MAY 2017 M. Sc. in Civil Engineering BAYDAA HAMDI SALIH

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INFLUENCE OF INFILL WALL OPENINGS ON PUSHOVER RESPONSE OF REINFORCED CONCRETE MOMENT RESISTING FRAME STRUCTURES

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BAYDAA HAMDI SALIH

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Influence of Infill Wall Openings on Pushover Response of Reinforced Concrete Moment Resisting Frame Structures

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

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Baydaa Hamdi SALIH

ABSTRACT

INFLUENCE OF INFILL WALL OPENINGS ON PUSHOVER RESPONSE OF REINFORCED CONCRETE MOMENT RESISTING FRAME STRUCTURES

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In this study, the pushover behavior of reinforced concrete (RC) moment resisting frame structures with masonry infill walls having different opening percentages was investigated. The equivalent diagonal compression strut method was used to model the infill panels. The essential point in this method is the equivalent strut width. Firstly, five different expressions suggested by the researchers exist in the literature and one formulation given in a provision of FEMA 306 for calculating the width of compression diagonal strut were examined by analyzing 2, 4, 6 and 8 storey RC frames as full-infill frames. Then, various opening contents in infill walls have been studied. The openings in the walls were located at the center. The opening was formed at the percentages of 20, 40, 60, 80, and 100. A reduction coefficient proposed in the literature was adopted to find the equivalent width of the diagonal strut for the cases of the infill wall with opening. Thus, a total of 144 different RC frame models were evaluated through nonlinear static (pushover) analysis. The results on the capacity, initial stiffness, strength, and plastic hinge formation at mechanism as well as the energy dissipated by the buildings were discussed comparatively.

KEYWORDS: Equivalent strut, Infill wall, Reinforced concrete frame, Opening, Nonlinear analysis.

ÖZET

DOLGU DUVAR BOŞLUKLARININ BETONARME ÇERÇEVELİ YAPILARIN İTME ALTINDAKİ TEPKİSİNE ETKİSİ

SALIH, BAYDAA HAMDI
Yüksek Lisans Tezi, İnşaat Mühendisliği
Tez Yöneticisi: Doç. Dr. Esra METE GÜNEYİSİ
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120 sayfa

Bu çalışmada, betonarme çerçeveli yapıların itme altındaki davranışına dolgu duvar boşluklarının etkisi araştırılmıştır. Duvar panellerinin modellemesinde eşdeğer diagonal basınç çubuğu metodu kullanıldı. Bu yöntemde en önemli nokta eşdeğer çubuğun genişliğidir. İlk olarak, literatürde araştırmacılar tarafından önerilen beş farklı denklem ve FEMA 306 tarafından verilen bir denklem diyagonal basınç çubuğunun genişliğinin hesaplanmasında yararlanıldı. Analizlerde 2, 4, 6 ve 8 katlı betonarme çerçeveler tam dolgu duvarlı çerçeveler olarak kullanıldı. Daha sonra, boşluğun duvar ortasında oluştuğu düşünülerek, çeşitli boşluk miktarlarının etkisi incelendi. Boşluklar %20, %40, %60, %80 ve %100 olarak dikkate alındı. Boşluklu duvar için eşdeğer çubuğun genişliğini bulmak için, literatürde önerilen azaltma katsayısı kullanıldı. Böylece, toplam 144 farklı betonarme çerçeve modeli doğrusal olmayan statik (itme) analizi kullanılarak değerlendirildi. Elde edilen sonuçlar kapasite, başlangıç rijitliği, mekanizmada plastik mafsal oluşumu ve yapıların enerji dağıtması üzerine karşılaştırmalı olarak tartışıldı.

Anahtar Kelimeler: Eşdeğer çubuk, Dolgu duvar, Betonarme çerçeve, Boşluk, Doğrusal olmayan analiz.

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LIST OF SYMBOLS

W or b_w	Width of diagonal strut
d	Diagonal length of infill panel
E_c	Young's modulus of concrete
E_w	Young's modulus of masonry infill
F_m	Maximum force of infill
F_r	Residual force of infill
f_{tp}	Tensile strength of masonry infill
F_y	Yield force corresponding to the first crack in infill panel
G_w	Shear modulus of masonry infill
H	Height of panel frame
H_w	Height of infill panel
I_c	Second moment of area of the column
I_b or I_T	Second moment of area of the beam
K_1	Initial shear stiffness of the un-cracked panel
K_2	Axial stiffness of the equivalent strut
L_w	Length of infill
S_m	Displacement corresponding to the maximum force
S_r	Residual displacement of infill panel at failure
S_y	Displacement of infill at yielding
t_w	Thickness of infill
θ	The angle made by the strut with horizontal
λ	Stiffness reduction factor
α_w	The opening percentage of infill (area of opening/area of infill)

CHAPTER 1

INTRODUCTION

1.1 General

Masonry infilled frames are a composite structural system composed of a steel or reinforced concrete (RC) frame with the portal space containing masonry, which is typically unreinforced. The behavior of composite structure results in higher flexibility than the masonry infill walls alone. Also, numerous studies show that integral masonry infill increases the stiffness and strength of the composite system in comparison with the bare structure. Several stages of response occur during the loading of a masonry infilled frame. At first, the infill wall acts as a monolithic cantilever panel. Little concentrations of stresses take place at corners, whilst the central region of the panel evolves a state of pure shear stress approximately. When in-plane loading increases gradually, separation occurs at the interface of the masonry and the frame members and it can be observed at the off-diagonal corners. Once a gap is formed (Figure 1.1), the stresses at the tensile corners are relieved while those near the compressive corners are increased (Tucker, 2007).

As the load continues to increase, additional separation takes place between the frame and the masonry wall, which result in contact at the frame sections near the loaded corners only. This condition of contact leads to the composite behavior which represents the braced frame. This has led to the concept of replacing the masonry infill with an equivalent diagonal strut when modeling the behavior of the system as shown in Figure 1.1. The induced stresses in the masonry panel produce various cracking patterns depending on the combination of the tensile strength of the masonry units, the shear strength of the mortar joints, and the relative values of the normal and shear stresses. Failure of masonry infilled frames can be classified into three basic modes: flexural cracking, shear cracking, and compression failure. Flexural cracking failure is scarce due to separation at the frame-masonry interface commonly takes place firstly; then, the horizontal force is resisted by the truss mechanism of the diagonal strut. Shear cracking can be divided into two types: diagonal tensile cracking, and cracking along the mortar joints which involves horizontal cracks and stepped cracks. The compression failure mode consists of the failure of the diagonal strut and the crushing of the masonry in the loaded corners at the diagonal. The concept of diagonal strut is developed within the infill wall depending on the result of diagonal tensile cracking (Tucker, 2007).

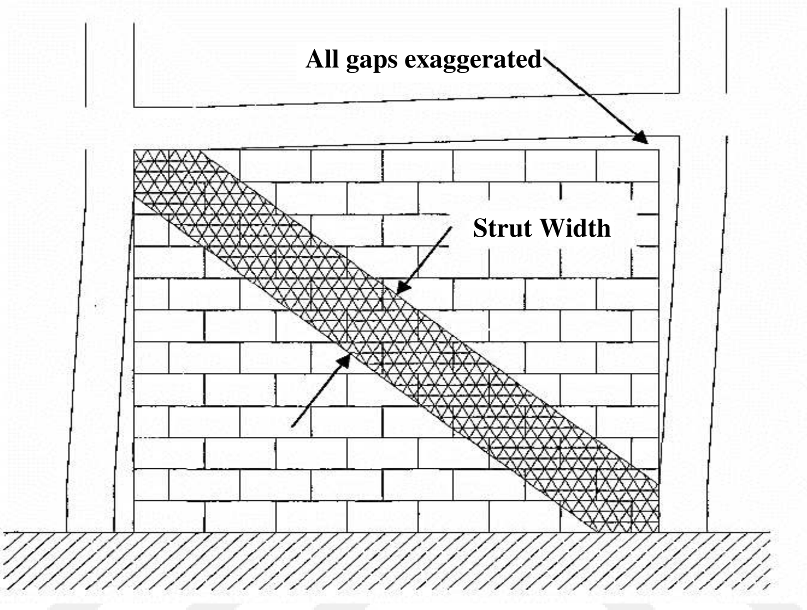


Figure 1.1 Deformed shape with equivalent diagonal strut (Tucker, 2007)

To indicate a composite structure constituted from the infill walls and a moment resisting frame, the term ‘infilled frame’ is used. Depending on the connection between the frame and the infill, 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 infilled panel frame imparts lateral strength and stiffness to the building (Davis et al., 2004).

When a non-integral infilled panel frame is under a racking load the infill panel and surrounding frame are separated largely along the length of interaction, only the two adjacent end compression corners remain diagonally as interface area of contact, as shown in Figure 1.2a. Depending on this behavior, and as a rough evaluation, the behavior of infill is assumed to be like a diagonal strut bracing, (Figure 1.2b). If it was possible to find the diagonal strength and stiffness values of the infill, appropriately can be expecting the lateral strength and stiffness of the infill frame structure by assuming to replace every infill wall panels and substitute the equivalent compression diagonal struts. Each members of the frame are to be rigidly connected to the other frame members, but the infill panel has non-integral connection with the bounding frames, and it is an isotropic material which has two types of failure: a brittle tensile failure and the plastic compression failure, same as to medium strength concrete (Smith, 1967).

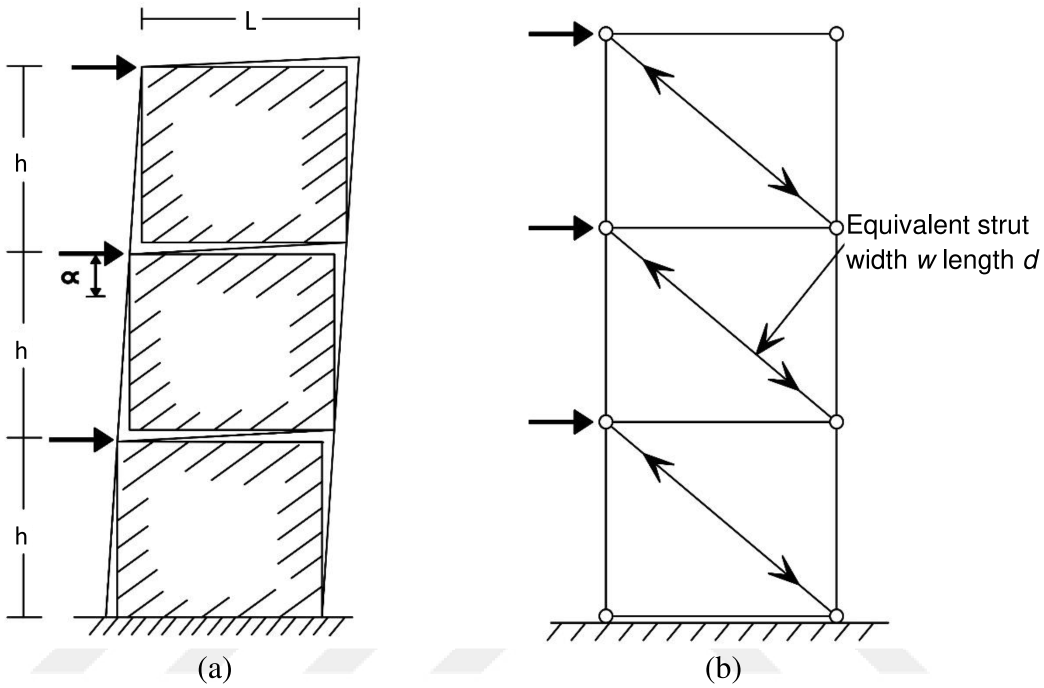


Figure 1.2 Elevation view of (a) Infilled frame and (b) Equivalent frame (Smith, 1967)

If the infill walls are properly considered in the design and properly distributed throughout the structure, then they usually have a useful effect on the seismic response of structures. Furthermore, irregular positioning of the infill walls in plan can be caused negative effects, and especially in elevation (Dolšek and Fajfar, 2008).

The presence of infill walls in the RC frames can significantly change earthquake behavior of buildings in some cases produce undesirable (potentially negative) effects like dangerous collapse mechanisms, torsional effects induced by in plan-irregularities, short-column effects and soft-story effects induced by irregularities, etc. The masonry infill walls have other significant effects of increasing the seismic resistance capacity of the building (increase the global strength and stiffness of the building) (Agrawal et al., 2013; Korkmaz et al., 2007). Nevertheless, it may not be suitable to ignore its existence and declare the output of design as conservative.

Observed infill induced failure in buildings in the past earthquakes detects the defects in the current bare framework approach (Wakchaure and Ped, 2012).

In several countries, infill masonry wall in RC frames constitute a structural system often used with relatively strong seismic force. The unanticipated damage caused by earthquakes has been demonstrated as the vulnerability of these seismic structures. Therefore, it is not a realistic approach to ignore masonry infill walls in the design procedure (Samoilã, 2012). Moreover, the pushover curve of masonry walls infilled frame is quite different when compared to that of the corresponding bare frame, with respected to the role played by infill walls in the nonlinear response of the structure (Uva et al., 2012).

Numerous methods have been prepared in the literature for modeling masonry infill walls, and they are grouped in two main categories (Alguhane et al., 2015):

- Macro-models, based on the method of the equivalent compression diagonal strut, and

- Micro-models, based on the methods of the finite element, by discretized infill masonry wall panel into several elements, etc.

In general, the micro-models which based on the method of the finite element require more computing abilities and more time-consuming than the macro-models which based on the method of equivalent diagonal compression strut, which these macro-models in evaluating the lateral strength and initial stiffness for masonry infilled RC frames have a sufficient accuracy. Conveniently, it can be modeled the infill masonry walls by using a diagonal 1-strut, 2-struts, 3-struts, and multi-number of struts. Several researchers have investigated the use of 1-strut model for infill masonry walls (Kaushik et al., 2008).

The micro-models approach was first discussed by Mallick and Severn (1967) using the method of finite element for the 2D infilled frame analysis. On the other hand, there are many studies in the literature on using a micro-model. Among them, the following researchers could be mentioned (Papia, l988; El Haddad, 1991; May and Naji, 1991; Lotfi and Shing, 1994; Mehrabi and Shing, 1997; Lourenço and Rots, l997; Singh et al., 1998; Van Zijl, 2004). However, as reportedly, this micro-model approach that mentioned above is more sophisticated, too time-consuming and too complex for using in a broad and practice oriented analysis as needed (Kaushik et al., 2008).

In RC frame, infill walls are used based on the architectural point of view and to make partition and other aspect. The vertical loads usually taking place, in multistory buildings dead or live loads, do not pose many problems, but the horizontal loads due to seismic motion or wind are an issue of considerable concern and required a special attention in the design of those buildings under lateral loads. These lateral forces set up undesirable vibrations, can generate the critical stress in a building, and furthermore, cause lateral sway of the building which can amount to a stage of worry to the residents (Wakchaure and Ped, 2012; Jinya and Patel, 2014).

In plenty of countries located in seismic zones, RC buildings are filled partly or completely by brick masonry walls having openings (Asteris, 2003). Windows and doors openings are invertible parts of infill panels for practical reasons which results in deceasing of load carrying ability and stiffness of infill walls. Presently, documents like ATC-40 and FEMA-273 include provisions for the calculation of stiffness of solid panels in infilled structures fundamentally by modeling the panels as a "diagonal strut". However, those provisions are not fitted for the infilled structures with openings (Mondal et al., 2008).

Generally, most national codes do not include the effect of openings on the stiffness and strength of masonry infill walls or present procedures about how to model wall openings. Eurocode-8 specifies that only the solid infills or infills with one window or door opening are supposed to be imparting any considerable strength to the building and gives some details about the openings. However, researches state that the two main parameters affecting the properties of equivalent strut represents infill walls with openings are size and position of opening (Mohamed, 2012).

The location and size of the openings have an important influence on the gross behavior of the constructional system. This could result in major difficulties in the modeling of infilled buildings, since in the majority cases of infill walls are not solid, but they have one or more openings which alters their conduct. So far, infill panels are being modeled as diagonal struts, which cannot represent infill panels having openings, unless a relation is found between the strength and stiffness reduction of the panels and the size and location of the openings. Stiffness degradation and multiple-strut strength models, such as that one described in the study of Chrysostomou (1991), have used to represent the influence of the openings by developing the strength diagram and area of hysteresis loops, to account for the effects of openings (Asteris, 2011).

The presence of openings, such as doors and windows, in masonry infill walls can reduce strength and stiffness of the infilled structures and hence change the expected behavior. A numerous commercial and residential buildings have soft storeys at the first floor due to the absence of masonry infill walls in this storey, inside and on the sides of major streets. This happens because the first storeys have been usually used for shopping and trade purposes, heavy masonry infill walls begin directly above the soft storey. Inspections of latest damages of earthquake, in addition to the results of several analytical researches, have revealed that structures with a soft storey can cause severe troubles during intense earthquake ground vibration. The existence of a soft storey could result in considerably growing demands of deformation, and makes the burden of energy dissipation on the columns at first storey (Mohamed, 2012).

The openings result in massive rising in bending moment and lateral force in the infilled buildings. By using various opening percentages it was concluded that as the size of opening increases the lateral force decreases and the bending moment increases. In case of opening situated outside the main diagonal, the opening role in reducing strength and stiffness is less important in comparison with that of the opening upon the main diagonal. The openings may interfere with diagonal bracing action if they are large and located at the center. Thus, shear failure can early happen in the sections on either side of the diagonal. Thereby, it is recommended to use small sizes of opening, and should be located out off the main diagonal (Motwani et al., 2015).

