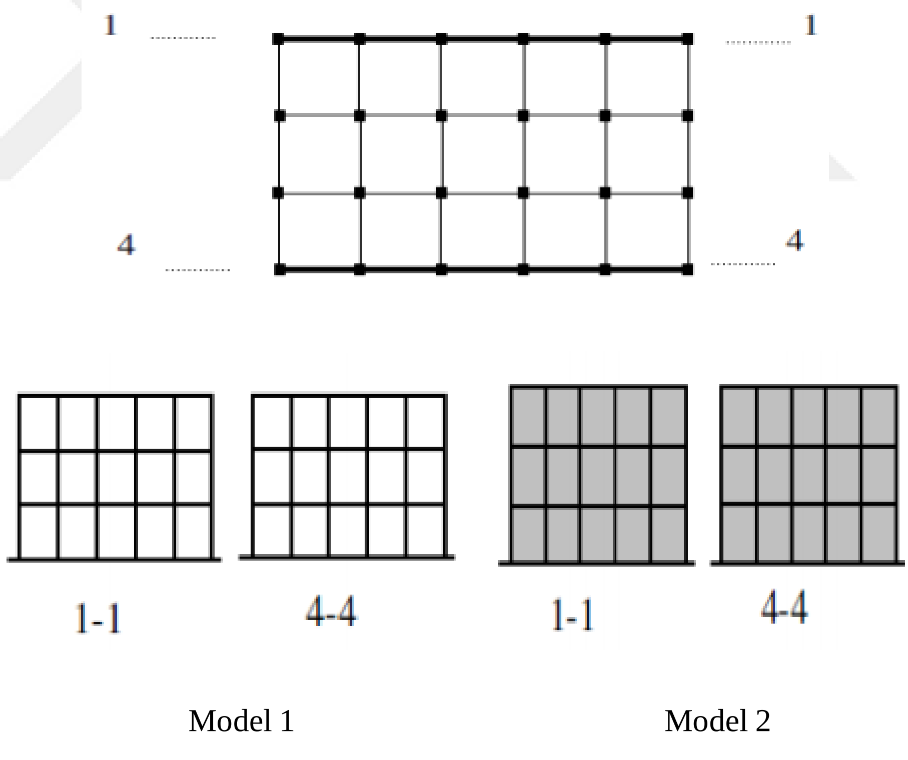
The literature shows a comprehensive variation of testing procedures, frame structure dimensions, and materials used in the study of masonry infill frames. Approaches adopted and assumptions made by various researchers have also varied widely and as a conclusion, there exists a spacious spectrum of analytical techniques for predicting the stiffness and strength of infill frame structures (Seah, 1999). The approaches to the problem have varied widely and, given and considering the different assumptions, it is not surprising that the strength and stiffness predictions have also varied widely; this was particularly true for earlier theories. More recent research studies have produced better correlation with each other and with experimental evidence (Smith and Carter, 1969).

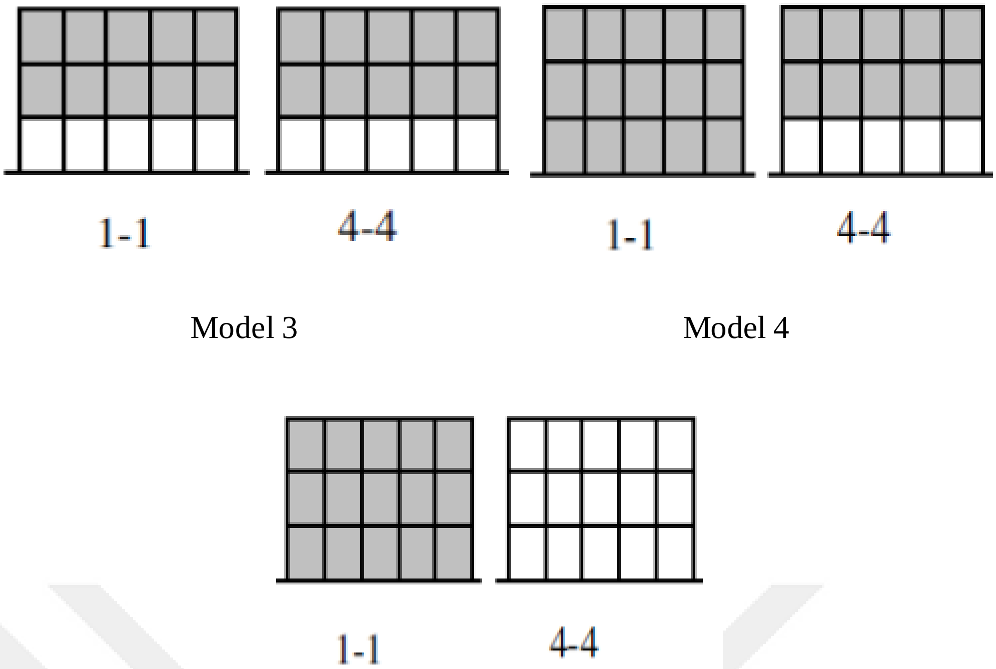
While for more than four decades' significant research has been devoted to the study of infill building frames, definitely there is no widely suitable design method for such Structures, because the panels are often considered to be inactive structurally they are rarely taken into account during the design development. This is partially explained by the complication of the interaction between the frame and the infill and the many variables that affect the behavior of such a composite structure. Significant parameters affecting strengths of the system can be divided into those which are quantifiable and those which are not. The first category includes variables such as geometry and strength of infill, relative infill to frame stiffness, plastic bending moment capacity of the frame members, strength and rigidity of the joints, beam to column location to relative stiffness, infill reinforcement, geometry and location of openings, together bays effect and the upper floors, and the frame type. The second category encompasses parameters including, among others, workmanship, climatic effects, grout and mortar variations due to working conditions, work stoppage, random variation of materials, and human error (Dukuze, 2000).

In the study of Korkmaz et al. (2007), the reinforced concrete building frame include three story with different distribution of masonry infill walls considered and observed that the effect of infill walls on seismic response. The masonry infill walls were modeled as a diagonal strut. Capacity curves were obtained for the frame structures using nonlinear analyses alternative of by using SAP2000 program. Nonlinear static analyses are realized to draw pushover curves and results are existing in comparison and the effects of irregular relationship of masonry infill wall on the performance of the structure were observed. The capacity curves, story displacements, comparative story displacements, maximum plastic rotations are determined by them. Regarding with the results of the analysis, effects of irregularities were determined in the structural behavior under seismic activities. The modeling of typical building of the applied study schematically is described in Figure

2.2 the outcome of their study showed that the reliability and stability of RC frames are improved with masonry infill walls.

Uva et al. (2012) presented a paper concerning the analysis of an existing building with reinforced concrete frame situated in a high earthquake risk region in Calabria (Southern Italy), which is constructed to the early 1970's, provide only the vertical loads were presence, an extensive experimental test operation was performed on the building so as to evaluate the condition and quality of both infill walls and reinforced concrete elements. Many non-linear static pushover analyses were performed on structural models of the building frames, considering the bare and infilled frame structure, as shown by the investigations carried out in their study, an important problem that must be accurately faced is the recognition of the mechanical properties correctly of the exact masonry infill, while the effect on the overall structural reactions is quite responsive to the variation of the parameters.





Model 3 Model 4

Model 5

Figure 2.2 Models with and without infill walls (Korkmaz et al., 2007)

Davis et al. (2004) studied the inference of the common practice of ignoring the infill stiffness with regard to performance under seismic loading. Two typical existing buildings are recognized. Features like plan irregularity and vertical irregularity (soft storey) are found in one of the buildings, while the other is moderately symmetric. Infill walls were modeled as the equivalent diagonal struts. Static analysis response spectrum analysis and nonlinear pushover analysis were performed. They observed that the seismic demand at the soft storey level is considerably large when infill stiffness is considered, with larger displacements and larger base shear. This effect, however, is not found to be significant in the symmetric building (without soft storey). Seismic performance was compared in the pushover analysis for the two cases. They concluded that the lateral load resisting of the masonry infilled structure fundamentally different from original bare frame. They observed that the existing buildings with soft ground storey are incomplete and it need of retrofitting.

2.2.2 Modeling and simulation of infill walls

2.2.2.1 Briefly historical development

In the United Kingdom, Wood (1958) conducted an experimental testing, and test results provided ample evidence that a relatively weak infill could contribute significantly to the stiffness and strength of an otherwise flexible frame structures. Although a number of finite element models have been developed to anticipate the response of frames infilled. Dhanasekar and page (1986) explained the use of finite element (micro-model) mechanisms to evaluate the contribution of the infill to the behavior of infilled frames. The influence of the properties of the masonry infill on the behavior of infilled frames is evaluated by means of a parametric study. It is shown that the behavior of the composite frame depends not only on the relative stiffness of the frame and the infill and the frame geometry, but is also critically affected by the magnitudes of the shear and tensile bond strengths relative to the compressive.

This micro-modeling procedure is to time-consuming for analysis of large structures as an alternative, a macro-model permits treatment of the entire infill panel as a single unit (Dukuze, 2000). Holmes and Malcolm (1961) suggested to replacing the infill by an equivalent pinjointed diagonal compressive strut of the same material with a width equal to one third of the infill"s diagonal length.

Smith and Carter (1969) suggested a theoretical relationship for the width of the diagonal strut connected to infill-frame stiffness parameter λh. And it is revealed that the effective width of the strut is not a constant value for a particular infill but decreases as the loading is increased.

A multi-strut model known as the (compression-only three struts model) was also investigated by Abel (1992). Modeled infill wall as shown in Figure 2.3 by six strength and stiffness degradation struts inclined compression alone, at any time during the analysis only three of the six struts are active, the struts are switched to the contrary direction whenever they reach zero forces.

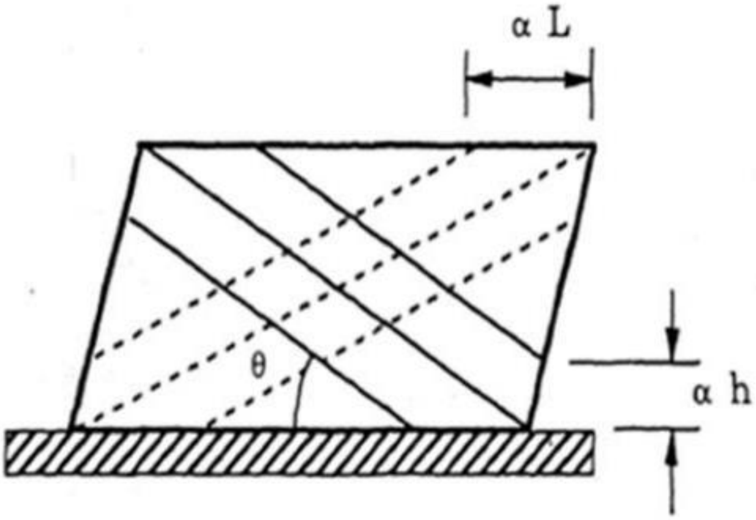


Figure 2.3 Six-strut idealization of infill walls (Abel, 1992)

Mander et al. (1993) performed cyclic pseudo dynamic tests on masonry infilled subassemblies framed. The result of the report presents the observed strength and deformation limit states as well as the characteristics of hysteretic behavior such as strength and stiffness degradation due to repeated load reversals. The report presents in a condensed form the crucial in-plane failure modes of masonry infilled frames, which include: (1) tension failure of the tension column due to overturning moments; (2) flexural or shear failure of the columns; (3) compression failure of the diagonal strut (diagonal compression); (4) diagonal tension cracking of the panel; and (5) sliding shear failure of the masonry a long horizontal mortar beds. And provided engineering formulation for capacity values corresponding to the studied of failure modes for the purpose of design. The mechanism load resisting of infilled frames was idealized as a combination of a pin-jointed truss system formed by the infill panel and a moment resisting frame system formed by the frame.

Wood (1978) presented a paper dealing with the plasticity, the composite action, and collapse design of frames with unreinforced shear panels. Four different collapse styles represented rely on observations of full-scale as well as model tests (Shear mode S; Shear rotation mode SR; Diagonal compression mode DC; Corner crushed mode CC). As shown in Figure 2.4 the first three modes were expected by combining standard plastic theory for frames and an idealized plastic yield criterion, for membranes which either are crushed at a constant yield stress or crack at zero (constant) tensile stress, with criterion plastic theory for frameworks, mode S, SR and DC can be anticipated in appropriate order of decreasing relative frame strength. The ultimate change to mode CC is anticipated but a more complicate theory is needed to speculate the extent of crushing. This failure mode relied on the associate strength of the frames to the infill described by a parameter. This parameter was defined as the ratio of the column bending strength to the compressional strength of the panel. Since the masonry walls did not exhibit a perfect plastic behavior, proposed the use of an empirical penalty factor, artificially decreasing the efficient crushing strength of the infill. Wood researched each collapse situation and simplified his results into a form appropriate for practical design.

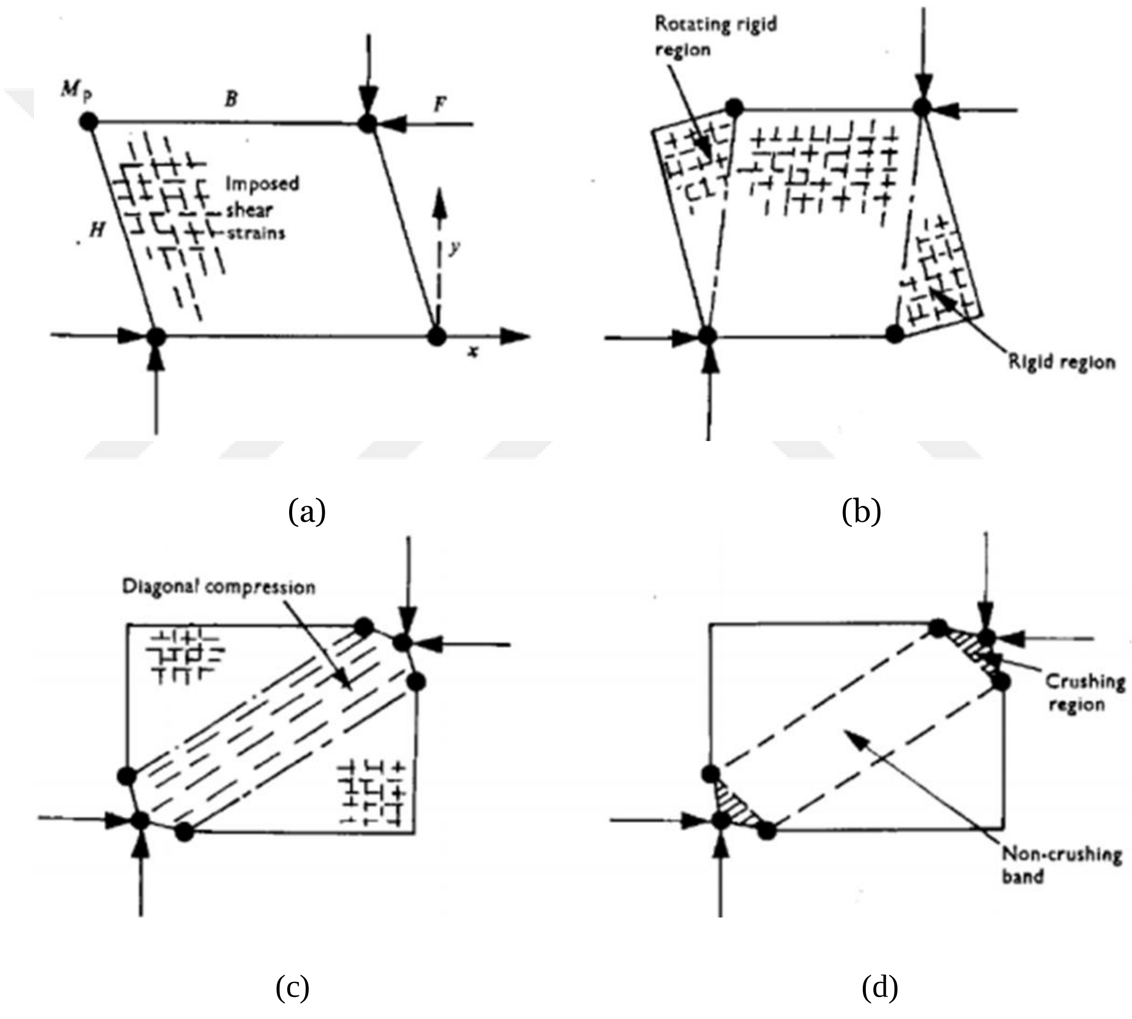


Figure 2.4 Idealized plastic failure modes for wall frame panels (Wood, 1978)

This concept is extended by Liauw and Lee (1977) to multistory, multi-bay infilled frames. Based on detected failure modes, it is illustrated for stress distribution due to the growth of cracks or crushing of the composite system at the collapse state. In addition, the shear strength at the panel-to-frame interface was taken into account which might be developed by connectors. It has been identified four patterns of failure and derived corresponding ultimate strengths. And it was defined the ultimate racking load as the minimum of the values that were obtained from all the kinematic failure modes.

Caliò and Pantò (2014) presented a macro modeling approach for the seismic evaluation of infilled frame structures. The complex nonlinear behaviour of infilled frame is analyzed according to an original approach in which the bounding frame is modeled using concentrated plasticity beam–column elements while the infills are described by plane macro-elements. This element is described by a simple mechanical system, Figure 2.5 formed by the articulated quadrilateral that four edges was rigid and connected by a hinge and a couple of non-linear springs diagonal. Each face of the quadrilateral can interact with the frame element face through a separate distribution of non-linear springs, denoted as interface. The interfaces are described by n nonlinear orthogonal springs, perpendicular to the panel side, and an additional longitudinal spring, parallel to the panel edge. In spite of its simplicity, like the basic mechanical system is capable to simulate the main in-plane failures of a masonry wall portion exposed to in-plane horizontal and vertical loads.

These failure mechanisms, namely the flexural failure, the shear-diagonal failure and shear-sliding failure as shown in Figure 2.6, also sketched the typical crack patterns. Figure 2.7 shows how the proposed plane macro-element allows a simple and realistic mechanical simulation of the corresponding failure mechanisms.

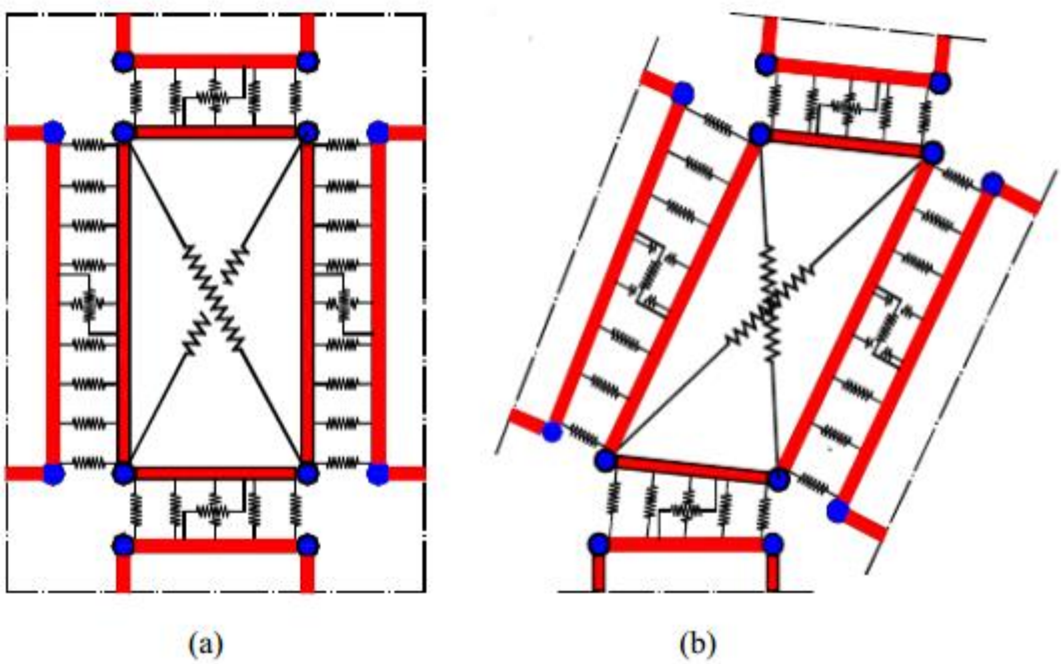


Figure 2.5 The basic macro-element for the masonry infill: (a) undeformed configuration; and (b) deformed configuration (Caliò and Pantò, 2014)

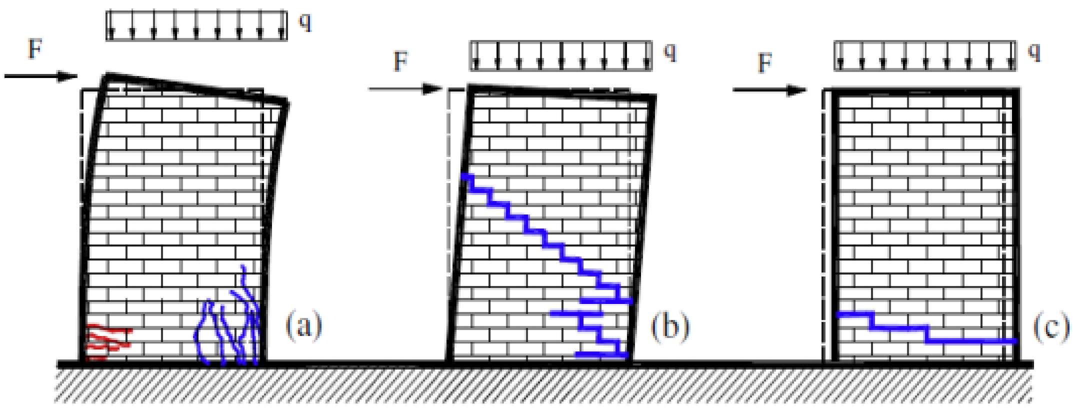


Figure 2.6 Main in-plane failure mechanisms of a masonry portion: (a) flexural failure, (b) shear-diagonal failure, and (c) shear-sliding failure (Caliò and Pantò, 2014)

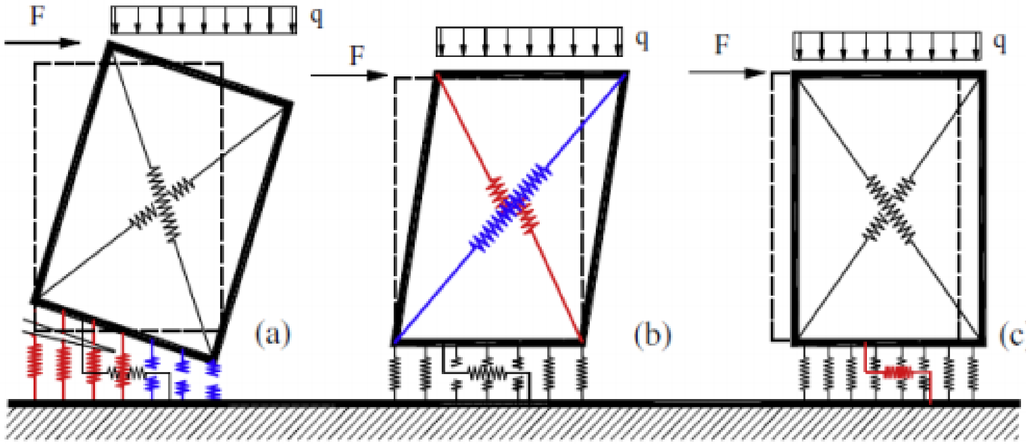
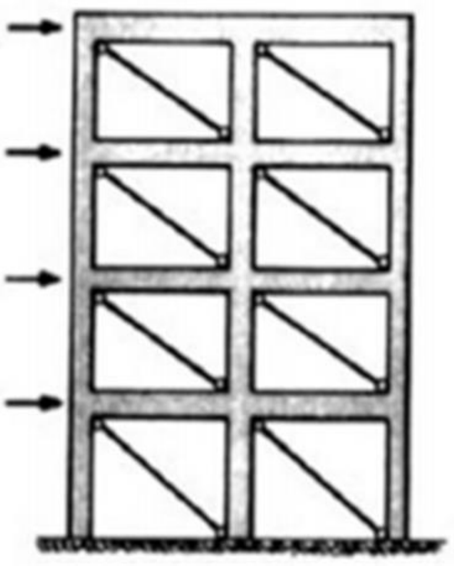


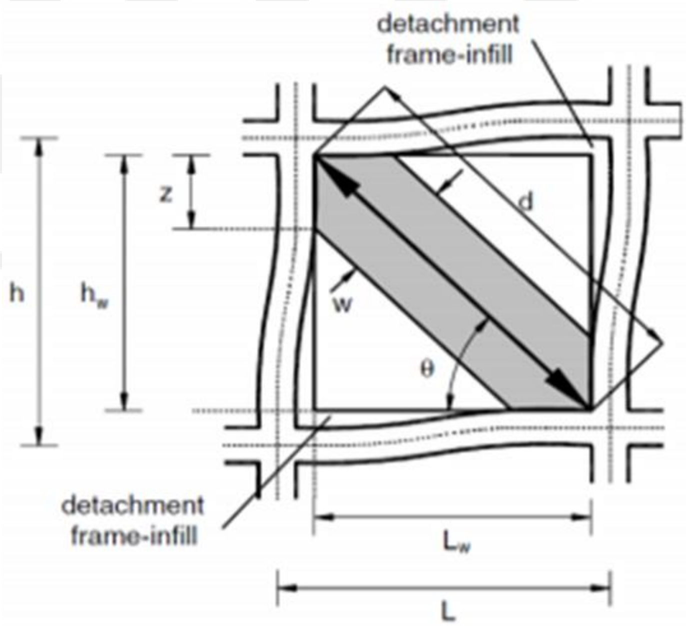
Figure 2.7 Simulation of the main in-plane failure mechanisms of a masonry portion by means of the macro-element: (a) flexural failure, (b) shear-diagonal failure, and (c) shearsliding failure (Caliò and Pantò, 2014)

2.2.2.2 Equivalent diagonal strut method

The widespread approach and a particularly effective for representing the response of RC frame building with infill walls under the action of seismic load, is the use of equivalent diagonal compression struts (Figure 2.8a). By increasing loads gradually, infill wall panels and the frame (coupled with a slip along the contact surfaces, both horizontal and vertical) is separated at the nodes (Figure 2.9b), observe that the stress in the panel is increased. Determine progressively modified the initial shear behavior of the masonry infill walls, because the axial stress in the compression corners that are still in attached with the frame becomes relevant. It is thence the infilled frame is reasonable to model as a braced system by diagonal compression equivalent struts that simulate the axial behavior of the panels (Uva et al., 2012).



(a)



(b)

Figure 2.8 the equivalent strut model (Uva et al., 2012)

The equivalent compression strut model depends on some essential factors, which would be mentioned the historical review in detail in the following (Uva et al., 2012):

- The width of the strut,
- The constitutive relationship of the panel, and

- The number of struts introduced.

Wide information can be found in the literature about the geometric properties of the infill wall panel in order to obtain the equivalence in terms of stiffness and strength. Usually assume the strut thickness same as to the masonry infill wall thickness, whereas the researchers prepare different proposals for determining the width. Fundamentally, there are two main categories. A first approaches simply take the diagonal length of the panel as a fundamental parameter to define, focus on the geometric properties as an essential than to the mechanical ones. The second approach, instead, take both the geometry and the mechanical properties of the infilled frame for defines the parameters of the equivalent strut and providing more refined numerical formulations (Uva et al., 2012).

The first researchers, who have dealt with this issue (Polyakov, 1958), as reported by Mallick and Severn (1967), based on elastic theory performed one of the first analytical studies. From his study, complemented with tests on masonry walls diagonally loaded in compression, he suggested a diagonal strut concept that could be equivalent to the effect of the masonry panels in infilled frames subjected to lateral loads as shown in Figure 2.9.

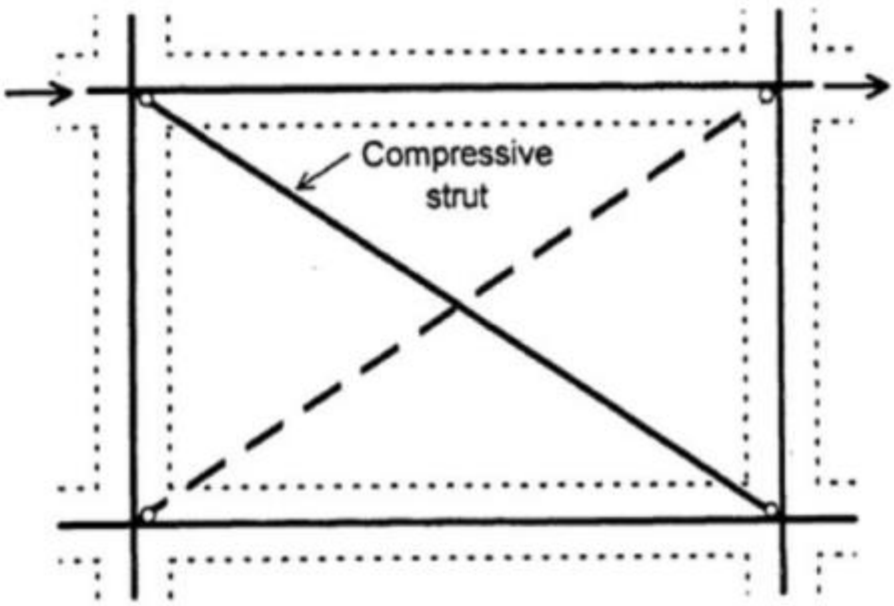


Figure 2.9 Diagonal strut model for infilled frame (Abdelkareem et al., 2013)

Holmes (1961) suggested a method for anticipating the strength and deformations of an infilled frame building based on the concept of the equivalent diagonal compression strut, by substituting the equivalent diagonal pin jointed strut instead of the infill masonry wall, which having the same thickness and made of the same material as the infill wall panel and randomly assumed the one-third of the diagonal length between the two compressed corners for the width of the equivalent strut. As shown in Figure 2.10.

$$\mathrm{w} = \frac{1}{3}\mathrm{d} \tag{2.1}$$

Where,

d = Diagonal length of infill panel

w = Depth of diagonal strut

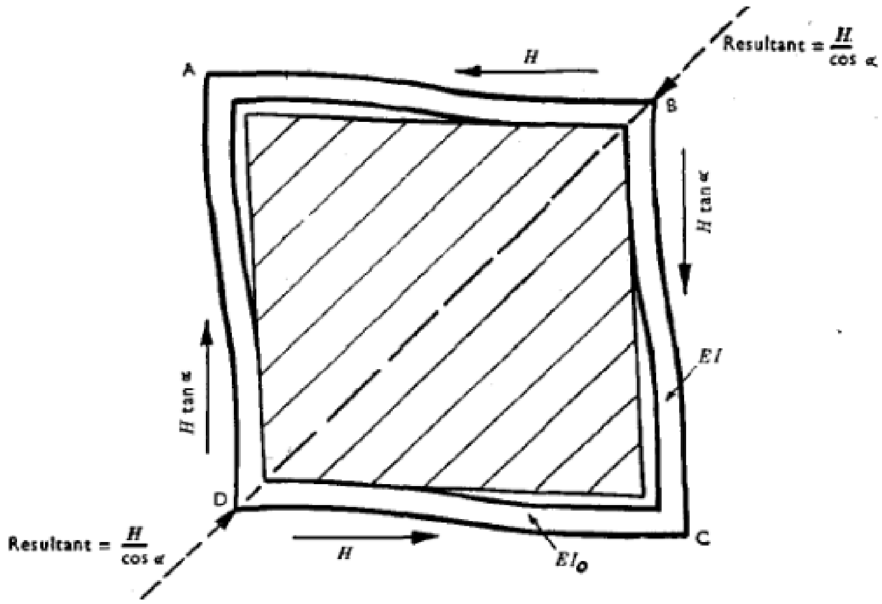


Figure 2.10 Shear force on steel frame with infill (Holmes, 1961)

Holmes (1961) also concluded that, at failure, the lateral deflection and displacement of the infilled frame building is small when compared with the deflection and displacement of the corresponding bare frame. Also, up to the failure load the frame members remained in elastic behavior. By equating the elastic deformation at failure of the frame diagonal to the shortening of the equivalent diagonal strut.

At the beginning level of in-plane force, the infill wall panel and the surrounding frame will act together as a fully composite model, as a structural wall with boundary elements. When the lateral deformation is increase, the behavior becomes more sophisticate, because the infill panel tried to deform in a shear mode while the frame tried to deform in a flexural mode, as shown in Figure 2.8(b). As a result, the infill panel and surrounded frame is separate at the two corners on the tension diagonal, and on the compression diagonal contact between infill panel and surrounded frame occurs for a length z, as shown in Figure 2.8(b).

The effective width of the diagonal Compression strut after separation W is less than that of the full panel, as shown in Figure 2.8(b). Analytical expressions have been evolved based on a beam-on elastic foundation analogy modified by the experimental results showing that the active width W of the diagonal compression strut depends on the stress-strain curves (strength) of the infill-frame materials, the level of load on infill-frame and, the relative stiffness of panel and surrounding frame. However, the stiffer structure has a high value of W, and therefore potentially have a higher seismic response, it is sensible to take a conservatively high value of; W = 0.25d. This value can be used as an approximate estimate of the elastic period of the infilled frame (Paulay and Priestley, 1992).

Later, Holmes (1963) for anticipating the behavior of infilled frames subjected to vertical and horizontal loadings prepared a semi-empirical method. Were described this method based on the tests on model steel frames with concrete infilling.

In the other cases, so as to include into account the actual variability of the infill mechanical properties, for defining the strut width besides the diagonal length d includes, the mechanical characteristics of both the masonry infill panel and the frame. Consider the cracking of the panel and the level of degradation failure it is particularly important.

The first researcher that takes the mechanical property of infill panel was Smith and Carter (1969) that by experimental study. It is revealed that the strength and stiffness of the diagonal infilling panel not only depend on physical properties and diagonal length but also depend on the contact length of infill panel with the bounding frame.

This contact length, α, is grasped by the relative stiffness of the infill panel to the surrounding frame and is given by the equation approximately:

$$\frac{\alpha}{h} = \frac{\pi}{2\gamma h} \tag{2.2}$$

λ_h is a non-dimensional parameter which expressing the relative stiffness of the infill panel to the frame, where:

$$\lambda_p = \sqrt[4]{\frac{E_w \,.\, t_w \sin(2\theta)}{4E_c I_c H_w}} \tag{2.3}$$

Considering α, to be the length of the infill panel contact with the column, and equation (2.2) is used to plot α/h as a function of λ_h, as shown in Figure 2.11.