1.2 Objectives of the Thesis

The essential objective of the present study is to investigate the effect of different percentages of central opening in masonry infill walls on the seismic response of RC buildings. Another objective is to study comparatively the influence of different formulations exist in the technical literature to model infill wall as a diagonal compressive strut. Differences in those formulations are pointed out by comparing some response parameters in order to determine the most effective expression for calculating the strut width. For this, 2, 4, 6, and 8 story RC frames with and without infill walls have been examined. Thus, bare and full-infill walled frames as well as those with central openings within masonry walls up to 80% are studied analytically. All frame models (a total of 144 different models) are subjected to the nonlinear static analysis. Results obtained from the parametric study in terms of load carrying capacity, initial stiffness, strength, plastic hinge formation at mechanism, and energy dissipated by the buildings are discussed comparatively.

1.3 Outline of the Thesis

Chapter 1 Introduction: A general background and the main objective of the present investigation are provided in this chapter.

Chapter 2 Literature review: This chapter includes an overview of the previous studies about the effect of infill masonry walls on the seismic response of RC structures, failure modes of infilled RC frames, some formulations which have been used for modeling the infill wall as a diagonal compression strut, and the effect of openings on the behavior of infill walled frames.

Chapter 3 Case study: The description of analytical models of buildings, the modeling methodology of masonry infill walls with and without openings, and the nonlinear analysis method are presented in this chapter.

Chapter 4 Results and Discussion: In this chapter, the results obtained from the nonlinear static analysis of infilled frames with and without openings are presented and discussed.

Chapter 5 Conclusions: Some conclusions are presented in this chapter depending on the results obtained from this study.

CHAPTER 2

LITERATURE REVIEW

2.1 Infilled Reinforced Concrete Buildings

2.1.1 Experimental Research

In the literature, several experimental studies on the behavior of reinforced concrete (RC) frame buildings with and without infill walls exist. For example, Fiorato et al. (1970) tested three series of infilled frames of RC building as eight one-bay one-story specimens, thirteen one-bay five-story specimens, and six three-bay two-story specimens. The effect of different geometrical configurations of infill wall, the reinforcement ratio, and the vertical load were studied. Both flexural and shear failure of columns were observed on the one-bay one-story specimens in which the mid-story crack in the infill wall resulted in un-braced columns to fail by shear when the column was under tension and by flexure when the column was under compression. Shear failure was also observed at first-story columns subjected to a tensile axial load (in multi-story specimens) where the infill wall provided continuous bracing along its full length which resulted in shearing the column before a major crack formed in the wall. The failure mode observed in specimens with openings was the same as the ones without openings except for the noticeable flexibility resulting in an obvious trend toward flexural failure. Properties of the openings affected the flexural failure location (represented by hinges). The presence of openings decreased the strength but not in a manner proportional to the reduction in infill wall area.

Leuchars and Scrivener (1976) tested three single-bay single-story frames; a bare frame, a frame infilled with unreinforced grouted hollow bricks, and a frame infilled with reinforced grouted hollow bricks. This study confirmed that more data were required on the behavior of the infilled structure and the masonry material properties. Stiffness prediction based on beam theory presented by Fiorato et al. (1970) and a diagonal strut model presented by Smith and Carters (1969) provided a good match with Leuchars and Scrivener's experimental results but the strength estimation was not accurate. High ductility ratios were estimated but the authors confirmed that severe damage was attained at these levels of ductility.

Liauw and Kwan (1985) tested three series of infilled frames; without connectors between the frame and infill wall, with connectors provided along the beam-infill wall interface only and a 4mm gap was left between columns and wall, and with connectors between all infill wall and frame interfaces. Models tested in the first series showed an initial shortage of contact between the infilled frame and the wall. After that, the system was loaded and the wall became in contact with the frame along the compressive corners until a firm contact was reached. Peak strength was reached as infill corners were crushed. Models tested in the second series underwent inclined cracks at 45°. The peak strength was reached when the connections between the beam and the wall yielded. The load then dropped until a firm contact was achieved and resulted in a gaining back of part of the strength. High stiffness, strength, and energy dissipation were observed in the last series of models. An inclined crack at 45o was noticed until shear failure occurred at the beam-wall connection and crushing occurred at the corners under compression.

Schmidt (1989) tested RC frames infilled with brick walls and weak mortar along bed joints. The mortar weakness resulted in a large amount of slipping over the bed joints and plastic hinges at the ends of the columns.

Mehrabi et al. (1994, 1996) tested 14 half-scale specimens to study the major parameters that control the behavior of RC frames infilled with either hollow or solid concrete brick walls. These parameters include the relative stiffness and strength of an infill wall to those of the RC frame, the lateral load history, the aspect ratio of infill wall, the applied vertical load magnitude and distribution, and the number of bays. In these tests, most of the common failure modes identified by the Mehrabi et al. (1994, 1996) were observed. Specimens subjected to cyclic loading showed an improvement on energy dissipation capability of frames due to the presence of infill walls. However, a reduction in lateral strength and an increase in the strength reduction rate were observed for these specimens when compared to monotonically loaded frames. Mehrabi et al. concluded that infill walls improved the seismic performance of RC frames. It should be noted that adequate shear (transverse) reinforcement was recommended by these researchers to prevent brittle, shear failure of the columns especially when relatively strong infill walls were used.

Al-Chaar (1998) tested five, one story half-scale, non-ductile RC frames. Four of these specimens are infilled with either a concrete masonry unit (CMU) wall or a clay brick wall. One of the infilled frames was a single bay with CMU infill, one was a single bay with brick infill, one was a two-bay with CMU infill, and one was a three-bay with brick infill. The single-bay frame infilled with a CMU wall failed mainly by shear. A shear crack observed at the top of the windward column and a shear crack formed at the bottom of the leeward column. In the single-bay frame infilled with a brick wall, they observed a hinge in the middle of the windward column, a hinge in the beam at the windward joint, and a separation between the leeward column and the base because of the inability of the reinforcement to develop resistance. The failure mechanism of the two-bay specimen was by the formation of two hinges on the windward and center columns, shear cracking in the infill wall, and shear cracking at the base of the leeward column. The failure of the specimen with three bays was dominated by shear cracks in all four columns.

Colangelo (2005) tested 13 half-scale infilled RC frames. Pseudo-dynamic tests were performed on single-bay single-story specimens designed to represent the first story in a four story building. Each specimen was subjected to a Friuli Italy 1976 earthquake displacement record twice. The tested specimens belonged to three groups based on the seismic design consideration, namely; seismic loads are not considered, seismic loads are considered and the design complies with previous design code, and seismic loads are considered and the design is based on Eurocode 8 (1988). Inclined cracks were observed at the top of the columns in the frames not designed for lateral loads and had an aspect ratio of 0.57. Increases in stiffness, peak strength, and ductility demand were found compared to these of a bare frame.

Hashemi and Mosalam (2006) tested a one story-one bay structure (3/4 scale) representing a segment of a five story RC structure infilled with a masonry wall. The changes in load path and distribution in the RC frame due to the presence of infill walls were described. They estimated the equivalent viscous damping ratio to be in the range of 4% to 12% depending on the level of excitation.

Stavridis (2009) conducted a two phase large-scale experimental study. The first phase consisted of four 2/3-scale infilled RC frames with different opening configurations tested quasi-statically. He stated that the presence of openings in the infill wall affected the behavior of the system in terms of strength, stiffness, and mode of failure. He found that shear failure could take place when the developed struts in the solid masonry wall acted against the RC frames. Stavridis suggested that these brittle failures could be avoided if openings were located such that the behavior was controlled by flexure. The second phase of Stavridis' study consisted of testing of a three-story two-bay infilled RC frame on a shake table. The specimen maintained its lateral strength until a drift ratio of 1% at which shear failure of columns occurred. It was concluded that the infill wall improved the performance of the whole system in the range of the design earthquake level. The effect of vertical irregularity and torsional loads encountered were not investigated in this experimental study.

Zovkic et al. (2013) tested ten single-bay single-story specimens built at a 1:2.5 scale as a model for the first story middle-bay of a three-bay seven-story frame. The effect of the infill wall type on the behavior of the RC frames, designed for seismic loads based on Eurocode 8, under lateral loads was investigated. The damage was observed to be concentrated in the infill wall and accordingly it was concluded that the behavior of frames designed for seismic loads was improved irrespective of the infill wall type.

2.1.2 Analytical and Computational Research

Polyakov (1956) was the first to study and develop a simulation for the infilled frame as a braced frame with a strut, and quantify the width of the strut. Holmes (1961) replaced the infill panel by an equivalent diagonal strut made of the same material and jointed with pins at each tip and having a width one third of the infill panel diagonal length. Smith (1967) developed a method to evaluate the strength and lateral stiffness of frames infilled with homogenous walls that have no bond with the frames. Smith's method depends primarily on the relative stiffness of frame to that of the infill wall.

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 equivalent strut width is not a constant value for a particular infill but decreases as the loading is increased.

An equivalent beam model was proposed by Fiorato et al. (1970) to estimate the cracking load of infilled RC frames. Failure mechanisms observed in their tests were classified as a flexural failure mechanism and a shear failure mechanism. The former mechanism resembles the infilled frame to a beam and the failure is attained by yielding reinforcement at the column under tension or crushing of concrete at the column under compression even though the strain distribution is different than that in a monolithic beam. The latter mechanism is defined as shear cracks taking place in the infill wall and separating the wall into two pieces. They used a knee-braced frame concept to describe the behavior of infilled RC frames after the initiation of shear cracks. This concept is based on columns braced by a cracked wall. The short column behavior observed in their tests is also part of their knee braced frame model. Even though the mechanism is identified as shear failure, the columns could fail in flexure or shear.

Smolira (1973) presented an approach to analyze the response of an infilled frame to lateral loads based on a force-displacement method. The indeterminate parameters were the forces and linear displacements but not the joint rotations. The principle of superimposition (the material was assumed linear elastic), compatibility (the deformations were assumed to be small), and equilibrium using the un-deformed geometry were enforced in this method. The comparison between the proposed analysis procedure and experimental results were stated to be difficult due to many parameters that control the response of infilled frames.

Mehrabi et al. (1994) defined the most widespread types of failure of infilled RC frames. Analytical methods were developed to predict the lateral strength capacity of these failure mechanisms. In Mehrabi’s tests, the mechanism associated with the critical (i.e. lowest) calculated strength in a given specimen compared well with the mode of failure observed in the laboratory test.

Mohamed (2012) studied the lateral behavior of masonry infilled RC buildings by a nonlinear numerical investigation. Variety of parameters for both masonry infill (MI) walls and buildings are considered. Various arrangements of MI walls, MI wall thickness and absence of MI walls in the first storey, opening size of MI wall are studied. The application buildings are either dual shear wall-moment resisting frames (SWMRF) or moment resisting frame (MRF) buildings. The MRF buildings have 6 floors, while the SW-MRF buildings have 5 different heights represented by the number of floors (from six to twenty floors). Equivalent strut method was used and developed to model the conduct of infill walls taking into account the influence of opening sizes. Non-linear static pushover analysis was accomplished for the utilized case study buildings. It is found that MI walls can highly increase the base shear capacity of either building types while significantly reduce the displacement capacity of MRF buildings, RC shear walls can resist this negative effect. The influence of MI walls fades as the building height increases.

Rajesh et al. (2014) investigated the performance of RC frame buildings with and with-out infill walls. Here analyses and designs the of masonry infill walls using equivalent diagonal strut method were done to assess their participation in seismic resistance of RC buildings. The two different buildings with and without infill walls were modeled and designed and the analysis done for gravity and seismic loads using software (SAP2000). It was concluded that strut model buildings are stiffer and safer during the earthquakes than the bare-frame models. Also, strut model buildings give better and best performance than bare-frame model buildings in the high seismic prone areas.

A multi-strut model known as the (compression-only three struts model) was also investigated by Abel (1992). Infill walls were modeled as shown in Figure 2.1 by six inclined compression struts located at strength and stiffness degradation points. At any time during the analysis only three of the six struts are effective; the struts are switched to the contrary direction whenever they reach zero forces.

By using the equivalent diagonal strut method, it can model the global behavior (force-displacement) of the infilled structure, but it is not able to model the interaction between frame and masonry wall in local sense, particularly the changing in the shear diagram and moment along the column length because of the presence of the infill wall (Crisafulli, 1997). Many researchers have attempted to enhance the infill-frame interaction by proposing models with different orientations and number of struts. Syrmakezis and Vratsanou (1986) initiated one of the early attempts to consider this infill-frame interaction by using five parallel compressive struts in parallel direction, as shown in Figure 2.2a, to study the effect of the contact length on the moment distribution of the frame.

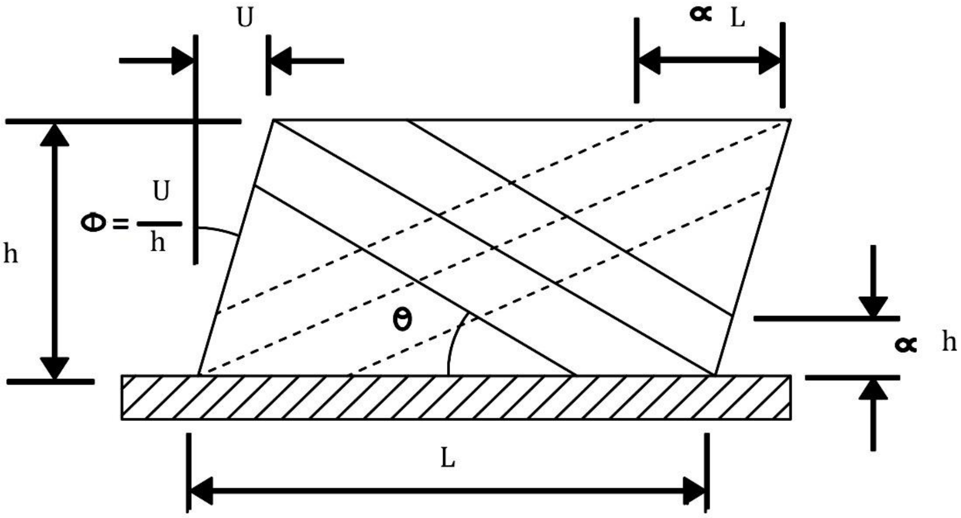


Figure 2.1 Six-strut idealization of infill walls (Abel, 1992)

Zarnic and Tomazevic (1988) attempted to consider this infill-frame interaction by changing the orientation of the struts. They developed an equivalent compressive strut model in which the strut is offset from the diagonal, as shown in Figure 2.2b. They developed this model based on the results of many cyclic tests conducted on infilled RC frames. Schmidt (1989) combined the idea of off diagonal struts and increased the number of struts, proposing a strut model with offsets at both ends, as shown in Figure 2.2c. Chrysostomou (1991) further altered the orientation of the struts, in order to model the response of the infill wall with three parallel compressive struts (one diagonal and two off-diagonal) as shown in Figure 2.2d. All of these models can represent the interaction between the frame and infill wall more accurately by increasing the number of the points connecting the infill panel to the columns or by changing the location at which the infill transfers load to the columns. However, the complexity and computational efforts have been increasing.

More recently, El-Dakhakhni et al. (2003) have expanded the idea of multiple struts by using three non-parallel struts, as illustrated in Figure 2.3, to reproduce the appropriate moment diagram of the columns in an infilled structure because of the interaction between the structure and masonry also to explain the corner crushing failure mechanism visibly.

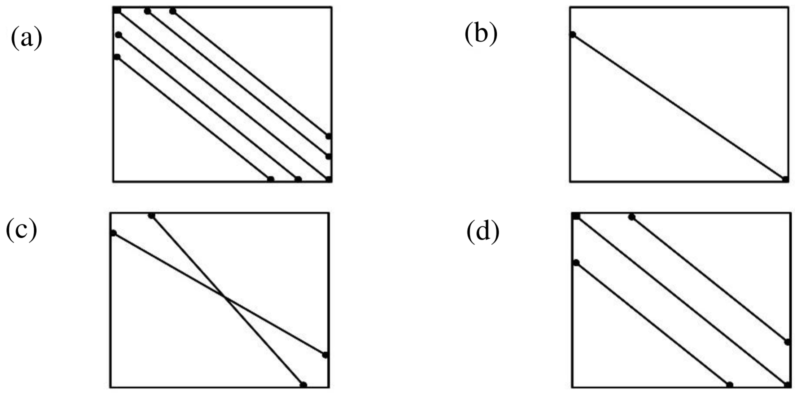


Figure 2.2 Alternative models with off-diagonal struts to model the frame infill interaction proposed by (a) Syrmakezis and Vratsanou (1986), (b) Zarnic and Tomazevic (1988), (c) Schmidt (1989), (d) Chrysostomou (1991)

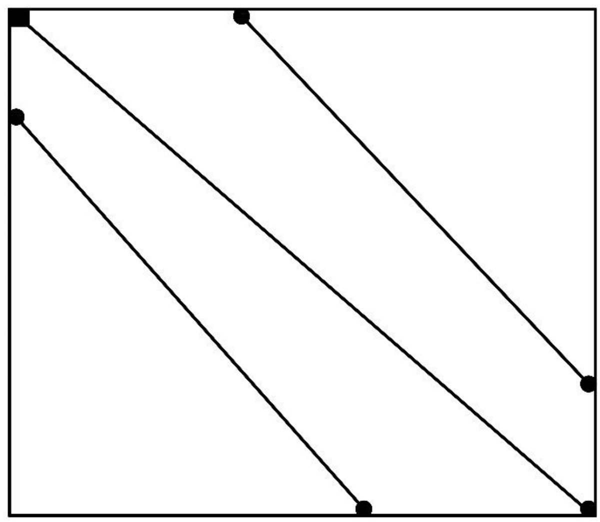


Figure 2.3 Three-diagonal strut model proposed by (El-Dakhakhni et al., 2003)

To better represent the infill wall shear failure mechanism, Crisafulli (1997) proposed a different kind of multi strut with spring model, which is implemented in a 4-node element. The model was accounted for the compressive and shears behavior of the infill panel using a nonlinear double-strut and one elasto-plastic shear spring as shown in Figure 2.4. The shear strength of the spring is evaluated based on the shear-friction mechanism that can represent the shear strength as a function of the maximum permissible shear stress, axial load, and thickness and length of the infill. The area of the struts in this model decreases as the axial strut displacement increases, because of the reduction of the length of contact between infill walls and frame and, also, due to cracking of the masonry infill. Crisafulli (1997) defined associated hysteretic rules.