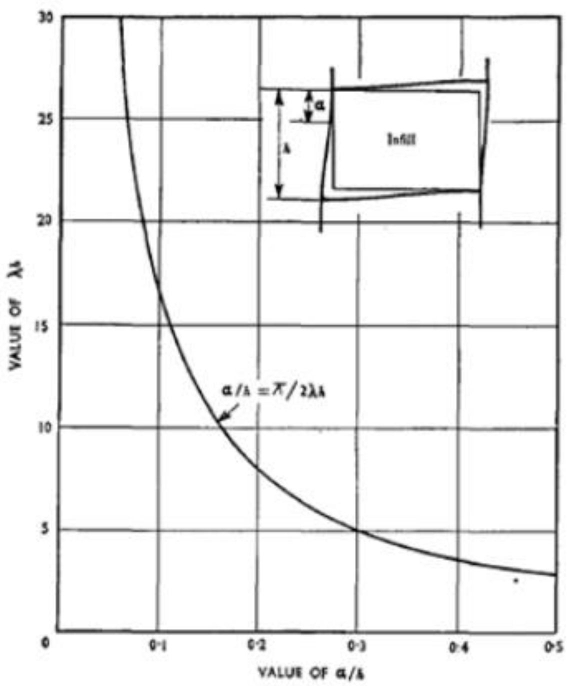
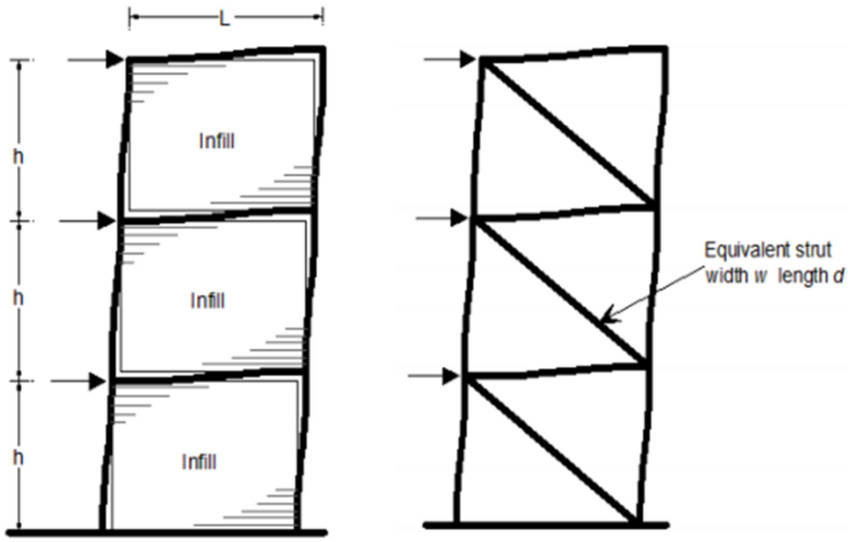


Figure 2.11 Length of contact as a function of λ_h (Smith and Carter, 1969)

They are replacing the infill walls to equivalent compression struts. As shown in Figure 2.12. The equivalent strut width is:

$$w = 0.58 \left(\frac{1}{H_w}\right)^{-0.445} (\lambda_h H)^{0.335 d_z \left(\frac{1}{H_w}\right)^{0.064}} \tag{2.4}$$



(a) (b)

Figure 2.12 view of (a) laterally loaded infilled frame and (b) equivalent frame (Smith and Carter, 1969)

Mainstone and Weeks (1972) considers the relative flexibility of infill panel to frame for appraisal of the equivalent strut width of the infill panel. Prepared an empirical relationship based on analytical and experimental data, for calculation the width of the equivalent compression strut:

$$b_w = 0.175 d_{inf} (\lambda_h H)^{-0.4} \tag{2.5}$$

Klingner and Bertero (1976), executing a laboratory test on models by scale (1:3) that composed infill panels with reinforced concrete frames, were infill panels made by hollow concrete brick masonry. They proposed a relationship for the equivalent strut width, as a function of the stiffness parameter λ, as the following expression:

$$b_w = 0.175 d_{inf} (\lambda_h H)^{-0.4} \tag{2.6}$$

Later Durrani and Luo (1994) modified the equation (2.7) on the basis of detailed numerical finite element method analyses, proposing a relationship in which the geometry of the frame is introduced by means of the coefficient parameter m:

$$\gamma = 0.32\sqrt{\sin 2\theta}\left(\frac{E_w t_w H^4}{m I_c E_c H_w}\right)^{-0.1} \tag{2.7}$$

Where m is given by;

$$m = 6\left(1 + \frac{6E_cI_bH}{\pi E_cI_cL}\right) \tag{2.8}$$

$$b_w = \gamma\sqrt{L^2 + H^2}\sin 2\theta \tag{2.9}$$

According to Abdul-Kadir (1974), actually the width of the equivalent strut not only affected by the adjacent columns, but also influenced by the top beam, and dividing the character λ, into two different factors and, which are correlated to the adjacent columns and to the upper beams, respectively. In particular, the parameter of λ_T, is determined by equation (2.10) as well, while is proposed by following expression:

$$\lambda_T = \sqrt[4]{\frac{E_w \,.\, t_w \sin(2\theta)}{4E_cI_TH_w}} \tag{2.10}$$

The width as shown in Figure 2.13, of the equivalent diagonal strut, as a function of the above defined parameters, is defining by following equation:

$$b_w = \frac{\pi}{2}\sqrt{\left(\frac{1}{\lambda_T}\right)^2 + \left(\frac{1}{2\lambda_p}\right)^2} \tag{2.11}$$

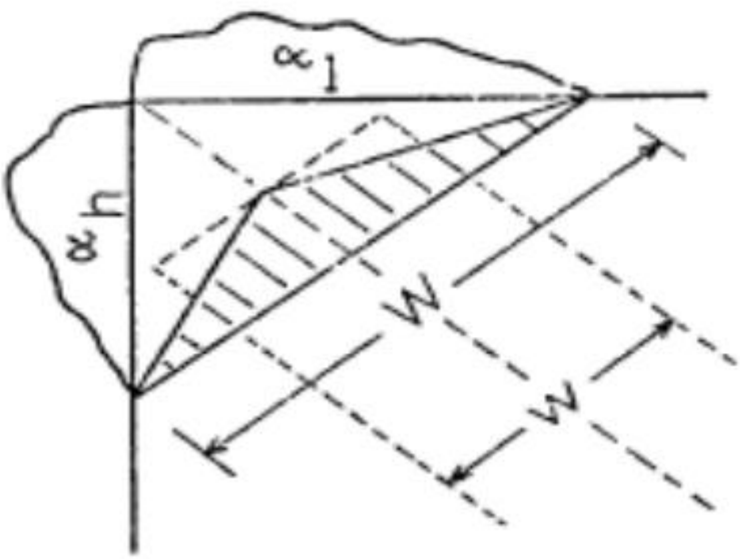


Figure 2.13 Width of the equivalent strut (Abdul-Kadir, 1974)

FEMA (1998) suggested that the equivalent diagonal strut is represented by the diagonal length (d) and the actual infill panel thickness that is in contact with the frame (t_w) and the equivalent strut width, is defining by following expression:

$$b_w = 0.175 d_{inf} (\lambda_h H)^{-0.4} \tag{2.12}$$

According to Al-Chaar (2002) the frame members and the equivalent diagonal masonry strut is to be connected by this form as shown in Figure 2.14, assumed that the column is mainly resisted the infill masonry forces, and are placed the struts according to this idea. The column and the strut are connected at a distance from the face of the beam and should be pin connected.

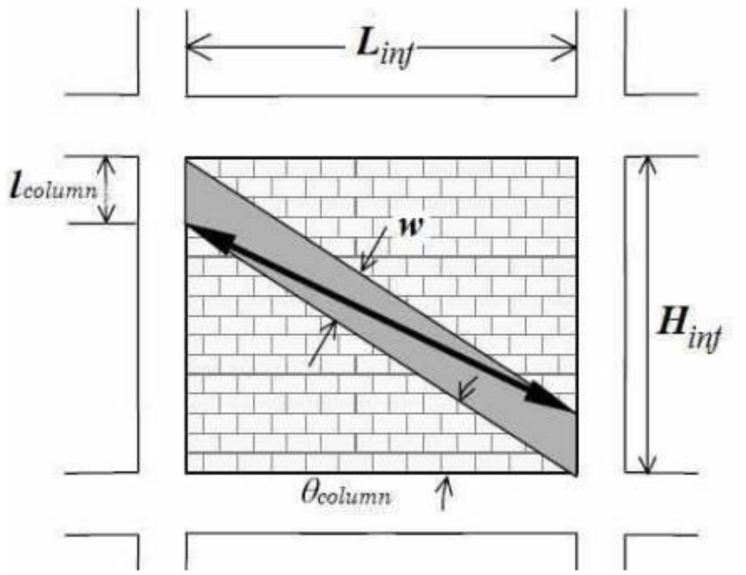
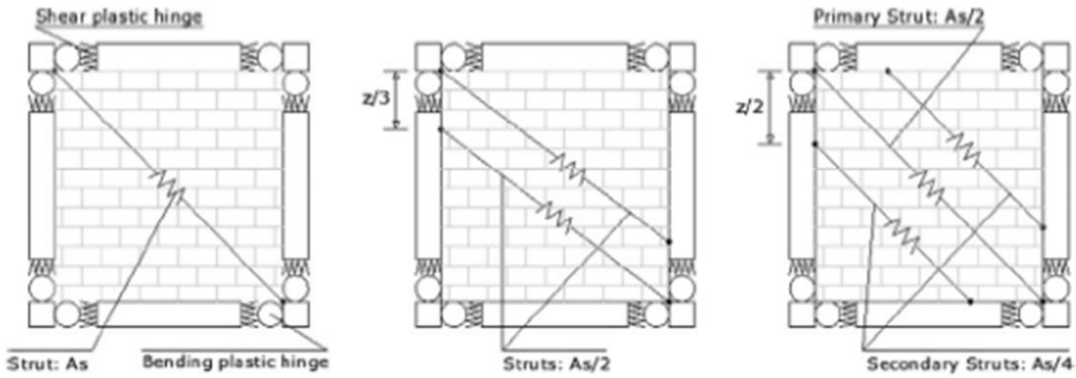


Figure 2.14 Placement of strut (Al-Chaar, 2002)

In a comparable study, Samoilă (2012) assessed different relationship that available in literature for determining the width of the equivalent compressed strut, and recommends the use of (Paulay and Priestley, 1992) relation. Take several models and study the effect of infill panel on frame members, as finite element models, the three-strut model and, the single-strut model. Revealed that; in the analysis related to the general behavior of infilled frames the single-strut model is the appropriate approach, because it can be accepted as correct idea and due to its simplicity, while for determining the local effects of frame infill interaction it is better to be used the three-strut model.

In fact, the main critical problem in the use of macro-models consists in the difficulty of obtaining the mechanical properties of the panels and the geometrical features of the equivalent diagonal struts with an acceptable level of accuracy (Uva and Fiore, 2012).

Crisafulli (1997) Investigate the restriction of the single equivalent strut model, as the rational simplest representation which is used for the analysis of infilled frame. Furthermore, investigate the structural response of the infilled frame when the infill panel is represented by different multi-strut model, with particular interest in the strength increased in the bounding frame and in induced the stiffness of the frame structure. As shown in Figure 2.16. The total width of the equivalent masonry struts was the same in all the cases. In three-strut model Figure 2.15c, it was assumed that the width of the off-diagonal strut was half of that corresponding to the central struts.



(a) (b) (c)

Figure 2.15 Equivalent model with: (a) single strut, (b) two struts, and (c) three strut, (Uva et al., 2012)

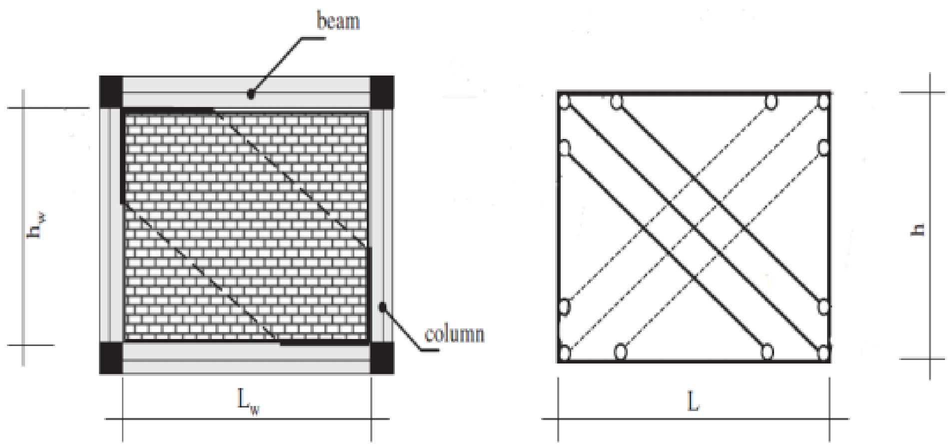


Figure 2.16 Three-strut macro-model adopted for infill panel (Verderame et al., 2011)

According to Uva et al. (2012), with respect to the specific model that used multiple struts model for infill wall panel, but, it was not appropriate to prefer one and indicated as better than from the other. In fact, a great level of variability has been noticed between the different formulations that have been tested, as an objective unquestionable indication better one cannot be provided, because so far cannot find experimental comparison for explaining the real structural behaviour. The realistic assessment of the mechanical parameters of masonry that used in introduced the strut models is paramount, and is a significant aspect to be considered. Often, literature values are easily assigned, while a great sensitivity of the results only related to this various mechanical parameter. Instead, it will be enough to collect the basic mechanical information for infill walls to achieve a significant improvement.

2.2.2.3 Numerical methods

In recent years, finite element methods had been used successfully in solving various engineering problems. dhanasekar and page (1986) investigate the effect of brick masonry infill properties on the behaviour of infilled frame buildings, simulate the behavior of infilled frames subjected to racking loads by using a finite element method. Gradual advancement of knowledge in the technique of finite element in structural analysis has significant effect on many researchers to use this method for studying the complicated composite behaviour of infill masonry panel with surrounding frame. Among these researchers were (Mallick and Severn, 1967; King and Pandey, 1978; Liauw et al 1983; Dhanasekar and Kleeman, 1985). Numerous difficulties that have appeared in the composite behavior of infilled frame can be solved by using this type of analysis. Among these are separation between frame and infill under the effect of lateral loading, bond strength, interface friction, geometry and the effects of any components that tend to tie the infill and frame together.

Mallick and Severn (1967) defined an iterative method of performance which makes use of a finite element concept whereby the points of separation between the infill panel and frame, and also the stress that distributed along the contact length between the infill panel and frame, which achieved as an integral part to solving problem. It also taken slip between the frame and infill into account. Were for modelling the frame they used Standard beam elements while for the infill wall panel they used plane stress rectangular elements. By initially assuming that frame nodes and infill have the same displacement the contact problem have been solved. The nodal displacements have been determined, and find the load along the surrounding of the infill walls and checked for tension. When a tension force is founded assume separation had occurred and allowed to move the corresponding nodes on the frame and infill wall independently in the next iteration. This procedure is continuous repeated until a pre-described tolerance for convergence is achieved. Consider the effect of interface friction and slip by defining shear forces along the contact length. In spite of this, they ignored the axial deformation of columns in their formulation. Barua and Mallick (1977) used finite elements to analyses infill frame buildings and used this method for determine boundary nodes stiffness coefficients of infill panel. And permitted the separation between infill and surrounding frame and included the effect of slip. they analysis infilled frame depend on the technique of matching displacements between infill panel and the bounding frame at a finite number of nodes, assuming infill wall is a homogeneous, an isotropic and elastic material, were this infill panel tightly fitted inside the surrounding frame, when the shear force in the contact area is equal to or greater than ε times the normal force the slip had occurred.

King and Pandey (1978) depend on the finite element technics define a method for analyze infilled framed structures. Particularly, revealed that; interface surface between infill panels and frames can be modeled with cleverness and rigorousness using modified friction elements. Where accounts an element of the gap for the separate and contact surface and, in the same time, also assesses the friction between the infill panel and bounding frame.

Liauw et al (1983) analyzed a typical model structures and compared with experimental results to show the accuracy and versatility and non-linear analysis of structures. In order to predict failure and investigate a full range of load deflection behavior, cracking, crushing, and non-linear stress-strain behavior of the infill and yielding of the frame are all taken into account, to incorporate nonlinear stress-strain behaviour of infill and frame.

Dhanasekar and Kleeman (1985) describes the development of a criterion for the inplane failure of brick masonry which can be used in conjunction with the finite element method to predict failure in local regions of a wall and thus allow the more realistic analysis of masonry shear walls, infill frames, composite wall-beam action and so on. Micro-modelling is a more complex method based on dividing the masonry panel and the concrete frame into several elements. This modelling can provide an accurate computational representation of both material and geometrical aspects, but is too time-consuming to be used in large and practical-oriented analysis (Samoilă, 2012). Koutromanos et al. (2011) successfully simulated a three-story, two-bay, frame by a finite element model, tested the inelastic seismic response of this frame on a shake table in terms of the response time histories, base shear vs. first story drift hysteresis curves, and failure mechanism. Explain the ability of nonlinear finite element models to capture the response of masonry infilled reinforced concrete frames under cyclic loading. Investigate the numerical results revealed that; the inelastic behavior of the infilled frames is not very sensitive to the lateral loading history but a monotonically increasing load could lead to higher peak strength than fully reversed load cycles. Furthermore, additional analyses have been conducted to demonstrate that even though infill walls are considered as nonstructural elements in design, they can contribute to the seismic resistance of a structure significantly.

2.2.3 Effect of infill wall on strength and stiffness of structures

Because the infill wall in tall buildings would lead to sophisticates in evaluating the behaviour of the frame structures, according to Wood (1958) the composite action of infill walls in stiffening tall buildings is neglected. He suggested the use of the collapse design methods for design the frame structures which would take into account the contribution of infilling panels. It seems that while it is essential to put the provisions of a possible loss of carrying capacity of frames due to instability, and general conduct practical frames prefer to “collapse” design, especially if it is some simple, in this modest meanwhile, the contribution from composite action can be devised. For anticipating the strength of a frame with the wall as a diagonal brace, Holmes (1961, 1963) developed a semi-empirical method. His tests were performed on one sixth scale models of steel frame structures infilled either with brickwork panels or reinforced concrete walls. He recommended one-third of the length of the diagonal infill be taken as the width of the diagonal strut. To predict the ultimate strength of his samples he assumed that the infill failed when it reached a predefined average strain along the compression diagonal. Then, it could be determined the strength of the composite system as the sum of the frame strength and that of strut. He pointed out that display diagonal strut be taken as a third of the length of the diagonal infill. By predict the ultimate strength of the samples, he is supposed to infill failure when the average strain was already pressure along the diagonal.

The mechanical properties of the diagonal strut it was very significant to determine. Based on the analogy of a beam on an elastic foundation, Smith (1967) developed the theory of equivalent strut width. The width of the equivalent diagonal strut relied on different parameters such as the beam and column moments of inertia, the infill and frame modulus of elasticity, and the lengths of contact between infill and frame structures. The relationship between the stiffness of the frame and the panel was used to determine the width of the equivalent strut.

The strength and the stiffness of the infill walls were then included in design charts as functions of the relative stiffness of the column and infill. In addition to relative stiffness of the composite system, the panel aspect ratio was found to importantly affect the stiffness and the strength of infilled frames. Since most of the tests were initially conducted on square specimens to incorporate rectangular frames, were later updated these design charts by Smith and Carter (1969). They also pointed out that unlike the constant value suggested by Holmes (1961), the strut equivalent width actually varied with the level of loading.

Dawe and Seah (1989) reported that for ultimate capacity the contribution of joint reinforcement was unimportant. By packing mortar between the panel and the frame improving the interface mechanical properties or by using mechanical ties had small effect on major crack levels or the ultimate load of infilled frame structures. Sattar and Liel (2010) researched to evaluate the seismic performance of masonry infilled RC frames, including a building with configurations of various infill frames. An increase in energy dissipation in the infilled frame, compared to the bare frame, initial strength, and stiffness, appears in the results of pushover analysis in spite of the walls brittle failure pattern. Likewise, dynamic analysis outcomes indicate that the bare frames are found to be the most vulnerable to earthquake-induced collapse and fully-infilled frame has the lowest collapse risk. The better collapse performance of fully-infilled frames is associated with the larger strength and energy dissipation of the system, associated with the added wall.

The seismic response and failure mechanisms of infilled reinforced concrete frame structures investigated by Yuen and Kuang (2015). It was revealed that serious stress and damage localization incurred by irregular arrangements of infill walls. The damage of the unbraced soft-storey columns can be seen by abrupt changes in the vertical stiffness of the infilled frame with a soft first storey, and cause to reducing the vertical load carrying capacity. As reflected by the modeling, after shear and/or flexural damage the axial-load carrying capacity of the columns can be significantly degraded.

Agrawal and Raut (2013) attempts to detect the performance of infilled masonry reinforced concrete frame buildings. It is observed that, the deflection has decreased drastically if the effect of masonry infill wall is considered. And deflection is very small in case of infill frame with opening as compare to that of bare frame. And also because earthquake force acting on last story more effectively the deflection at it is greater.

Korkmaz and Sivri (2007) investigates the effect of infill walls on the earthquake response of frame structure with different amount of masonry infill. It is found that infill walls increase strength and stiffness of frames and they have significant effect on the seismic response of the structural system.

Davis et al. (2004) illustrated the influence of masonry infill wall on the response of multi-story frame building under earthquake loading. By the presence of infill masonry panels, a change in the structural force distribution is point out. The total story shear force increases greatly as the strength and the stiffness of the building frame increases in the presence of infill masonry.

The presence of an infill masonry strong wall, endowed with high stiffness and strength, and dramatically alters the structural response of the horizontal action. In fact, in addition to the expected changes in terms of resistance to shear V and ductility μ, a considerable change of the collapse mechanisms is caused, for both low-intensity and high-intensity earthquake loads. It was noticed that, the infill masonry walls significantly interact with the frame building primary elements, by increasing horizontal loads, participating to the distribution of forces and determining an advance failure. In other words, the infill‟s masonry walls, in this situation, have a truly primary behavior, which is a typical circumstance in existing structures designed to vertical actions (Uva and Fiore, 2012).

Dolšek and Fajfar (2008) studied the effect of masonry infill walls on the seismic behavior of a RC frame. By using the N2 method, were it based on pushover analysis and the inelastic spectrum approach, it was recently extended in order to make it applicable to infilled reinforced concrete frames. This method is summarized and applied to the deterministic seismic assessment of a four-storey reinforced concrete frame with masonry infill walls, with openings and without them. Use the compressive diagonal strut techniques for modelling of the masonry infill walls. The results of the analyses indicate that; the infill walls can completely change the distribution of damage throughout the structure. The infill walls can have a beneficial effect on the structural response, provided that they are placed regularly throughout the structure, and that they do not cause shear failures of columns.

Fiore et al, (2012) Performed a finite element analyses, in order to evaluate the local effects on the frame and underline the influence of the coefficient of friction at the infill–frame interface. Successively a suitable macro-model to simulate the infill behaviour is proposed, at the aim to reach acceptable results both in global and local analyses: two non-parallel struts are set in each frame according to the most significant parameters affecting the behaviour of an infilled structure. The obtained results are applied to a partially infilled building, in order to underline the peculiarities of the proposed approach and to compare it to other methods available in literature and in codes.

2.2.4 Effect of infill wall on fundamental period of structures

Kose (2009) studied the effects of some parameters i.e. ratio of area of shear walls to area of floor, ratio of infilled panels to total number of panels, building height, number of bays and type of frame on the fundamental period of RC frame buildings. Commonly, by a modal analysis the fundamental period of RC frame buildings is determined, by getting rid the effect of infill‟s masonry wall on the rigidity and total mass of the building frame structure. He generated 189 building models and analyzed with selected parameters in 3D model using a finite element method. For determine the fundamental period of models was used an iterative modal analysis, due to the nonlinear conduct of infill masonry walls; It was observed that RC frame buildings without infill masonry walls had a greater period, when compared with RC frame buildings with infill masonry walls regardless of whether this building had shear walls or not. The presence of shear walls also led to a reduction in the fundamental period, between models with and without infill masonry walls.

Jinya and Patel (2014) investigate the effect of infill masonry walls on seismic response of frame buildings. He is illustrating that through the elastic response phase, the in plane lateral stiffness of the building is increased when infill masonry walls stiffness is considered and lead to reduce the fundamental period of the building, and as a result, it leads to greater shear forces. Davis et al. (2004) investigate and illustrate the influence of infill masonry wall on the multi-story frame buildings response under earthquake loading by taken a typical example. It is observed that by a presence of infill masonry wall stiffness, the structure attracts more base shear and the fundamental period of the structure reduces.

2.2.5 Effects of openings in infill wall on the rigidity of structures

Dawe and Seah (1989) observed that in masonry infill panel when opening is existence, the initial major crack load is reduced while the ultimate load may not be affected significantly. When a doorway opening location is appeared near the loaded side of the panel, the ultimate load is greater affected when compared to the opening is far from the applied ultimate load. It seems that if the opening must be presence and cut off the diagonal compression path load, it is better that it is placed so that as much panel material as possible is between it and the point of load. This gives a diagonal strut effect an opportunity to develop in that part of the panel. Since in practical situations, the load would come from in the two end of infill panel, it means that the appropriate place for a doorway opening is at the center of the panel. The initial stiffness increases by the presence of vertical reinforcement around an opening, but does not increase ultimate strength of the panel and the major cracking load. It is believed that if the vertical reinforcement is integrally tied to the floor beam, roof beam, and the lintel to form a configuration of subpanel, ultimate strength and the cracking load would be higher.

Asteris (2003) demonstrates analytical investigate results on the effect of the opening location and its size on the infill masonry wall in earthquake response of infilled masonry frame buildings. He uses the estimated stiffness reduction factor λ (λ is the ratio of the stiffness of infill panel with opening to the stiffness of infill panel without opening) for comparison with bibliographical data. He analyzed a different configuration model of one-bay one-story infilled frame building structures. In order to completely understand the influence of openings on the lateral stiffness and behavior of infilled plane frames under lateral seismic loading presented some parameters and some cases as; the percentage of opening (opening area/infill wall area), the presence (or not) of opening in the infill panel, and the location of opening to the compressed diagonal), investigated for the following three cases:

Case A: opening is underneath the compressed diagonal,

Case B: opening is upon the compressed diagonal, and

Case C: opening is above the compressed diagonal.

Figure 2.17 shows the variation of the λ factor as a function of the opening percentage (ratio of the area of opening to the total area of infill panel) for an opening upon the compressed diagonal (Case B) of the infill masonry wall (with dimensions' proportion is the same as the infill wall dimensions' proportion). As expected, by increasing the opening percentage the frames stiffness is decreased. Figure 2.18 illustrated the variation of the stiffness reduction factor λ of the masonry infilled frame as a function of the opening percentage (the effect of opening percentage on the reduction infilled frames stiffness) for the three different positions (as mentioned above) studied. When the opening is upon the diagonal compressed (Case B) the higher value of stiffness reduction of the frame building can have observed when compared to the (Case A and C). Because of the compressed diagonal action of the infill wall interrupted in this case (Asteris, 2003).

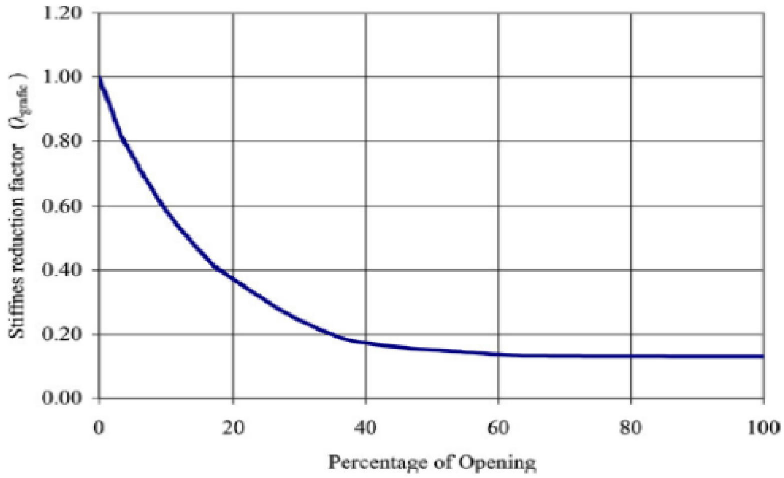


Figure 2.17 Stiffness reduction factor λ of infilled frame in relation to opening percentage (case B: opening upon compressed diagonal) (Asteris, 2003)

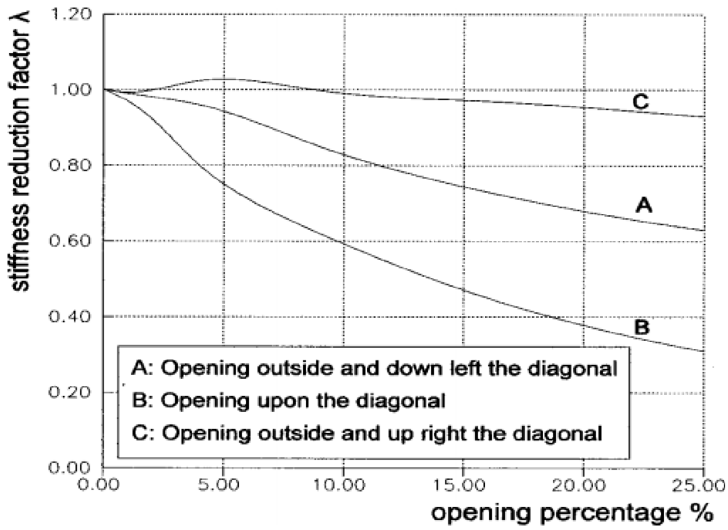


Figure 2.18 Stiffness reduction factor λ of infilled frame in relation to opening percentage for different positions of opening (Asteris, 2003)

The obtained values of the stiffness reduction factor λ from these diagrams (Figures 2.7 and 2.8) can be used to make better estimation calculation of the width of the equivalent diagonal compressed strut. For practical design purposes, Asteris (2003) propose the estimation of the equivalent strut width based on the Mainstone"s (1971) formula, by means of the following simplified equation:

$$b_w = 0.175 d_{inf} (\lambda_h H)^{-0.4} \quad (2.13)$$

Where λ = stiffness reduction factor (Figures 2.10 and 2.11) and

$$\lambda_h = h \sqrt[4]{\frac{E_b \ t \ \sin(2\theta)}{4 E_s \, I \, H}} \quad (2.14)$$

There are still great difficulties unresolved regarding the modeling approach for frames infilled with openings. Openings of door and window in the infill masonry wall reduce the effective width of the equivalent diagonal compression struts. As a result, the lateral load bearing capacity of the infilled frame buildings is decreased by the presence of openings in the infill masonry walls. Lateral stiffness of the infilled frame building with openings, i.e. window and door opening, under in-plane horizontal loading, relies on the size of the opening, and the position of the opening with respect to the equivalent diagonal compression strut. The location of the opening with respect to diagonal strut can be classified as over, under or above the equivalent diagonal compression strut (Kose, 2009).

Yuen and Kuang (2015) studies the response of the earthquake and mechanisms of failure of infilled frame RC structures with five different infill configurations which include infill's with door openings and infill's with window openings. The presence of openings in the infill masonry panels obviously reduces the incurred bracing action against the surrounding frame and quite significantly reduced the seismic forces resistance of the frame building and revealed that; Infilled frames with openings of door or window exhibit shakedown or ratcheting phenomena, respectively, in hysteresis behavior due to the spread of sliding cracks started from the regions of the stress concentration around the corners of the openings. Escalation of deformation, and consequently, extortion, is halted by the kinematic restrict imposed by continuous infill"s under the window openings after full slippage of the sliding cracks. However, Escalation unrestricted deformation of infilled frames with door openings bear serious asymmetric damage patterns and mechanisms for the transfer of lateral force.

Agrawal and Raut (2013) attempt to highlight the performance of masonry infilled reinforced concrete (RC) frames. It is revealed that; by increase the opening percentages the lateral stiffness of infilled frame is decreased. Jinya and Patel (2014) also investigate the effect of infill walls on seismic behavior of RC frame buildings by taken different model of infill wall with and without strut and with central outer opening were compared with two different percent 15% and 25%. It is revealed that by taken diagonal strut the seismic performance of RC frame building would change. Increased axial force in column, decrease the story drift and story displacement and increase the base shear with higher stiffness of infill. The soft story effect can be minimized if in the ground level at least exterior wall is provided. The lateral stiffness well decrease by increasing the opening percentage.

CHAPTER 3
CASE STUDY

3.1 Description of RC frame building models

In this study, the effects of masonry infill wall on the behavior of reinforced concrete (RC) frame building were investigated. For this purpose, as case study structures, seven storey and four bays RC frames regular and irregular in elevation with and without masonry infill walls have been inspected. Bare frames (RF, IRF1, IRF2, IRF3, and IRF4) and those with three different infill wall arrangements (PI-DB, PI-QB, and FI) were considered. In the investigation, RF and IRF indicate the regular and irregular frames, respectively while PI-DB, PI-QB, and FI explain the frames with partially infill (double bay infill or quadruple bay infill), and fully infill cases, respectively.

The elevation views of regular and irregular case study frame models are shown in Figure 3.1 while those with infills (single strut in one direction) are given in Figures 3.2 and 3.3, respectively. Moreover, Figures 3.4 and 3.5 show the regular and irregular frame models with infills in the case of single strut in two directions, respectively. For the nonlinear pushover analysis, the analytical models associated with single strut in one direction were used. However, a pair of compression strut was adopted to model the behavior of infill panels in nonlinear time history analysis.