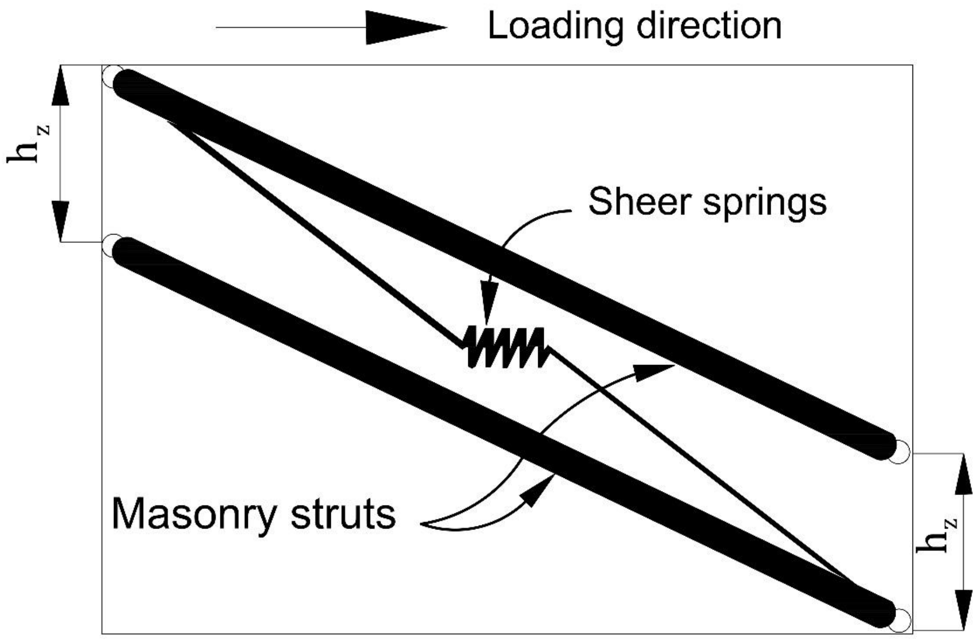
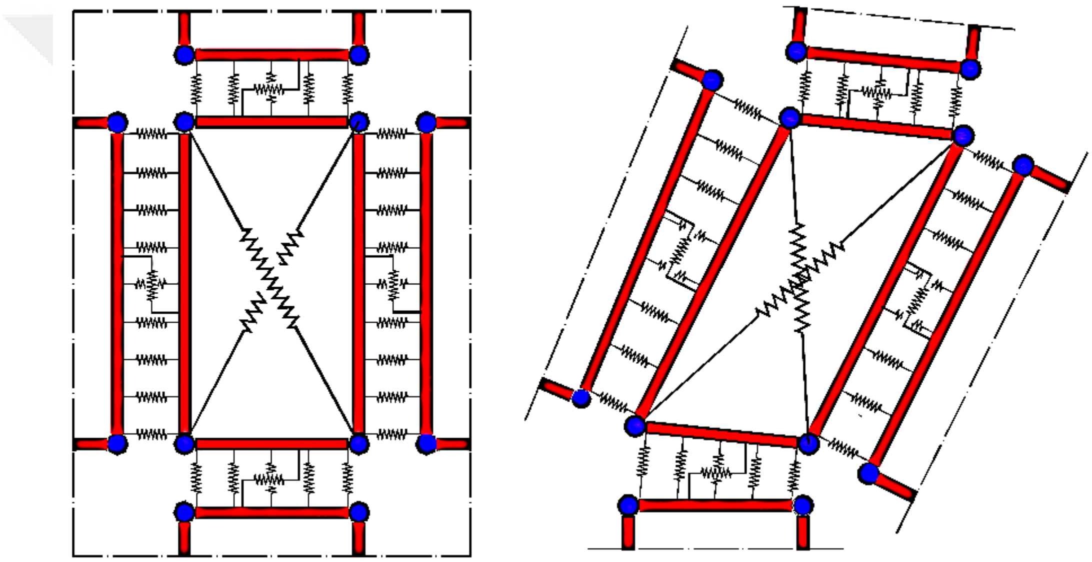


Figure 2.4 Multi-strut with shear spring model (Crisafulli 1997)

Stavridis (2009) developed a backbone curve for infilled RC frames tested in their experimental study to estimate the specimen behavior under cyclic loadings. In Stavridis's model, the backbone curve features were defined in six steps. The drift at peak strength estimation was based on a parametric computational study in which finite element models were used. This drift was calculated in terms of aspect ratio while the drift at residual strength was estimated as a factor of the drift at peak strength. The developed backbone curve was compared with their tests and the results matched well for infilled frames with solid panels. The backbone curve does not include the failure mechanisms in detail. The Loading-unloading features were not considered in the backbone curve proposed by Stavridis (2009).

Caliò and Pantò (2014) presented a macro-modeling approach to evaluate the seismic behavior of infilled structures. The nonlinear complex behavior of infilled structure is analyzed depending on an original approach in which the bounding frame is modeled using concentrated plasticity beam-column elements whilst the infills are described by plane macro-element. 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 located diagonally. Each face of the quadrilateral can interact with the frame element face through a separate distribution of nonlinear springs, denoted as interface. The interfaces are described by nonlinear orthogonal springs, perpendicular to the panel side, and an additional longitudinal spring, parallel to the panel edge. In spite of its simplicity, the basic mechanical system is capable to simulate the main in-plane failures of a masonry panel portion exposed to in-plane vertical and horizontal loads. These failure mechanisms are 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 suggested plane macro-element permits a real and simple mechanical simulation of the identical failure mechanisms.



(a) (b)

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

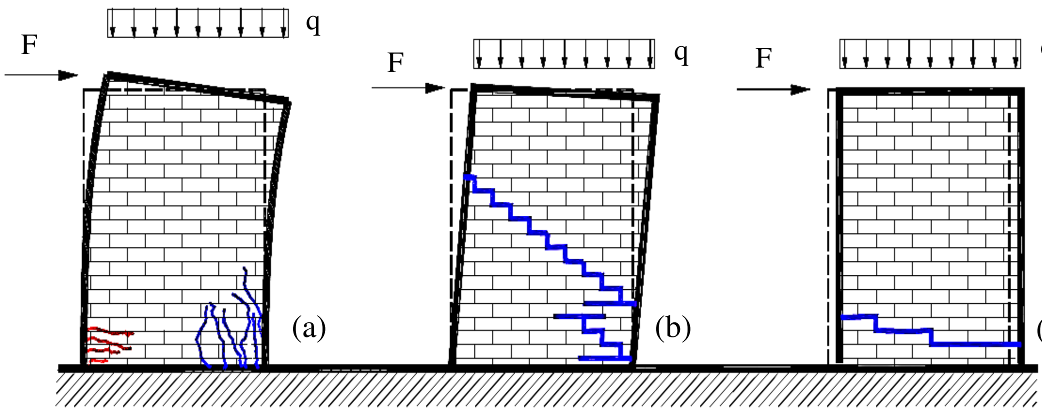


Figure 2.6 Main in-plane failure mechanisms of a masonry portion: (a) flexural failure, (b) shear-diagonal failure, and (c) shear-sliding failure (Caliò and Pantò, 2014)

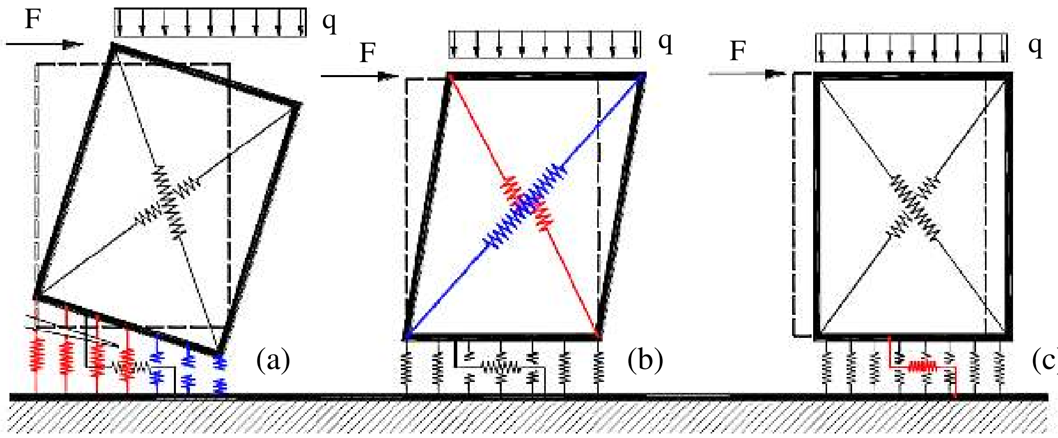


Figure 2.7 Simulation of the main in-plane failure mechanisms of a masonry portion by means of the macro-element: (a) flexural failure, (b) shear-diagonal failure, and (c) shear-sliding failure (Caliò and Pantò, 2014)

In the literature, linear and nonlinear finite-element (FE) models were also used in modeling components and interfaces in RC frames with infill walls. Riddington and Smith (1977) used linear elastic finite element analysis to simulate RC frames with unreinforced masonry infill panels. The possibility of separation between the infill panel and frame and the subsequent loss of friction along the remaining lengths of contact were considered. The influence of a range of parameters on the behavior of infilled frames was investigated. These parameters included boundary conditions, aspect ratios, frame stiffness, beam to column connection rigidity, and multistory and multi-bay configuration. King and Pandey (1978) used the finite element method with friction elements at the interface to model the lateral response of infilled frames.

The change in friction due to the change of normal stress or the separation at the interface was not considered. Coarse finite element meshes were used to determine the lateral stiffness with good agreement with experimental results.

Dhanasekar and Page (1986) constructed iterative non-linear finite element analysis to study the behavior of masonry infilled frames exposed to lateral loading. The material model for the infill wall included elastic and inelastic stress-strain relations and it was capable to simulate progressive cracking and final failure of the infill wall. The model was verified by comparing with experimental results. The model was then used to study the effect of mechanical properties infill wall on the behavior of infilled frame and it was shown that this behavior depended on the relative stiffness of infill wall to the frame, the aspect ratio, and the strength of the infill wall.

Lotfi and Shing (1991) assessed the applicability of representing masonry infills by smeared-crack finite element model. A J2 plasticity model was adopted for un-cracked masonry and nonlinear orthotropic constitutive models for cracked masonry. The model outcomes were compared with experimental data. Their model was capable of capturing the flexure-dominated behavior but not the brittle, shear behavior. A parametric study was also conducted to inspect the influence of different modeling parameters on the behavior of the system.

Computational models were developed by Mehrabi and Shing (1997) to simulate the conduct of masonry infilled RC structures using smeared-crack finite elements for frame concrete and infill wall bricks. They developed a constitutive model that accounts for physical characteristics of interfaces to simulate the conduct of mortar under cyclic loading. Even though their computational model could estimate the nonlinear behavior of infilled frames at times, it failed to capture some of the failure mechanisms observed in their tests.

Citto (2008) developed an interface constitutive model for fracture initiation and propagation in masonry walls under normal and shear stresses and implemented it in ABAQUS (2007). A compressive cap was included to account for crushing failure. Al-Chaar and Mehrabi (2008) performed numerical simulations on DIANA (v. 8.1) to study the nonlinear behavior of infilled frames. The mortar joints between bricks and frame and between the bricks themselves were modeled as cohesive interfaces to account for shear behavior along the mortar joints. A smeared-crack model was used for frame concrete and brick units. The results were compared with experimental outcomes. It was shown that using interface elements in the column ends overcomes the inability of the smeared-crack model to model the shear behavior.

Stavridis and Shing (2010) used smeared and discrete crack models to simulate the behavior of infilled frames. They used 51 parameters to define mortar, infill brick units, concrete, and reinforcing bars for their computational models. Their discretization method was unique and developed to overcome the stress locking problem associated with smeared-crack model while modeling the shear behavior in concrete members and brick units. They assessed the influence of material properties in their numerical results and suggested that initial (un-cracked) the mortar shear strength had the highest influence on results.

Koutromanos et al. (2011) demonstrated the ability of finite elements to simulate the behavior of RC frames infilled with masonry walls. Nonlinear FE models were modified to simulate the cyclic closing and opening of cracks, the reversible shear behavior, stiffness and strength degradation in tensile region, and hardening–softening behavior in the compressive region. They developed a new cohesive-crack interface model and enhanced the smeared-crack model developed by Stavridis and Shing (2010) to capture the cyclic behavior of infilled RC frames. Their FE model for the three-story two-bay RC frame with infill walls tested on a shake table by Stavridis were able to capture the experimental response observed. In their models, the expected shear crack initiation and propagation planes need to be predefined manually. This approach requires a tedious calibration process and the models needs heavy computational effort.

2.2 Failure Modes of RC Buildings with Infill Walls

Observations from past earthquakes (including 1999 Kocaeli, Turkey earthquake, the 2008 Sichuan, China earthquake, and the 1999 Chi-Chi, Taiwan Earthquake) show severe damage to these types of structures, as illustrated in Figure 2.8. Damage patterns include partial or full failure of masonry panels, shear failure of columns, column plastic hinges, soft story mechanisms, short column shear failures, etc. (Li et al., 2008; Sezen et al., 2003).

The mechanism of failures of the masonry infilled structures is complicated because of the large number of parameters involved in the seismic response of the frame like the configuration, material property, and relative stiffness of the structure to the infill walls, detailing, etc. The results of experiments have shown that masonry infilled structures can experience a vast assortment of the failure modes, as shown in Figure 2.9. Mehrabi (1994) defined twenty four different mechanisms of in-plane failure for infilled frames, based on the experiment specimens of fourteen ½-scale single-story frames. These mechanisms can be minimized to three as proposed by Stavridis (2009).

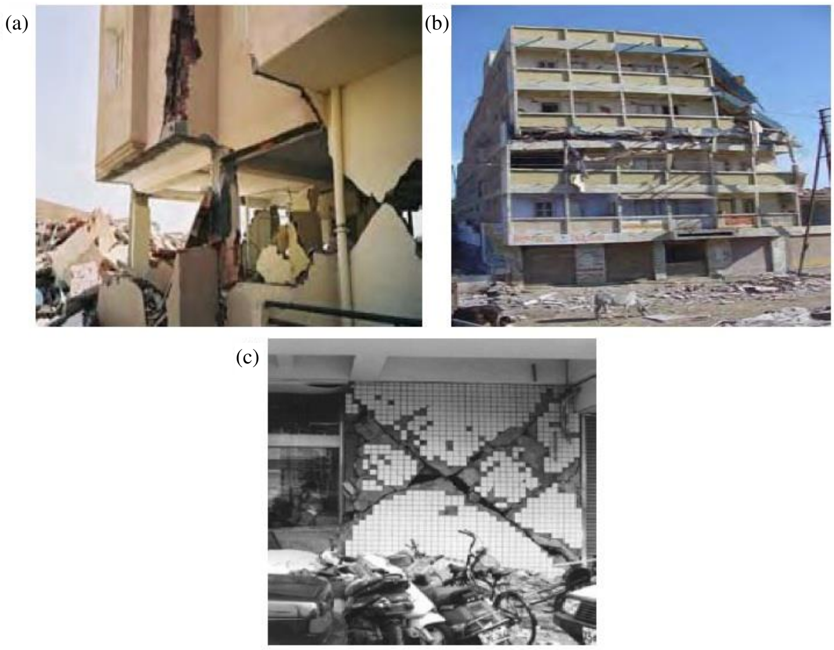


Figure 2.8 Different failure mechanisms in: (a) Algeria (2003 Boumerdes Earthquake), masonry wall failure (photo: S. Brzev); (b) India (2001 Bhuj earthquake), (photo: EERI, 2001); (c) Taiwan, (1999 Chi-Chi earthquake) (photog: Charleson, 2008)

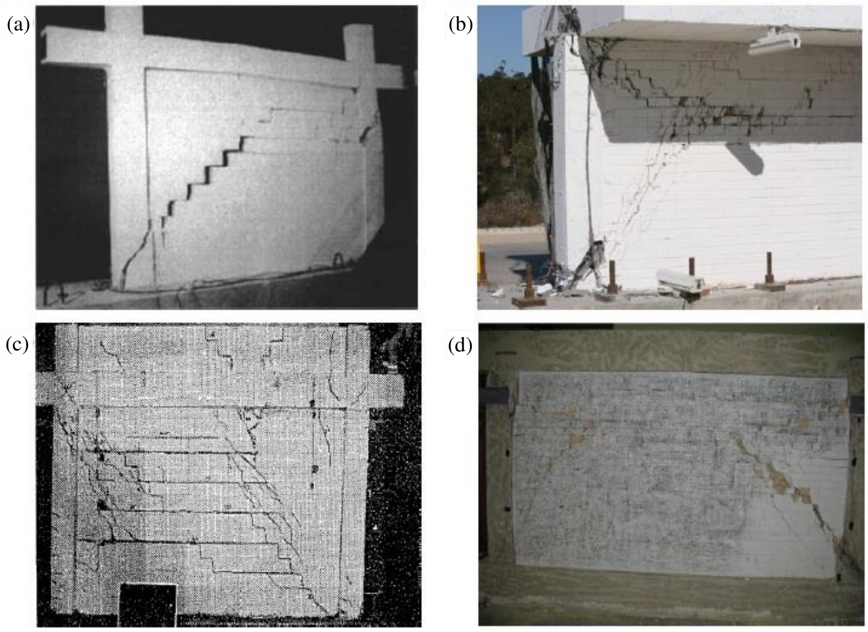


Figure 2.9 Failure mechanisms of the infilled frames observed in the experiments conducted by: (a) Al-Chaar et al. (2002); (b) Stavridis (2009); (c) Mehrabi (1994); and (d) Blackard et al. (2009)

These three mechanisms have been defined such that they describe the full range of behavior exhibited in Mehrabi’s tests, as shown in Figure 2.10:

i. Diagonal cracking in the infill panel with shear failure in columns or, more rarely, plastic hinges in columns. This failure typically takes place in weak/non-ductile buildings with powerful infill panel;
ii. Horizontal sliding of the masonry with shear or flexural failure of the columns. Infill crushing is sometimes observed in these tests. This failure mechanism was observed in the weak frames with weak panels and also in the strong and ductile frames with weak infill panels;
iii. Infill corner crushing with flexural failure in the columns. This mechanism is most likely in strong and ductile frames with strong infill.

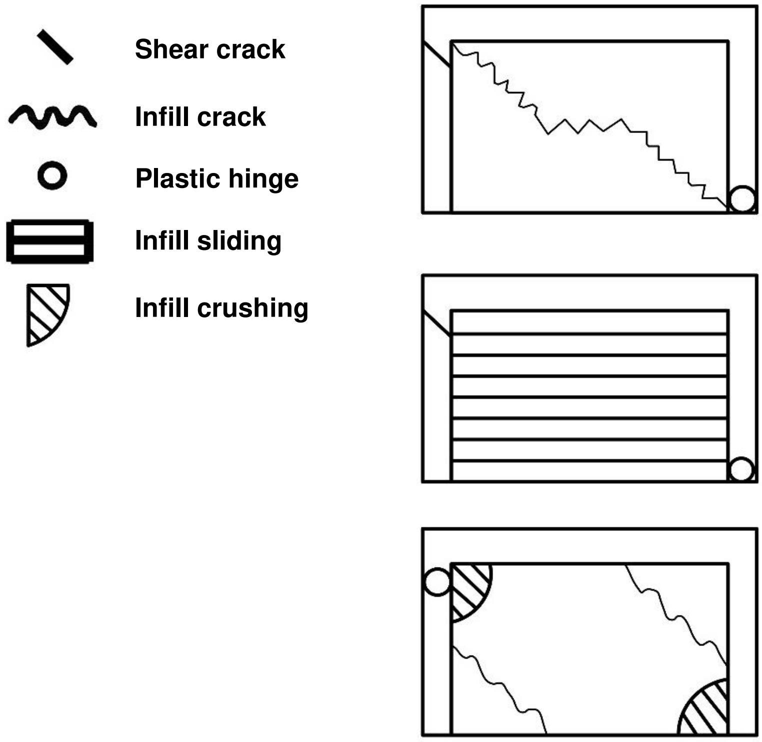


Figure 2.10 Failure mechanisms of infilled frames (Mehrabi, 1994)

Similarly, (El-Dakhakhni et al., 2003) classified the mechanisms of failure of masonry infilled structures into five distinguished modes, illustrated in Figure 2.11:

- Corner crushing failure, which is associated with strong frame with weak infill (similar to failure mode (iii) above).
- Sliding shear failure, associated with weak mortar joint infill bounded with strong frame (same as failure mode (ii), above).
- Diagonal compression failure, associated with slender flexible infill walls.
- Diagonal cracking failure, associated with weak frame with relatively strong infill (similar to failure mode (i) above).
- A frame bending failure mode which is associated with weak frame with weak infill.

Crisafulli (1997) reported the most common failure mode of the masonry panel from his experimental data is the shear failure of the masonry panel by stepped debonding (mode (i) in Figure 2.10 or mode (d) in Figure 2.11).

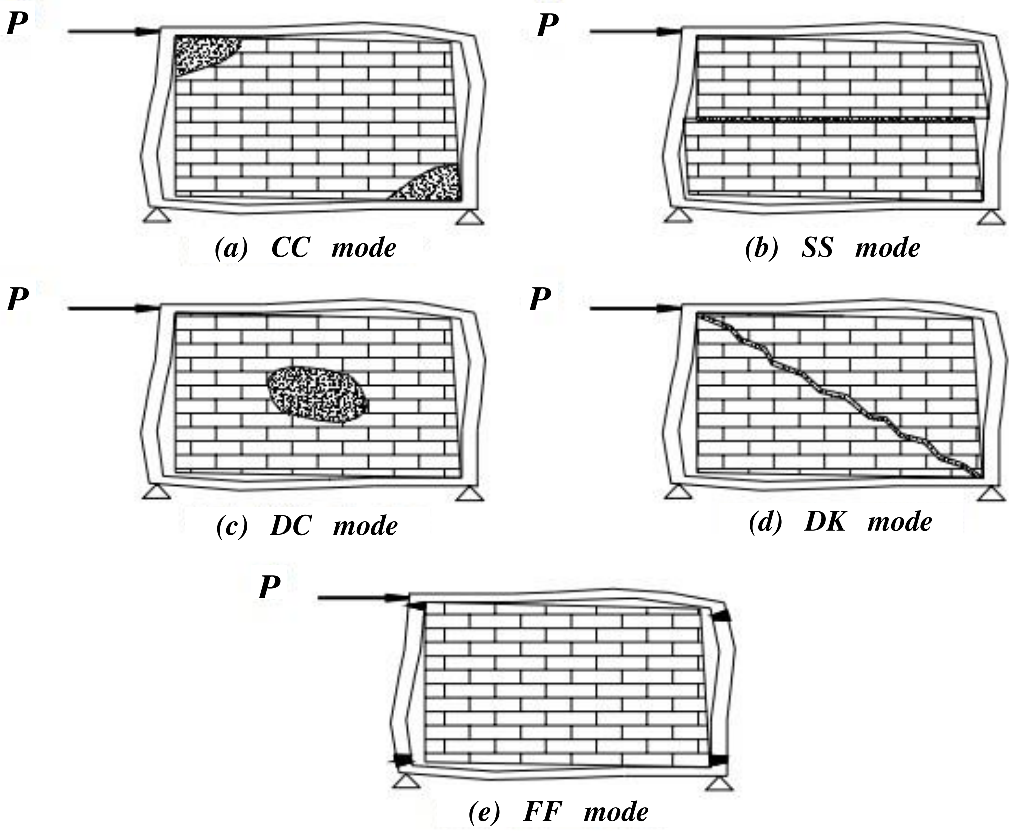


Figure 2.11 Different failure modes of the infilled frames: (a) corner crushing; (b) sliding shear; (c) diagonal compression; (d) diagonal cracking; and (e) frame bending failure (El-Dakhakhni et al., 2003)

The failure modes of in-filled frames with openings were investigated experimentally by Asteris et al. (2011). The experimental study was based on testing ten specimens of one-bay, one-story of RC structures built at scale of 1/3. Each structure involved an infill of clay brick with opening. This study investigated three parameters, i.e. the location of opening, the shape of opening, and the size of opening (as percentage of the infill panel area) within the structure. The crack patterns of the tested specimens were compared (Figures 2.12 and 2.13) and it was concluded the following:

- The failure mechanism of the structure in case of weak infill with window opening is controlled by shear sliding of masonry below and above the window, internal crushing of masonry parts between window and columns, and plastic hinges at ends of the columns.
- The failure mechanism of the structure in case of weak infill with door opening is controlled by shear sliding of masonry above the door, plastic

hinges at ends of the columns, internal crushing of the other masonry parts between door and column, corner–toe crushing due to rocking of masonry parts between the door and the column in tension (in case of eccentric door).

- As predicted, the opening build upon the diagonal of infills leads to the abolishment of the failure modes of Diagonal Cracking (DK) and Diagonal Compression (DC), which is interpretable assuming that the major compression strut is not composed.

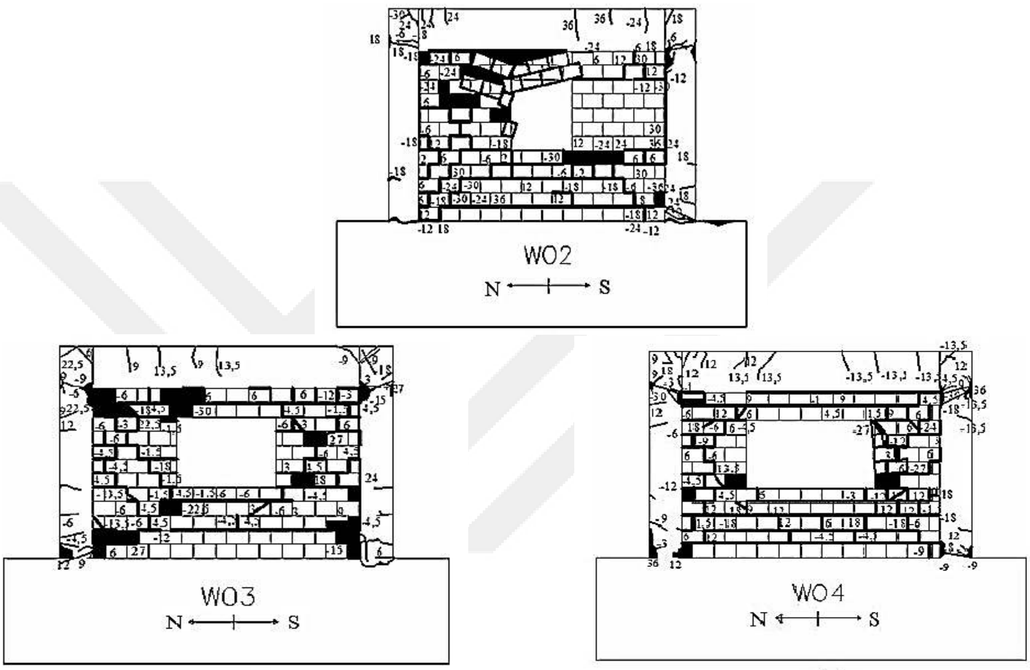


Figure 2.12 Failure modes of specimens with three different percentages of window openings (Asteris et al., 2011)

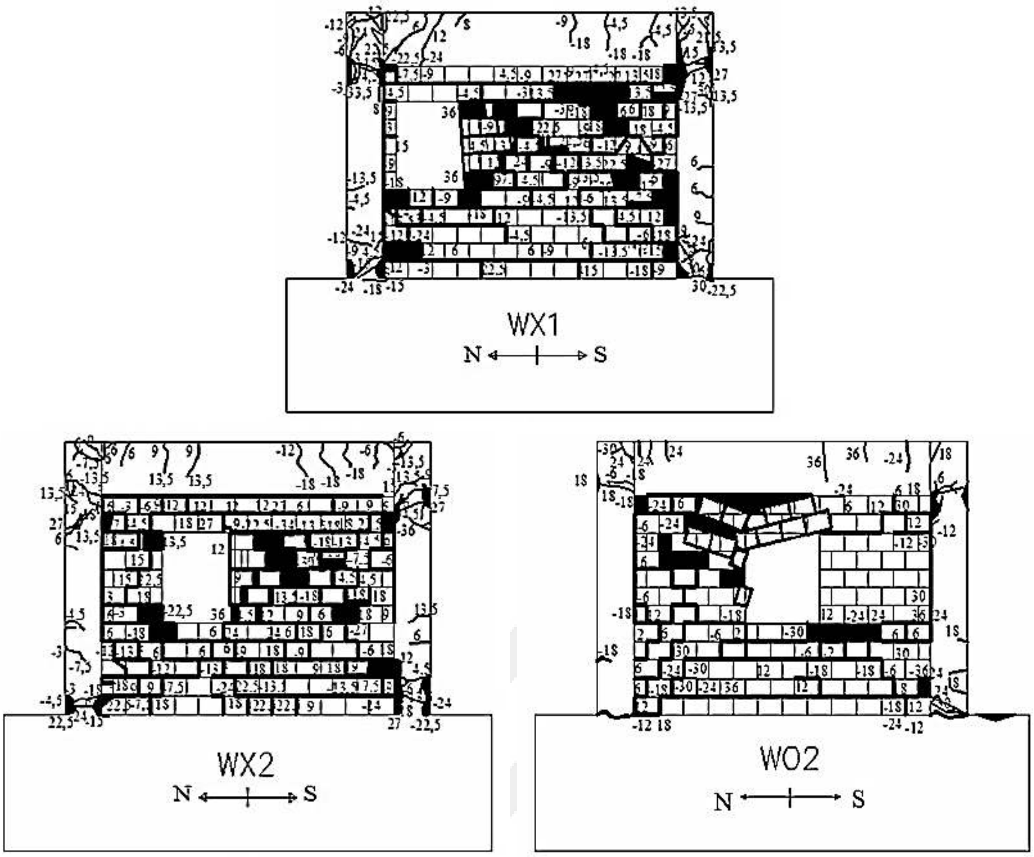
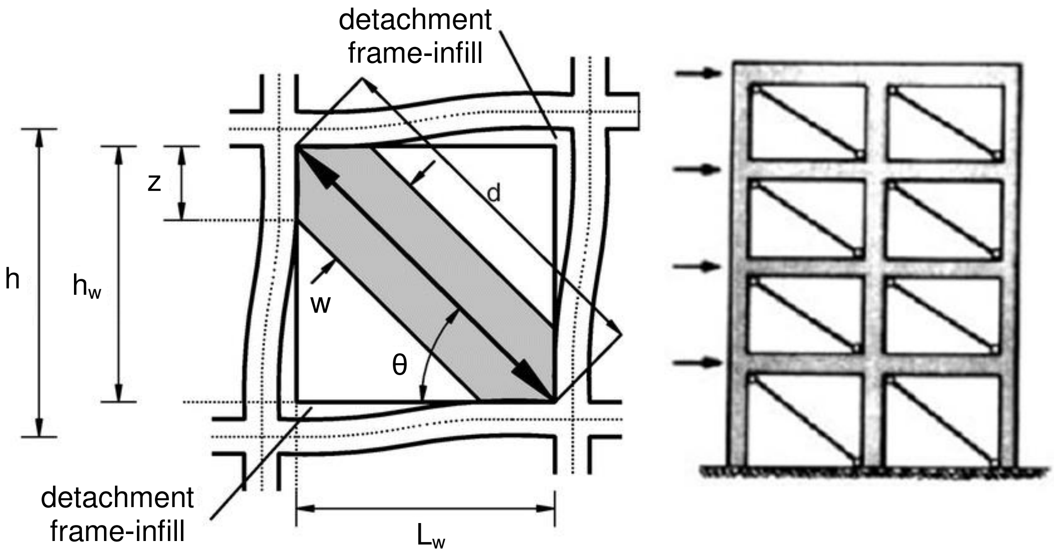


Figure 2.13 Failure modes of in-filled frame specimens with window openings and various opening locations (Asteris et al., 2011)

2.3 Equivalent Diagonal Strut Model for Infill Panels

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 using the equivalent diagonal compression struts (Figure 2.14b). By increasing loads gradually, infill wall panels and the frame (coupled with both vertical and horizontal slip for the large dimension interaction surfaces) is separated at the nodes (Figure 2.14a), observing that the stress in the panel is increased. This separation determine progressively the behavior of infills of initial shear, due to the axial stress in the corners which is in compression and infills still have interaction with frame. Because of this the idea to model the infilled structure as a braced system is reasonable, that the axial behavior of the panels is simulated by equivalent diagonal compression struts (Uva et al., 2012).



(a) (b)

Figure 2.14 the equivalent strut model (Uva et al., 2012)

The model of the equivalent diagonal compression strut depends on some essential factors, where the details would be mentioned the historical review as the following:

- Equivalent strut width b_w,
- The panels constitutive relationship, and
- Number of introduced equivalent struts.

In the literature, there are many studies dealt with the geometric properties of the infill walls to represent them in terms of the stiffness and the strength properties equivalence. Usually the strut thickness assumed and taken as same as the masonry infill wall thickness, whereas the researchers prepare different proposals for determining the width b_w. Fundamentally, there are two main categories. A first approach simply take the diagonal length of the panel as a fundamental parameter to define b_w, focuses on the geometric properties as an essential than to the mechanical properties. Instead, the second approach takes both the geometry and the mechanical properties of the infilled frame for introducing diagonal struts and providing more refined numerical formulations (Uva et al., 2012).

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 increase, the behavior becomes more sophisticate, because the infill panel tried to deform in a shear mode whilst the frame tried to deform in a flexural mode, as shown in Figure 2.14(a). As a result the infill panel and surrounded frame is separate at the two corners on the tension diagonal. On the compression diagonal, contact between infill panel and surrounded frame occurs for a length z, as shown in Figure 2.14(a). The effective width of the diagonal compression strut after separation, W, is less than that of the full panel, as shown in Figure 2.14(a). 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 relies 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 has a higher seismic response. It is sensible to take a conservatively high value of equivalent strut width. Thus, a moderate value of a quarter of the diagonal length was proposed as strut width (W = 0.25d). This value can be used as an approximate estimation of the elastic period of the infilled frame (Paulay and Priestley, 1992).

The first researcher who has dealt with this issue was 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 infill walls loaded in compression diagonally, he suggested a diagonal strut concept that could be equivalent to the effect of the masonry panels in infilled structures exposed to lateral loads as shown in Figure 2.15.