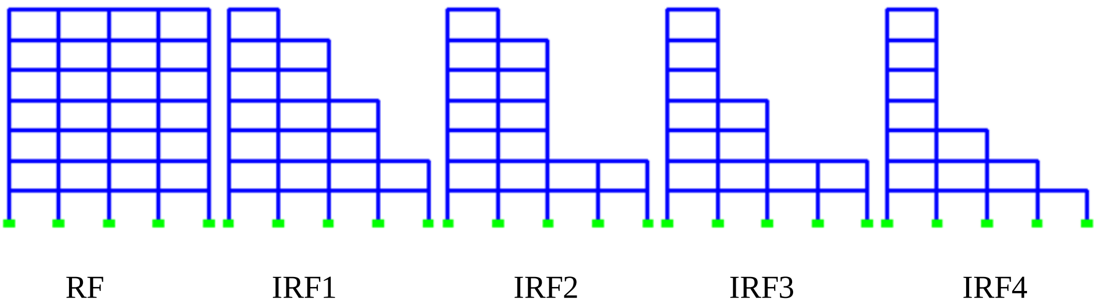
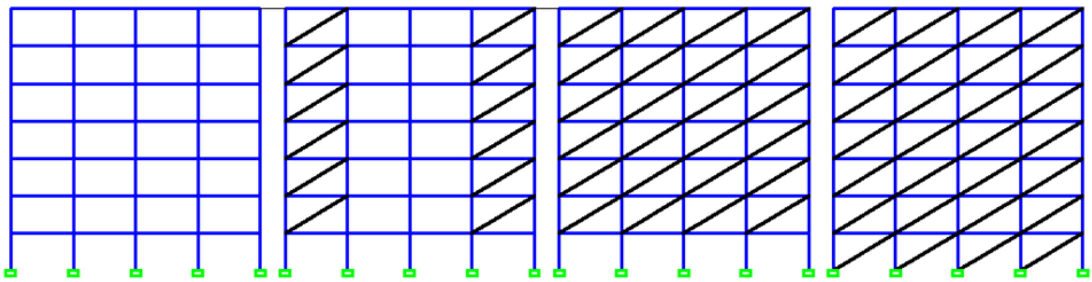
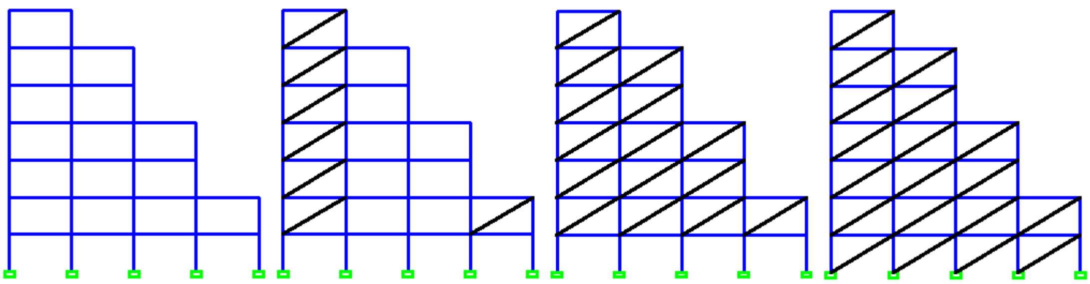


Figure 3.1 Regular and irregular frames in elevation



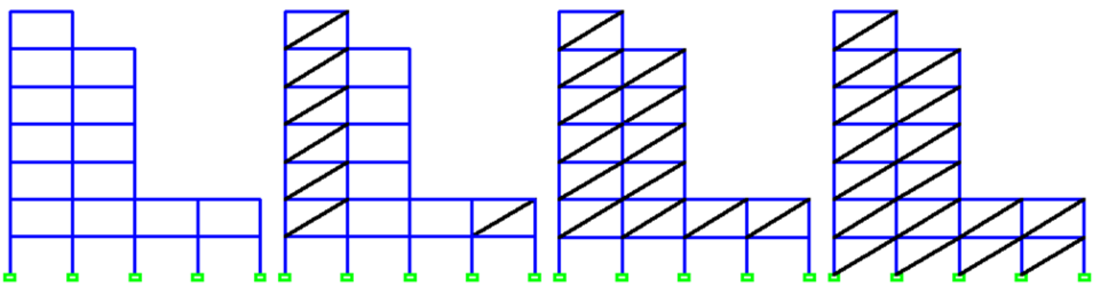
RF RF-PI-DB RF-PI-QB RF-FI

Figure 3.2 Regular frames with and without infills (single strut model in one direction)



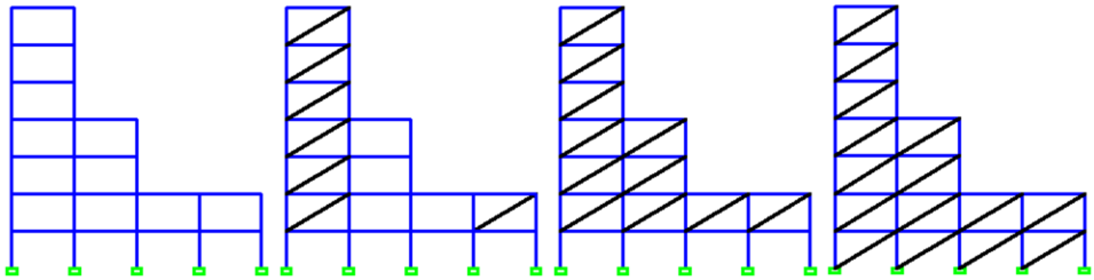
IRF1 IRF1-PI-DB IRF1-PI-QB IRF1-FI

a)



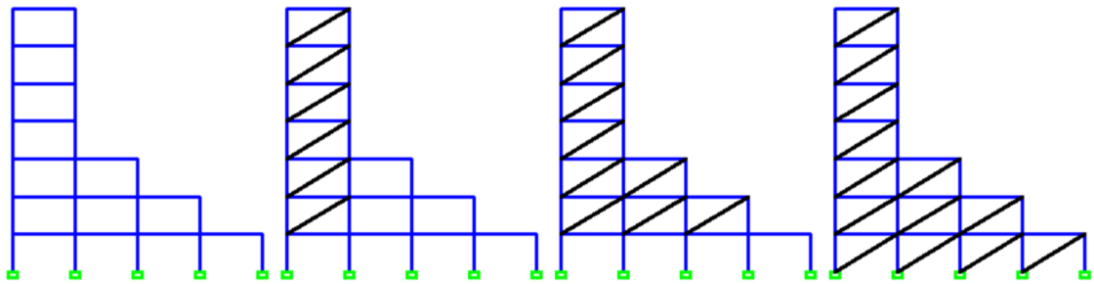
IRF2 IRF2-PI-DB IRF2-PI-QB IRF2-FI

b)



IRF3 IRF3-PI-DB IRF3-PI-QB IRF3-FI

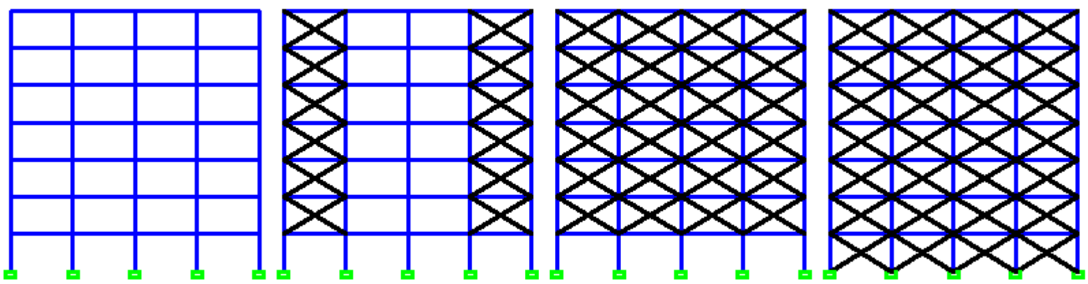
c)



IRF4 IRF4-PI-DB IRF4-PI-QB IRF4-FI

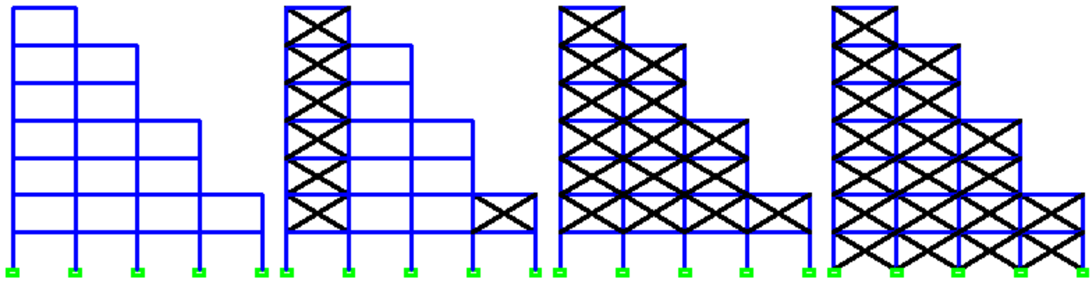
d)

Figure 3.3 Irregular frames with and without infills (single strut model in one direction)



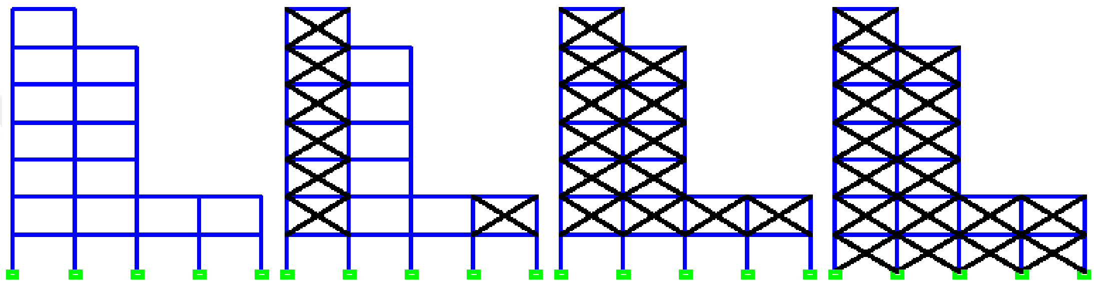
RF RF-PI-DB RF-PI-QB RF-FI

Figure 3.4 Regular frames with and without infills (single strut model in two directions)



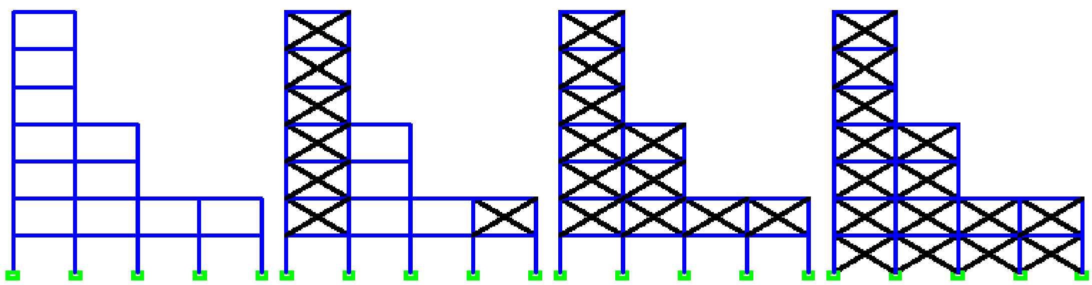
IRF1 IRF1-PI-DB IRF1-PI-QB IRF1-FI

a)



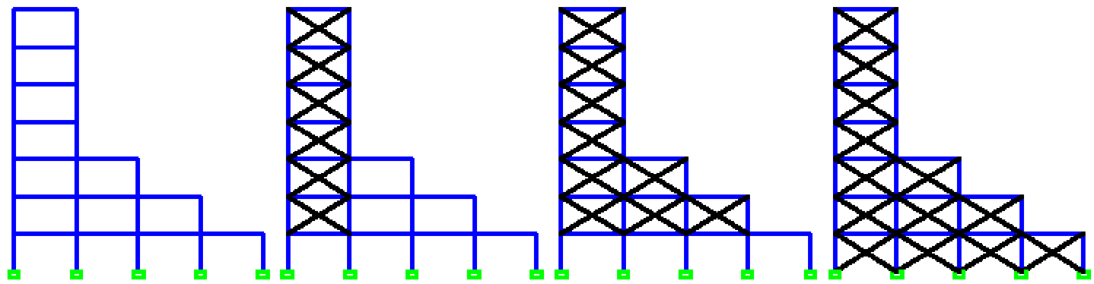
IRF2 IRF2-PI-DB IRF2-PI-QB IRF2-FI

b)



IRF3 IRF3-PI-DB IRF3-PI-QB IRF3-FI

c)



IRF4 IRF4-PI-DB IRF4-PI-QB IRF4-FI

d)

Figure 3.5 Irregular frames with and without infills (single strut model in two directions)

The frame models had the storey height of 3 m at each level and four equal bays of 5 m. The compressive strength of concrete was taken as 30 MPa, and the yielding strength of the steel was obtained as 350 MPa. Frame column sections were considered as 0.40x0.50 m while beam sections were 0.30x0.50 m. The RC elements were modeled using frame elements. The nonlinear hinges were assigned to beams as moment (M) hinge and to columns as axial load- moment (P-M) hinge. The dead load and live loads were applied as distributed on each beam; for the gravity loading in the analysis full dead load and 0.3 of the live load were taken into account.

The design codes of IS 1893 (2002) and ASCE 7 (2005) describe the vertical geometric irregularity in building. The former mentions that the building will be vertically irregular in the case that the lateral dimension of the maximum offset at the top level exceeds 25% of the lateral dimension of the building at the base while the latter explains the limitation that if the lateral dimension of the building in any storey is more than 130% of that in an adjacent storey. In the current study, all irregular frame buildings exceed the limitation of the codes and have the same amount of geometric irregularity of 75% and 200% as per IS 1893 and ASCE 7, respectively.

Two dimensional analytical models of the structures and the nonlinear analysis were carried out by using SAP 2000 software.

3.2 Building frame model with infill walls

3.2.1 Equivalent strut model

Infill masonry walls can be modeled using a diagonal 1-strut, 2-struts, 3-struts, or similarly more number of struts. Although, many analytical models for masonry infill walls are available in literature but guidelines on selection of a mathematical model are not available. In the past, several researchers have used 1-strut diagonal model for modelling of masonry infill walls (Saneinejad and Hobbs 1995; Thiruvengadam 1985; Reflak and Fajfar 1991; Buonopane and White 1999).

The local effects resulting from the interaction of the infill walls with the bounding frame could not be totally included in the analysis by 1-strut models, because of this, macro-models based on two, three or multiple diagonal struts were proposed and investigated by many researchers (Andreaus et al., 1985; Abel et al., 1992; akhakhni et al., 2003; Crisafulli and Carr 2007). Despite of increasing complexity, the significance of these models is their ability to indicate local effects and to reflect the actions in the frame more accurately (Samoilă, 2012).

3.2.2 Determination of equivalent strut width

In macro-modeling, one or a few diagonal strut elements simulate each infill panel. One of the first authors, who dealt with this issue, and suggested equivalent diagonal strut instead of the masonry panels in infilled frames under lateral load is Polyakov (1958) (Mallick and Severn, 1967). Holmes (1961) proposed replacing the infill by an equivalent pin-jointed diagonal strut of the same material assuming that its width was the third part of the diagonal between the two compressed corners. Later, Smith and Carter (1969) suggested a theoretical relationship for the width of the diagonal strut connected to infill-frame stiffness parameter λ_h; it is revealed that the effective width of the strut is not a constant value for a particular infill but decreases as the loading is increased. An alternative proposal was given by Mainstone (1971). For evaluation of the equivalent strut width of the panel, Mainstone and Weeks (1972) considers the relative infill to frame flexibility, based on analytical and experimental data, proposed relationship for the calculation of the equivalent strut width.

According to Abdul-Kadir (1974), actually the width of the strut is influenced not only by the adjacent columns, but also influenced by the top beam, by dividing the parameter λ, into two different factors and, that are correlated to the adjacent columns and to the upper beam, respectively, and prepared a relation for the width of the equivalent strut, as a function of the two different factors. The first model simulating the infill behavior with non-linear diagonal struts was proposed by Klingner and Bertero (1976) by performing laboratory tests, they also suggested an expression for the width of the equivalent strut, as a function of the stiffness parameter λ. Paulay and Priestley (1992), reasonably take a conservative value of as one-fourth of the diagonal length, this value can be used in order to provide an approximate estimate (in a conservative sense) of the elastic period of the infilled frame. Later, Durrani and Luo (1994), on the basis of detailed numerical FEM analyses, proposed a formula in which the dependence on the geometry of the frame is introduced by means of the coefficient. FEMA (1998) proposed the equation for the equivalent strut width, which is represented by the diagonal length (d_{inf}) and the actual infill thickness that is in contact with the frame (t_{inf}).

Amato et al. (2008) introduced that the strut size not only depended on the lateral stiffness of the bare frame, but also depended on the axial stiffness of the RC elements, defined The equivalent width by introducing the elastic modulus (E_{Wh}) of the masonry - evaluated along the diagonal (θ = inclination angle) and the Poisson coefficient.

The width of the equivalent diagonal strut can be found by using a number of expressions in the literature that given by different researchers and the design code as mentioned above, some of them are summarized in the Table 3.1.

In this study, applying the expressions as mentioned in Table 3.1 to a panel example of the case study frame buildings as shown in Figure 3.6 the length of the strut is given by the diagonal distance (d) of the panel. And the thickness of the strut is usually assumed to be the same of the panel (for this objective the width of strut is 24 cm). The material properties of the masonry infill walls refer to the results of an extensive experimental investigation performed by Porco et al. (2004) and Uva et al. (2012) were studied in the main types of masonry walls used in Calabria in the 1970s, and investigated by experimental laboratory testing, including diagonal compression tests Figure 3.7. The tensile strength of masonry wall was taken as 0.36 MPa. And the diagonal elastic modulus (young’s modulus) of infill was taken as 1495 MPa, and the transversal elastic modulus (shear modulus of) of infill was taken as 598 MPa.

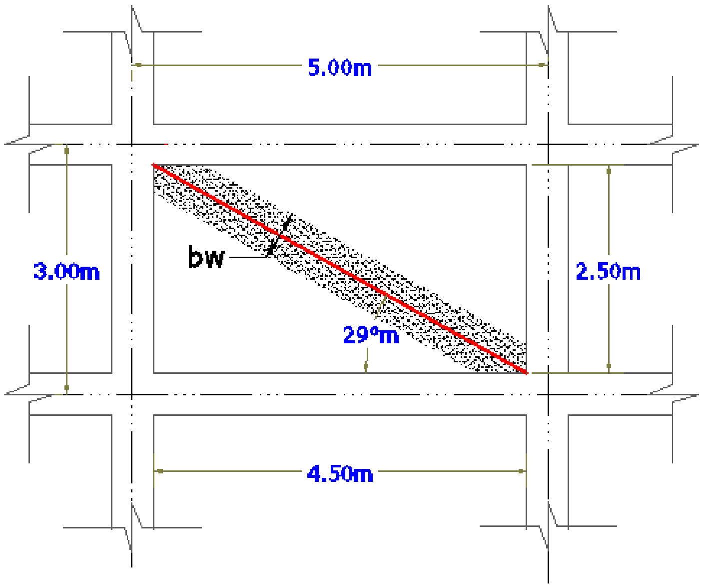


Figure 3.6 Equivalent strut compression model in this study

Figure 3.7 Diagonal compression test of a masonry infill sample (Uva et al., 2012)

Table 3.1 Different proposals with regard to the width of the strut

Eq. No.	Researcher/code	Width of the equivalent strut *bw*
1	Holmes (1961)	$w = \frac{1}{3}d$
2	Smith and Carter (1969)	$w = 0.58\left(\frac{1}{H_w}\right)^{-0.445}(\lambda_h H)^{0.335 d_z\left(\frac{1}{H_w}\right)^{0.064}}$
		$\lambda_p = \sqrt[4]{\frac{E_w . t_w \sin(2\theta)}{4E_c I_c H_w}}$
		$I_c = \frac{b * h^3}{12}$
		$I_b = \frac{b * h^3}{12}$
3	Mainstone (1972)	$b_w = 0.175 d_{inf}(\lambda_h H)^{-0.4}$
4	Klingner (1976)	$b_w = 0.175 d_{inf}(\lambda_h H)^{-0.4}$
5	Durrani (1994)	$b_w = \gamma\sqrt{L^2 + H^2}\sin 2\theta$
		$\gamma = 0.32\sqrt{\sin 2\theta}\left(\frac{E_w t_w H^4}{m I_c E_c H_w}\right)^{-0.1}$
		$m = 6\left(1 + \frac{6E_c I_b H}{\pi E_c I_c L}\right)$
6	Abdul-Kadir (1974)	$b_w = \frac{\pi}{2}\sqrt{\left(\frac{1}{\lambda_T}\right)^2 + \left(\frac{1}{2\lambda_p}\right)^2}$
		$\lambda_p = \sqrt[4]{\frac{E_w . t_w \sin(2\theta)}{4E_c I_p H_w}}$
		$\lambda_T = \sqrt[4]{\frac{E_w . t_w \sin(2\theta)}{4E_c I_T H_w}}$
7	Paulay (1992)	$W = 0.25d$
8	FEMA (1998)	$b_w = 0.175 d_{inf}(\lambda_h H)^{-0.4}$
9	Amato et al. (2008)	$b_w = d_{inf}\frac{c}{z}\frac{1}{(\lambda^*)^{-\beta}}$
		$\lambda^* = \frac{E_w}{E_p}\frac{th}{A_C}\left(\frac{H^2}{L^2} + 0.25\frac{A_c}{A_b}\frac{L}{H}\right)$
		$\beta = 0.146 + 0.0073\nu$
		$c = 0.249 - 0.0116\nu + 0.567\nu^2$
		$z = 1 \ldots\ldots\ldots \text{ if } \frac{l_{inf}}{h_{inf}} = 1$
		$z = 1.125 \ldots\ldots \text{ if } \frac{l_{inf}}{h_{inf}} \geq 1.5$

The width of the struts that calculated according to equations proposed by different authors and design codes are given in Figure 3.8. The equation proposed by AbdulKadir (1974), Smith and Carter (1969), and Holmes (1961) provided the highest values for the strut width because AbdulKadir (1974), Smith and Carter (1969) considered the stiffness of the beams in addition to that of the columns. Mainstone and Week (1972), Klingner and Bertero (1976), and FEMA (1998) provided the lowest values for the strut width. The constant value was suggested by Paulay and Pristley (1992) provide results that are near to the mean value for the strut width. It was noted that in modeling the infill panels, among the approaches mentioned the method explain in FEMA (1998) was utilized for all analytical models.

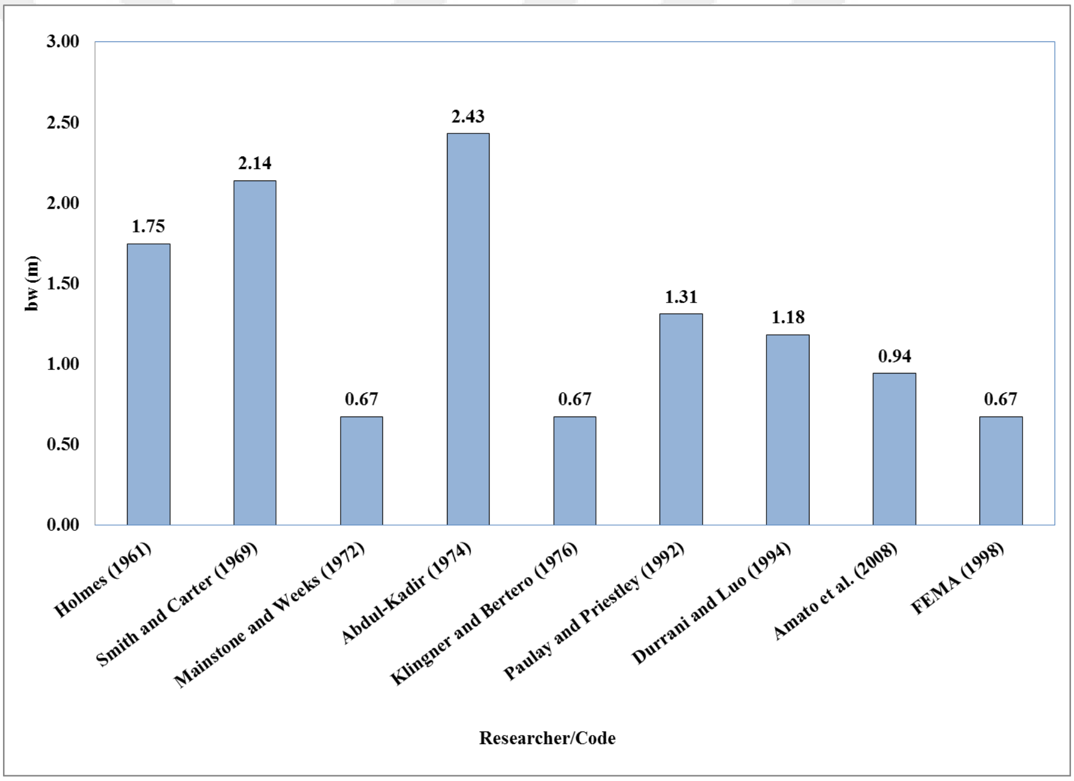


Figure 3.8 Variation of equivalent strut width with respect to different computation methods

3.2.3 Strength and stiffness

In the case of an infill wall located in a lateral load resisting frame, the stiffness and strength contribution of the infill are considered by modeling the infill as an equivalent compression strut. Because of its simplicity, several investigators have recommended the equivalent strut concept. In the present study, a trussed frame model is considered. This type of model does not neglect the bending moment in beams and columns. Rigid joints connect the beams and columns, but pin joints at the beam to column junctions connect the equivalent struts. Moment plastic hinges in which flexural (M) hinge is assigned at both ends of the beams, whereas axial forcebi-axial moment interaction hinges (P–M–M) are assigned at both ends of the columns. Axial plastic hinges, have been assigned at mid-length of the equivalent diagonal struts, simulating infill walls.

The scientific literature includes many different proposals for the determination of the strength and stiffness of infill. In this study, a constitutive relation proposed by Panagiotakos and Fardis (1996), were proposing a constitutive relation validated by experimental cyclic tests, obtaining the curve shown in Figure 3.9; this curve has mainly four segments. The first part shows the initial shear behavior of the un cracked panel. The second segment depends on the equivalent diagonal strut formation in the panel, after the separation of the infill from the bounding frame. The third part defines the infill walls softening behavior after the critical displacement and is characterized by the slope. The final part which is a horizontal line describes the final behavior of the infill walls. In literature, the final part can be ignored by assuming a softening line that reaches a zero residual strength, as shown in Figure 3.9.

Initial shear stiffness of the un-cracked panel:

$$k_1 = \frac{G_w t_w L_w}{H_w} \tag{3.1}$$

Yielding force corresponding to the first cracking of the panel:

$$F_y = f_{tp} t_w L_w \tag{3.2}$$

Displacement at the yielding:

$$S_y = \frac{F_y}{K_1} \tag{3.3}$$

Axial stiffness of the equivalent strut:

$$k_2 = \frac{E_m t_w L_w}{d} \tag{3.4}$$

Due to uncertainties in the modeling and the lack of data, it was necessary to randomly assume some mathematical parameter for the models. In this study, in order to simplify some of the parameters: the displacement corresponding to the maximum force was calculated as:

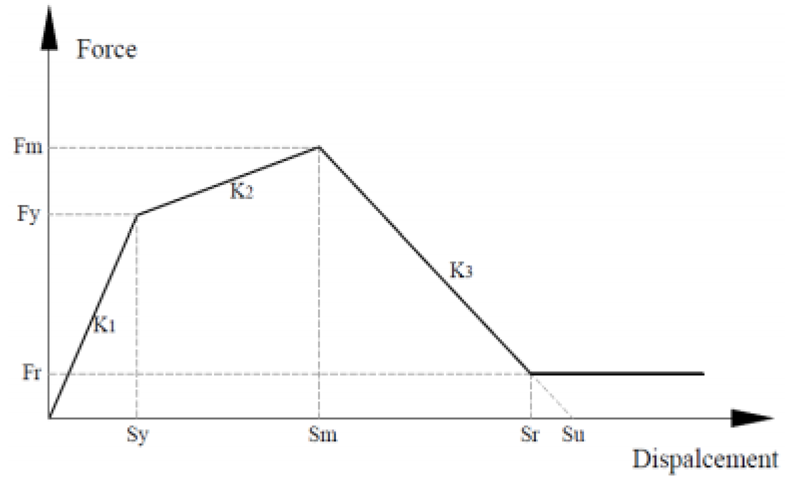


Figure 3.9 Force–displacement relationships for the equivalent strut model by (Panagiotakos and Fardis, 1996)

3.3 Nonlinear methodology

Nonlinear pushover analysis is performed for assessing the structural seismic response. In pushover analysis, the structure is subjected to incrementally increasing lateral forces with a uniform height-wise distribution until a target displacement is reached (Kadid and Boumrkik, 2008). The base shear and roof displacement could intend to create the pushover curve. It provided an indication of the maximum base shear that the structure was capable of resisting at the time of the earthquake. For regular buildings, it could provide a rough idea about the global stiffness of the building. The ATC-10 and FEMA 356 documents have established modeling techniques, analysis procedures, and acceptance criteria for pushover analysis. These documents refer to force-deformation criteria for hinges used in pushover analysis pronounced the acceptance criteria depending on the plastic hinge rotations by considering different performance levels. Each plastic hinge is designed as a separate point binge. All plastic deformation is displacement or rotation, develops within the point hinge (Nel and Ozmen, 2006).

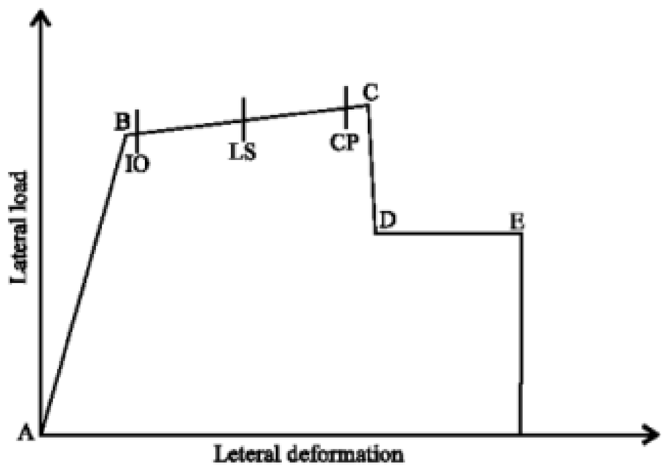


Figure 3.10 Curve of force vs. deformation (Bansal, 2011)

Five points considered A, B, C, D, and E describe the force deflection behavior of the hinge as shown in Figure 3.10. Point A refers to unloaded state and point B represents yielding of the element. The ordinate at C corresponds to nominal strength and D corresponds to the deformation at which substantial strength degradation starts. Three points termed LO, LS, and CP represented immediate occupancy, life safety, and collapse prevention, respectively. The principles for assigning values for each of these points differ liable on the form and type of member such many other parameters defined in the ATC-10 and FEMA-356 documents (Bansal, 2011).

Time history analysis of the structure has been successfully used to analyze the structure. Even though time history analysis consumes time, it is the only method capable of giving results closer to the actual one. In the time history analysis, the loads are applied over a period of time and the response is obtained at different time intervals (Muthukumar and Kumar, 2014). Nonlinear time history analysis was carried out on the same analytical models with the same hinge properties so as to estimate the actual nonlinear behavior of the structural systems. In the dynamic analysis, ALTADENA-1.TH record was used as a ground motion. Figure 3.11 shows the acceleration time plot of the earthquake.

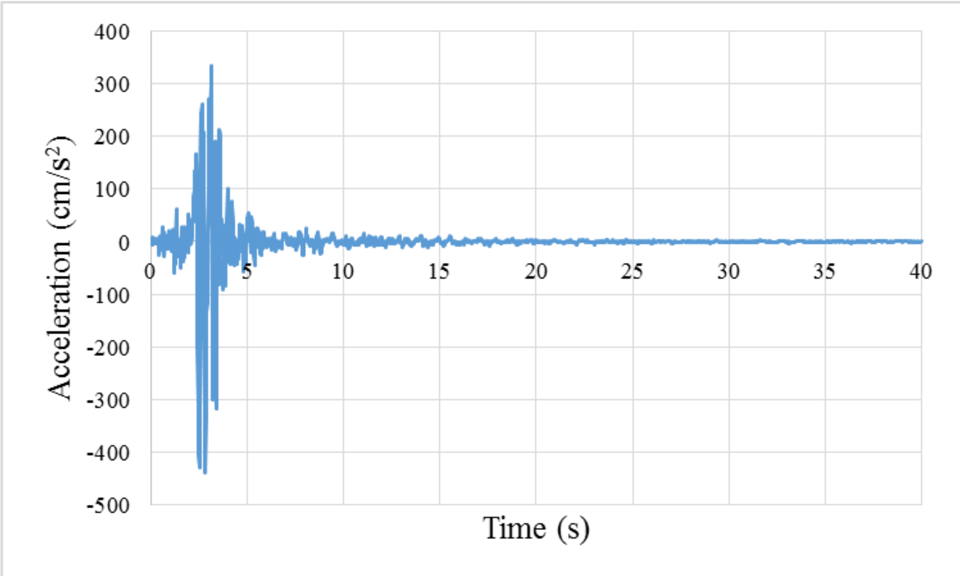


Figure 3.11 Acceleration time plot of the earthquake ground motion

CHAPTER 4
RESULTS AND DISCUSSION

4.1 General

This chapter presents the results of analysis of RC frame system. The nonlinear static pushover and time history analysis were carried out using SAP2000. For this purpose, as case study structures seven storey and four bays RC frames regular and irregular in elevation with and without masonry infill walls have been inspected. Bare frames (RF, IRF1, IRF2, IRF3, and IRF4) and those with three different infill wall arrangements (PI-DB, PI-QB, and FI) were considered. In the investigation, RF and IRF indicate the regular and irregular frames, respectively while PI-DB, PI-QB, and FI explain the frames with partially infill (double bay infill or quadruple bay infill), and fully infill cases, respectively.

4.2 Evaluation of RC frame with infills based on nonlinear static analysis

4.2.1 Pushover curve and normalize value

Pushover curves obtained by nonlinear analysis for the regular frame (RF) and four irregular frames (IRFs) in elevation are shown in Figure 4.1. The regular frame indicated the maximum value of base shear in comparison to the frames having the irregularity in elevation. It was observed that the base shear capacity of the frame decreased up to about 50% due to the introduction of the irregularity. Among IRF cases, it was also found that the initial stiffness of the frame building had a tendency to reduce significantly due to decreasing in the number of the vertical structural members and IRF4 gave the minimum value of base shear.

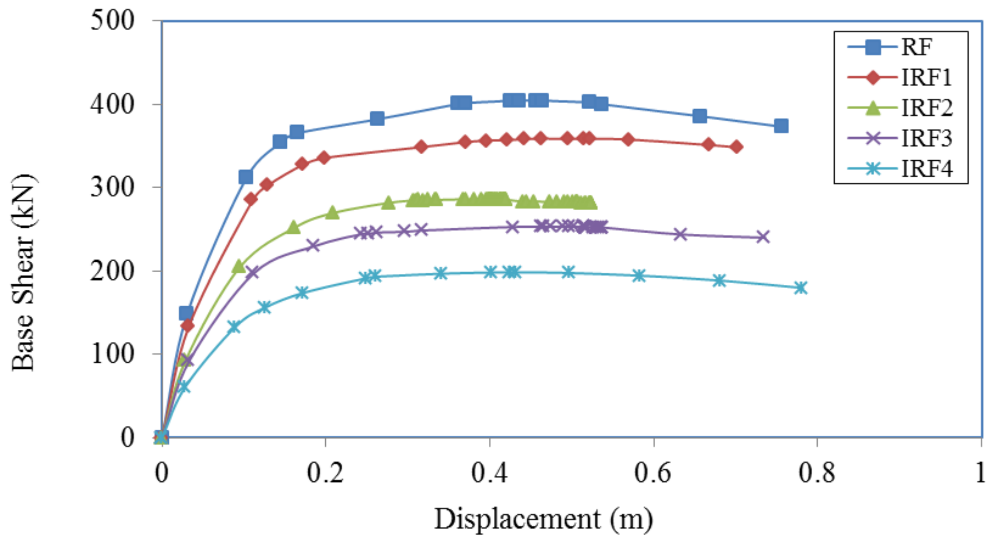


Figure 4.1 Pushover analysis results for regular and irregular frames without infills

On the other hand, it was observed that the presence of the infill walls increased both strength and stiffness of the RF and IRF systems as shown in Figures 4.2 to 4.6. However, its effect on RF and IRFs were different. For example, as seen in Figure 4.2, for the case of RF, the arrangement of the infill panel altered the behaviour of the structures subjected to lateral loading. For example, the fully-infilled frame (RF-FI) had the largest stiffness and provided the maximum value for base shear than the other cases. The stiffness of the partially infilled frame cases of RF-PI-DB and RF-PI-QB were very close to each other. However, the difference in their strength was more. The latter yielded about 15% higher base shear capacity value.

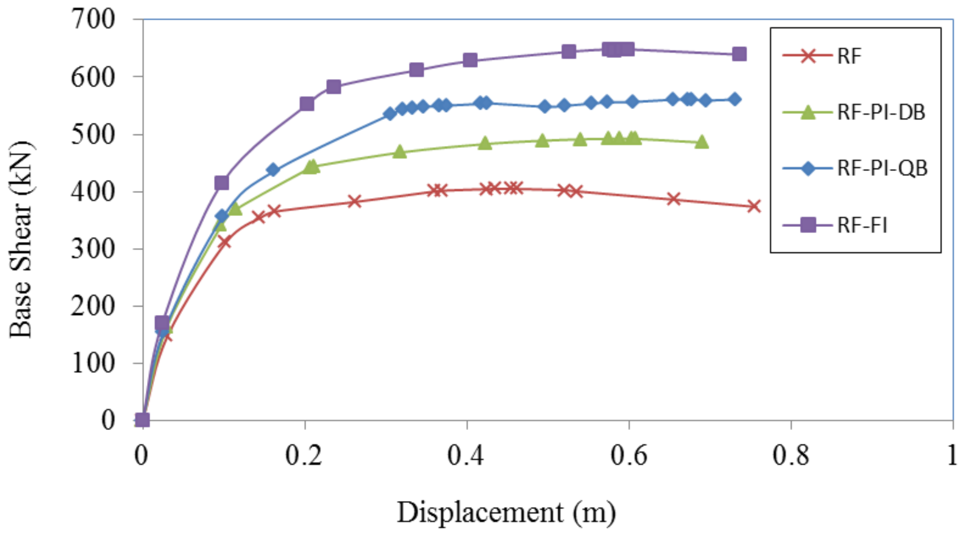


Figure 4.2 Comparison of the pushover curves of regular frames with different infill configurations

As seen in Figures 4.3 to 4.6, in the case of frames with irregular in elevation, similar to regular case, with the inclusion of infills, the strength and stiffness of all IRFs improved significantly. For example, IRF4-FI gave the highest stiffness and strength values while IRF4-PI-DB showed the lowest one as compared to the irregular frame without infill panel (IRF4). Similarly, for the IRF1-FI, the maximum base shear capacity was obtained as about 580 kN while for the IRF1-PI-DB and IRF1-PI-QB, it was found as approximately 400 and 450 kN, respectively.

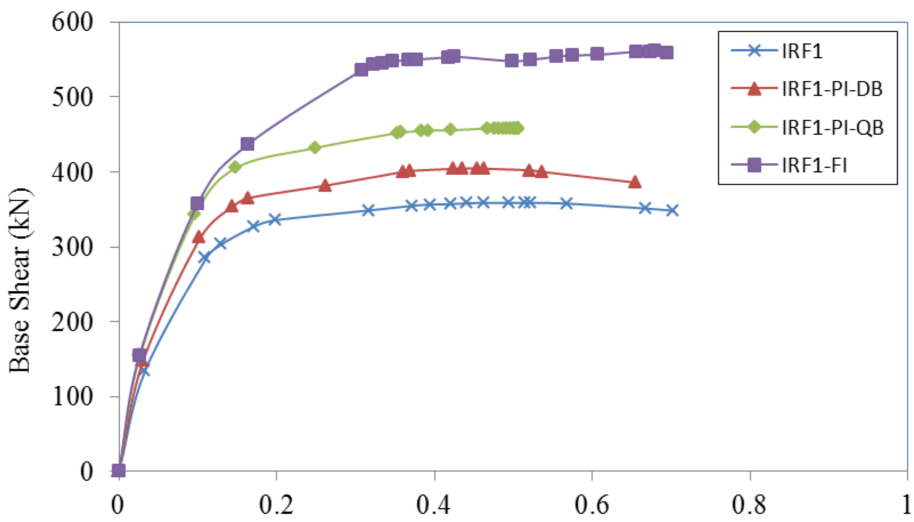


Figure 4.3 Comparison of the pushover curves of irregular frames case of IRF1 with different infill configurations

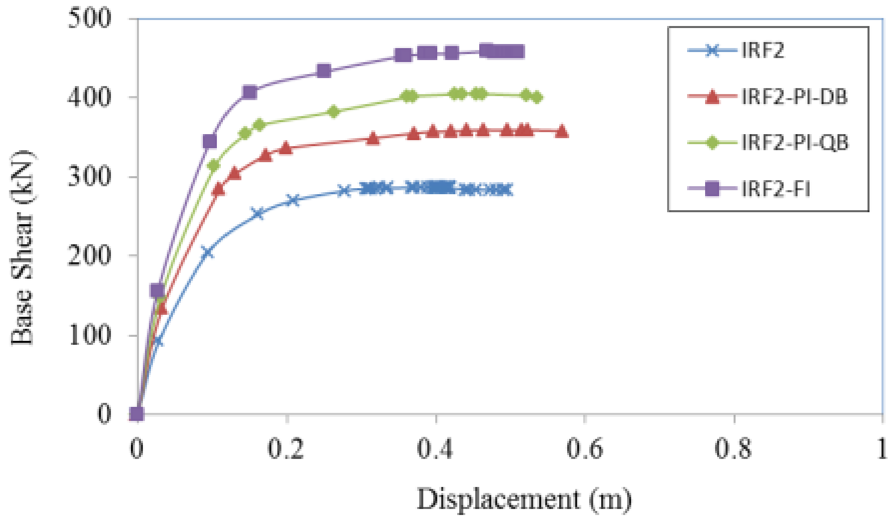


Figure 4.4 Comparison of the pushover curves of irregular frames case of IRF2 with different infill configurations

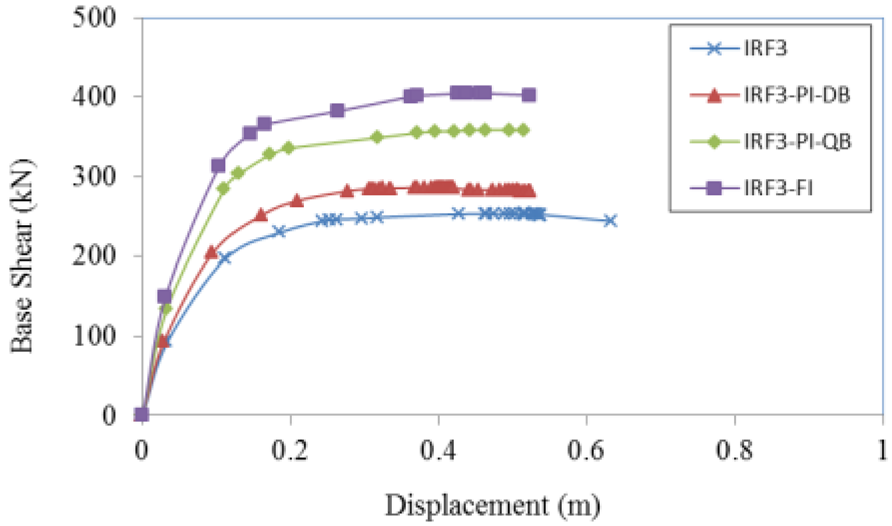


Figure 4.5 Comparison of the pushover curves of irregular frames case of IRF3 with different infill configurations

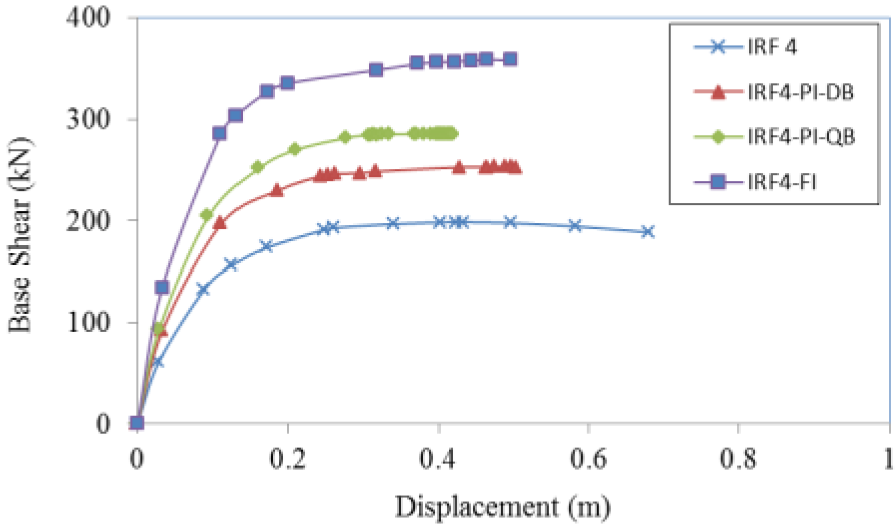


Figure 4.6 Comparison of the pushover curves of irregular frames case of IRF4 with different infill configurations

In Figure 4.7, the effects of infill configurations on the maximum base shear of RF and IRF cases were more clearly seen. Depending on the irregularities in the system, the increase in the base shear capacity of the frames with infills was varied from about 1.2 to 1.8 times. For example, for the IRF4-PI-DB, IRF4-PI-QB, and IRF4-FI, the normalized base shear values with respect to the frame without infills were obtained as 1.3, 1.4 and 1.8, respectively. In a similar trend, these values were evaluated as 1.3, 1.4 and 1.6 for the IRF2-PI-DB, IRF2-PI-QB, and IRF2-FI, respectively.

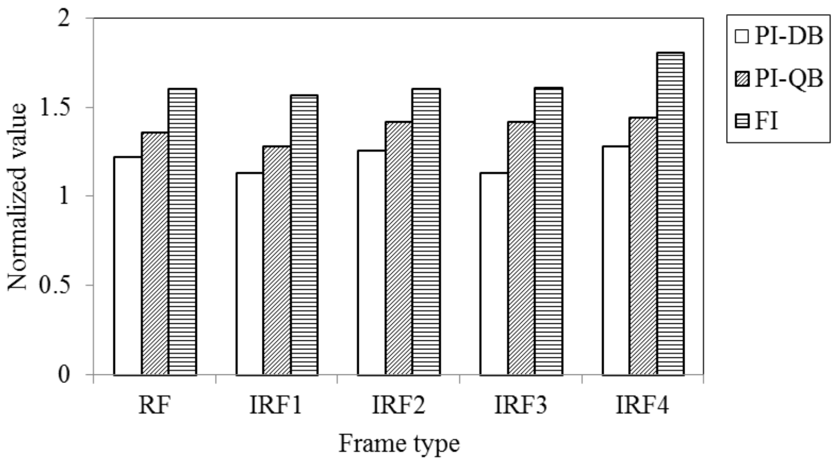
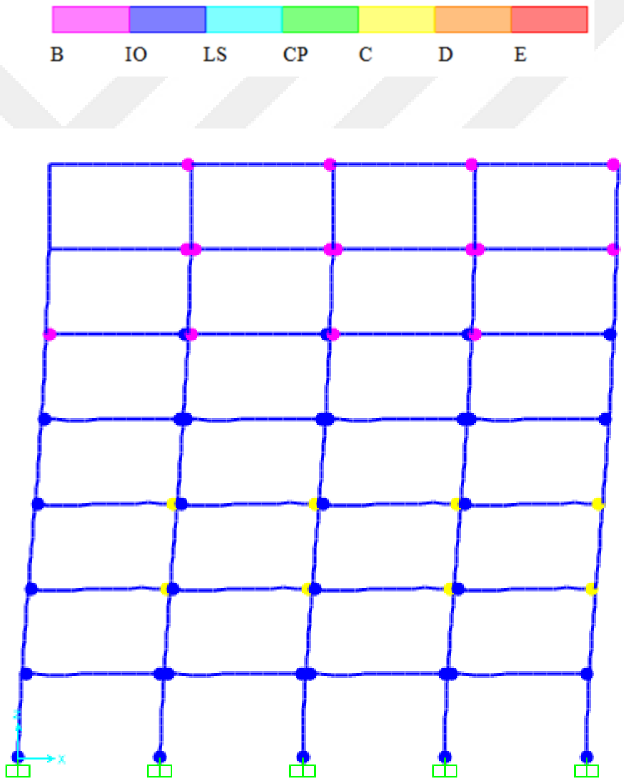


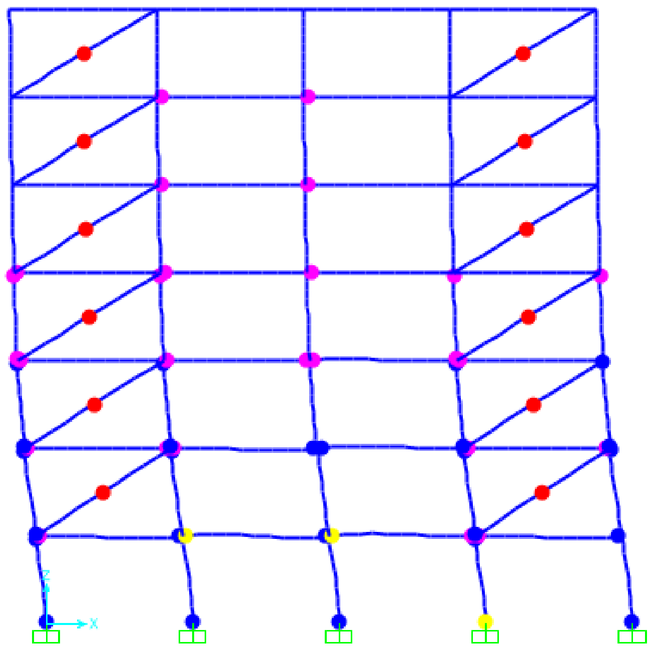
Figure 4.7 Variation of the normalized maximum base shear of infilled frame to the frame without infills

4.2.2 Plastic hinge distribution from nonlinear static analysis

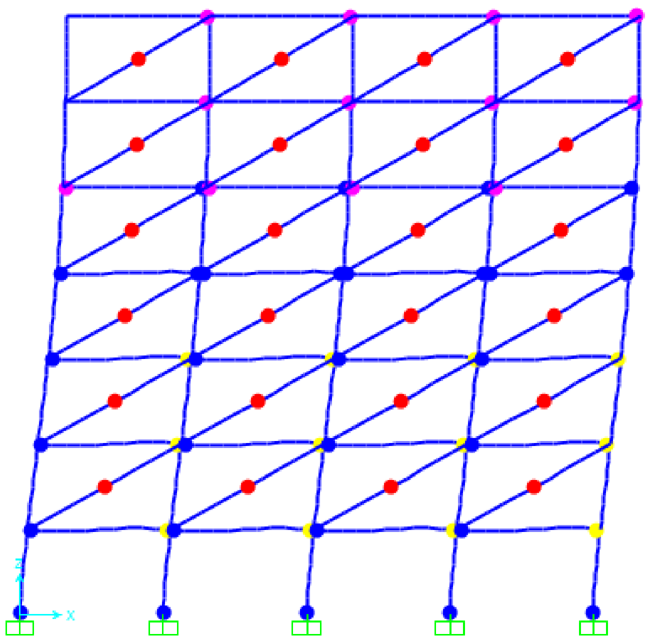
Masonry infill wall caused changing of distribution of plastic hinge and in some case changing of seismic performance levels. Also different infill model can develop different distribution of plastic hinge. For example, generally in regular frame and irregular frame with infill seismic performance level is IO which with comparing with frame without masonry infill. Masonry infille has positive effect and caused performance level reaching from C (frame without infill) to IO as shown in Figure 4.8 to 4.12 respectively. With comparing the plastic hinge distribution of frame without in-fill walls at the target displacement of infilled frame, the negative effect of masonry infille on around frame is observed.



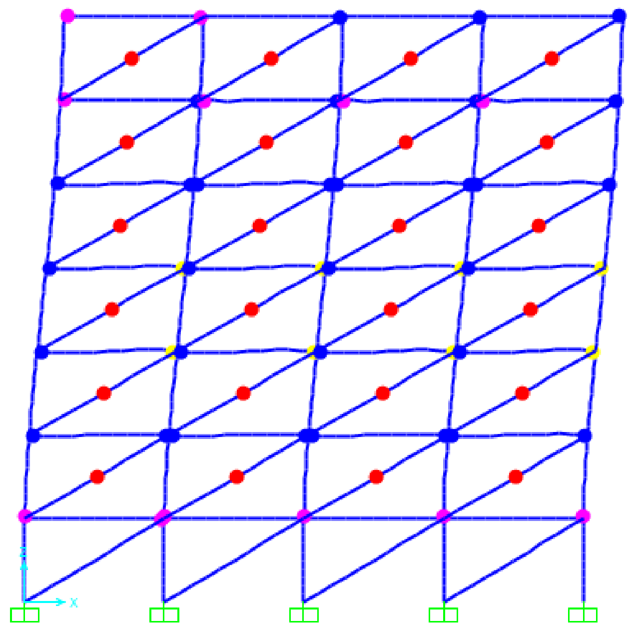
a)



b)

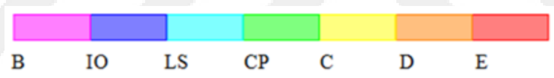


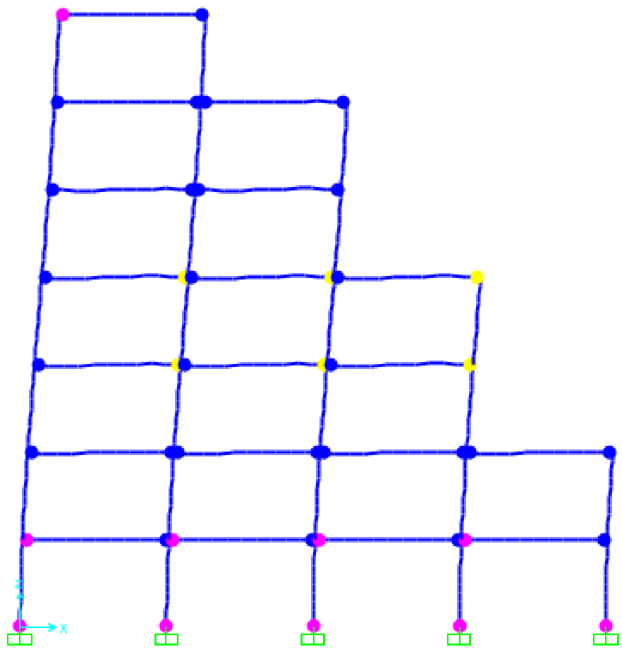
c)



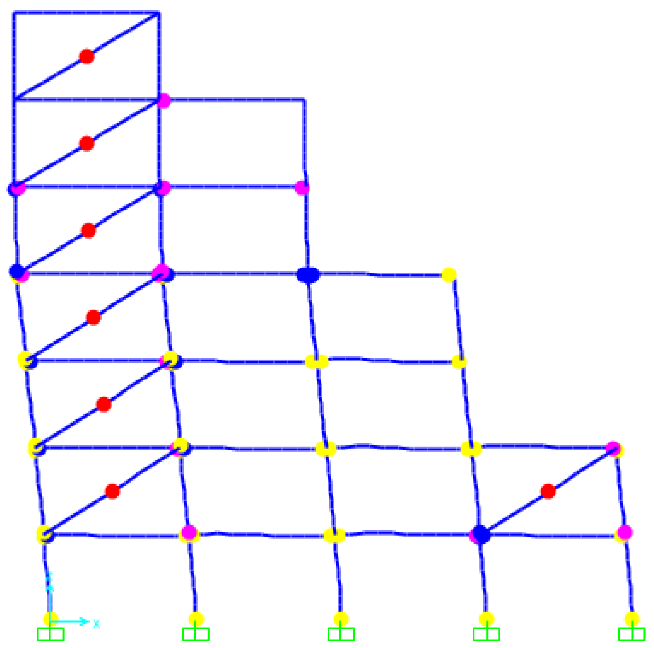
d)

Figure 4.8 Plastic hinge patterns of regular frames with different infill configurations

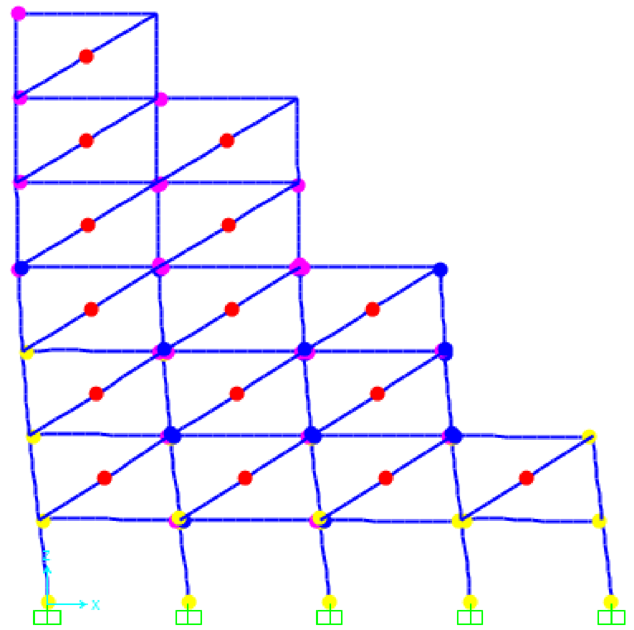




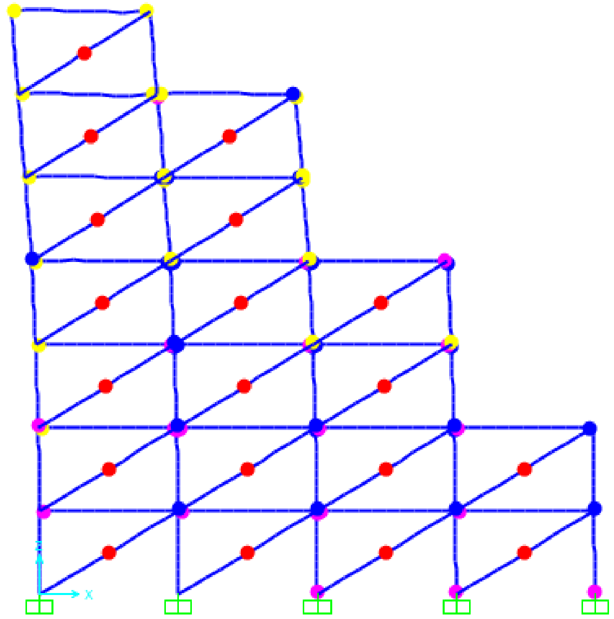
a)



b)

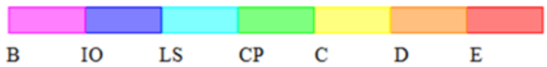


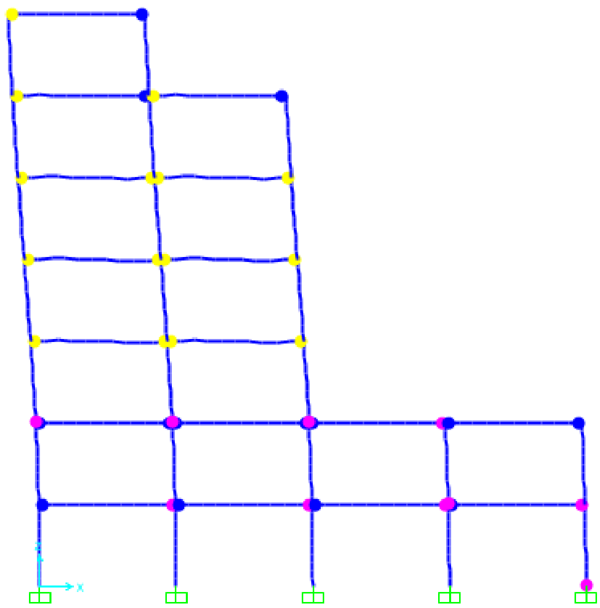
c)



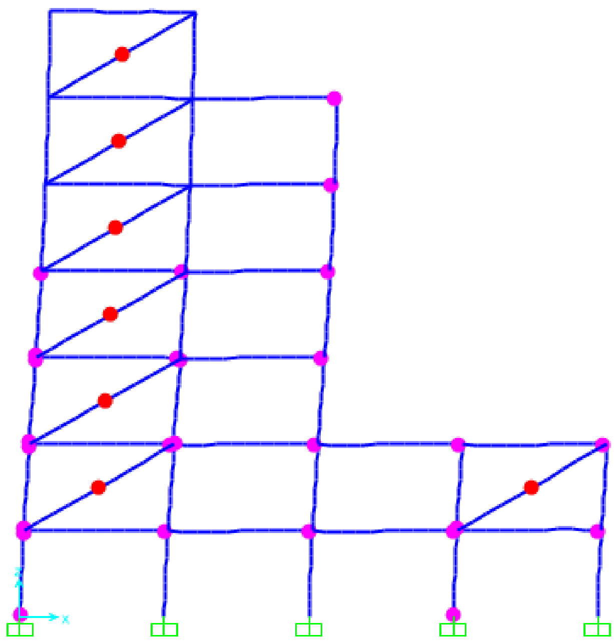
d)

Figure 4.9 Plastic hinge formations from pushover analysis of irregular frames case of IRF1 with different infill configurations

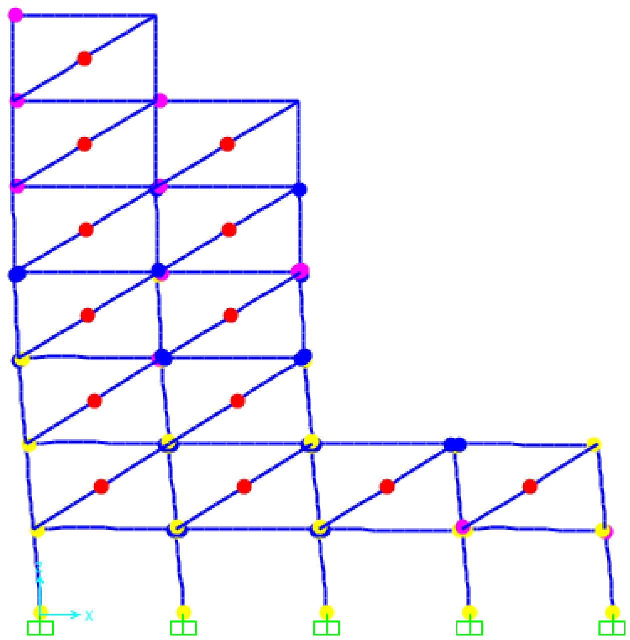




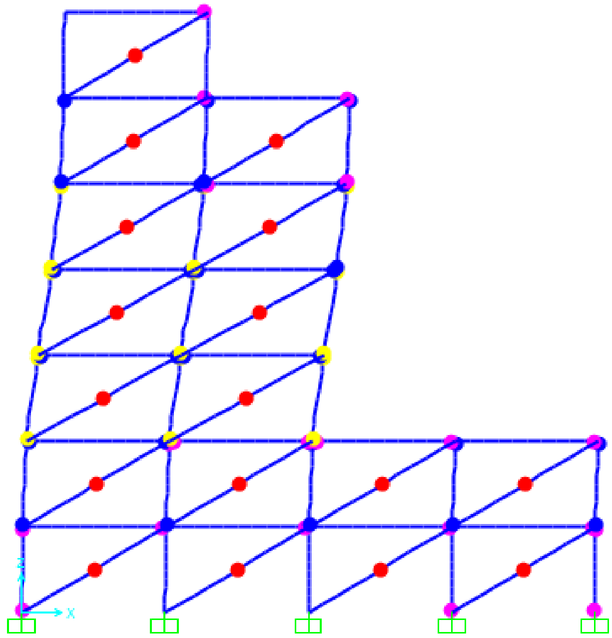
a)



b)

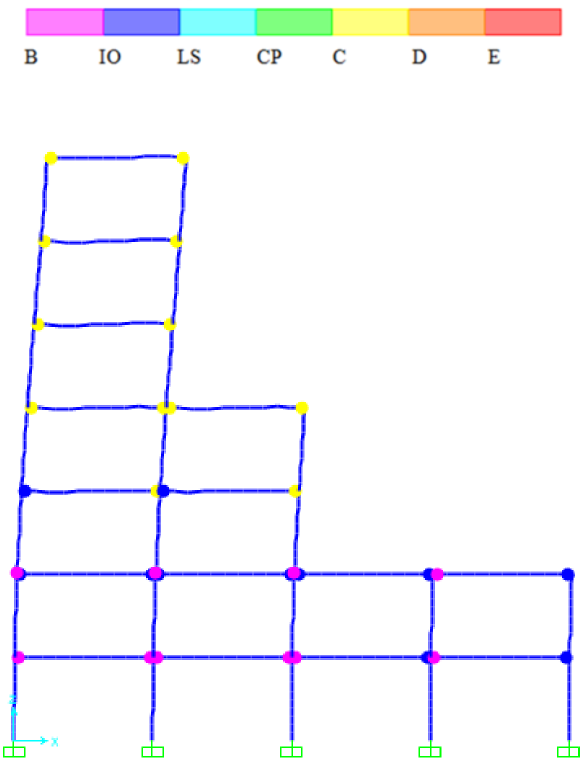


c)

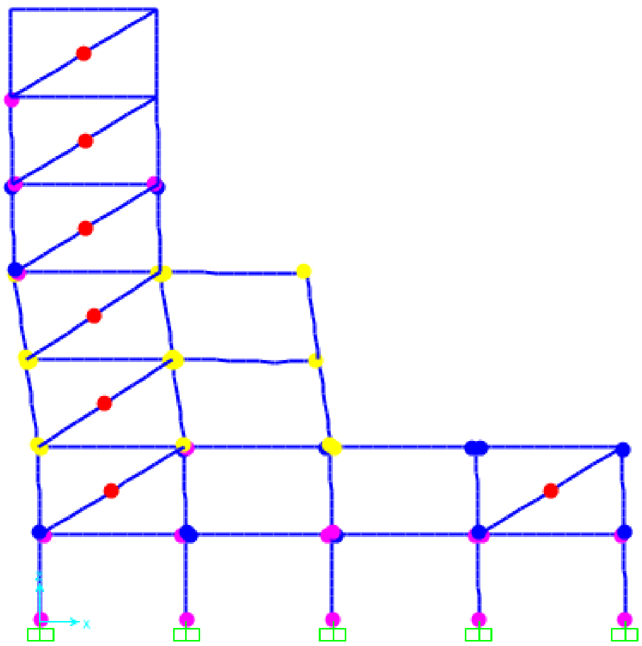


d)

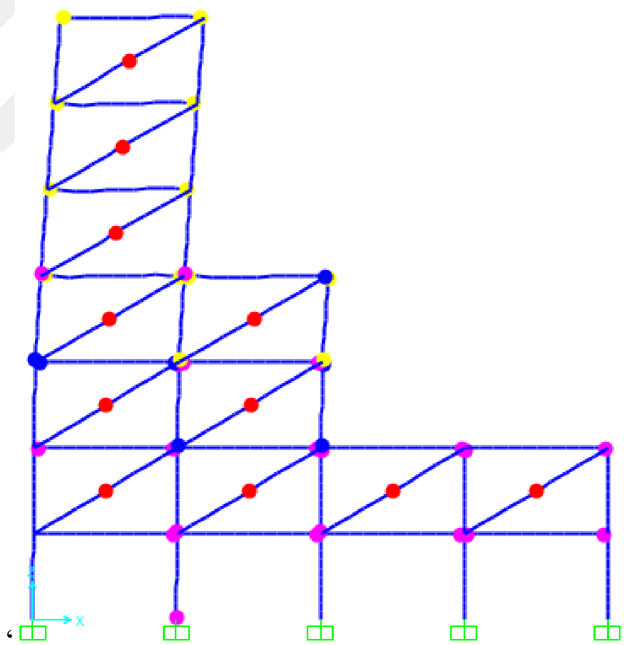
Figure 4.10 Plastic hinge formations from pushover analysis of irregular frames case of IRF2 with different infill configurations



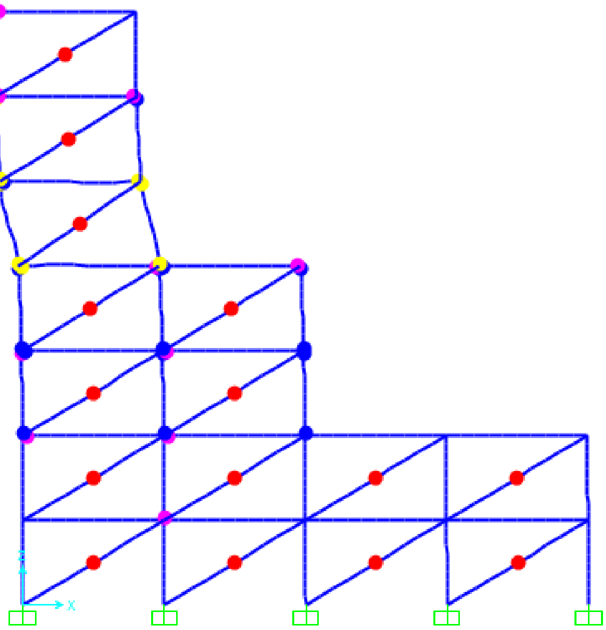
a)



b)

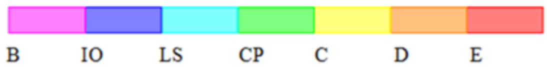


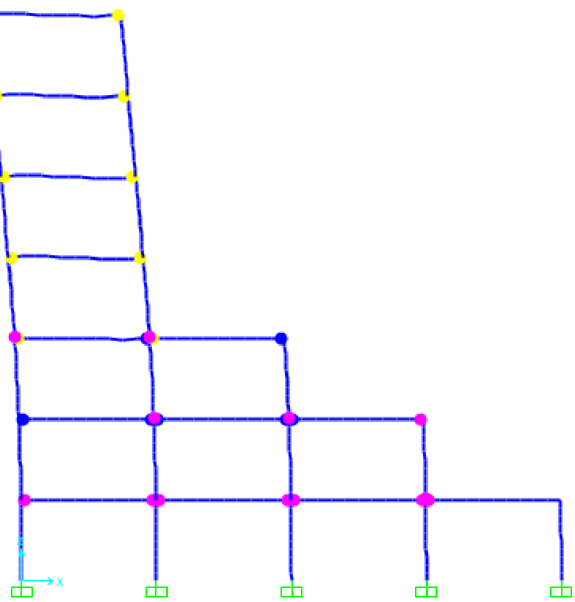
c)



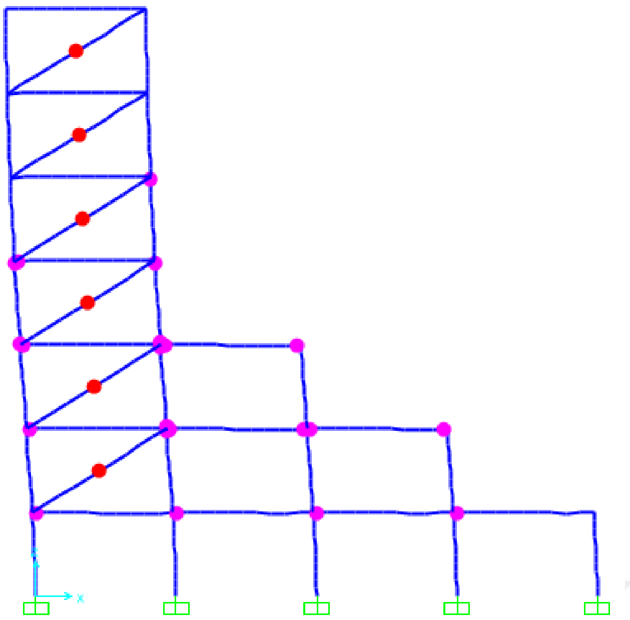
d)

Figure 4.11 Plastic hinge formations from pushover analysis of irregular frames case of IRF3 with different infill configurations

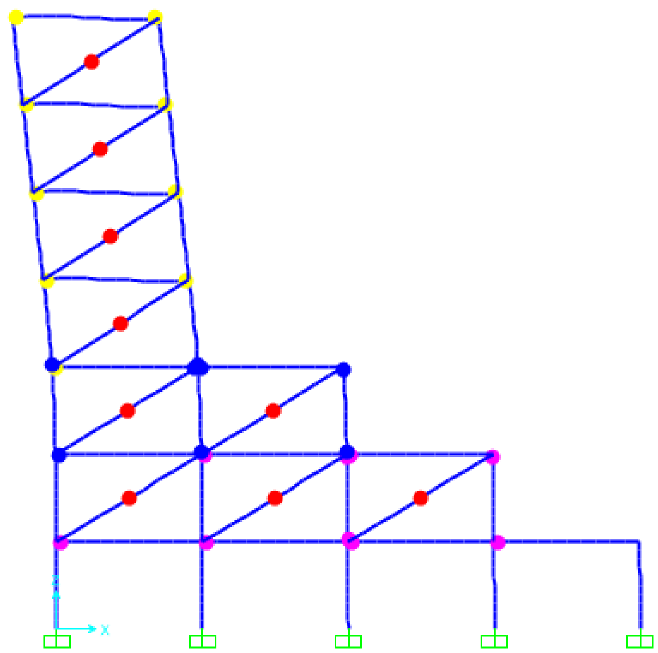




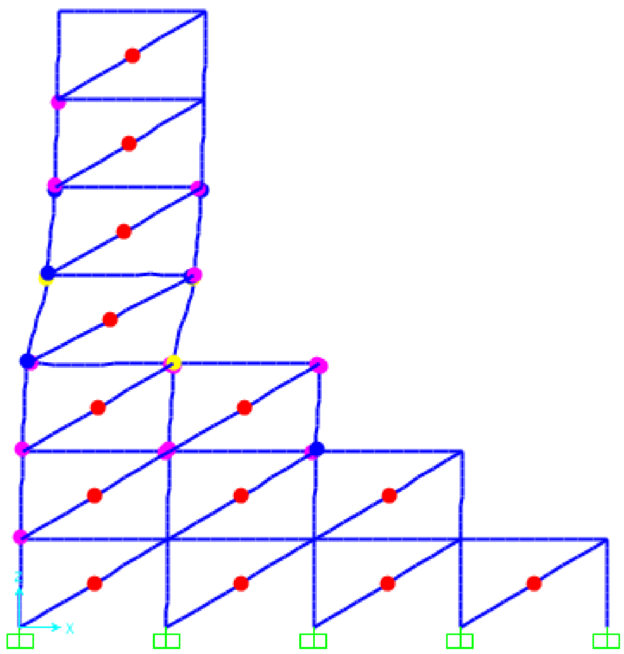
a)



b)



c)



d)

Figure 4.12 Plastic hinge formations from pushover analysis of irregular frames case of IRF4 with different infill configurations

4.3 Evaluation of RC frame with infills based on nonlinear time history analysis

The time history method is used when it is important to display inelastic response characteristics or to consider time dependent effects in the calculation of the structural dynamics. The earthquake ground motion which is used in the time history analysis of the structure is for both regular (RF) and irregular frame (IRF). The results were discussed below.

4.3.1 Lateral displacement

Maximum story displacement is always one of the most important considerations in analysis and design, it is very important to consider how much infill walls can play a role in the lateral displacement of frame systems under actual earthquake record. The results of maximum roof displacement obtained by nonlinear time history analysis for the regular (RF) and irregular (IRFs) frame system with and without infill walls are shown in Figure 4.13 to 4.17 respectively. From these values it observed that displacement is higher for the frame system without infill as compared to frame with infill, for all cases regular and irregular in elevation. The regular and irregular frames with fully infill indicated the minimum value of displacement in comparison to the frames without fully and partially infilled. It was observed that the displacement of the frame with infill walls decreased up to about 70% due to the introduction of the frame without infill.