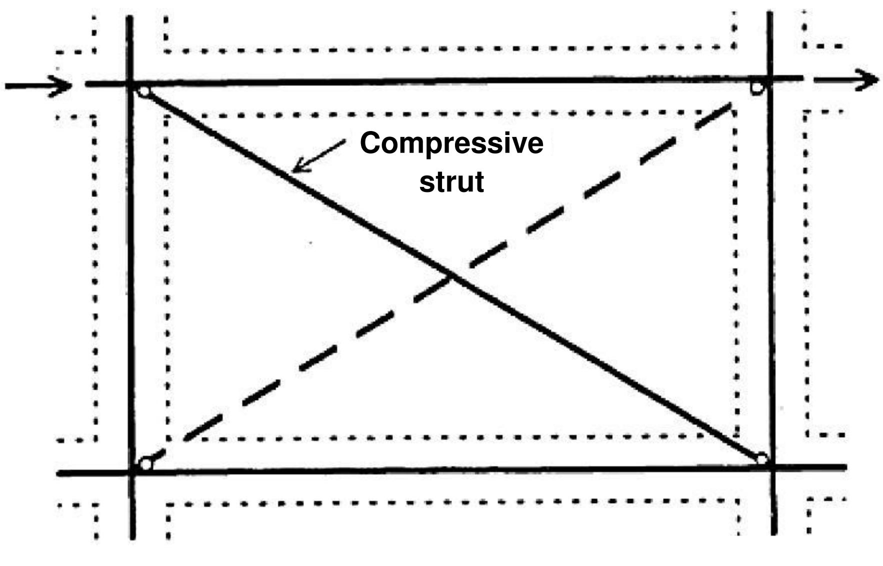


Figure 2.15 Diagonal strut model for infilled frame (Abdelkareem et al., 2013)

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 1/3 of the diagonal length between the two compressed corners for the equivalent strut width (Eqn. (2.1)) as shown in Figure 2.16.

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

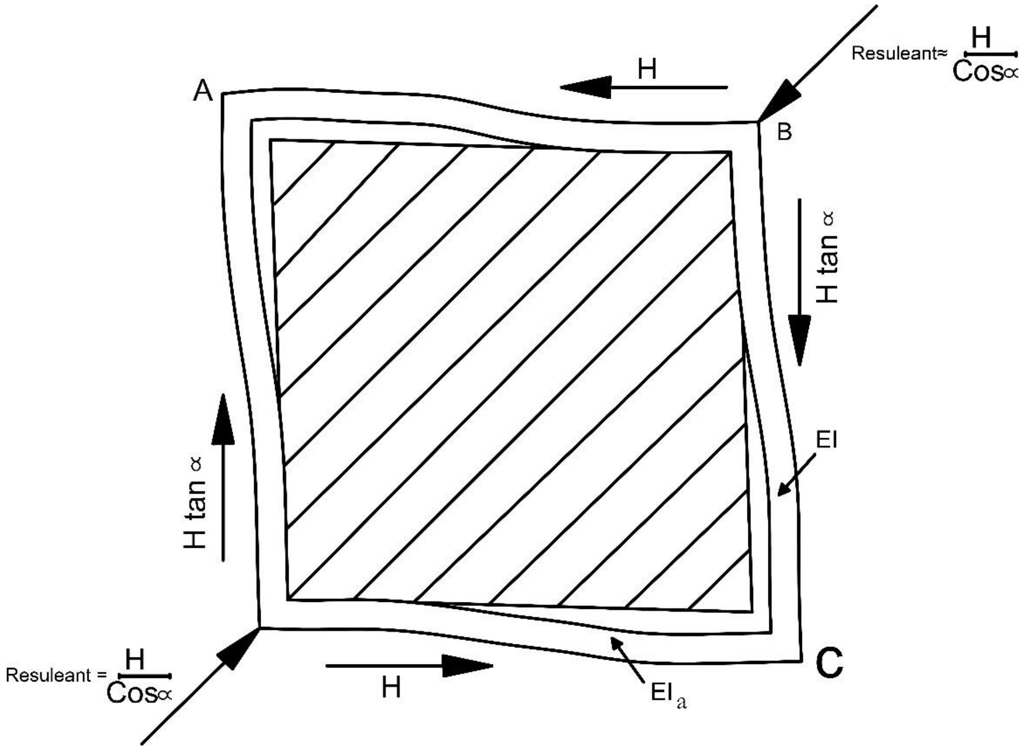


Figure 2.16 Shear force on steel frame with infill (Holmes, 1961)

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

Holmes (1963) prepared a semi-empirical method for anticipating the behavior of infilled frames exposed to vertical and horizontal loadings. He described this method based on the tests on model steel structures infilled with concrete walls. On the other hands, so as to participate the mechanical characteristics (actual variability) of the infill panel into account, for defining the strut width b_w besides including the diagonal length (d), the mechanical properties of both the surrounding frames and the masonry infill panel. Consider the infill panel cracking and the level of degradation failure is particularly important.

The first researcher who had taken the mechanical properties of infill panel was Smith and Carter (1969) by an experimental study. It is revealed that the strength and stiffness of the infilling panel are 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 wall to the surrounding structure and is given by an equation as follows:

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

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

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

Considering α, to be the length of the infill panel contact with the column, and Eqn. (2.4) is used to plot (α/h) as a function of λh, as shown in Figure 2.17.

They replaced the infill walls by equivalent compression struts as shown in Figure 2.18. The width of equivalent strut is defined as follows:

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

Mainstone (1974) considered the relative flexibility of infill panel to frame for an appraisal of the equivalent strut width of the infill panel. He suggested an empirical relationship based on analytical and experimental data, for calculation the width of the equivalent compression strut:

$$b_w = 0.175d\, (\lambda_h H_w)^{-0.4} \tag{2.5}$$

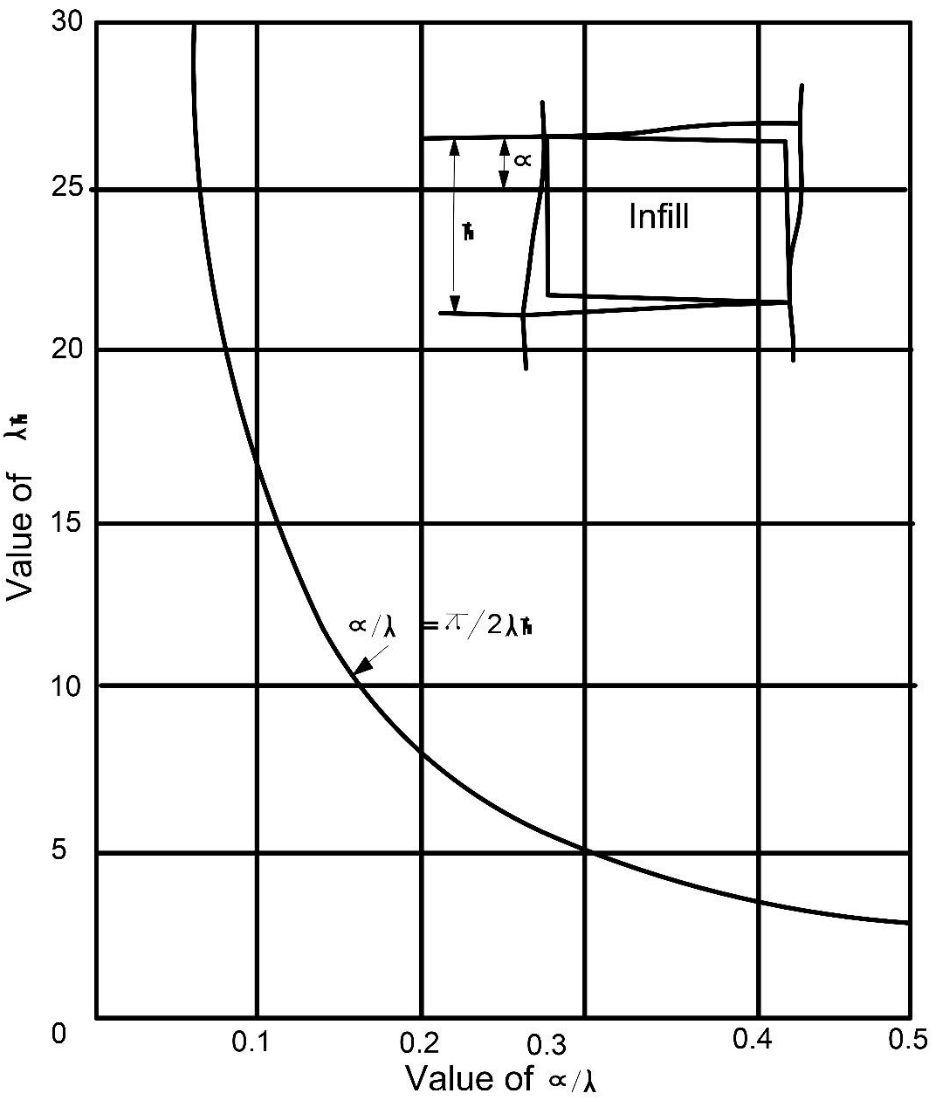


Figure 2.17 Length of contact as a function of λh (Smith and Carter, 1969)

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

$$b_w = 0.175d\ (\lambda_h H_w)^{-0.4} \tag{2.6}$$

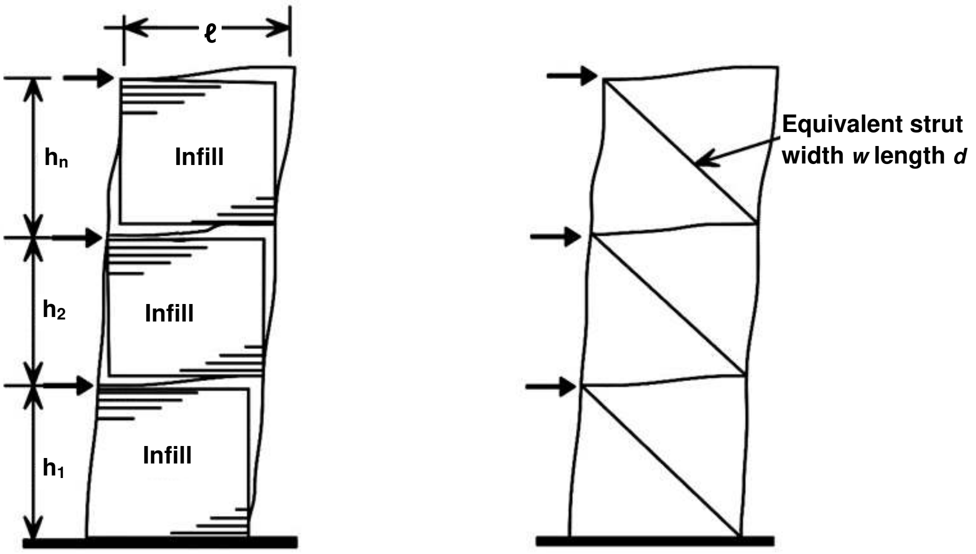
Later depend on the detailed numerical finite element analysis; Eqn. (2.5) was modified by Durrani and Luo (1994) who proposed a relationship in order to introduce the frame element geometry by means of the coefficient parameter m:

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

Where m is given by;

$$M = 6\left(1 + \frac{6\,E_c\,I_b\,H}{\pi\,E_c\,I_c\,\;L}\right) \tag{2.8}$$

$$b_w = \gamma\,\sqrt{L^2 + H^2}\,sin2\theta \tag{2.9}$$



(a) (b)

Figure 2.18 View of laterally loaded (a) infilled frame; and (b) equivalent frame (Smith and Carter, 1969)

According to Abdul-Kadir (1974), actually besides the adjacent columns, the top beam has an effect on the width of the equivalent struts. He divided the character λ into two different factors λ_P and λ_T, which are correlated to the upper beams and to the adjacent columns. Particularly, the parameter of λ_P, can be determined by Eqn. (2.3) as well, while λ_T is proposed by the following expression:

$$\lambda_T = \sqrt[4]{\frac{E_W\,t_W\,\sin(2\theta)}{4\,E_c\,I_T\,H_W}} \tag{2.10}$$

The width b_w of the equivalent diagonal strut (Figure 2.19), as a relationship of the above parameters described, is defining by following equation:

$$b_w = \frac{1}{2}\sqrt{\alpha_1^2 + \alpha_h^2}$$

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

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

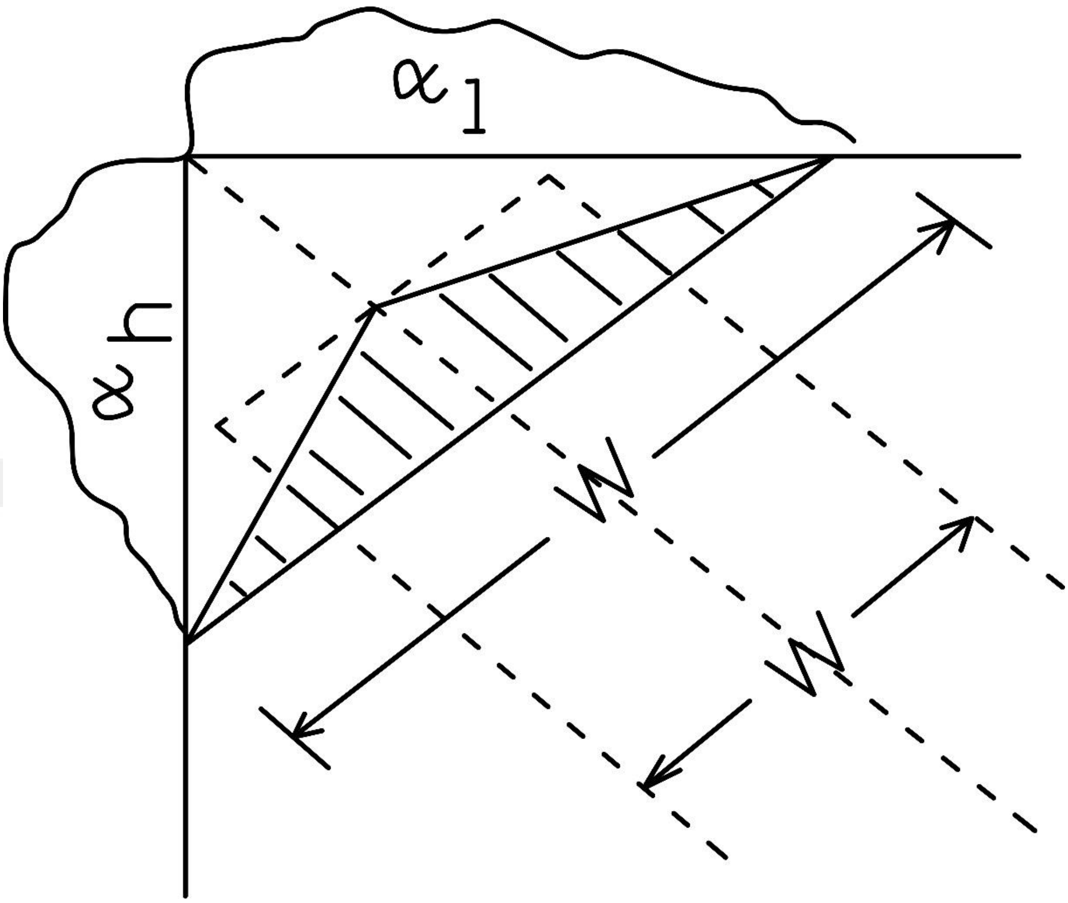


Figure 2.19 Width of the equivalent strut (Abdul-Kadir, 1974)

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). The equivalent strut width (b_w) is defined by the following expression:

$$b_w = 0.175d\,(\lambda_h H_w)^{-0.4} \tag{2.12}$$

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 2.20. 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 l_{column} from the beam face and should be pin-connected. This distance is given by the following relationship;

$$l_{\text{column}} = \frac{w}{\cos\theta_{\text{column}}} \tag{2.13}$$

$$\tan\theta_{column} = \frac{H_{inf} - \frac{w}{\cos\theta_{column}}}{L_{inf}} \tag{2.14}$$

Where Al-Chaar (2002) calculated the width of the strut (w) by using Mainstone's (1974) equation without any reduction factors.

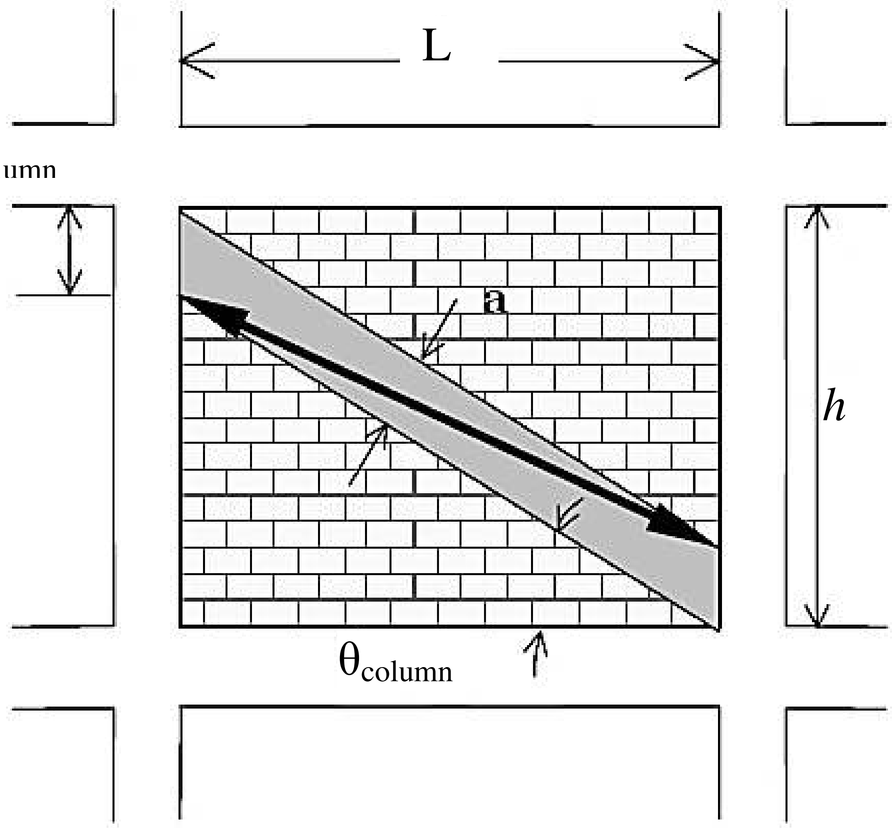


Figure 2.20 Placement of strut (Al-Chaar, 2002)

In a comparable study, Samoilă (2012) assessed different relations available in the literature for determining the width of the equivalent compressed strut. He recommended using the relation of Paulay and Priestley (1992). He had taken several models and studied the effect of infill panel on frame members, as finite element models, of the three-strut model and the single-strut model. It was revealed that in the analysis related to the general conduct of infilled structures, the single-strut model is the appropriate approach because it can be accepted as correct idea and due to its simplicity. Whereas for determining the local effects of frame infill interaction it is better to use the three-struts model.