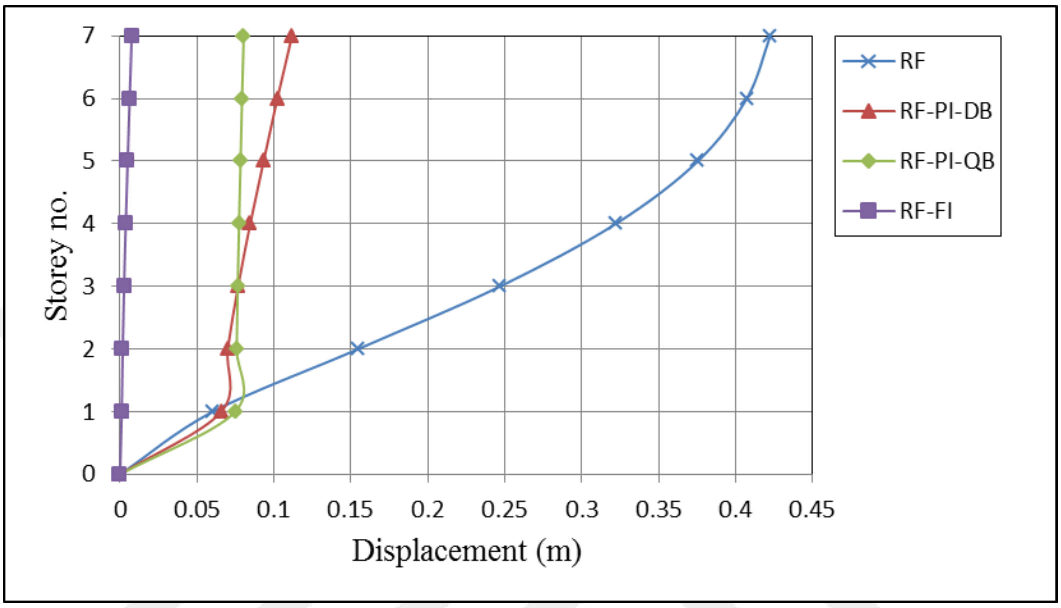


Figure 4.13 Peak displacement variations through frame height for RF with and without infills

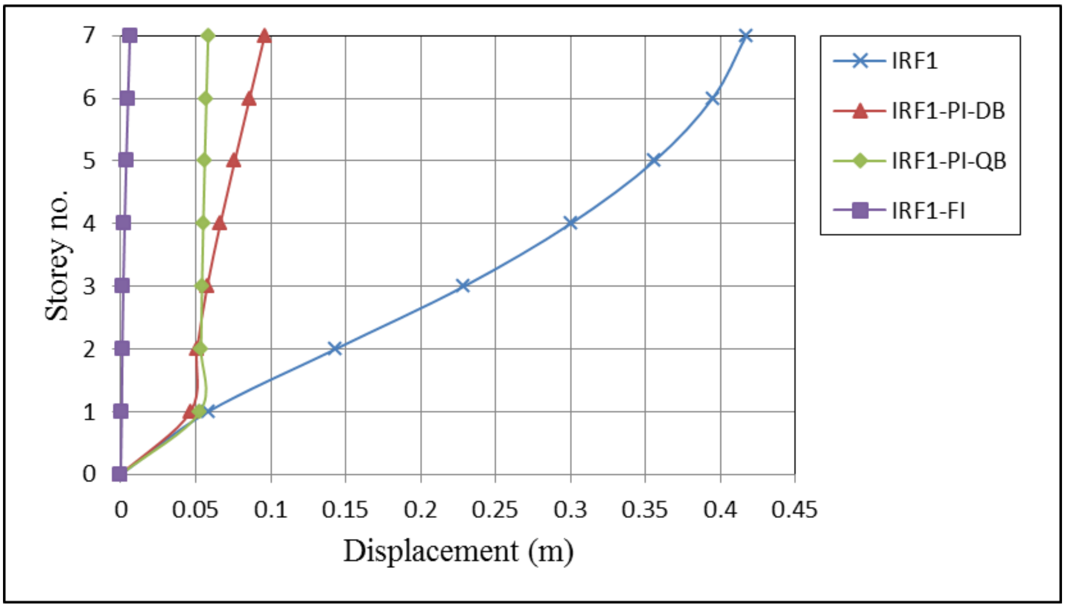


Figure 4.14 Peak displacement variations through frame height for IRF1 with and without infills

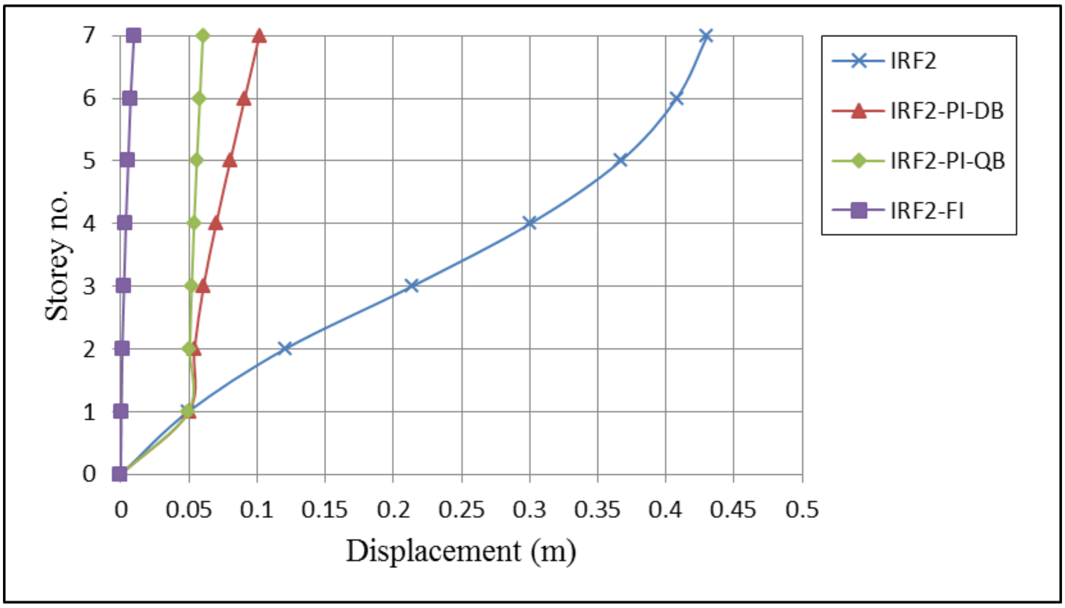


Figure 4.15 Peak displacement variations through frame height for IRF2 with and without infills

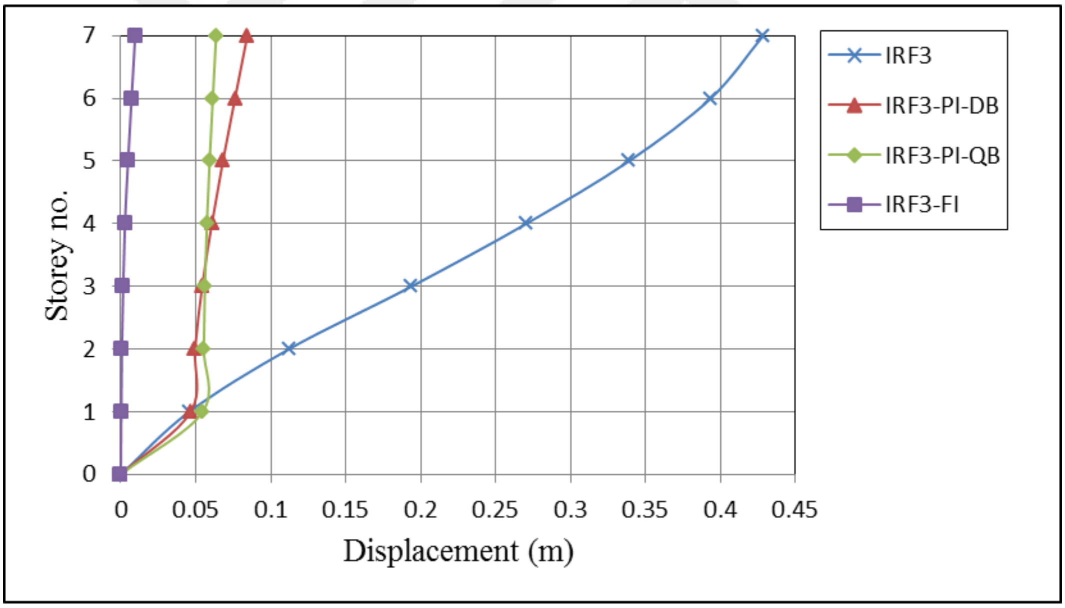


Figure 4.16 Peak displacement variations through frame height for IRF3 with and without infills

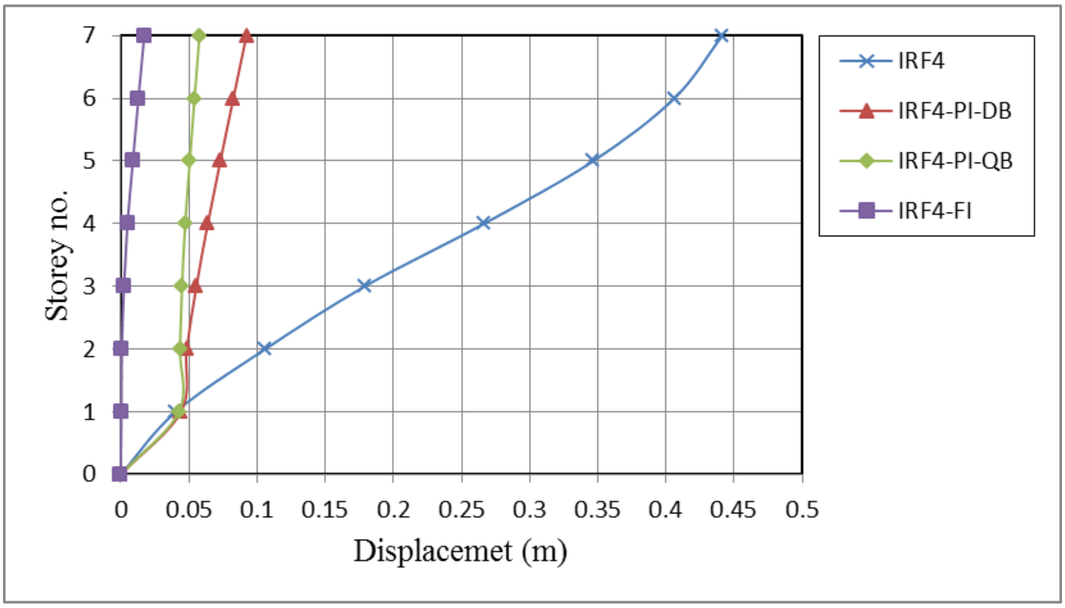


Figure 4.17 Peak displacement variations through frame height for IRF4 with and without infills

4.3.2 Story drift

Storey drift is the displacement of one level relative to the other level above or below. Drift also vary according to each direction. According to UBC 97, the story drift for buildings is limited to 0.020 times the story height, which was not exceeded in this analytical study for regular and irregular frame. Peak drift ratio obtained by nonlinear time history analysis for the regular (RF) and irregular (IRFs) frame system with and without infill walls are shown in Figure 4.18 to 4.22. There is a considerable difference in the drift ratio for the frame with and without infill walls, hence the lateral forces of the frame system with and without infill. Also considerable differ-ence is observed in the time period; natural frequency and storey drift of frame with and without masonry infill. It can be observed from the storey drift graphs represent-ed that the peak story drift of bare frame (frame without infill wall) is more than the frame with infill walls for all models (RF) and (IRF). It can also be observed that the stiffness is more at the 1st storey level as compared to the other stories for all the cases.

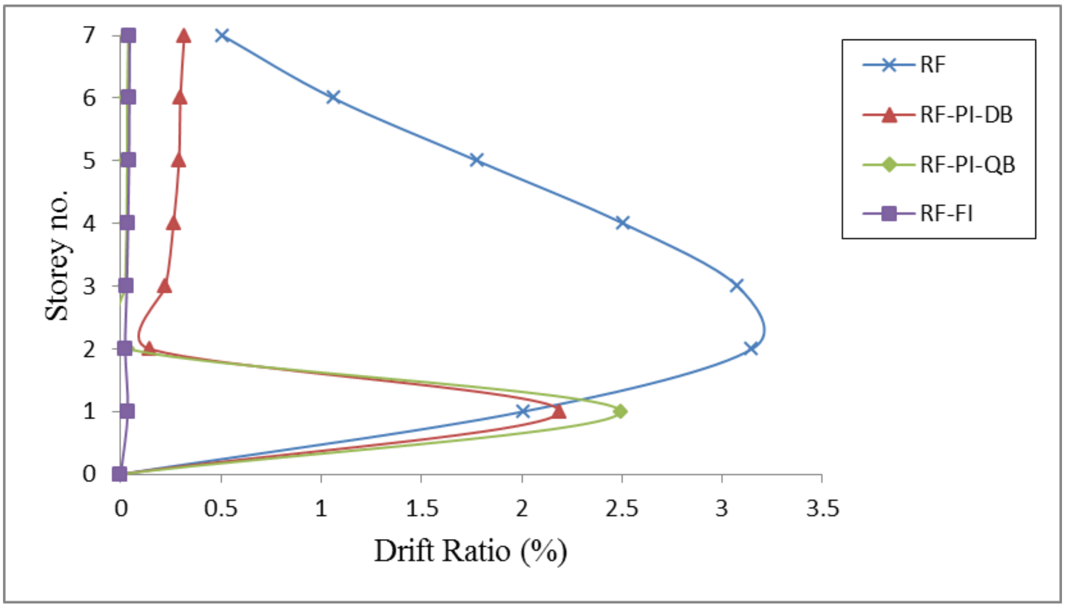


Figure 4.18 Peak drift ratio through frame height for RF with and without infills

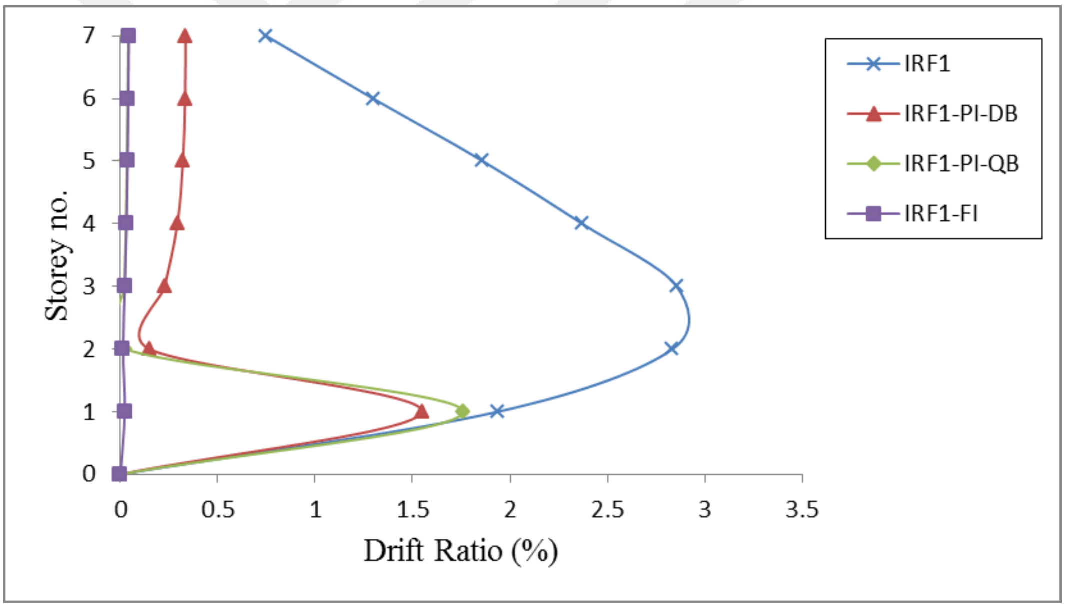


Figure 4.19 Peak drift ratio through frame height for IRF1 with and without infills

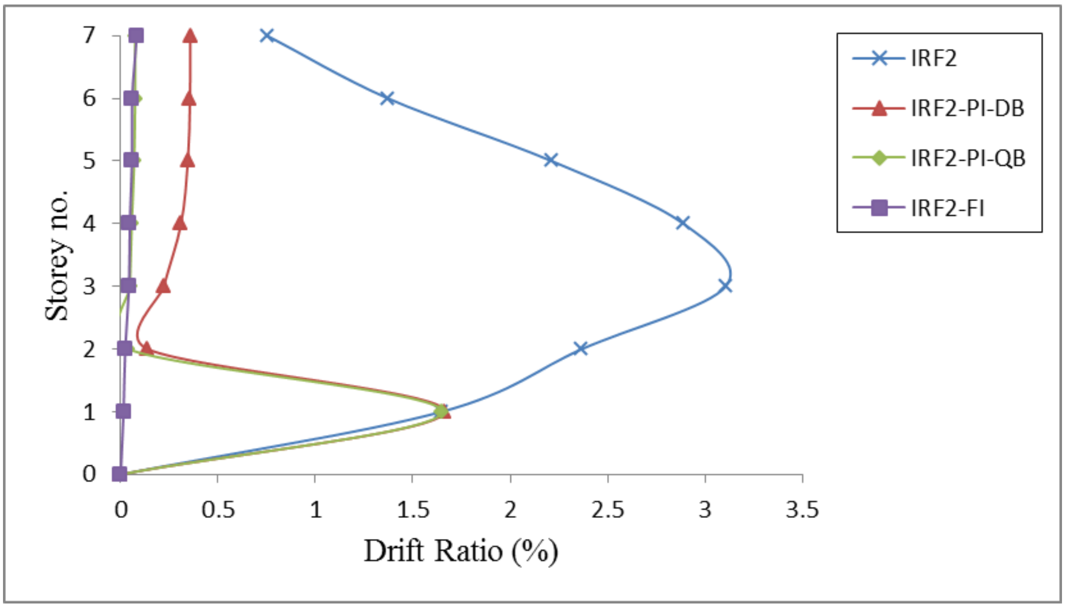


Figure 4.20 Peak drift ratio through frame height for IRF2 with and without infills

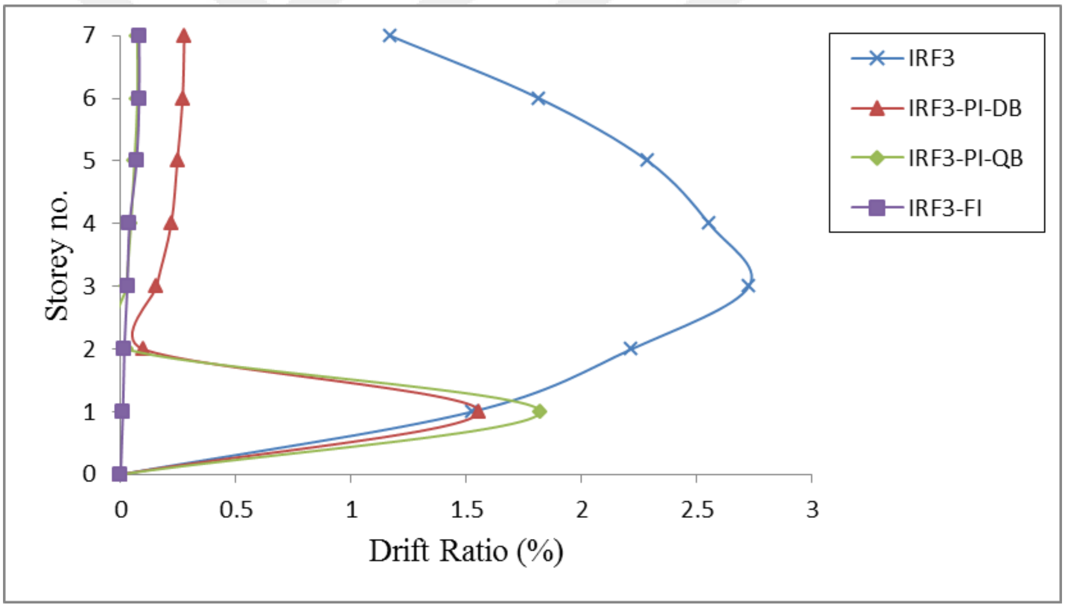


Figure 4.21 Peak drift ratio through frame height for IRF3 with and without infills

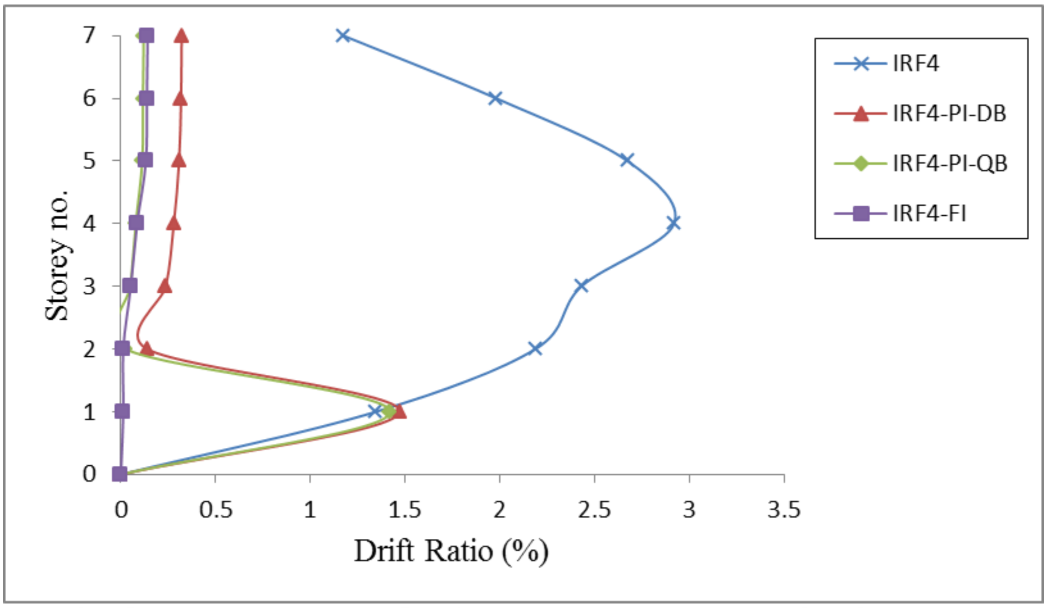
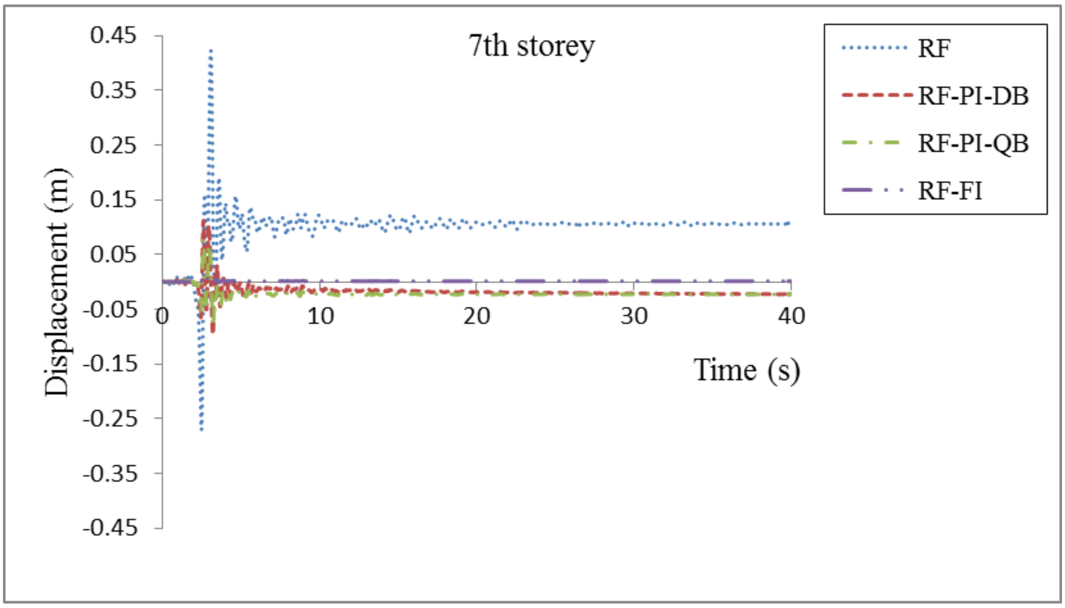


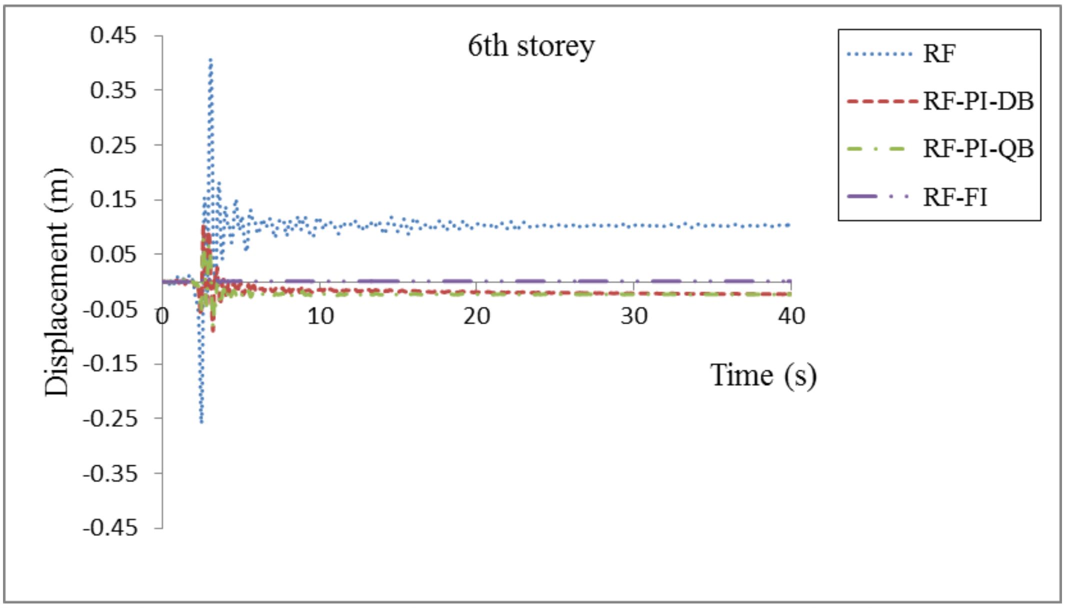
Figure 4.22 Peak drift ratio through frame height for IRF4 with and without infills

4.3.3 Time history plots for displacement

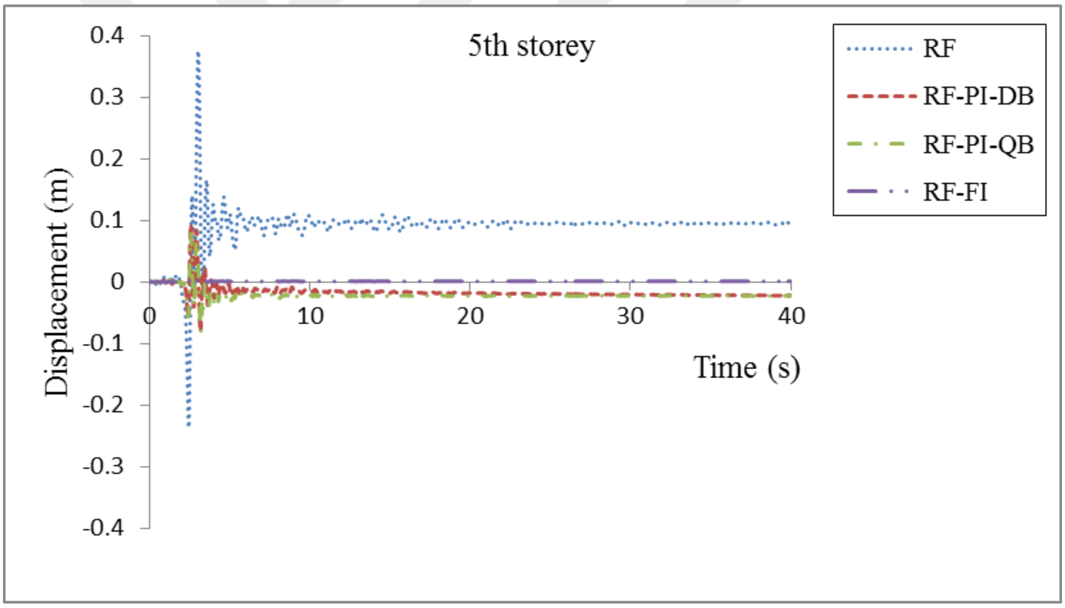
The nonlinear time history of displacements shown in Figures 4.123 to 4.27 indicates a general fluctuation of top lateral displacement responses for all of the models and storeys.



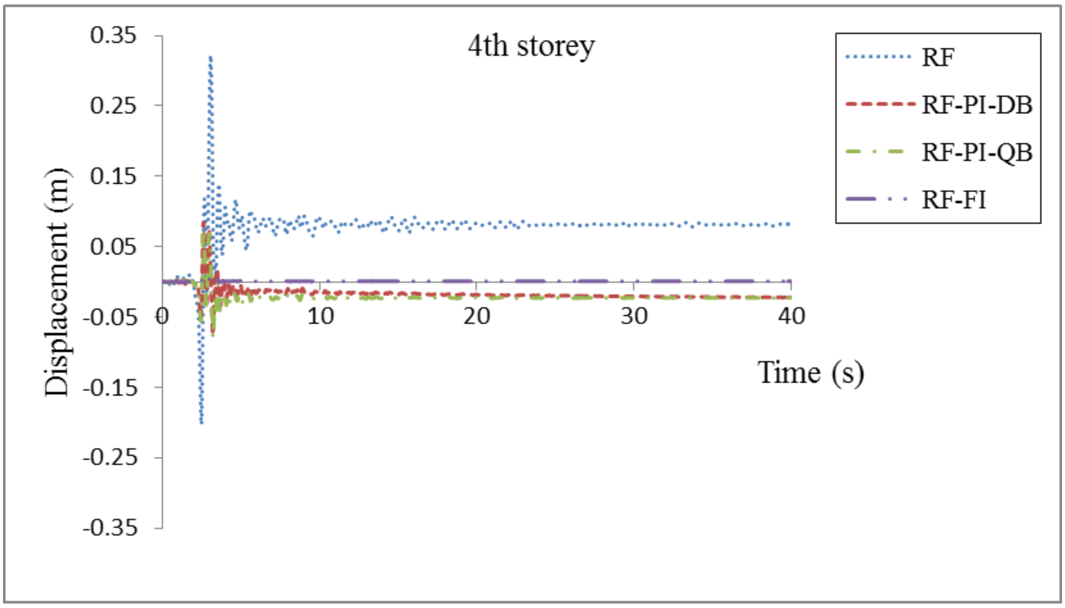
a)


6th storey
0.45
0.35
0.25
0.15
0.05
-0.05
-0.15
-0.25
-0.35
-0.45
Displacement (m)
0
10
20
30
40
Time (s)
RF
RF-PI-DB
RF-PI-QB
RF-FI

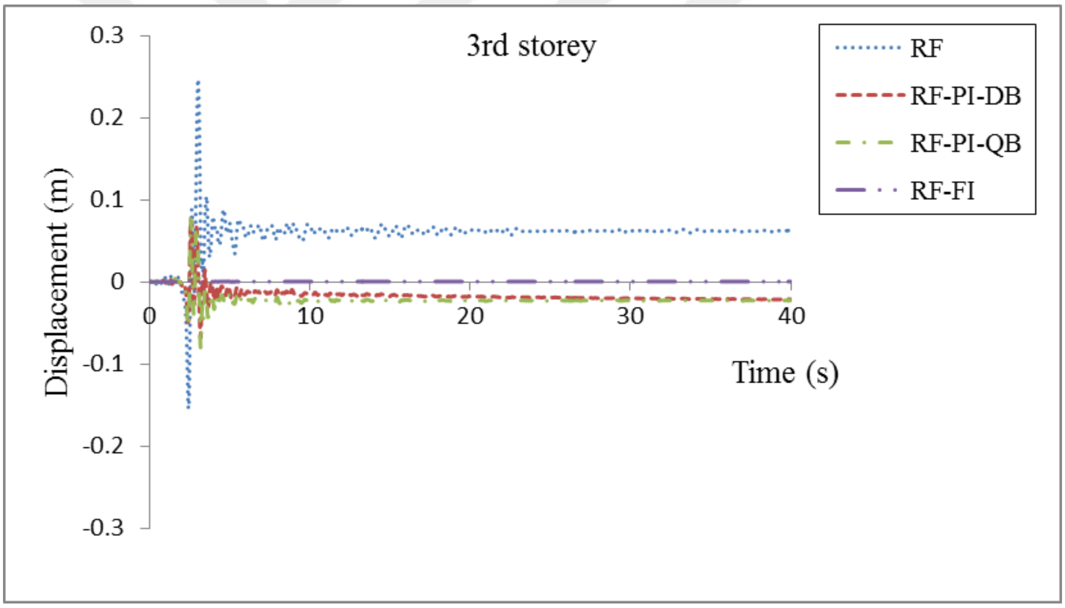
b)


5th storey
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
Displacement (m)
0
10
20
30
40
Time (s)
RF
RF-PI-DB
RF-PI-QB
RF-FI

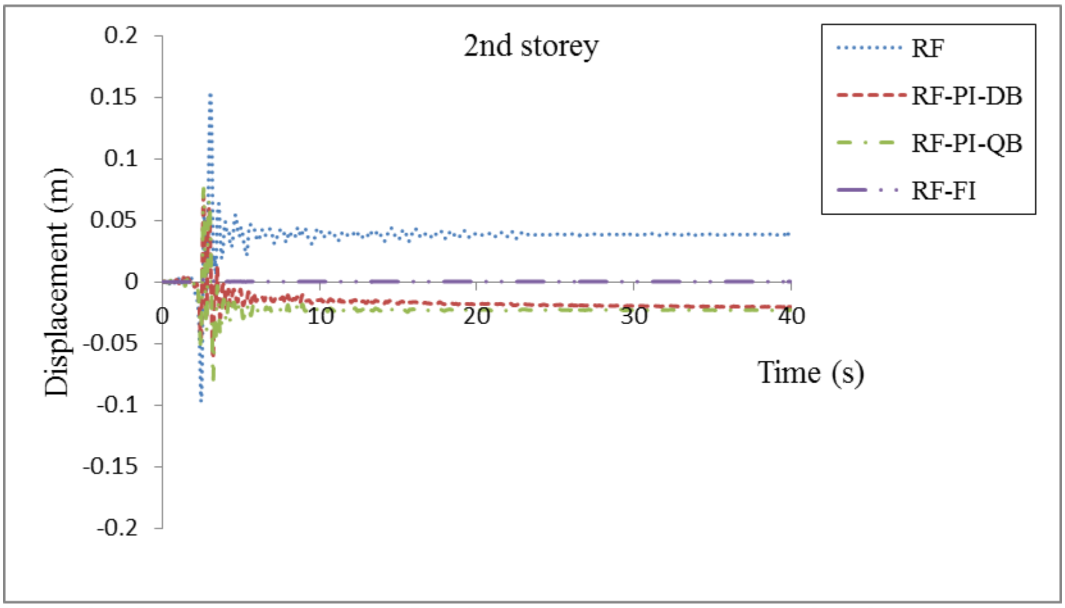
c)


4th storey
0.35
0.25
0.15
0.05
-0.05
-0.15
-0.25
-0.35
Displacement (m)
0
10
20
30
40
Time (s)
RF
RF-PI-DB
RF-PI-QB
RF-FI