In fact, the use of macro-models has a main critical trouble represented in the difficulty of getting the mechanical properties of the infills and the geometrical characteristics of the equivalent diagonal struts with an adequate accuracy (Uva et al., 2012).

According to Verderame et al. (2011), the infill wall must be represented by nonlinear modeling because of the damage suffered by the infill walls, and infill walls must be modeled by a three-strut nonlinear model. They revealed that the three-strut model is differing from the single-strut model and it allows taking into consideration local interacting between the infill panel and the bounding RC elements. They adopted six struts, three in each diagonal direction, to represent infill wall panel, which carrying load only in compression. In fact, the single- strut model is easy to be performed but it is unable to get the actual distribution of bending moments and shear forces in structure members due to the absence of local interaction with the infill panel. The central strut connected the two opposite comers along the diagonal of the frame bay and position of the two off-diagonal struts depends on the stretched contact area, that transfer the load between the RC structure members and the infill panel (see Figure 2.21). The length of this contact area depends on the amount of lateral load that act on the frame and the deflected shape and on the stiffness of the frame members.

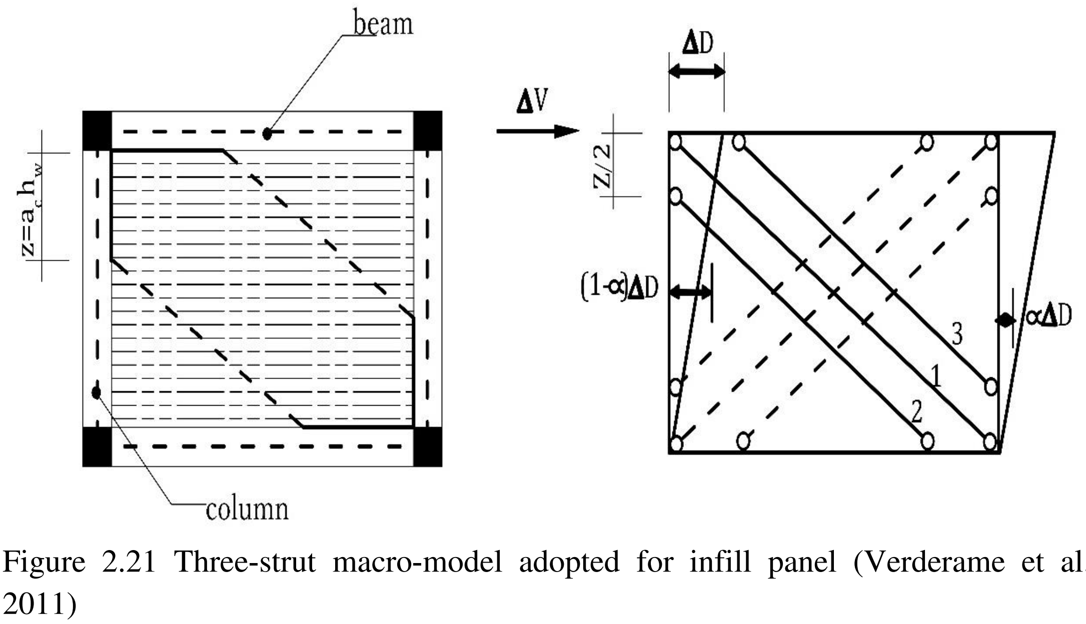


Figure 2.21 Three-strut macro-model adopted for infill panel (Verderame et al., 2011)

According to Uva et al. (2012), with respect to the specific model that used multiple struts for an infill wall panel, it was not appropriate to prefer one and indicate it as better than the others. In fact, a great level of variability has been noticed among various relationships that have been tested. Indisputable, as an objective, it cannot provide a better indication among them because so far cannot find an experimental expression for explaining the real structural behavior.

2.4 Infilled RC Buildings with Wall Openings

The role of openings in infill wall on the structural behavior of RC buildings is an important subject since the openings are required due to the functional requirements such as doors, windows, and other openings. In the literature, some experimental and numerical studies are available. For example, Fiorato et al. (1970) tested scaled specimens of brick masonry infilled RC structure in order to investigate the influence of several parameters like number of storeys and opening in infill walls. They concluded that the openings within infills somewhat decreased the stiffness of the structural system.

Mallick and Garg (1971) discussed the effects of the location of openings in the infill on lateral stiffness and ultimate load. They recommended suitable positions for door and window openings. In their study, the effects of openings in the infill and shear connectors between the infills and the frame were investigated. Openings at either end of a loaded comer without shear connectors reduce the stiffness by 75%, and they reduce the stiffness by 85 to 90% compared to infilled frames without openings. The stiffness is reduced by 60 to 70% for infilled frames with shear connectors and openings, versus one without openings. A central opening within the infilled panel reduced stiffness by 25 to 50% as compared to the frames without openings. Doors are best placed in the center of the lower half, and windows are best placed at mid-point of the height of left or right side, toward the vertical edge as possible.

In 1972 Liauw extended the equivalent diagonal strut concept to include frames with infills having openings. Liauw's work was based on the results of two elastic models of frame infilled with rubber and gelatine. The comparison had showed good agreement when the central opening is exceed 50% of the area of full infill wall.

Dawe and Seah (1989) observed that in masonry infill panel when opening is in existence, the initial main crack load is decreased whilst the maximum load may not be affected considerably. When a doorway opening location is appeared near the loaded side of the panel, the maximum load is greater affected when compared to the opening is far from the applied maximum load. It seems that if the opening must be presence and cut off the diagonal compression path load, it is better that it is positioned so that as much panel material as possible is between it and the load point. This produces an opportunity to diagonal strut effect to develop in that segment 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 door openings is at the central region 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 was concluded that if the vertical reinforcement is tied completely to the roof beam, floor beam, and the lintel to form an arrangement of subpanel, ultimate strength and the cracking load would be higher.

Mosalam et al. (1997) studied five specimens to investigate the behavior of steel structures infilled with unreinforced masonry and exposed to lateral cyclic loads. The powerful blocks had compressive strength of 30 MPa and the weakly ones were of 13 MPa strength. It was concluded the following from their study:

- Solid walls make the infilled structure more brittle than the wall with openings.
- The openings decreases the stiffness by 40% in case of lateral loads less than the cracking load, and causes the maximum load to be considerably higher than the cracking load.
- Solid walls and walls with window openings had similar maximum load capabilities whilst the door openings decreased the maximum load capability by 20%.

Asteris (2003) demonstrates analytical results which investigate the effects of the location and size of openings on the masonry infill wall in earthquake response of infilled masonry frame buildings. He used the stiffness reduction factor λ (λ is the ratio of the stiffness of infill panel having opening to the solid panel infills stiffness) for purpose of comparison with bibliographical data. He analyzed a different configuration model of one story and one bay infilled frame buildings. Presented some parameters and some cases for understanding the infill panel frames behavior under lateral seismic loading and the effect of openings on the infill frames lateral stiffness such as; (the openings percentage (opening area/area of infill panel wall), the existence or absence of openings in the infill wall, and the openings position on the diagonal compression strut), investigated for three cases as mentioned in the following:

- Case A: Openings position is undern the diagonal compression strut,
- Case B: openings position is upon the diagonal compression strut, and
- Case C: openings position is above the diagonal compression strut.

The variation of the λ factor is shown in Figure 2.22 as an openings percentage function (ratio of the opening area to the total area of infill panel) for openings position is upon the diagonal compression strut (Case B) of the infill masonry wall (with openings dimension proportion is the same as the infill walls dimension proportion). As expected, by increasing the opening percentage the frames stiffness is decreased. Figure 2.23 illustrates the studied for the variation of the stiffness reduction factors λ of the masonry infilled frame for the three different positions (as mentioned above) as a function of the percentage of the opening (the effect of opening percentage on the stiffness reduction of infilled frames). When the opening is upon the diagonal compressed (Case B) the stiffness reduction of the infill frame building can be observed as a higher value as compared to the (Case A and C). Because in this case (Case B) the diagonal compression strut action of the infill wall was interrupted (Asteris, 2003).

This stiffness reduction factors λ value that obtained from these diagrams (Figures 2.22 and 2.23) can be used to make better estimation calculation of the diagonal compression struts width. For purposes of practical design, Asteris (2003) propose the equivalent strut widths estimation based on the Mainstone's (1974) formula, by the following simplified equation:

$$w/d = 0.175\lambda\,(\lambda h)^{-0.4} \tag{2.15}$$

$$\lambda h = h^4 \sqrt{\frac{E_b\ t\ \sin 2\theta}{4\ E_s\ Ih}} \tag{2.16}$$

In Anil (2007) experimental study, the behavior of RC frames with partially RC infills was investigated. A test frame of single-bay single-storey with 1/3 scale under reversed cyclic loads. Six specimens of infill walled frames with various sizes and positions of opening were tested. The test results showed that the partial infilled frames had more strength and stiffness than the bare frame.

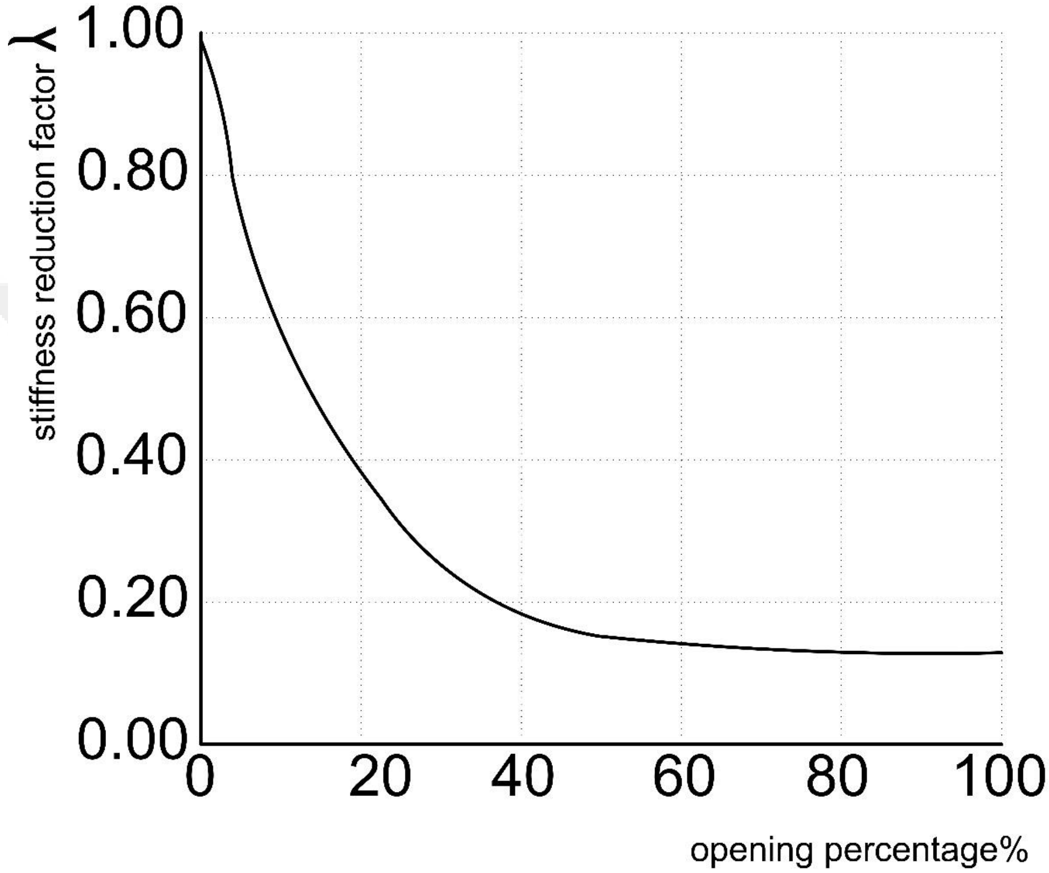


Figure 2.22 Stiffness reduction factor λ of infilled frame in relation to opening percentage (case B: opening upon compressed diagonal) (Asteris, 2003)

Mondal and Jain (2008) published their theory regarding central openings and pointed out that the original compressive strut cannot be considered with the same width anymore, but the new effective width has to be multiplied by a reduction factor. They studied three different frame models. These models were one storey, two storey and three storey frames with one bay. The analysis states the following three equations for the reduction factor:

$$\rho_w = 1 - 2.47\alpha_{co} \text{ (one storey)} \quad (2.17)$$

$$\rho_w = 1 - 2.56\alpha_{co} \text{ (two storey)} \quad (2.18)$$

$$\rho_w = 1\text{-}2.56\alpha_{co} \text{ (three storey)} \quad (2.19)$$

These equations show the insensitivity of strut-width reduction factor to the number of storeys.

Depending on the above equations and linear fit curve, the reduction factor for the infilled structure with opening can be simplified as:

$$\rho_w = 1\text{-}2.6\alpha_{co} \quad (2.20)$$

Where, α_{co} is the ratio of the opening area to the total area of the infill panel.

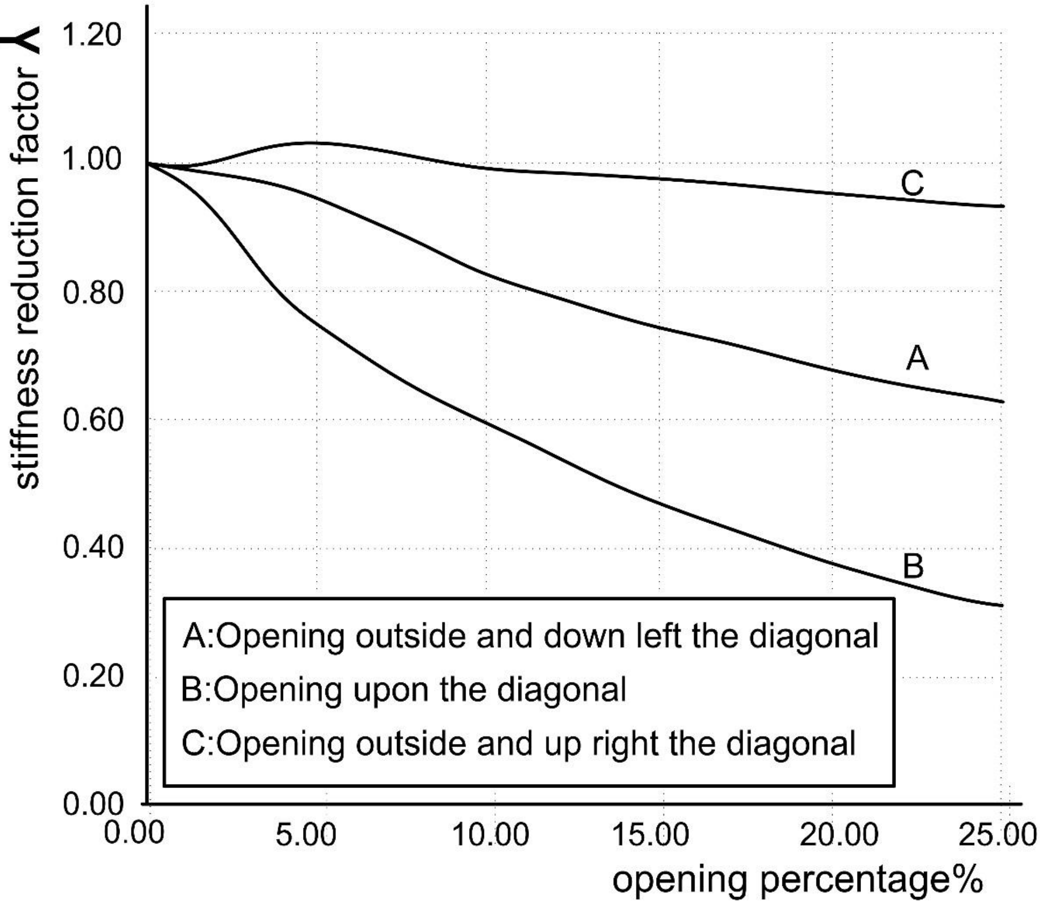


Figure 2.23 Stiffness reduction factor λ of infilled frame in relation to opening percentage for different positions of opening (Asteris, 2003)

The expression (Eqn. (2.20)) proposed by Mondal and Jain (2008) for the reduction factor is also given in Figure 2.24. If this curve is compared to the linear fit curve, the proposed equation shows a good estimation for initial lateral stiffness of the infilled structures having central openings. Thereby, they concluded that the contribution of stiffness of the infilled walls should be ignored if the area of the opening is greater than 40% of the total area of infill. Also, the presence of openings could be ignored if the opening area is less than 5% of the total area of infill wall, and that the reduction factor of the strut-width must be set as unit.