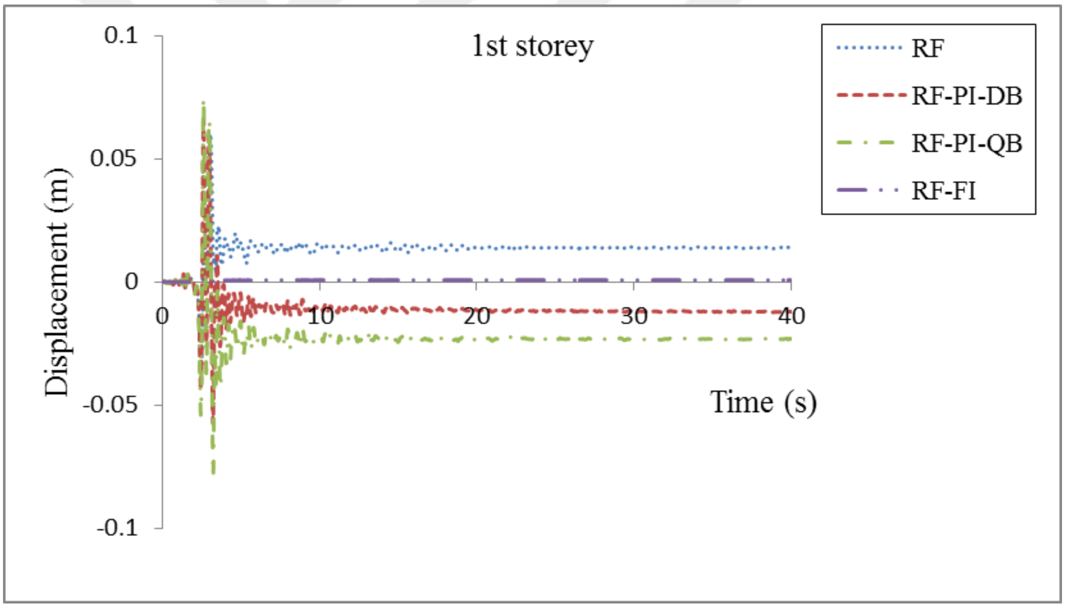
d)


3rd storey
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
Displacement (m)
0
10
20
30
40
Time (s)
RF
RF-PI-DB
RF-PI-QB
RF-FI

e)

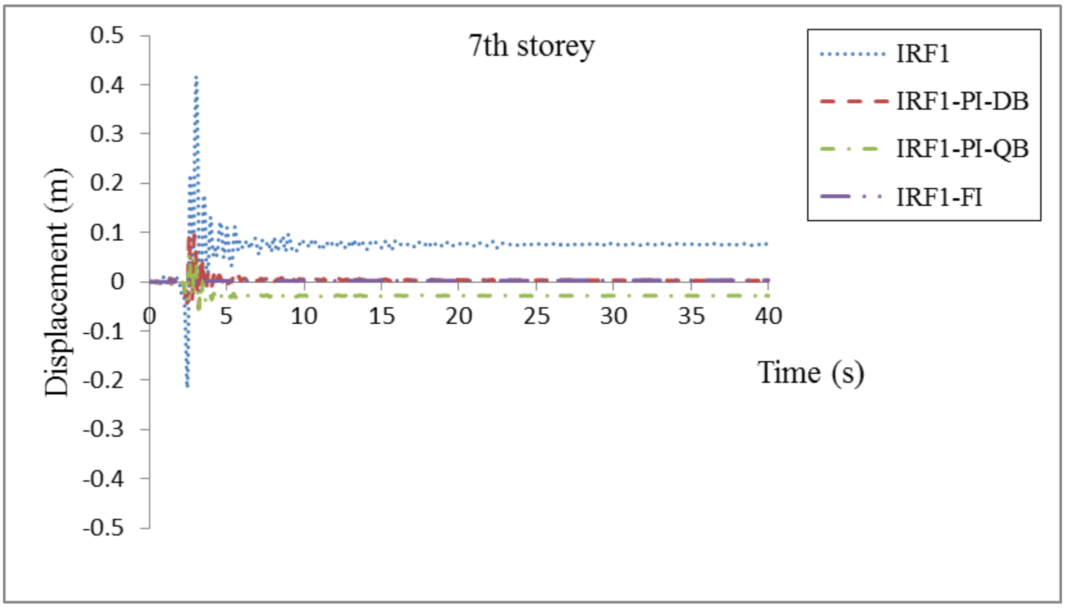


f)

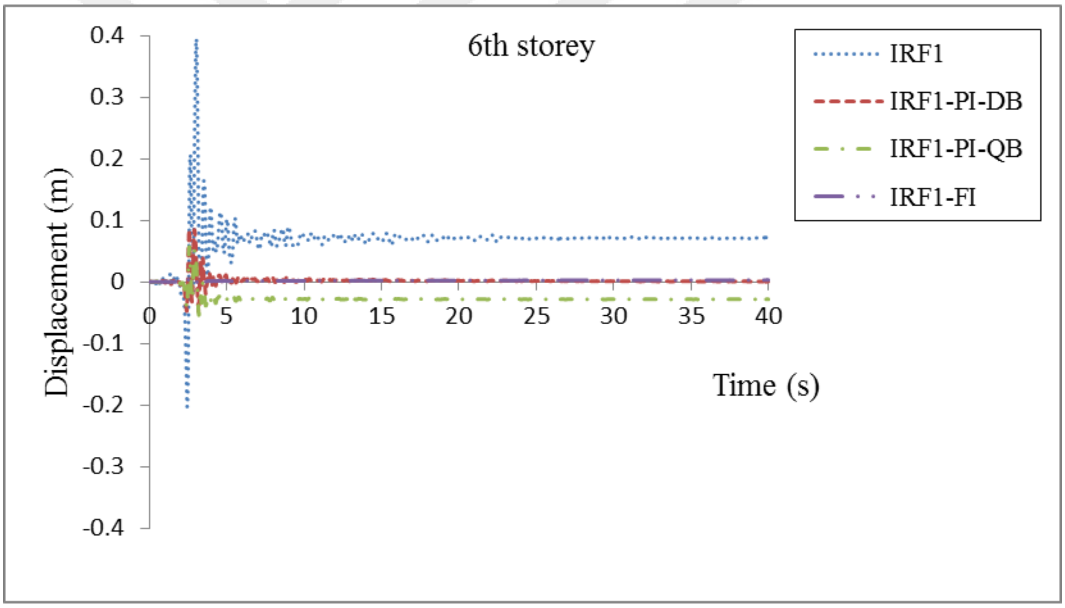


g)

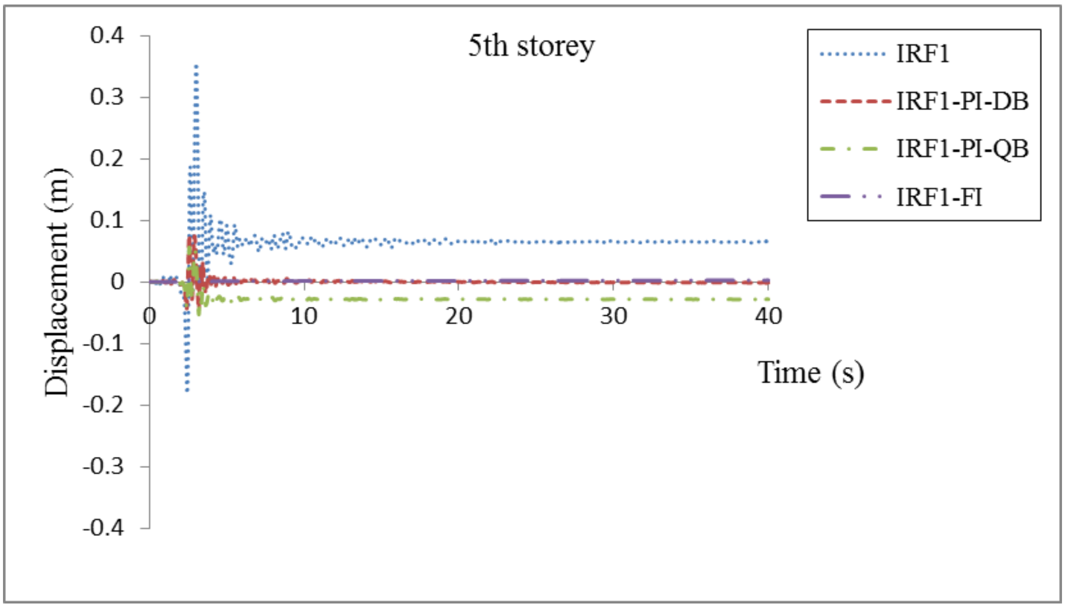
Figure 4.23 Displacement time history of RF with and without infills


7th storey
IRF1
IRF1-PI-DB
IRF1-PI-QB
IRF1-FI
Displacement (m)
Time (s)
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
0
5
10
15
20
25
30
35
40

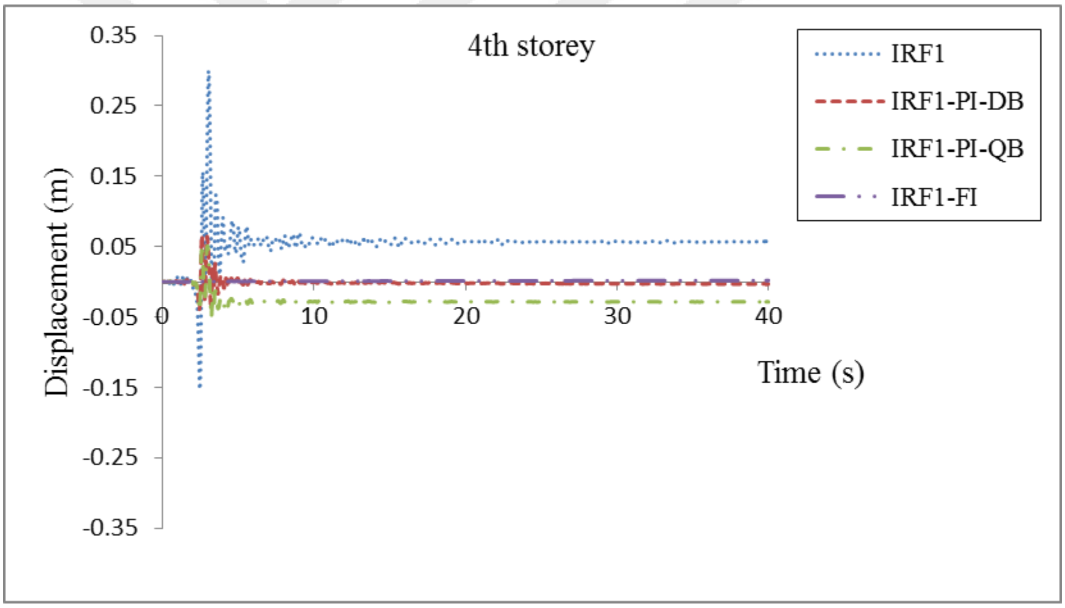
a)


6th storey
IRF1
IRF1-PI-DB
IRF1-PI-QB
IRF1-FI
Displacement (m)
Time (s)
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
0
5
10
15
20
25
30
35
40

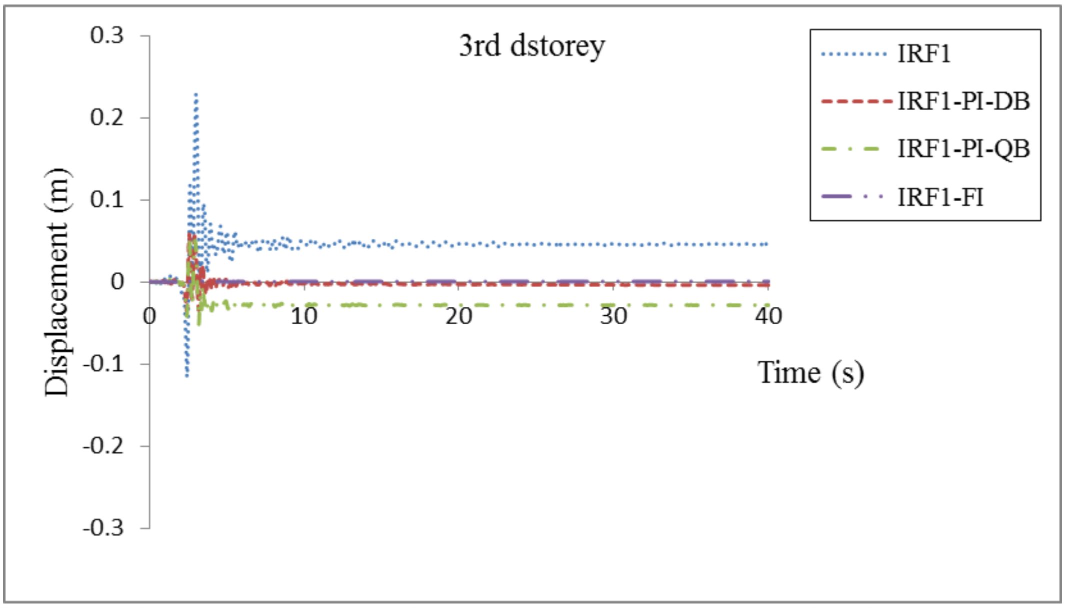
b)


5th storey
IRF1
IRF1-PI-DB
IRF1-PI-QB
IRF1-FI
Displacement (m)
Time (s)
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
0
10
20
30
40

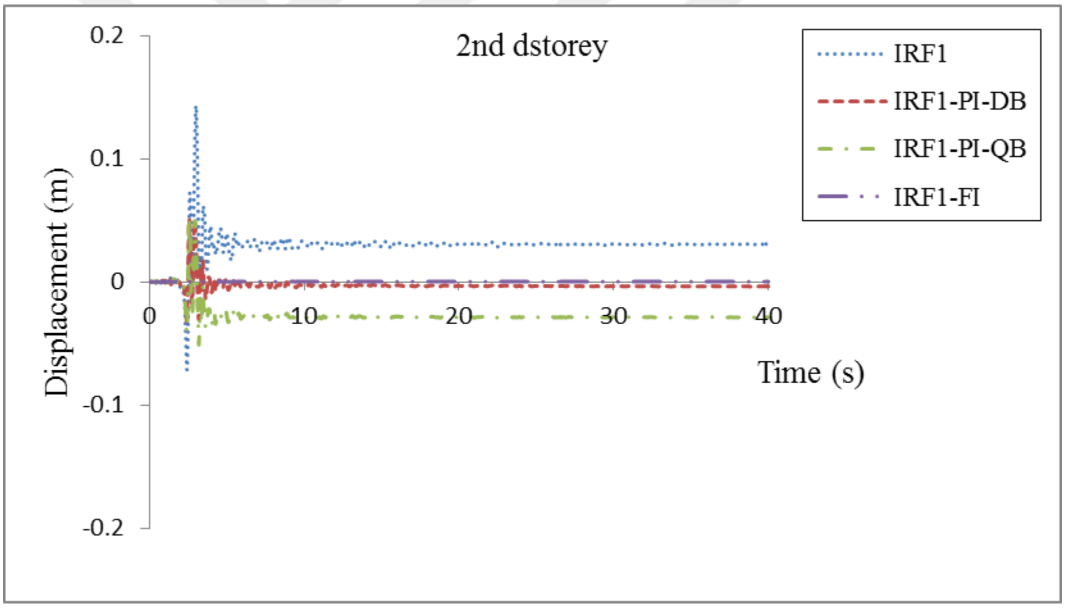
c)


4th storey
IRF1
IRF1-PI-DB
IRF1-PI-QB
IRF1-FI
Displacement (m)
Time (s)
0.35
0.25
0.15
0.05
-0.05
-0.15
-0.25
-0.35
0
10
20
30
40

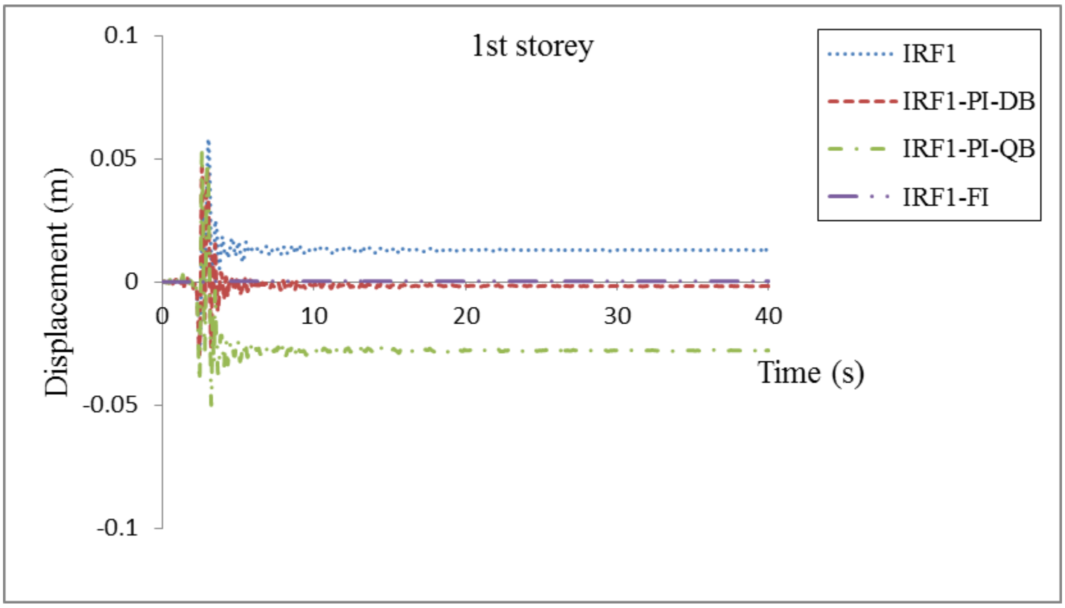
d)


3rd dstorey
IRF1
IRF1-PI-DB
IRF1-PI-QB
IRF1-FI
Displacement (m)
Time (s)
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
0
10
20
30
40

e)

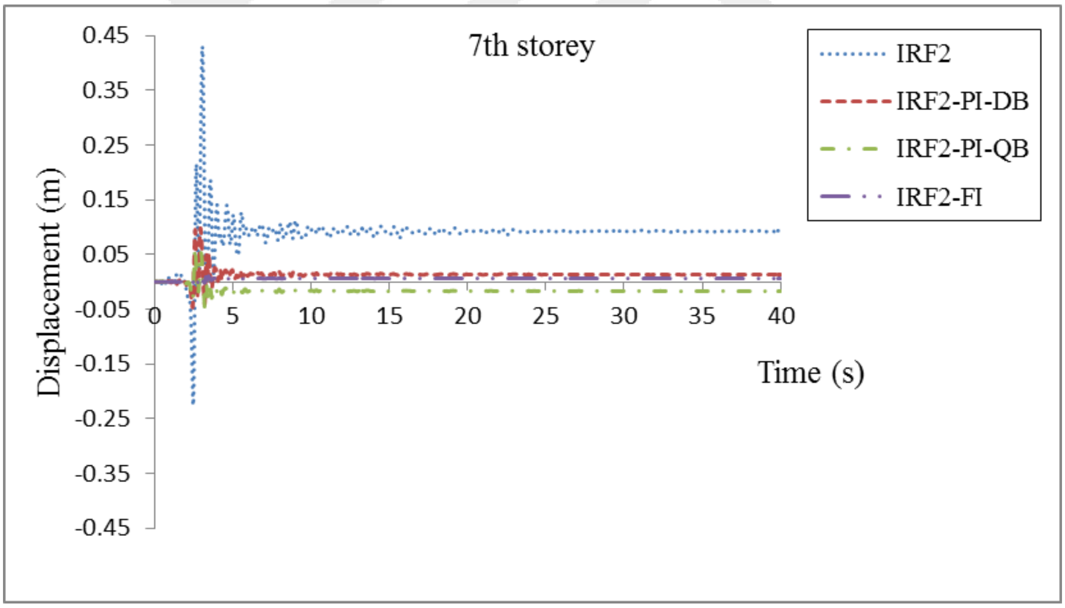

2nd dstorey
IRF1
IRF1-PI-DB
IRF1-PI-QB
IRF1-FI
Displacement (m)
Time (s)
0.2
0.1
0
-0.1
-0.2
0
10
20
30
40

f)

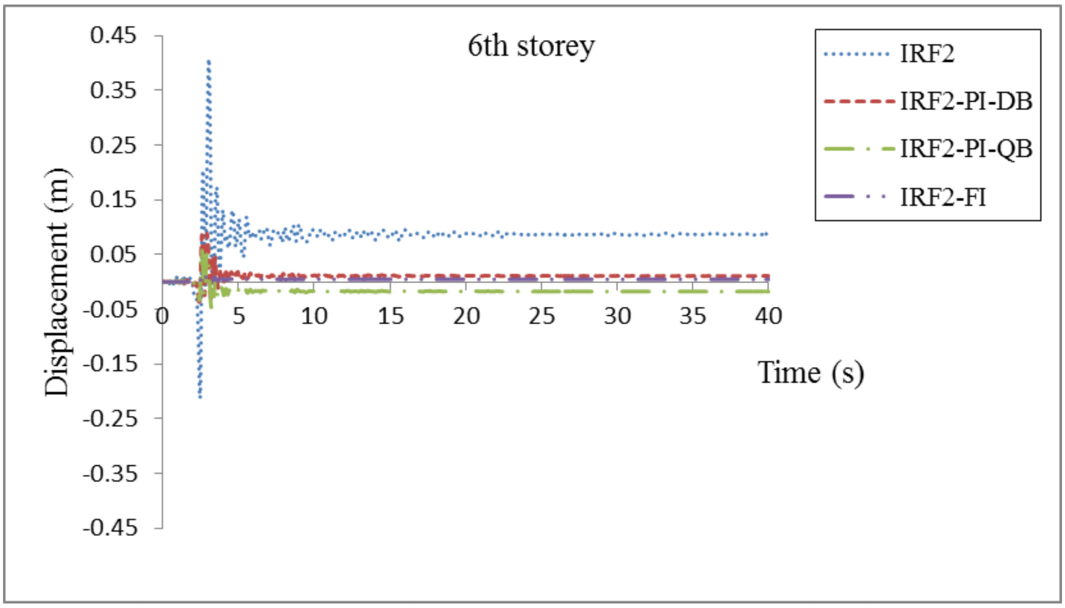


g)

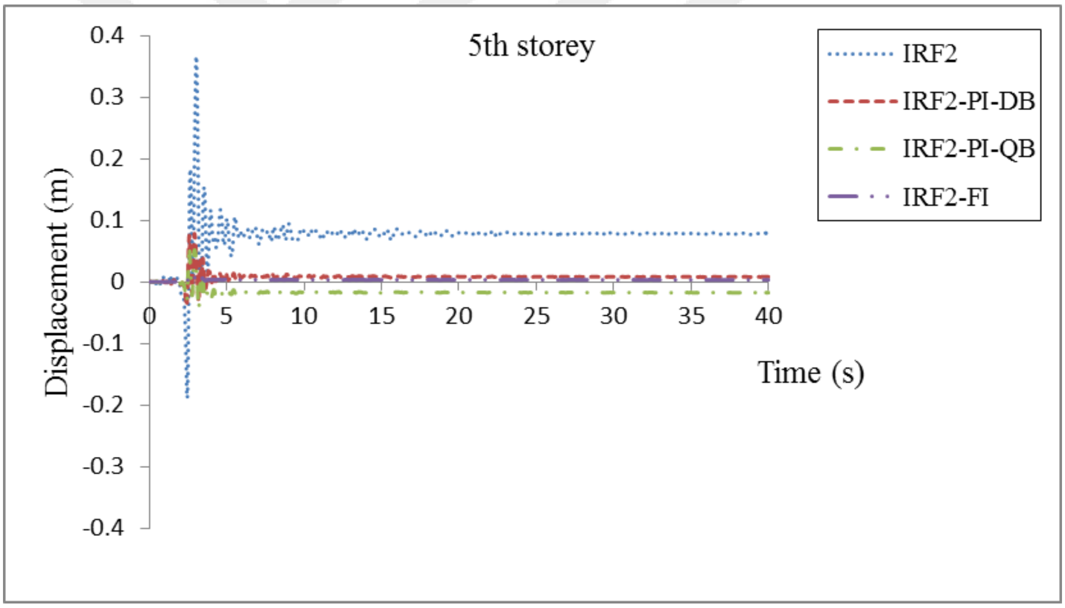
Figure 4.24 Displacement time history of IRF1 with and without infills



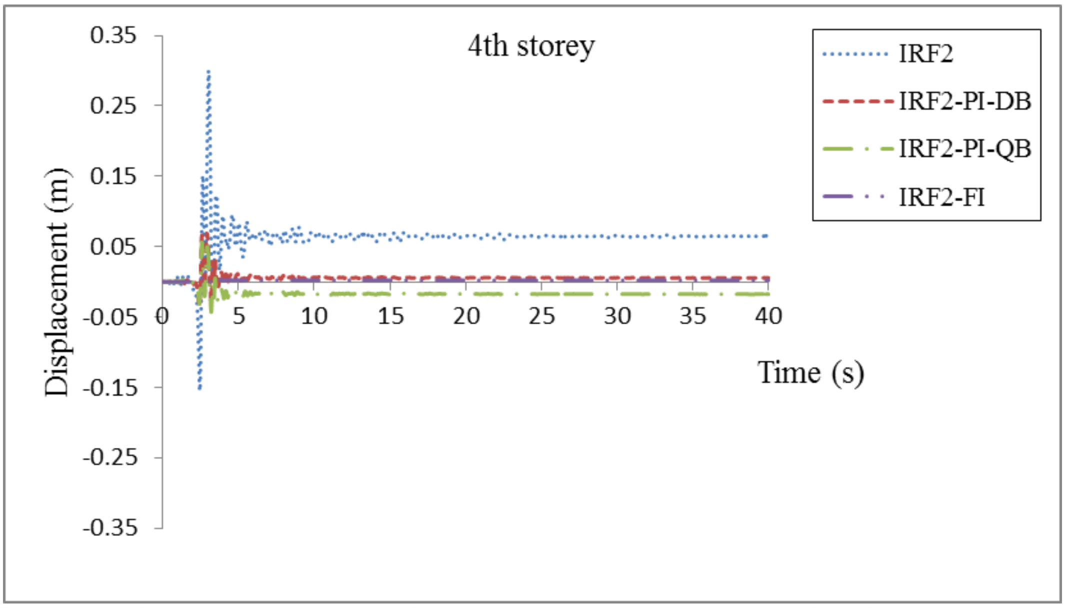
a)


6th storey
IRF2
IRF2-PI-DB
IRF2-PI-QB
IRF2-FI
Displacement (m)
Time (s)
0.45
0.35
0.25
0.15
0.05
-0.05
-0.15
-0.25
-0.35
-0.45
0
5
10
15
20
25
30
35
40

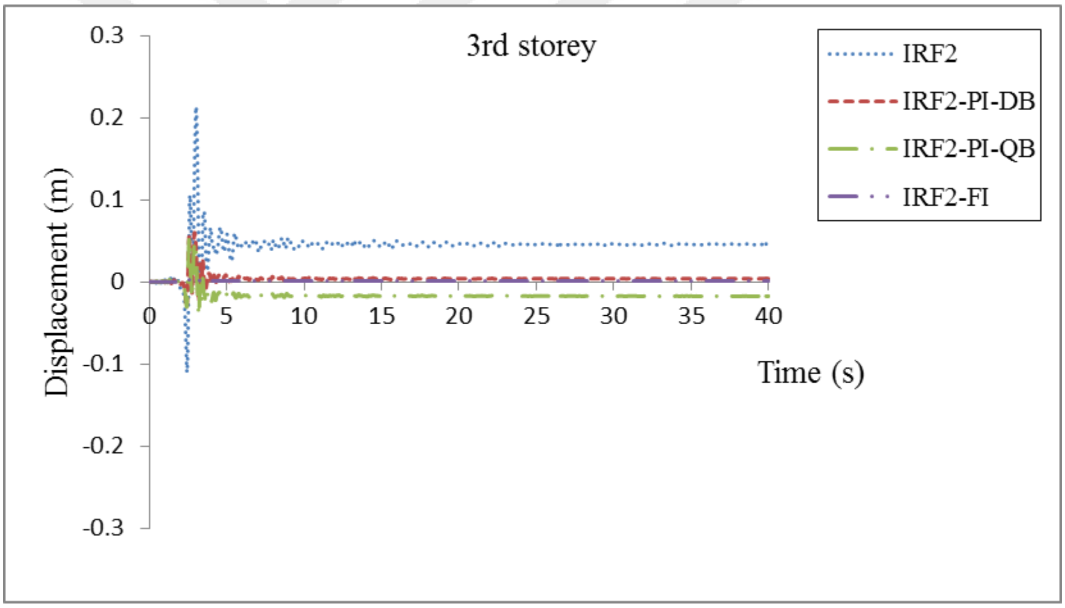
b)


5th storey
IRF2
IRF2-PI-DB
IRF2-PI-QB
IRF2-FI
Displacement (m)
Time (s)
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
0
5
10
15
20
25
30
35
40

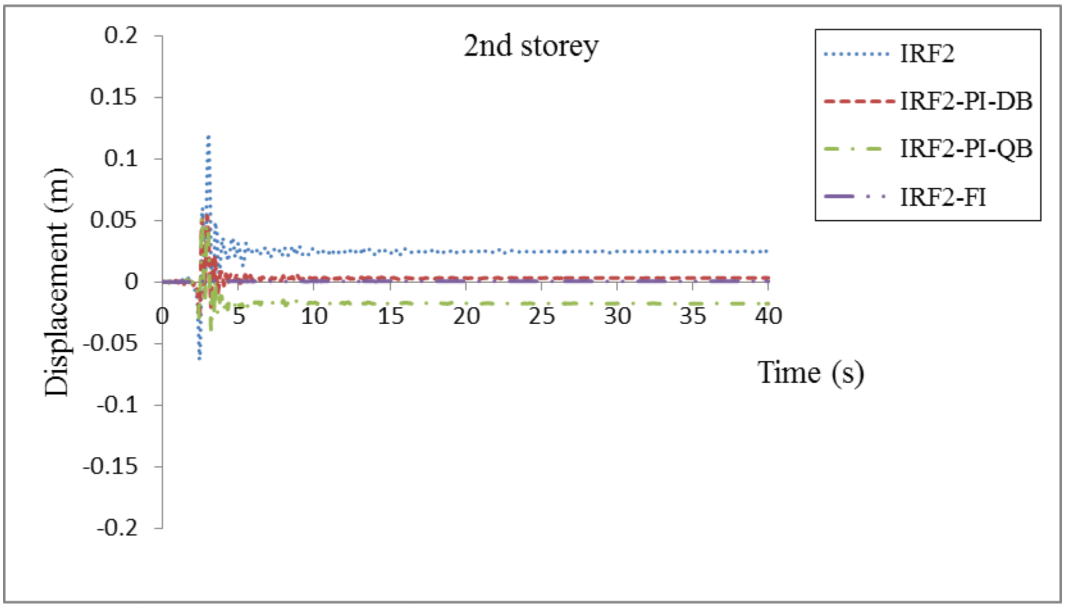
c)


4th storey
IRF2
IRF2-PI-DB
IRF2-PI-QB
IRF2-FI
Displacement (m)
Time (s)
0.35
0.25
0.15
0.05
-0.05
-0.15
-0.25
-0.35
0
5
10
15
20
25
30
35
40

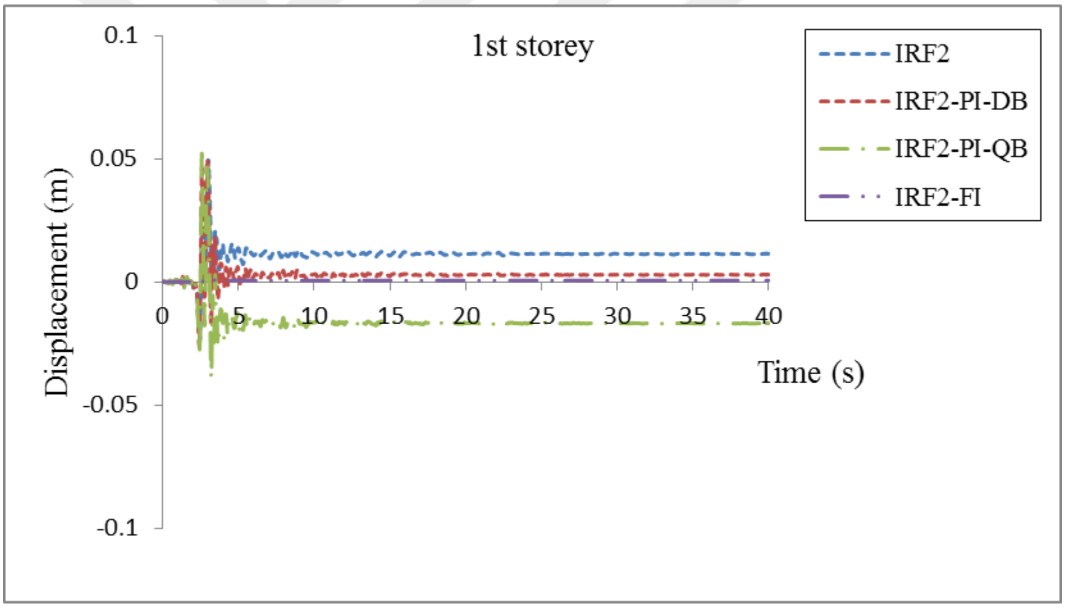
d)


3rd storey
IRF2
IRF2-PI-DB
IRF2-PI-QB
IRF2-FI
Displacement (m)
Time (s)
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
0
5
10
15
20
25
30
35
40

e)

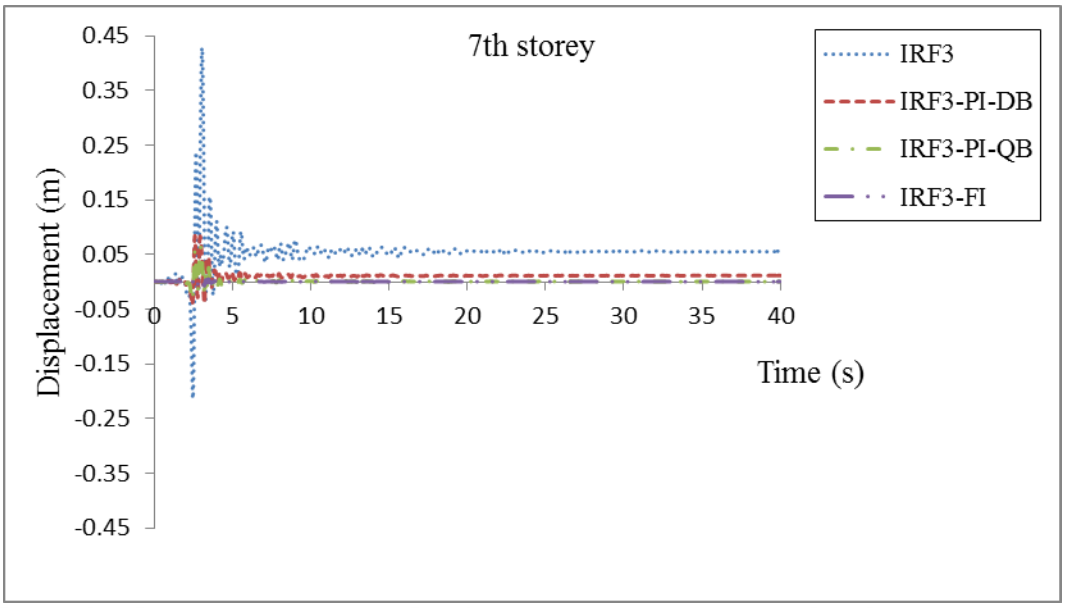


f)

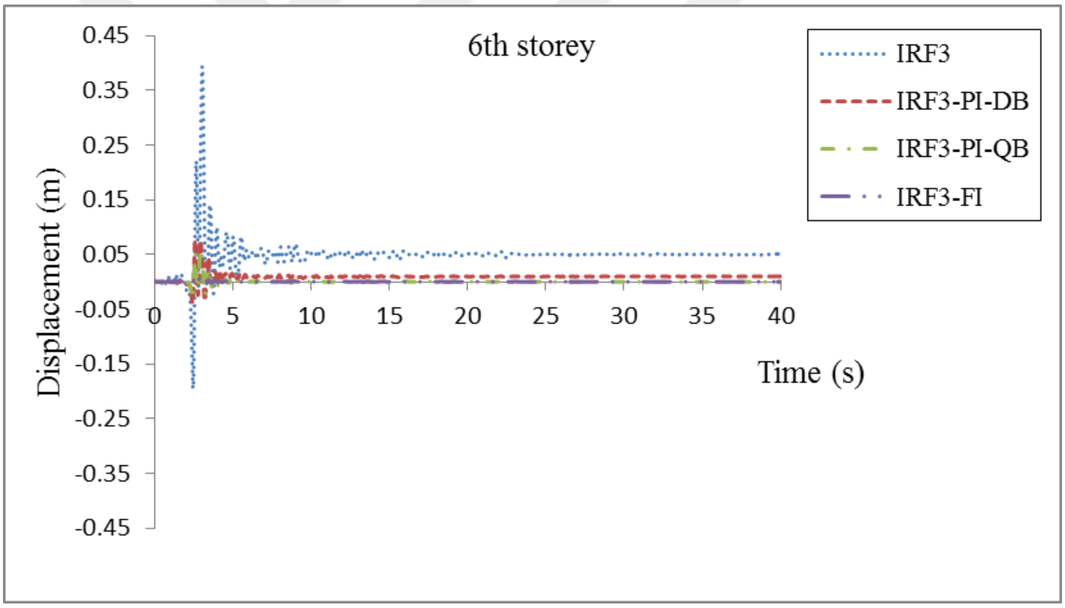


g)

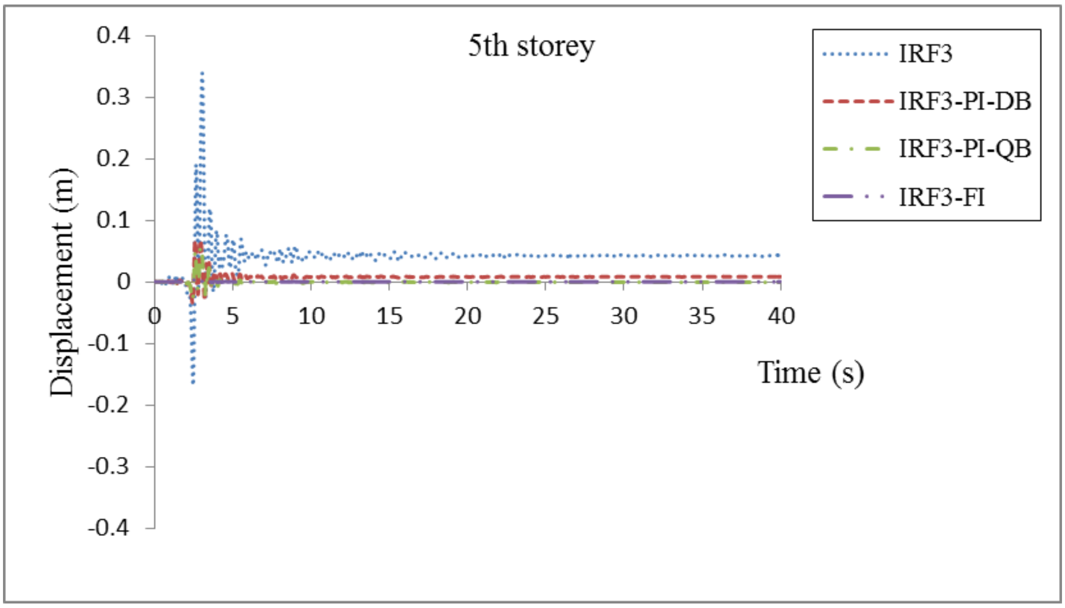
Figure 4.25 Displacement time history of IRF2 with and without infills


7th storey
IRF3
IRF3-PI-DB
IRF3-PI-QB
IRF3-FI
Displacement (m)
0.45
0.35
0.25
0.15
0.05
-0.05
-0.15
-0.25
-0.35
-0.45
0
5
10
15
20
25
30
35
40
Time (s)

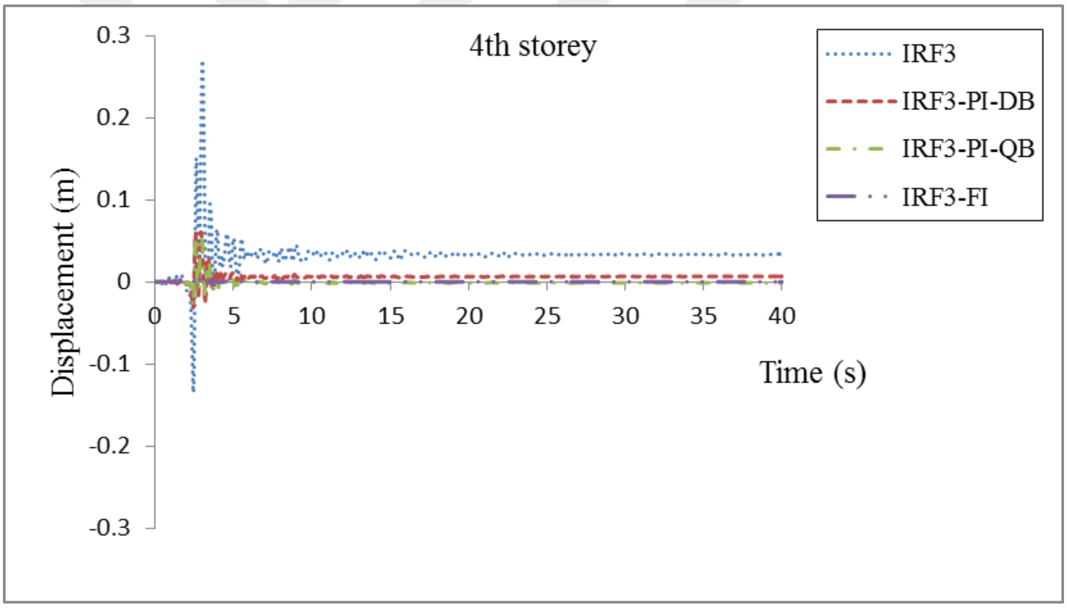
a)


6th storey
IRF3
IRF3-PI-DB
IRF3-PI-QB
IRF3-FI
Displacement (m)
0.45
0.35
0.25
0.15
0.05
-0.05
-0.15
-0.25
-0.35
-0.45
0
5
10
15
20
25
30
35
40
Time (s)

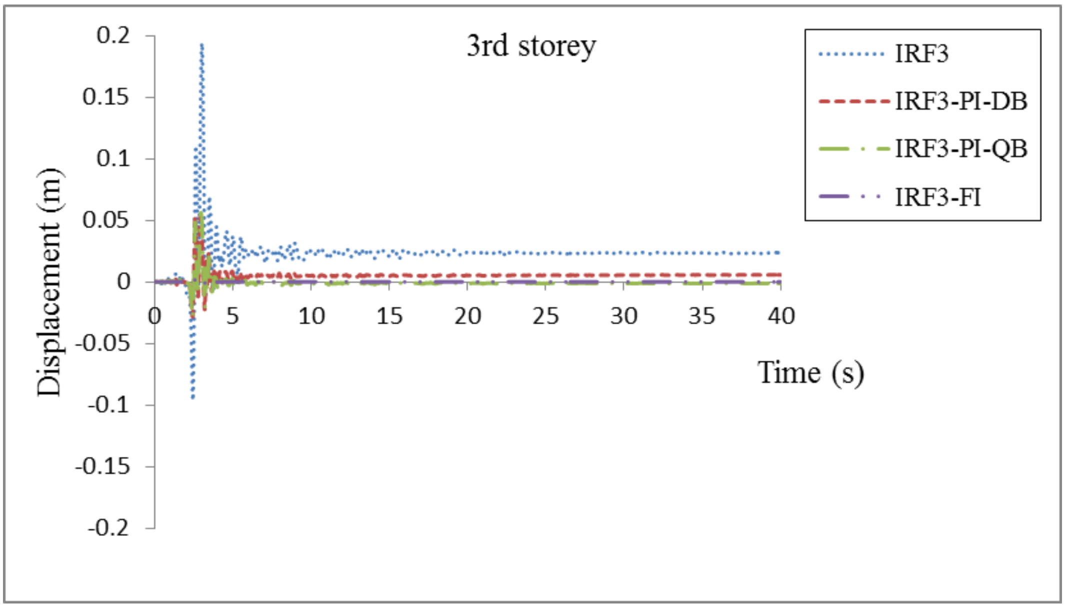
b)


5th storey
IRF3
IRF3-PI-DB
IRF3-PI-QB
IRF3-FI
Displacement (m)
Time (s)
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
0
5
10
15
20
25
30
35
40

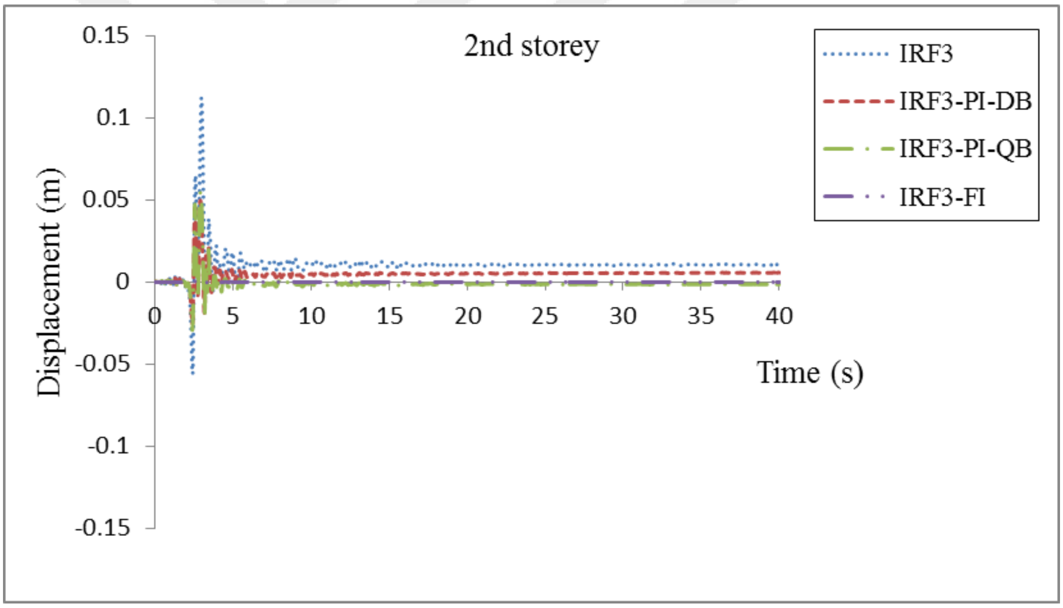
c)


4th storey
IRF3
IRF3-PI-DB
IRF3-PI-QB
IRF3-FI
Displacement (m)
Time (s)
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
0
5
10
15
20
25
30
35
40

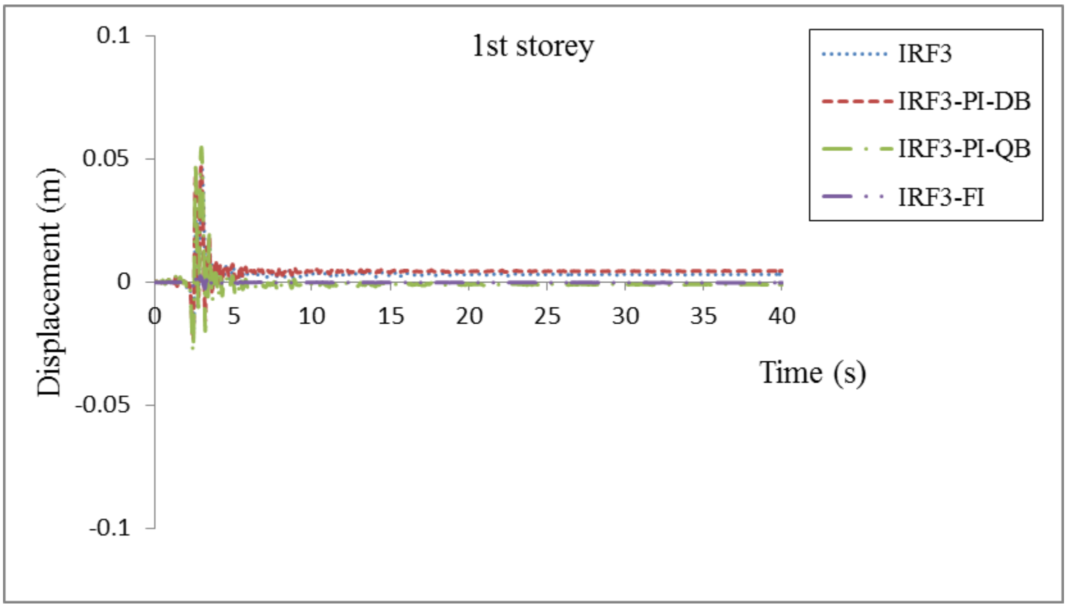
d)


3rd storey
IRF3
IRF3-PI-DB
IRF3-PI-QB
IRF3-FI
Displacement (m)
Time (s)
0.2
0.15
0.1
0.05
0
-0.05
-0.1
-0.15
-0.2
0
5
10
15
20
25
30
35
40

e)

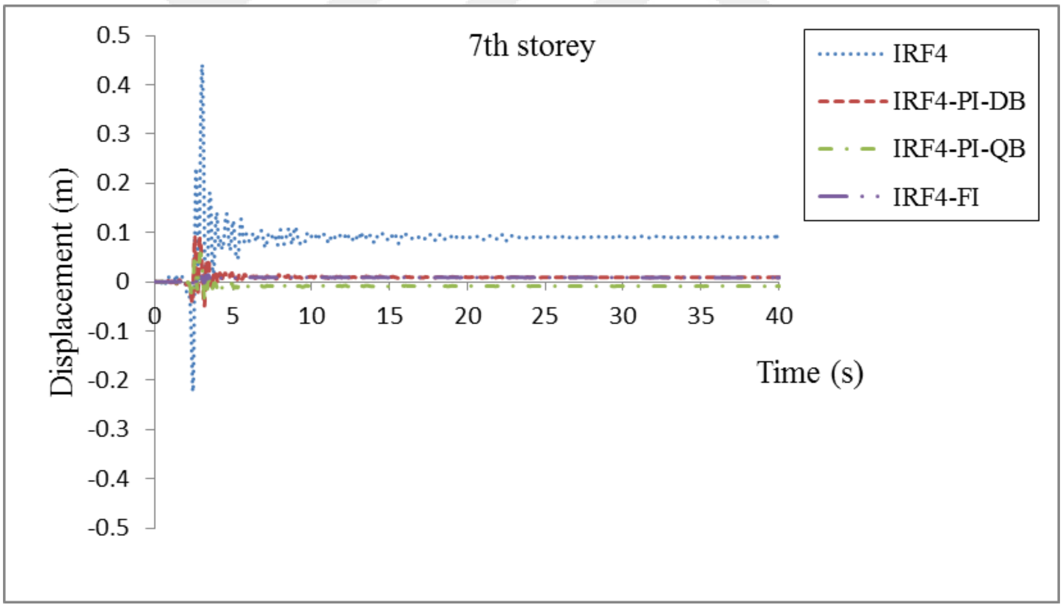

2nd storey
IRF3
IRF3-PI-DB
IRF3-PI-QB
IRF3-FI
Displacement (m)
Time (s)
0.15
0.1
0.05
0
-0.05
-0.1
-0.15
0
5
10
15
20
25
30
35
40

f)

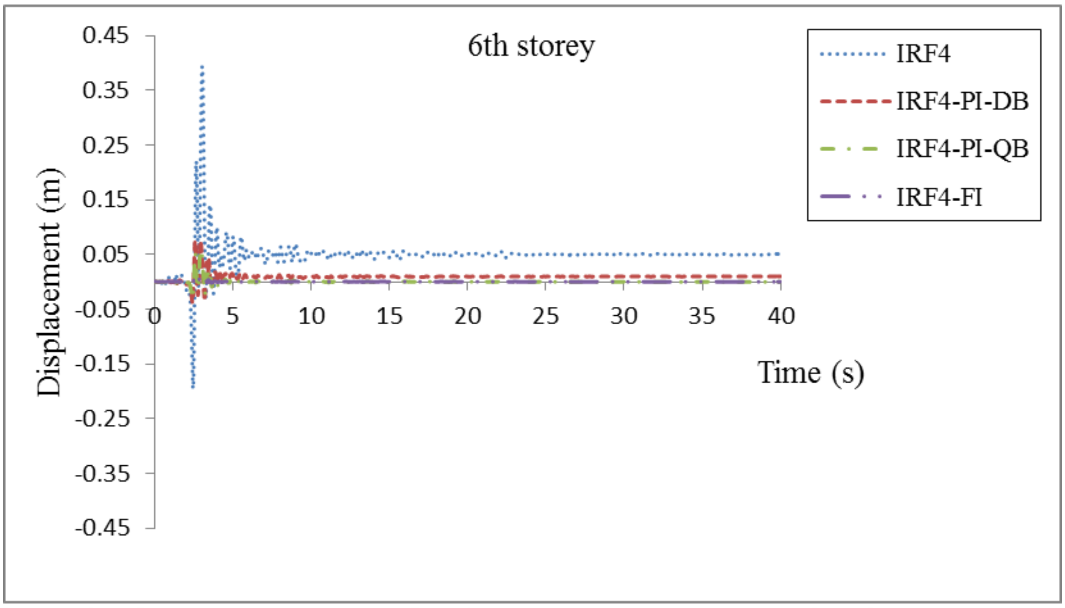


g)

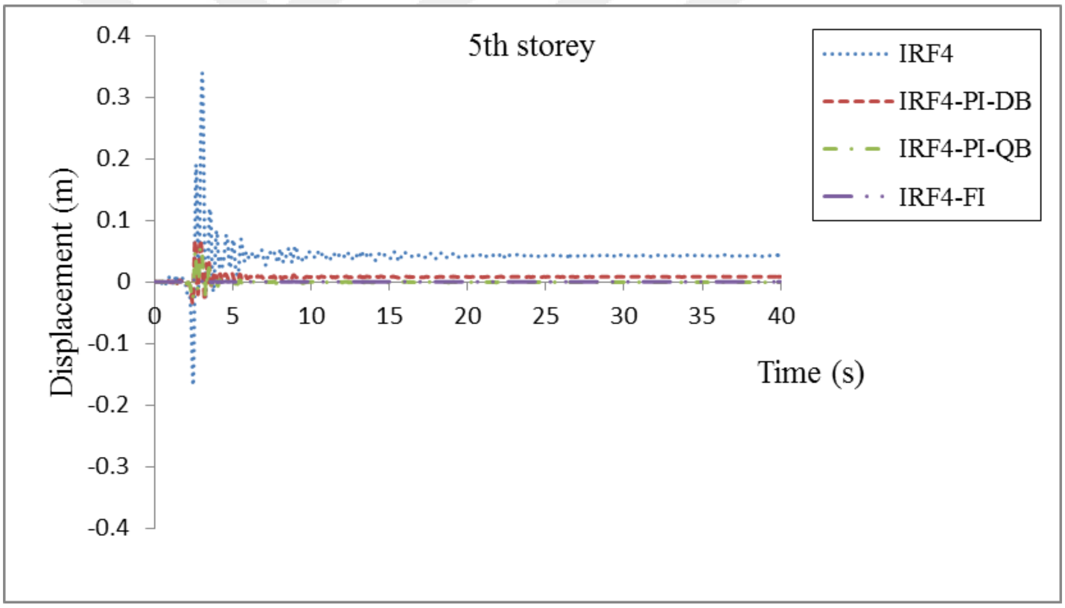
Figure 4.26 Displacement time history of IRF3 with and without infills



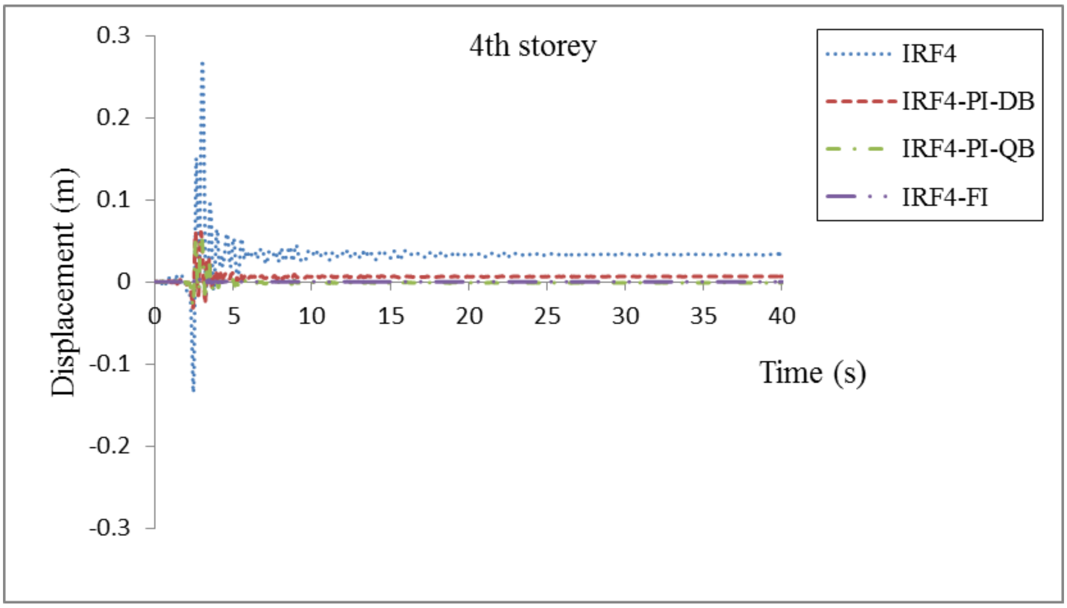
a)


6th storey
IRF4
IRF4-PI-DB
IRF4-PI-QB
IRF4-FI
Displacement (m)
Time (s)
0.45
0.35
0.25
0.15
0.05
-0.05
-0.15
-0.25
-0.35
-0.45
0
5
10
15
20
25
30
35
40

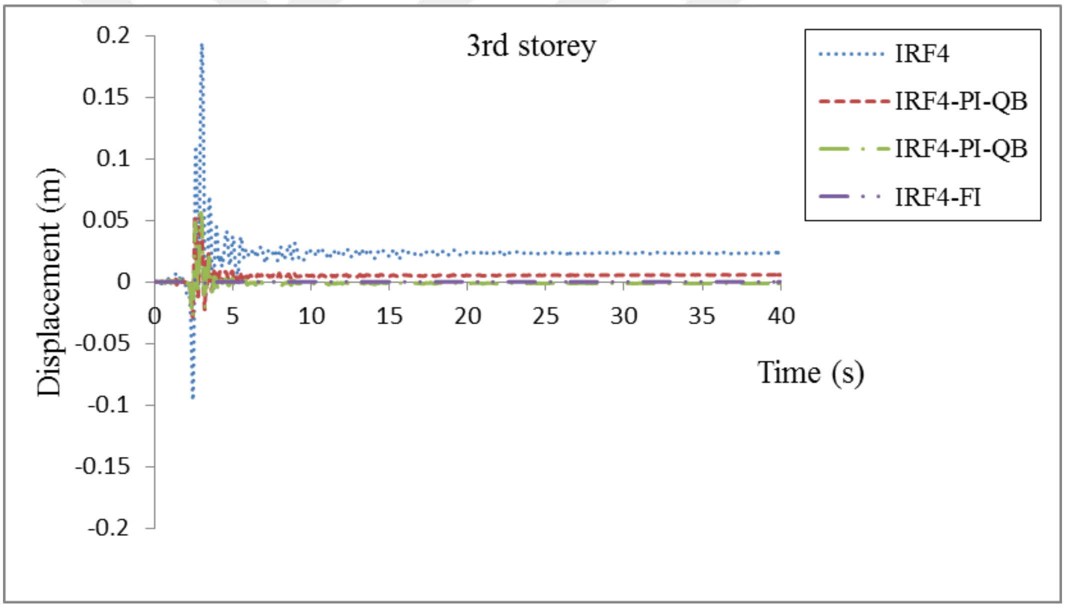
b)


5th storey
IRF4
IRF4-PI-DB
IRF4-PI-QB
IRF4-FI
Displacement (m)
Time (s)
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
0
5
10
15
20
25
30
35
40

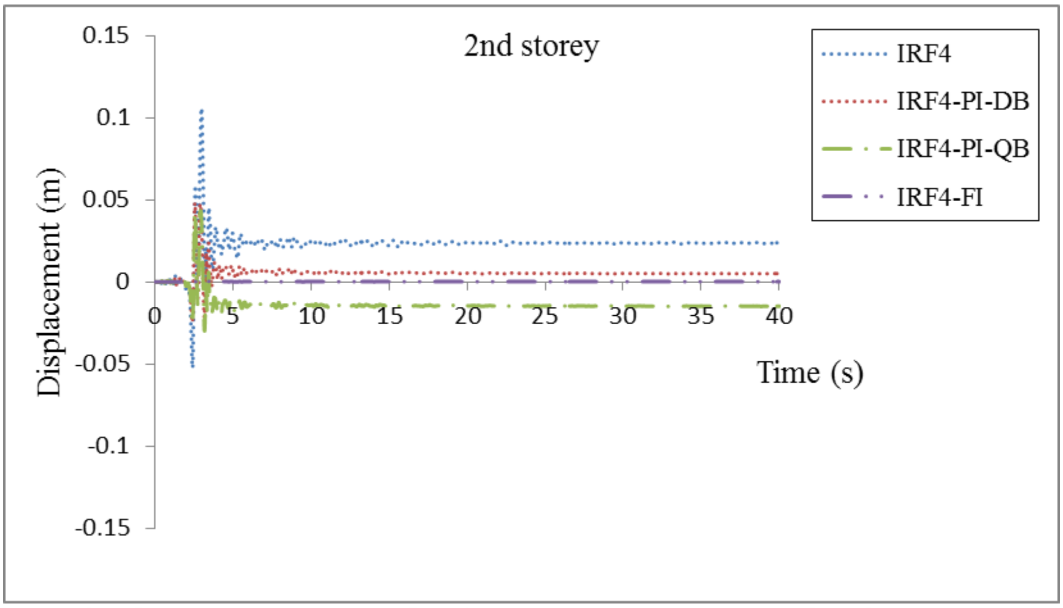
c)


4th storey
Displacement (m)
Time (s)
IRF4
IRF4-PI-DB
IRF4-PI-QB
IRF4-FI
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
0 5 10 15 20 25 30 35 40

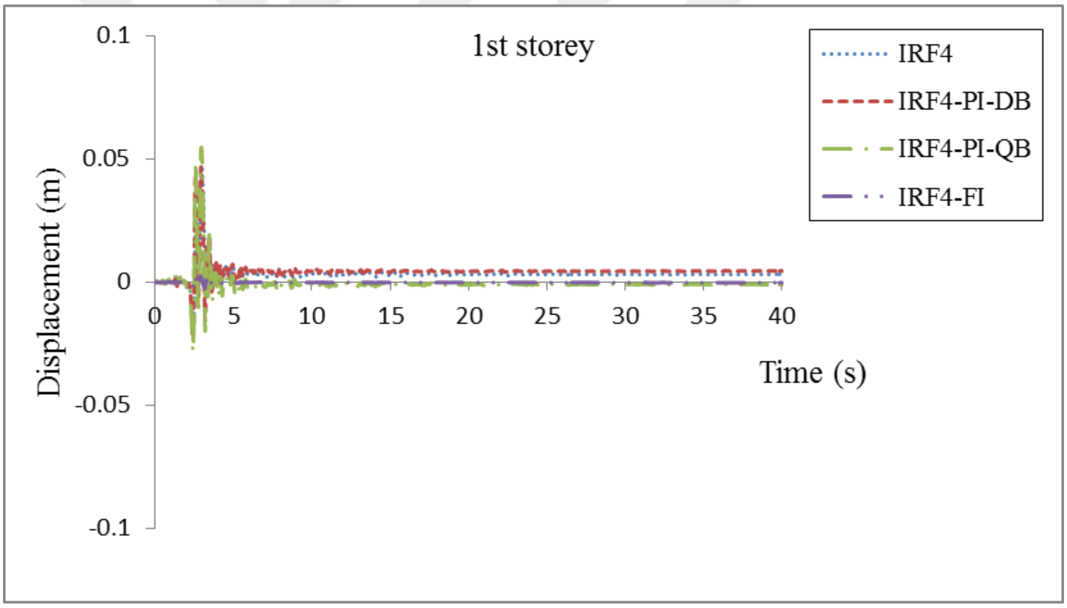
d)


3rd storey
Displacement (m)
Time (s)
IRF4
IRF4-PI-QB
IRF4-PI-QB
IRF4-FI
0.2
0.15
0.1
0.05
0
-0.05
-0.1
-0.15
-0.2
0 5 10 15 20 25 30 35 40

e)



f)

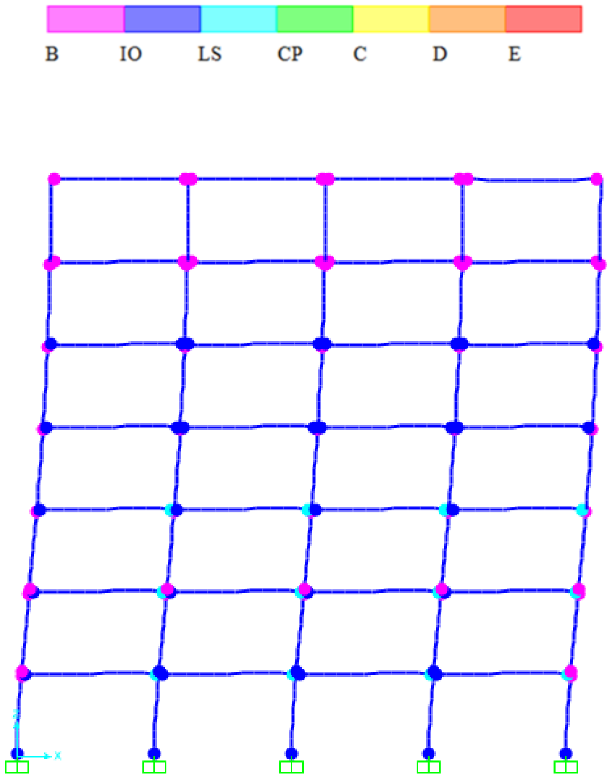


g)

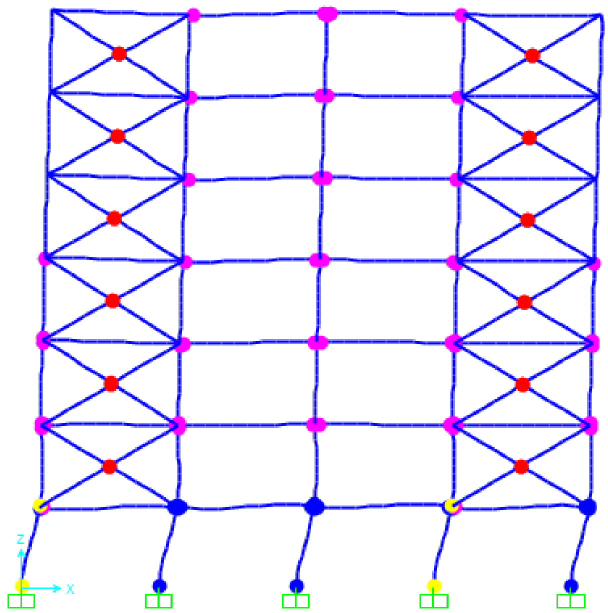
Figure 4.27 Displacement time history of IRF4 with and without infills

4.3.4 Plastic hinge distribution from nonlinear time history analysis

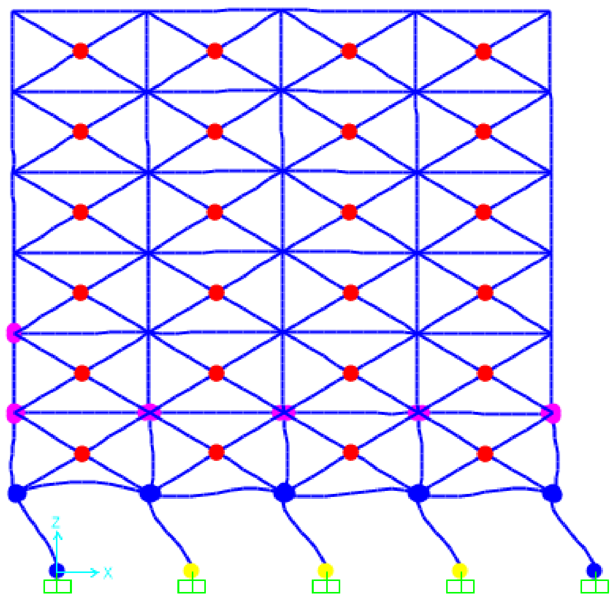
Masonry infill wall caused changing of distribution of plastic hinge and in some case changing of seismic performance levels. Also different infill model can develop different distribution of plastic hinge. For example, generally in regular frame and irregular frame with infill seismic performance level is IO which with comparing with frames without masonry infill. Masonry infille has positive effect and caused performance level as shown in Figure 4.28 to 4.32 respectively. With comparing the plastic hinge distribution of frame without infill walls at the target displacement of infilled frame, the negative effect of masonry infille on around frame is observed.