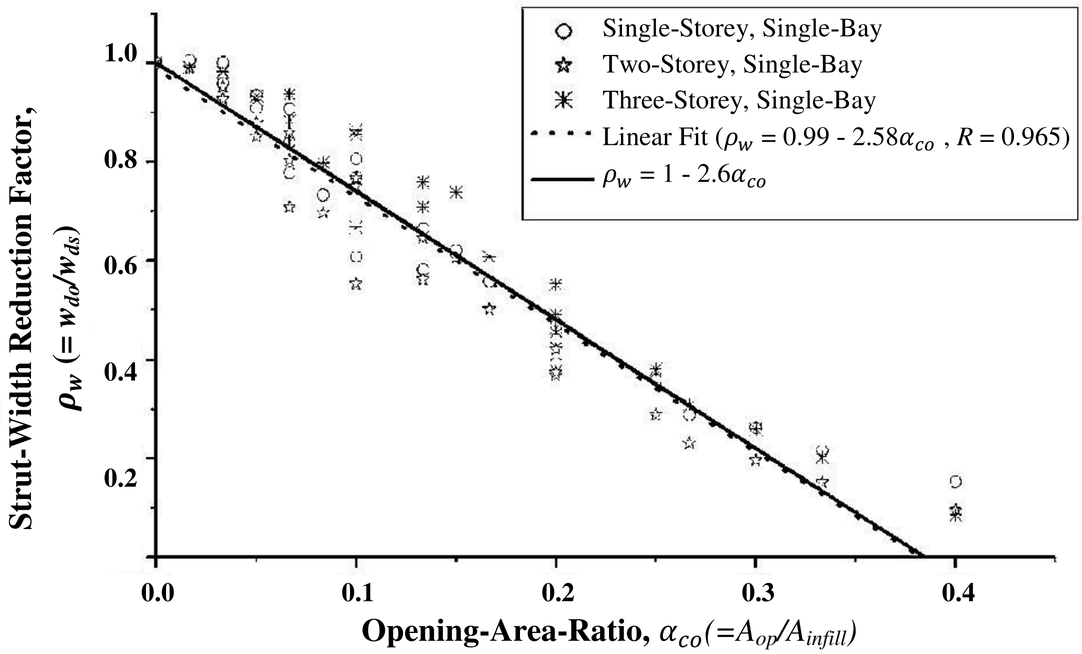


Figure 2.24 Linear trend line of all the merged data and comparison with the proposed equation of strut-width reduction factor (Mondal and Jain, 2008)

Dolšek and Fajfar (2008) investigated the earthquake behavior of infilled RC frames of four-story. The N2 method was used which is developed by Fajfar and based on nonlinear analysis technique. A fully-infilled frames (with solid infills) and partially-infilled frames (with infills having openings) were analyzed and compared to the case of a bare frame. The aim of designing such frames was to represent the typical building sorts built in Mediterranean and European countries forty to fifty years ago. The tested frames details are given in Figure 2.25. The infill panels of hollow clay bricks were modeled by the equivalent diagonal strut method. By using the results of previously implemented experiments of the identical frames, the analytical models were validated and calibrated. By comparing the analysis results, it was noticed that the structure of the infilled walls with openings has changed the damage distribution completely.

There are still great difficulties unresolved concerning the modeling approach of infilled structures with openings. Openings of door and window in the infill masonry panel decrease the effective width of the equivalent compression diagonal struts. As a result, the lateral load bearing capability of the infilled structure buildings is decreased by the existence 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 position and size of the opening with respect to the equivalent compression diagonal strut. The location of the openings with respect to diagonal strut can be classified as above, under, or over the equivalent compression diagonal strut (Kose, 2009).

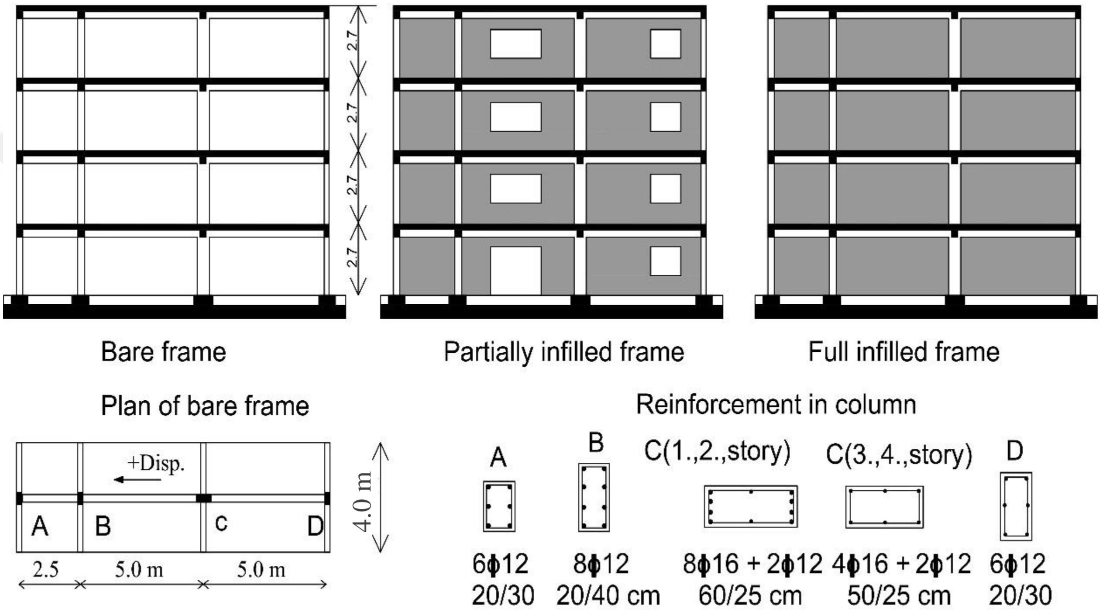


Figure 2.25 Layout of the frames and reinforcement details of the columns (Dolšek and Fajfar, 2008)

Mohammadi et al. (2010) performed a two-phase study to examine the effectiveness of various methods in order to enhance the flexibility of infilled structures. One of these methods called cornerless infill. The structures were infilled by panels of three-ply consist of concrete and masonry materials. The test specimens were exposed to five cycles of loading with drift ratio range of 0.5-2.5% and pushed, if possible, to failure. The cornerless infill method was attained by removing the masonry from the panel corners. This method was aimed to transfer the failure from corners to beams, because the beams are usually weaker than the columns. However, corner crushing failure took place in the upper corners beneath the openings. The stiffness and strength of the infilled system was noticed to be considerably less than those of the fully infilled frame.

Nwofor and Chinwah (2012) investigated the effect of brick masonry infills with various sizes and position configurations of openings on the shear strength of infilled structures. The finite element modeling technique is used to model those structures. The results achieved by the analytical study were compared with the results of experimental test in order to prove the validation of modeling technique. A one-strut model proposed by the researchers was adopted and developed, after obtaining the result of the finite element model, to take into account the effect of the case of window openings at the center on the shear strength behavior of infilled structures. This model could be easily used in seismic sensitivity analysis of existing buildings infilled with masonry walls having openings. The results have proved that the shear strength of infilled structures is decreased with the increase of the opening percentage in the infill wall. For a structure without infill walls, the reduction in the shear strength could reach 75% of the shear strength of full-infilled structure. Also, if the opening ratio exceeds 0.5 the reduction factor of shear strength may remain relatively constant.

Rathi and Pajgade (2012) studied masonry infilled RC structures with center and corner openings and without openings. They indicated that infill walls have a significant influence on the behavior of structures under earthquake excitement. It was found that the lateral stiffness of infilled structure decrease as the opening percentage increase. They concluded the following:

- In case of bare structure, deflection is larger than that of the case of infilled structure with and without opening.
- In case of central opening, deflection is greater than that of corner opening.
- Without considering the effect of infill walls, the value of bending moment, shear force, AST is very large in columns compared to full-infilled structure and infilled structure with opening.

Asteris et al. (2012) identified the influence of openings on the lateral stiffness of masonry infill panels. They used a finite element technique suggested by their other works (Asteris, 2003; 2008). The basic feature of this technique of analysis is that the contact stresses and contact lengths of the infill-frame are evaluated as an integrally segment of the solution, and are not considered in an ad hoc way. Analytical results, by using this technique, are presented on the effect of the opening dimensions on the seismic behavior of masonry infilled structures. Figure 2.26 shows the variation of the λ factor as a function of the opening ratio (opening area/infill panel area), for the case of openings upon the diagonal of infill panel (with same aspect ratio for the opening and the infill). As predicted, when the opening percentage increases the structure's stiffness decreases. Particularly, for the case of opening percentages higher than 50% the stiffness reduction factor tends to zero. This factor is not yet validated with respect to the experimental results. The proposed reduction factor λ can be employed with well-known formulations as a multiplication factor to estimate the reduced width of equivalent diagonal struts in order to be able to model the infill panels having openings. The same coefficient can be utilized in case of multi-struts in order to be able to idealize the nonlinear response of infill panels having openings.

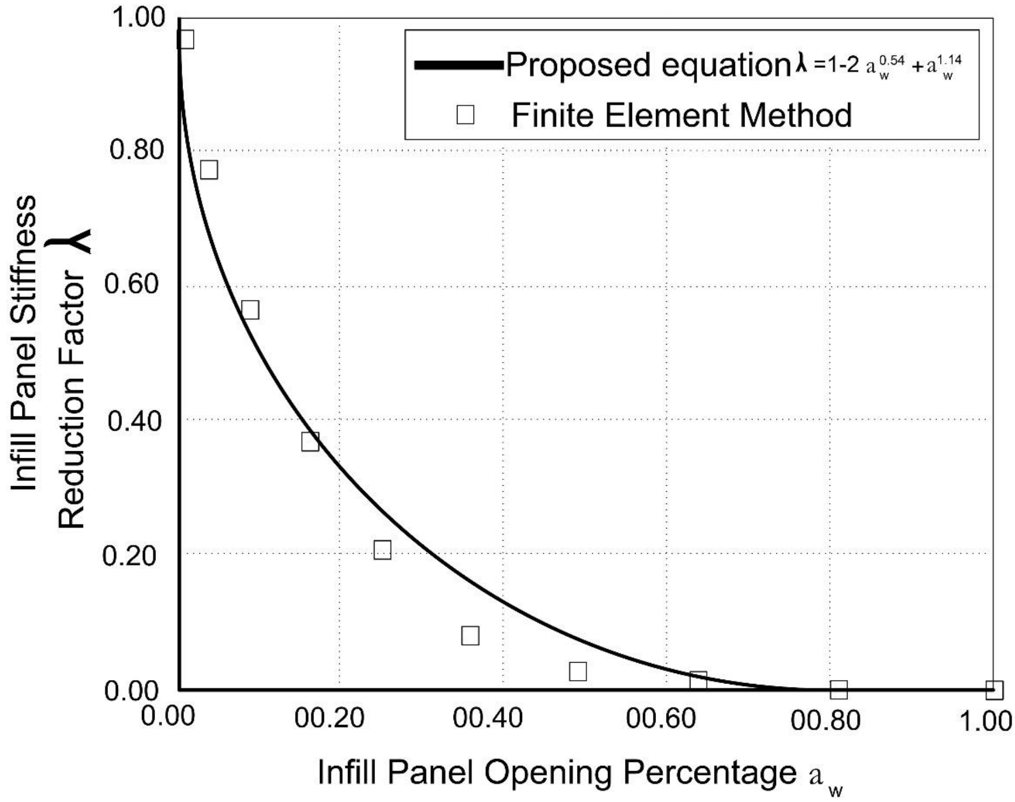


Figure 2.26 Infill panel stiffness reduction factor in relation to the opening percentage (Asteris et al., 2012)

Agrawal et al. (2013) also attempted to highlight the performance of masonry walls infilled RC structures. It is revealed that; by increase the opening percentages the lateral stiffness of infilled structure is decreased.

Sigmund and Penava (2013) studied the effect of the type and position of openings in masonry walls infilled RC frame on the lateral strength experimentally. They improved the equivalent diagonal compression strut modeling and proposed a modified expression for the strut width to take into account the opening's type and position. The corrected diagonal strut model could be used as the realistic estimation tool. It is based on the experimental results and correlates diagonal widths with infill's damage states, opening size, type and position.

Jinya and Patel (2014) investigate the effect of infills 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.

Caliò and Pantò (2014) conducted a cyclic static and nonlinear monotonic analysis on masonry walls infilled RC structure samples. They presented a macro-modeling approach which can be sufficient for the design of infilled structures and seismic evaluations since it provides satisfying accuracy, requires minimal computational sources, and allows simple explanation of results. This approach offers an easy modeling involving the existence of openings and can be employed on various scales (from macro-modeling to micro-modeling). They reported that the existence of window or door openings at the center, of different dimensions, reduce the base shear.

Motwani and Santhi (2015) investigated the behavior of brick infilled RC frames using finite element modeling. The objective of this study was to discover the influence of openings in brick walls. The existence of opening results in massive rising in bending moment and axial force in the infill-frame composite. By using various opening percentages it was concluded that as the size of opening increases the lateral force decreases and the bending moment increases. By changing 9% opening location in infill walls it was noticed that in the case of opening situated outside the main diagonal, the opening role in reducing strength and stiffness is less important in comparison with that of the opening upon the main diagonal. The openings may interfere with diagonal bracing action if they are large and located at the center. Thus, shear failure can early happen in the sections on either side of the diagonal. Thereby, it is recommended to use small sizes of opening, and should be located out off the main diagonal.

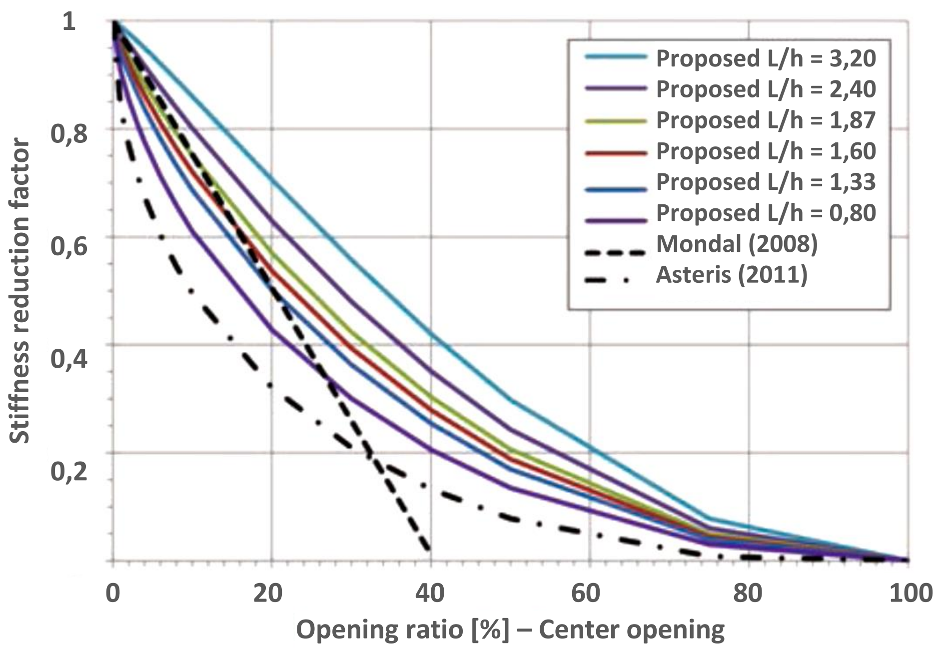
Cetisli (2015) analyzed the behavior of partly infilled RC frames, taking in consideration positions and sizes of openings. The influence of the position of the opening and the infill wall dimensions on the stiffness reduction factor was examined by a finite element analysis. A pushover analysis was implemented on a planar single-bay single-storey RC structure infilled with masonry walls with and without openings using the SAP2000 program, in accordance with rules included in FEMA-356. The results of study were analyzed so as to suggest a plain equation (Eqn. (2.21)) for calculating the stiffness reduction factor for the case of infill walls having openings. He proposed an equation to calculate stiffness reduction factor as follows:

$$\lambda = 1 - 2\alpha^{0.5\mathrm{k}_1\mathrm{k}_2} + \alpha^{\mathrm{k}_1\mathrm{k}_2} \tag{2.21}$$

Where, k_1 and k_2 are constants.

The above equation was predicted by summarizing the results so as to be able to idealize the strut influence of the infill panel having openings. The parameters influence the stiffness reduction factor were infill wall dimensions and position of the opening, in addition to the opening percentage. Although the stiffness reduction factor changes at each position, the position of the opening can be simplified by adopting two out of nine regions: opening at beam-column joint or opening at any other position (Figure 2.27).

Yuen and Kuang (2015) also studied the response of the earthquake and mechanisms of failure of infilled RC frame with five various infill arrangements which include infill with door openings and infill with window openings. The presence of openings in the masonry infill walls obviously decreases 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 ratcheting phenomena or shakedown, respectively. In hysteresis behavior, the spread of sliding cracks started from the regions of the stress concentration around the corners of the openings. However, unrestricted deformations of infilled frames with door openings bear serious asymmetric damage patterns and mechanisms for the transfer of lateral force. Their study also shows the necessity of the further researches on the effects of infill wall openings on the behavior of the structures under different conditions.