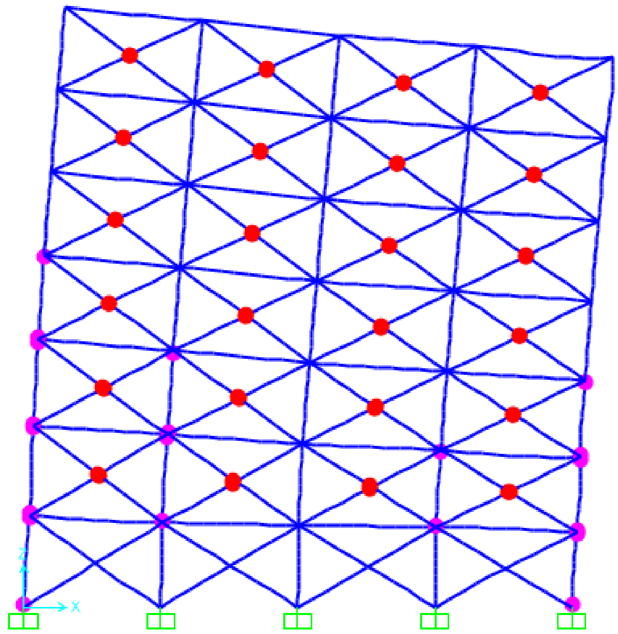
a)



b)

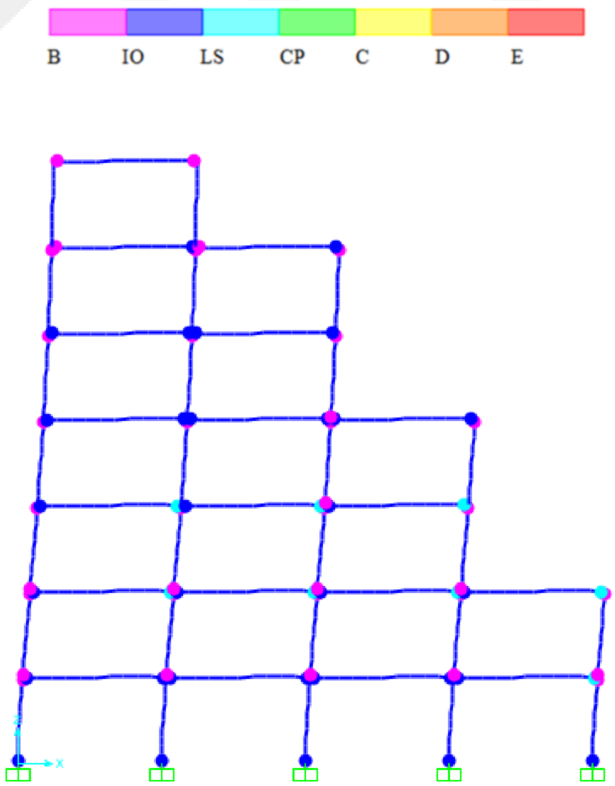


c)

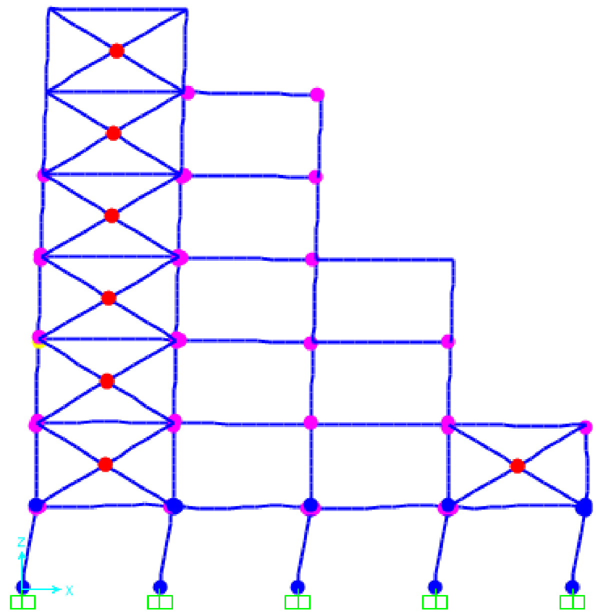


d)

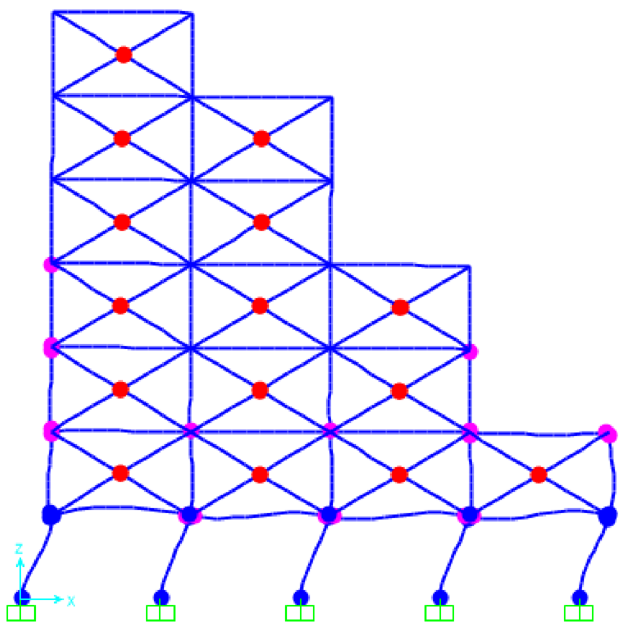
Figure 4.28 Plastic hinge formations from time history analysis of RF with different infill configurations



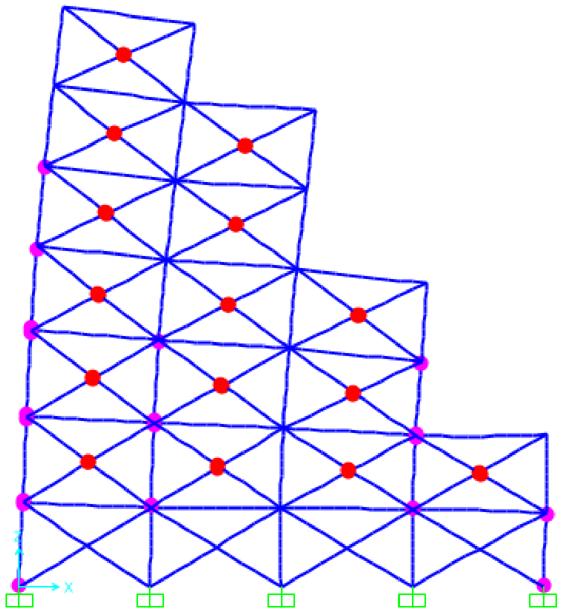
a)



b)

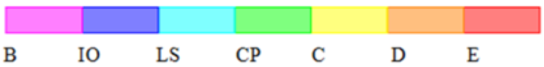


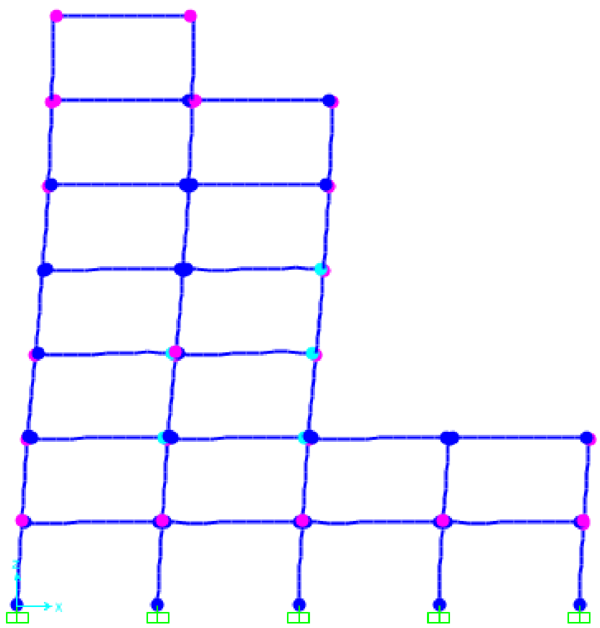
c)



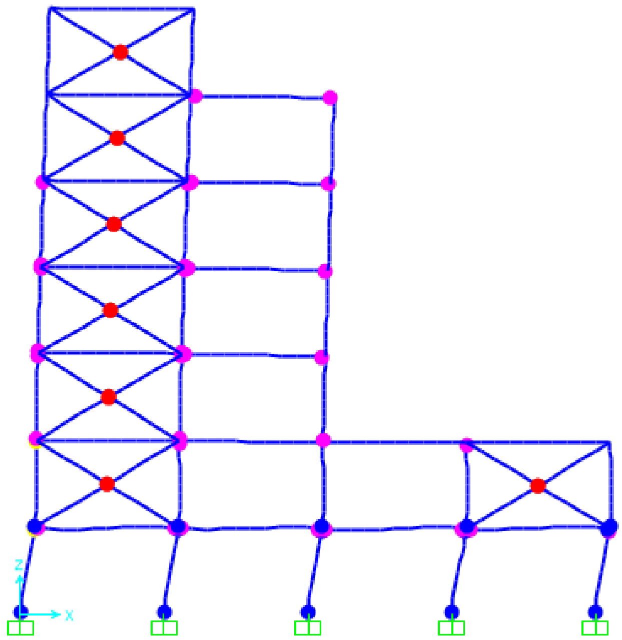
d)

Figure 4.29 Plastic hinge formations from time history analysis of IRF1 with different infill configurations

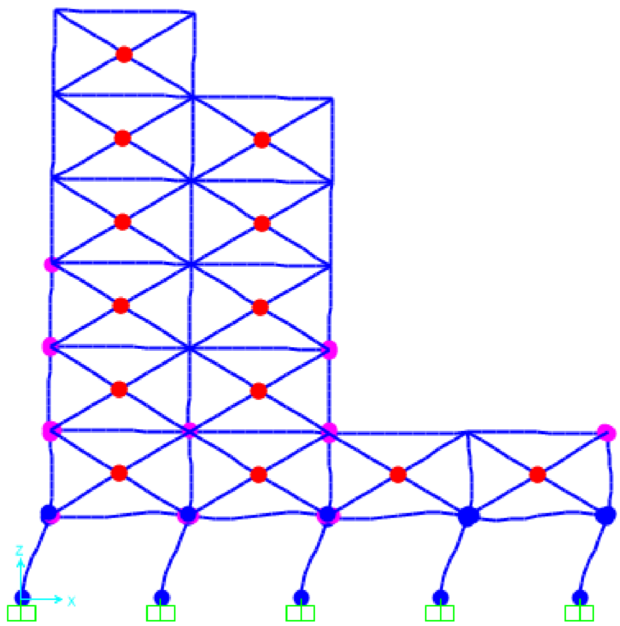




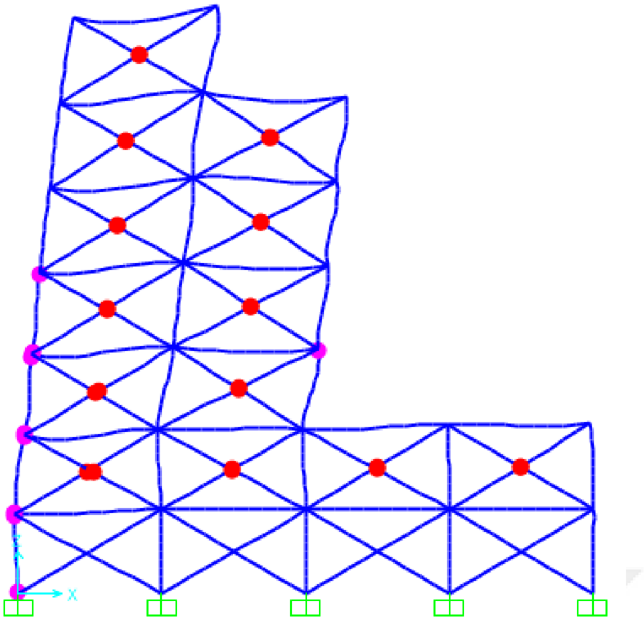
a)



b)

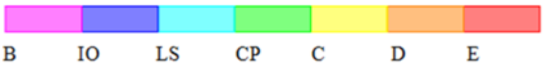


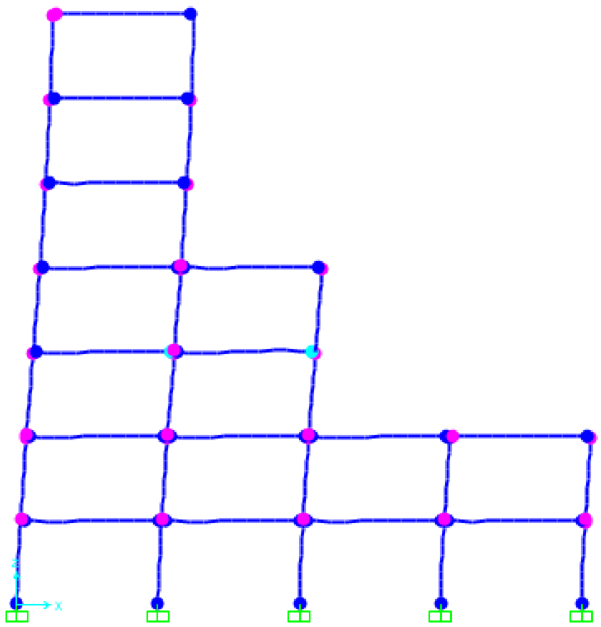
c)



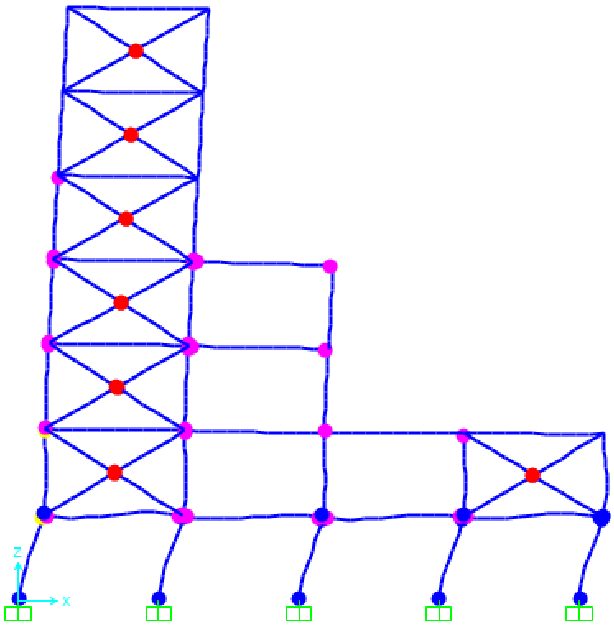
d)

Figure 4.30 Plastic hinge formations from time history analysis of IRF2 with different infill configurations

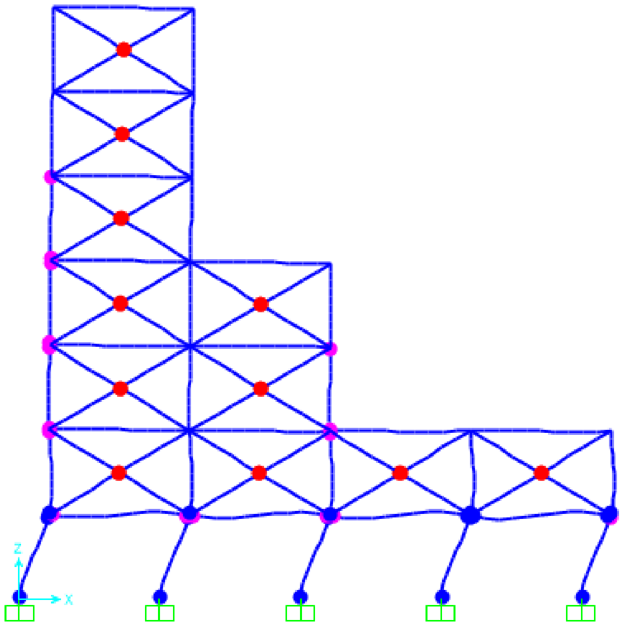




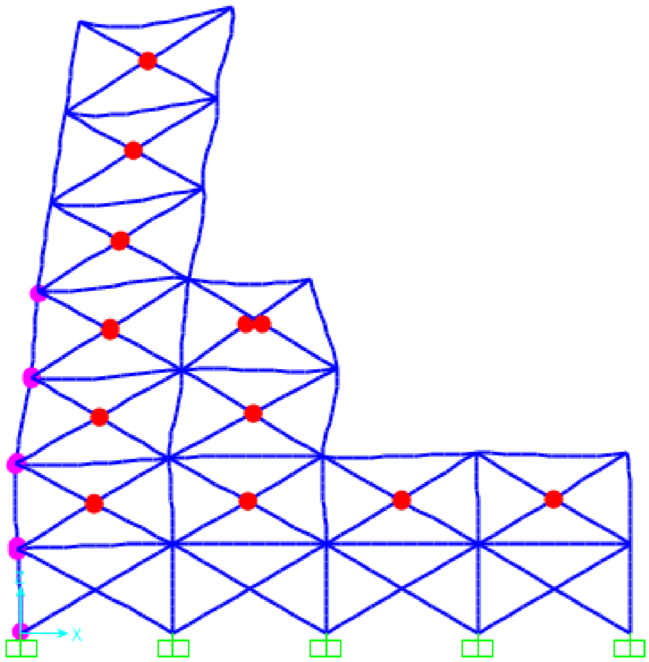
a)



b)

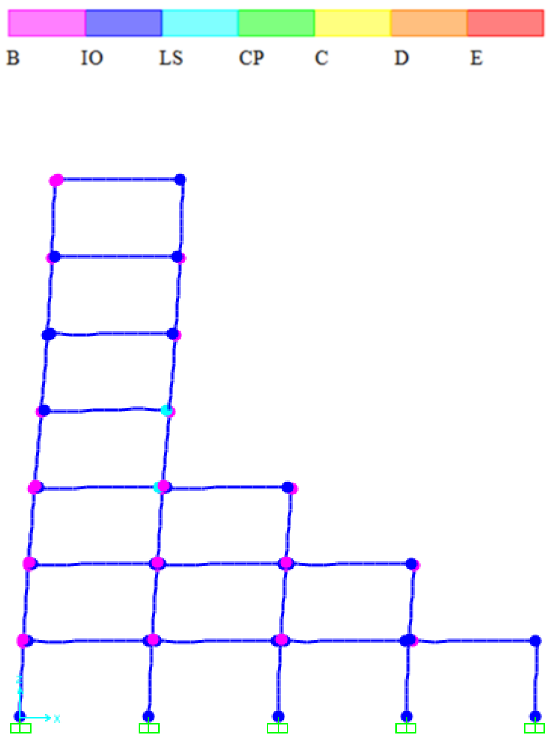


c)

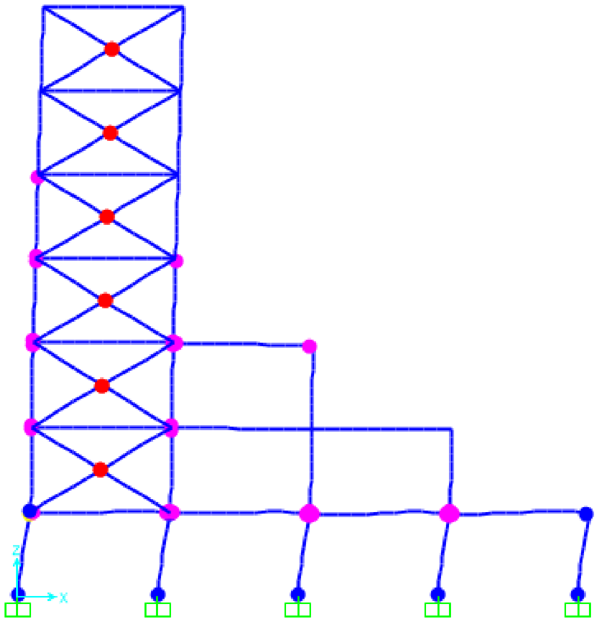


d)

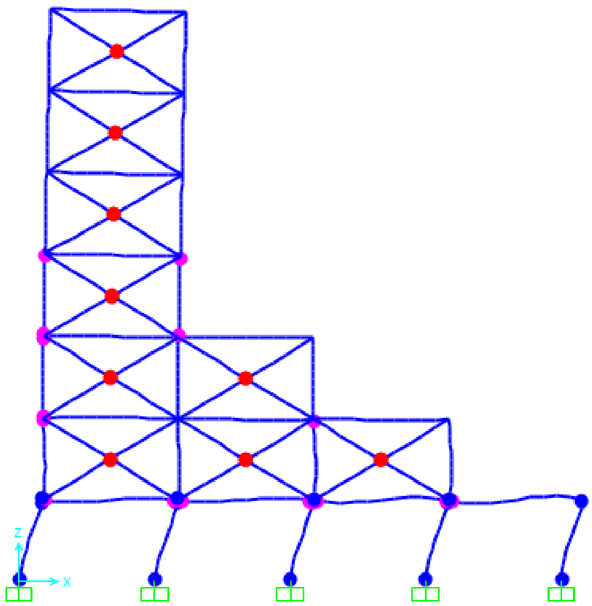
Figure 4.31 Plastic hinge formations from time history analysis of IRF3 with different infill configurations



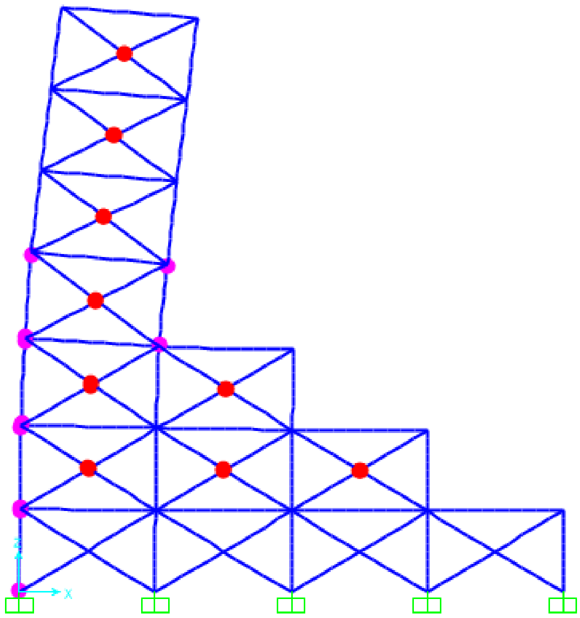
a)



b)



c)



d)

Figure 4.32 Plastic hinge formations from time history analysis of IRF4 with different infill configurations

CHAPTER 5
CONCLUSIONS

In this thesis, a detailed study has been carried out to examine the seismic performance of regular and four irregular frame models with and without three infill configurations of partially infill (double and quadruple bays) and fully infill. The following conclusions were obtained:

- The analysis of results indicated that the lateral load carrying capacity of the irregular frames remained quite low in comparison to the regular frame.

- Depending mainly upon the vertical irregularity, the irregular frame models resulted in up to about 2 time’s lower capacity. Similarly, their initial stiffness values were negatively affected. On the other hand, the results from the analytical models clearly showed the remarkable effect of the masonry infill on the frame response under lateral loads not only for the regular structure but also for the irregular ones.

- The arrangement of the infills also greatly upgraded the performance of the case study structures. In case of frame structure without infill walls, the displacement and storey drift is very large and may cause the collapse of structure during strong earthquake shaking than the frame structures infilled with masonry. Therefore, the infilled frame structures will be the better option to prefer in the seismic regions.

- The frame with the fully infill case was found to have more initial stiffness, strength and energy dissipation compared to the other infill cases, irrespective of being regular or irregular.

- It was also noted that the partially infilled frames failed in a soft story mechanism, because of much lower strength and stiffness of the ground floor, associated with the missing walls.

- Both nonlinear pushover and time history analysis indicated that the failure mode of the regular and irregular frames was remarkably affected by the existence of infill and also its configuration in the structure.

REFERENCES

Abdelkareem, K. H., Abdel-Saayed, F. K., Ahmed, M. H., AL-Mekhlafy, N. (2013). Equivalent Strut Width for Modeling RC Infilled Frames, *Journal of Engineering Sciences*, **41**(3), 851-866.

Abel, E. J. (1992). Nonlinear seismic response of infilled steel frames, *Earthquake Engineer 10th World*, **8**, 4435.

Agrawal, N., Kulkarni, P. B., Raut, P. (2013). Analysis of Masonry Infilled RC Framewith&without Opening Including Soft storey by using Equivalent Diagonal StrutMethod, *International Journal of Scientific and Research Publications*, **3**(9),1.

Ahmadi, S. H., Andersen, M. N., Plauborg, F., Poulsen, R. T., Jensen, C. R., Sepaskhah, A. R., Hansen, S. (2010). Effects of irrigation strategies and soils on field-grown potatoes: Gas exchange and xylem [ABA], *Agricultural Water Management*, **97**(10), 1486-1494.

Alavi, B., Krawinkler, H. (2000, January). Consideration of near-fault ground motion effects in seismic design, In *Proceedings of the 12th World Conference on Earthquake Engineering* (p. 8).

Amato, G., Cavaleri, L., Fossetti, M., Papia, M. (2008). Infilled frames: influence of vertical loads on the equivalent diagonal strut model, *Procedings of 14th WCEE, Beijing, China.* CD-ROM, 05-01.

Andreaus, U., di Paolo, A. L. D. O. (1988). 3-d Analysis of masonry columns with grouted reinforcement bars. In *Proceedings of the 8th International Brick and Block Masonry Conference, Dublin* (Vol. **3**, pp. 1507-1518).

Asteris, P. G. (2003). Lateral stiffness of brick masonry infilled plane frames, *Journal of Structural Engineering*, **129**(8), 1071-1079.

Asteris, P. G. (2008). Finite element micro-modeling of infilled frames, *Electronic Journal of Structural Engineering*, **8**(8), 1-11.

Athanassiadou, C. J. (2008). Seismic performance of RC plane frames irregular in elevation, *Engineering Structures*, **30**(5), 1250-1261.

Athanatopoulou, A. M., Avrimidis, I. E. (2008, September). Evaluation of EC8 provisions concerning the analysis of spatial building structures using two planar models, In *Proceedings of the 5th European Workshop on the Seismic Behaviour of Irregular and Complex Structure, 5EWICS* (pp. 16-17).

Barua, H. K., Mallick, S. K. (1977). Behaviour of mortar infilled steel frames under lateral load, *Building and Environment*, **12**(4), 263-272.

Caliò, I., Pantò, B. (2014). A macro-element modelling approach of infilled frame structures, *Computers & Structures*, **143**, 91-107.

Chopra, A. K., Goel, R. K., Chintanapakdee, C. (2004). Evaluation of a modified MPA procedure assuming higher modes as elastic to estimate seismic demands, *Earthquake Spectra*, **20**(3), 757-778.

Chrysostomou, C. Z. (1991). Effects of degrading infill walls on the nonlinear seismic response of two-dimensional steel frames (Doctoral dissertation, *Cornell University*.

Crisafulli, F. J., Carr, A. J. (2007). Proposed macro-model for the analysis of infilled frame structures, *Bulletin of the New Zealand Society for Earthquake Engineering*, **40**(2), 69-77.

Das, S., Nau, J. M. (2003). Seismic design aspects of vertically irregular reinforced concrete buildings, *Earthquake Spectra*, 19(3), 455-477.

Davis, R., Krishnan, P., Menon, D., Prasad, A. (2004, August). Effect of infill stiffness on seismic performance of multi-storey RC framed buildings in India, In *13th World Conference on Earthquake Engineering, Vancouver, BC, Canada*.

Dawe, J. L., Seah, C. K. (1989). Behaviour of masonry infilled steel frames. *Canadian Journal of Civil Engineering*, **16**(6), 865-876.

Dhanasekar, M., Page, A. W., Kleeman, P. W. (1985), The failure of brick masonry under biaxial stresses, *Proceedings of the Institution of Civil Engineers*, **79**(2), 295-313.

Dhanasekhar, M., Page, A. W. (1986). The influence of brick masonry infill properties on the behaviour of infilled frames, *Proceedings of the Institution of Civil Engineers*, **81**(4), 593-605.

Dinh, H. T., Kuever, J., Mubmann, M., Hassel, A. W., Stratmann, M., Widdel, F. (2004). Iron corrosion by novel anaerobic microorganisms, *Nature*, **427**(6977), 829-832.

Dolšek, M., Fajfar, P. (2008). The effect of masonry infills on the seismic response of a four-storey reinforced concrete frame a deterministic assessment, *Engineering Structures*, **30**(7), 1991-2001.

Dorji, J., Thambiratnam, D. P. (2009). Modelling and analysis of infilled frame structures under seismic loads, *Open Construction and Building Technology Journal*, **3**, 119-126.

Duan, L., Bozoglian, F., Mandal, S., Stewart, B., Privalov, T., Llobet, A., Sun, L. (2012). A molecular ruthenium catalyst with water-oxidation activity comparable to that of photosystem II, *Nature Chemistry*, 4(5), 418-423.

Dukuze, A. (2000). Behaviour of reinforced concrete frames infilled with brick masonry panels, *University of New Brunswick Department of Civil Engineering*.

Durrani, A. J., Luo, Y. H. (1994). Seismic retrofit of flat-slab buildings with masonry infills. In *Technical Report* (pp. 1-8), National Center for Earthquake Engineering Research.

El Haddad, M. H. (1991). Finite element analysis of infilled frames considering cracking and separation phenomena, *Computers & Structures*, **41**(3), 439-447.

FEMA30 6 Evaluation of Earthquake Damaged Concreteand Masonry Wall Buildings: Basic Procedures *Manual Federal Emergency Management Agency*.

Fiorato, A. E., Sozen, M. A., Gamble, W. L. (1970). *An Investigation of the Interaction of Reinforced Concrete Frames With Masonry Filler Walls*, University of Illinois Engineering Experiment Station, College of Engineering. University of Illinois at Urbana-Champaign.

Fragiadakis, D., Pissis, P., Bokobza, L. (2005). Glass transition and molecular dynamics in poly (dimethylsiloxane)/silica nanocomposites, *Polymer*, **46**(16), 6001-6008.

Gopalarathnam, M., Kumar, M. (2013). Nonlinear finite element dynamic analysis of squat shear walls with openings, *International Journal of Advanced Structural Engineering*, **5**(1), 1-20.

Güler, A. D., Ecker, J. L., Lall, G. S., Haq, S., Altimus, C. M., Liao, H. W., Hankins, M. W. (2008). Melanopsin cells are the principal conduits for rod–cone input to non-image-forming vision. *Nature*, **453**(7191), 102-105.

Holmes, M. (1961, August). Steel frames with brickwork and concrete infilling. In *ICE Proceedings* (Vol. **19**, No. 4, pp. 473-478), Thomas Telford.

Holmes, M. (1963, May). Combined loading on infilled frames, In *ICE Proceedings* **1**, 31-38.

Jinya, M. H., Patel, V. R. (2014). Analysis of RC Frame with and Without Masonry Infill Wall with Different Stiffness with Outer Central Opening, *IJRET: International Journal of Research in Engineering and Technology*, 2319-1163.

Kappos, A. J., Panagopoulos, G., Panagiotopoulos, C., Penelis, G. (2006). A hybrid method for the vulnerability assessment of R/C and URM buildings, *Bulletin of Earthquake Engineering*, **4**(4), 391-413.

Kaushik, H. B., Rai, D. C., Jain, S. K. (2008, October). A rational approach to analytical modeling of masonry infills in reinforced concrete frame buildings, In *Proceedings of The 14th World Conference on Earthquake Engineering* (pp. 12-17).

King, G. J. W., Pandey, P. C. (1978, December). The analysis of infilled frames using finite elements, In *ICE Proceedings*, **4**, 749-760.

Korkmaz, K. A., Demir, F., Sivri, M. (2007). Earthquake assessment of R/C structures with masonry infill walls, *International Journal of Science & Technology*, **2**(2), 155-164.

Kose, M. M. (2009). Parameters affecting the fundamental period of RC buildings with infill walls, *Engineering Structures*, **31**(1), 93-102.

Koutromanos, I., Stavridis, A., Shing, P. B., & Willam, K. (2011). Numerical modeling of masonry-infilled RC frames subjected to seismic loads, *Computers & Structures*, **89**(11), 1026-1037.

Liaum, T., Kwan, K. (1982). Non-linear analysis of multistorey infilled frames, In *ICE Proceedings*, **2**, 441-454.

Liauw, T. C., Lee, S. W. (1977, September). On the behaviour and the analysis of multi-storey infilled frames subject to lateral loading, In *ICE Proceedings*, **3**, 641-656.

Lodi, S. H., Mohammad, A. F. (2012). Nonlinear Static Analysis of an Infill Framed Reinforced Concrete Building, In *World Conference on Earthquake Engg, Losbia*.

Lotfi, H. R., Shing, P. B. (1994). Interface model applied to fracture of masonry structures, *Journal of Structural Engineering*, **120**(1), 63-80.

Lourenço, P. B., Rots, J. G. (1997). Multisurface interface model for analysis of masonry structures, *Journal of Engineering Mechanics*, **123**(7), 660-668.

Ma, D., Lu, L., Boneva, N. B., Warashina, S., Kaplamadzhiev, D. B., Mori, Y., Yamashima, T. (2008). Expression of free fatty acid receptor GPR40 in the neurogenic niche of adult monkey hippocampus, *Hippocampus*, **18**(3), 326-333.

Mainstone, R. J. (1971). Summary of paper 7360. on the stiffness and strengths of infilled frames, *Proceedings of the Institution of Civil Engineers*, **49**(2), 230.

Mainstone, R. J., Weeks, G. A. (1972). The influence of a bounding frame on the racking stiffness and strengths of brick walls, *Building Research Station*.

Mallick, D. V., Severn, R. T. (1967, December). The behaviour of infilled frames under static loading, In *ICE Proceedings*, **4**, 639-656.

Mander, J. B., Nair, B., Wojtkowski, K., Ma, J. (1993). An experimental study on the seismic performance of brick-infilled steel frames with and without retrofit, In *Technical Report, National Center for Earthquake Engineering Research (NCEER).*

Marcotullio, D., Magliulo, G., Pezone, T. (2002). Reinke's edema and risk factors: clinical and histopathologic aspects, *American Journal of Otolaryngology*, **23**(2), 81-84.

May, I. M., Naji, J. H. (1991). Nonlinear analysis of infilled frames under monotonic and cyclic loading, *Computers & Structures*, **38**(2), 149-160.

Mehrabi, A. B., Shing, P. B. (1997). Finite element modeling of masonry-infilled RC frames, *Journal of Structural Engineering*, **123**(5), 604-613.

Moehle, J. P., Elwood, K. J., Sezen, H. (2001). Gravity load collapse of building frames during earthquakes, *ACI Special Publications*, **197**, 215-238.

Moghaddam, H. A., Dowling, P. J. (1987). ESEE Research Report No. 87-2, The State of the Art in Infilled Frames, Imperial College of Science and Technology, *Civil Eng. Department, London, UK.*

Papia, M. (1988). Analysis of infilled frames using a coupled finite element and boundary element solution scheme, *International Journal for Numerical Methods in Engineering*, **26**(3), 731-742.

Papia, M., Cavaleri, L., Fossetti, M. (2003). Infilled frames: developments in the evaluation of the stiffening effect of infills, *Structural Engineering and Mechanics*, **16**(6), 675-693.

Perera, R. (2005). Performance evaluation of masonry-infilled RC frames under cyclic loading based on damage mechanics, *Engineering Structures*, **27**(8), 1278-1288.

Polyakov, G. D. (1958). Underfeeding as a cause of mortality in the wintering finger lings of carp (In Russian), In *Proceedings of Conference of Ichthyologic Committee of Academy of Sciences of USSR, Moscow*, **8**, 255-260.

Riddington, J. R., Smith, B. S. (1977). Analysis of infilled frames subject to racking with design recommendations, *Structural Engineer*, **55**(6), 263-268.

Rivero, C. E., Walker, W. H. (1982). An Analytical Study of the Interaction Frames and Infill Masonry Walls, *University of Illinois Engineering Experiment Station. College of Engineering, University of Illinois at Urbana-Champaign.*

Ruiz, S. E., Diederich, R. (1989). The Mexico earthquake of September 19, 1985 the seismic performance of buildings with weak first storey, *Earthquake Spectra*, **5**(1), 89-102.

Sattar, S., Liel, A. B. (2010, July). Seismic performance of reinforced concrete frame structures with and without masonry infill walls, In *9th US National and 10th Canadian Conference on Earthquake Engineering, Toronto, Canada.*

Sezen, H., Whittaker, A. S., Elwood, K. J., Mosalam, K. M. (2003). Performance of reinforced concrete buildings during the August 17, 1999 Kocaeli, Turkey earthquake, and seismic design and construction practise in Turkey, *Engineering Structures*, **25**(1), 103-114.

Singh, H., Paul, D. K., Sastry, V. V. (1998). Inelastic dynamic response of reinforced concrete infilled frames, *Computers & Structures*, **69**(6), 685-693.

Smith, B. S. (1967). Methods for predicting the lateral stiffness and strength of multi storey infilled frames, *Building Science*, **2**(3), 247-257.

Smith, B. S., Carter, C. (1969, September). A method of analysis for infilled frames, In *ICE Proceedings*, **1**, 31-48.

Te-Chang, L., Kwok-Hung, K. (1984). Nonlinear behaviour of non-integral infilled frames, *Computers & Structures*, **18**(3), 551-560.

Tremblay, R., Poncet, L. (2005). Seismic performance of concentrically braced steel frames in multistory buildings with mass irregularity, *Journal of Structural Engineering*, **131**(9), 1363-1375.

Uva, G., Porco, F., Fiore, A. (2012). Appraisal of masonry infill walls effect in the seismic response of RC framed buildings: a case study, *Engineering Structures*, **34**, 514-526.

Uva, G., Raffaele, D., Porco, F., Fiore, A. (2012). On the role of equivalent strut models in the seismic assessment of infilled RC buildings, *Engineering Structures*, **42**, 83-94.

Valmundsson, E. V., Nau, J. M. (1997). Seismic response of building frames with vertical structural irregularities, *Journal of Structural Engineering*, **123**(1), 30-41.

Van Zijl, G. P. (2004). Modeling masonry shear-compression: Role of dilatancy highlighted, *Journal of Engineering Mechanics*, **130**(11), 1289-1296.

Varadharajan, S., Sehgal, V. K., Saini, B. (2012). Review of different structural irregularities in buildings, *Journal of Structural Engineering*, **39**(5), 393-418.

Verderame, G. M., De Luca, F., Ricci, P., Manfredi, G. (2011). Preliminary analysis of a soft-storey mechanism after the 2009 L'Aquila earthquake, *Earthquake Engineering & Structural Dynamics*, **40**(8), 925-944.

Wakchaure, M. R., Ped, S. P. (2012). Earthquake Analysis of High Rise Building with and Without in filled Walls, *International Journal of Engineering and Innovative Technology (IJEIT)*, **2**.

Wood, R. H. (1958). The stability of tall buildings, *Proceedings of the Institution of Civil Engineers*, **11**(1), 69-102.

Wood, R. H. (1978, June). Plasticity, composite action and collapse design of unreinforced shear wall panels in frames, In *ICE Proceedings*, **65,** 381-411.

Yuen, Y. P., Kuang, J. S. (2015). Nonlinear seismic responses and lateral force transfer mechanisms of RC frames with different infill configurations, *Engineering Structures*, **91**, 125-140.

Zarnic, R. (1994). Experimental investigation of the R/C frame infill by masonry wall, *International Journal for Engineering Modelling*, **7**(1-2), 1994.

Zee, H. L., Moehle, J. P. (1984). Behavior of interior and exterior flat plate connections subjected to inelastic load reversals, *University of California, Earthquake Engineering Research Center.*

Zucchini, A., Lourenço, P. B. (2009). A micro-mechanical homogenisation model for masonry Application to shear walls, *International Journal of Solids and Structures*, **46**(3), 871-886.