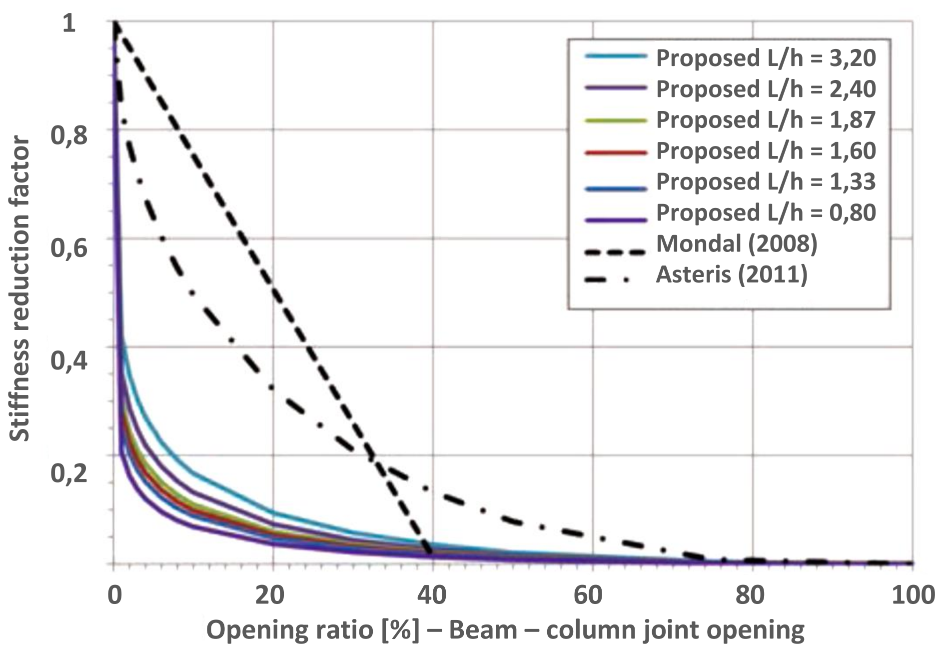


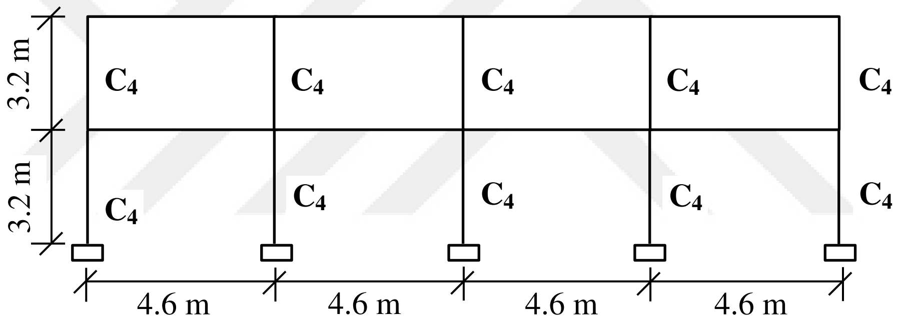
Figure 2.27 Comparison of the proposed model ($\boldsymbol{\lambda}$) and existing models in the literature (Cetisli, 2015)

CHAPTER 3

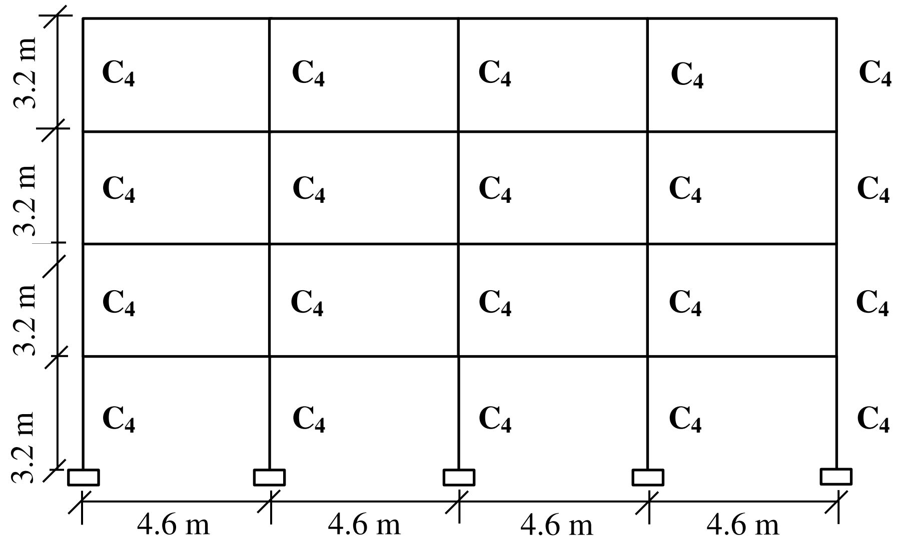
CASE STUDY

3.1 Description of Bare Frame

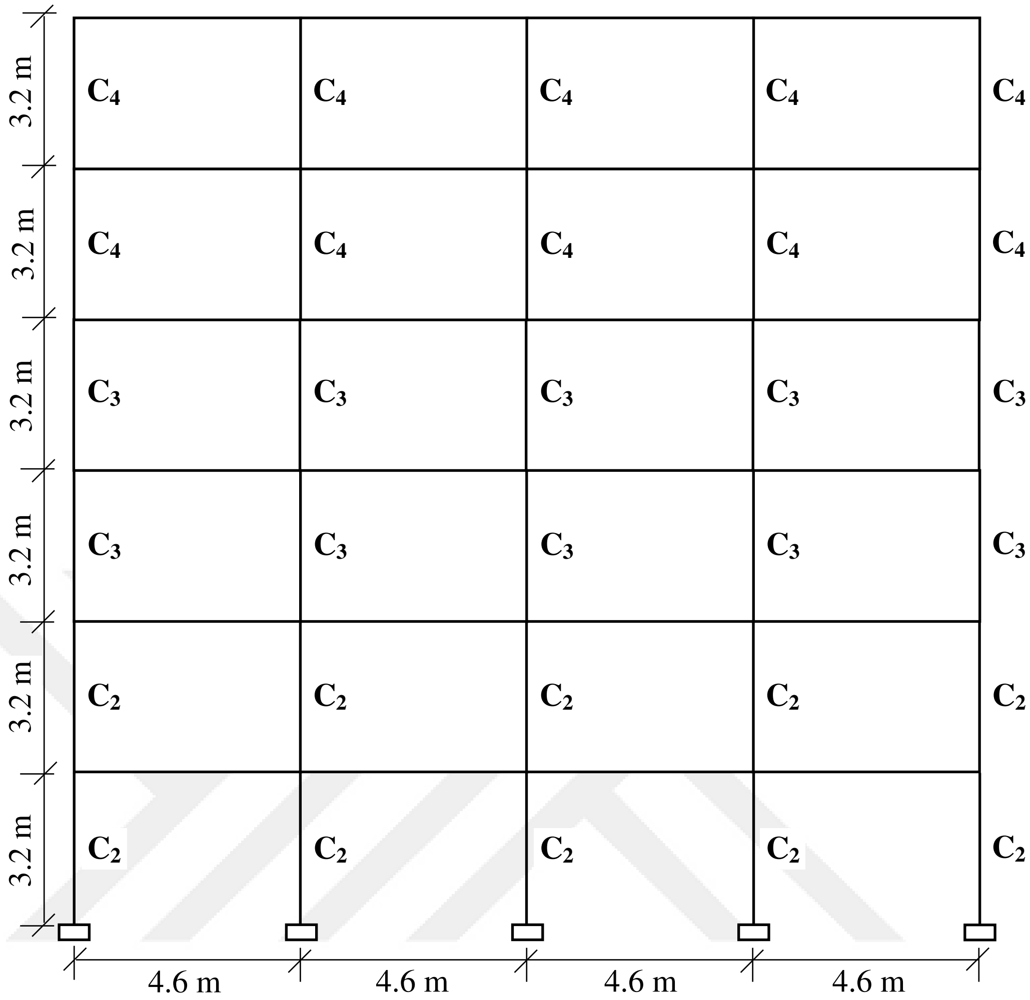
Two, four, six and eight storey reinforced concrete (RC) bare frames with four bays as shown in Figure 3.1 were considered as case study structures. As seen from the elevation views of each building, the columns are spaced at 4.6 m and storey heights are the same at all levels as 3.2 m. Figure 3.2 also demonstrates the plan of the buildings under investigation. First of all, the non-linear analysis of the bare frame model was performed to check the suitability of column and beam joint assignments.



(a) 2-storey



(b) 4-storey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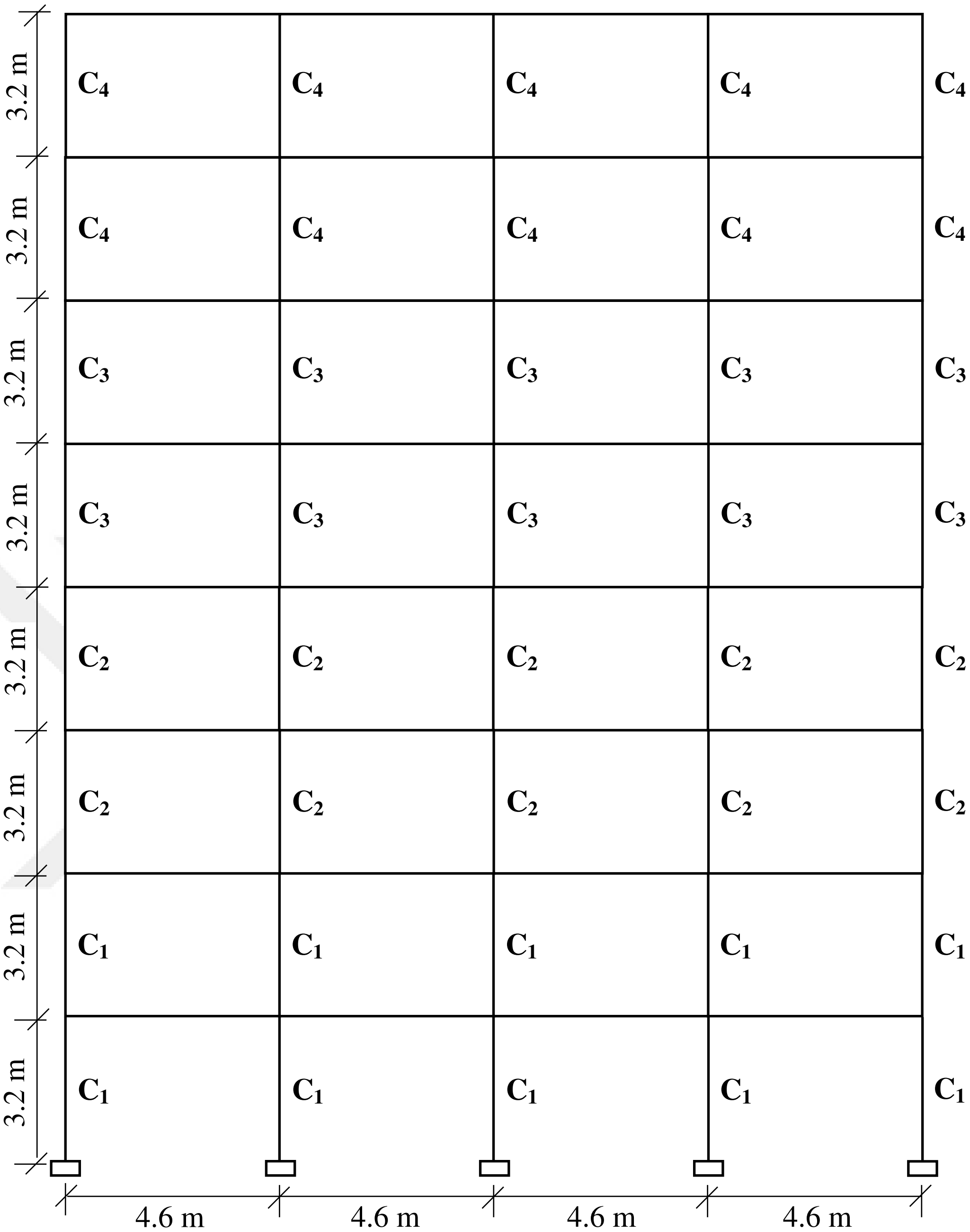
4.6 m

(c) 6-storey



(d) 8-storey

Figure 3.1 Elevation view of RC bare frames

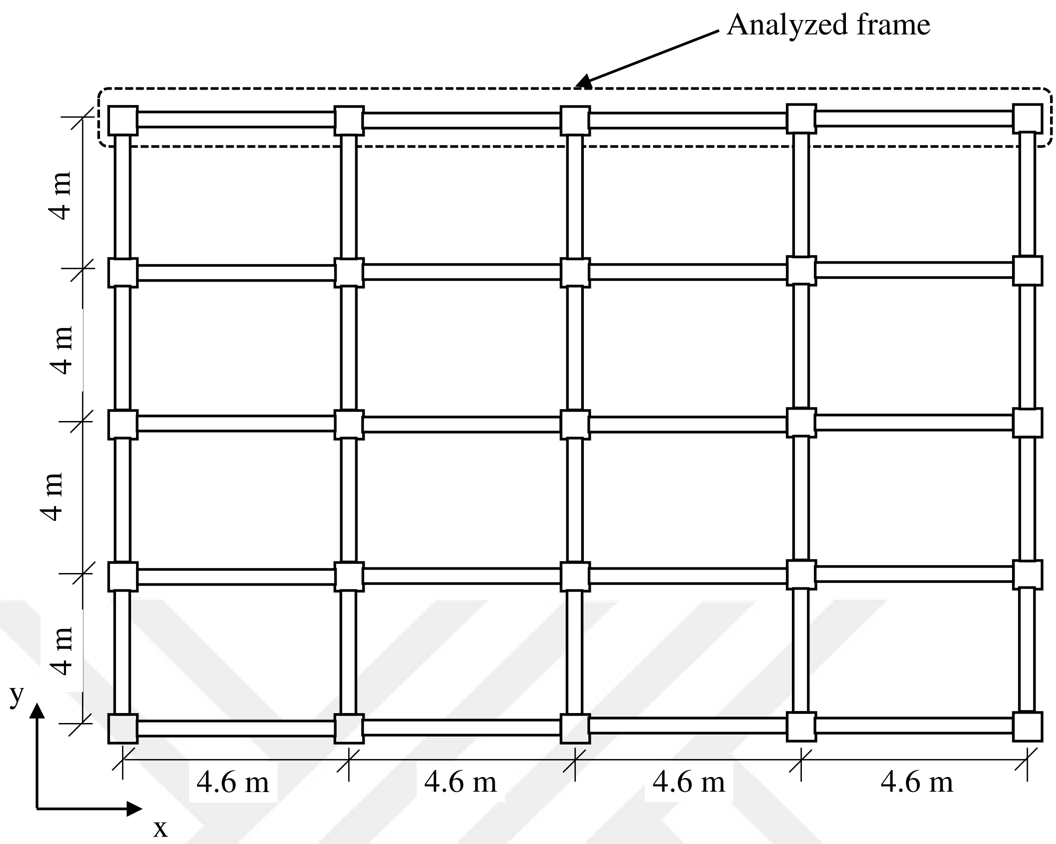


Figure 3.2 Plan view of RC frame buildings

The cross-sectional properties of the structural members are shown in Table 3.1 and the typical reinforcement in columns and beams are shown in Figure 3.3. The compressive strength of concrete used in the buildings is considered as 30 MPa and the yield strength of steel is considered as 365 MPa. The modulus of elasticity (E_c) of the concrete was taken as 25742 MPa and calculated by Eqn. (3.1) (Hassoun and Al-Manaseer, 2015).

All frames considered in this study were subjected to vertical loads of 11 kN/m for super dead load and 8 kN/m for live load.

$$E_c = (4700\sqrt{f_c}) \tag{3.1}$$

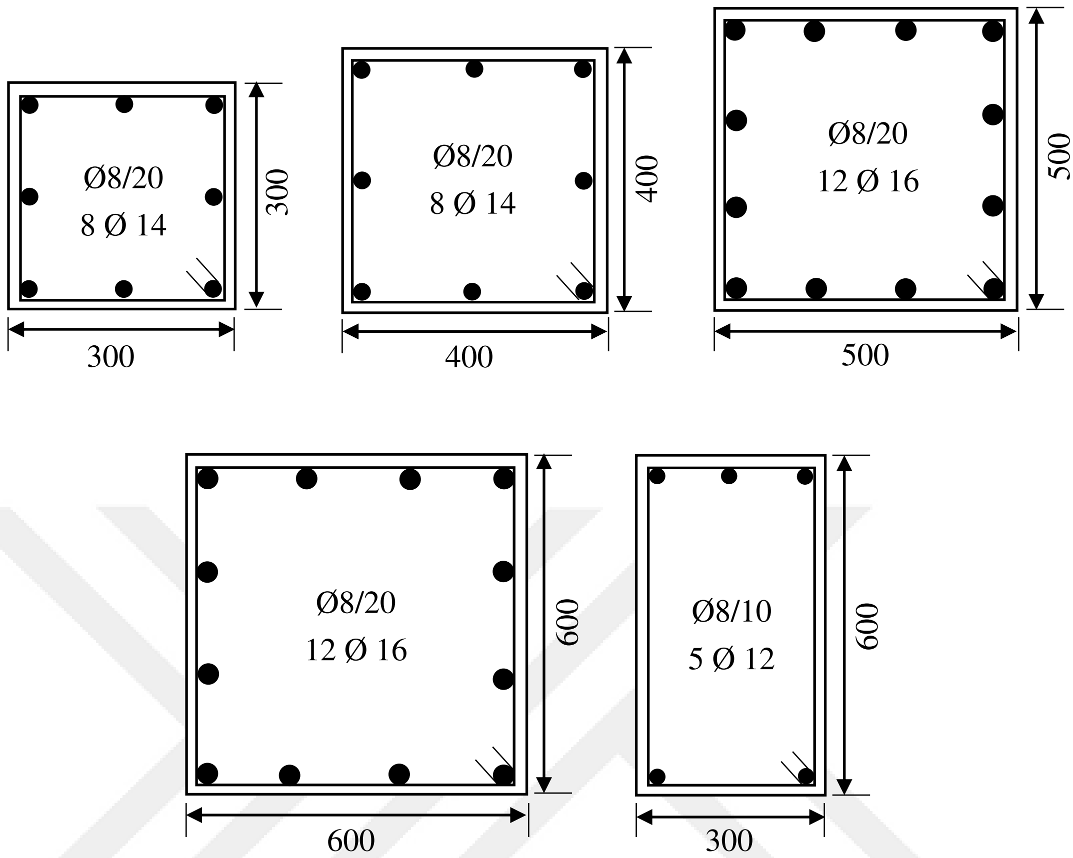


Figure 3.3 Typical reinforcement in columns and beams

Table 3.1 Cross-sectional properties of the structural members of 2, 4, 6 and 8 storey frames

Structural member	Dimension	Units
Beam width	300	mm
Beam depth	600	mm
Column width C_1	600	mm
Column depth C_1	600	mm
Column width C_2	500	mm
Column depth C_2	500	mm
Column width C_3	400	mm
Column depth C_3	400	mm
Column width C_4	300	mm
Column depth C_4	300	mm

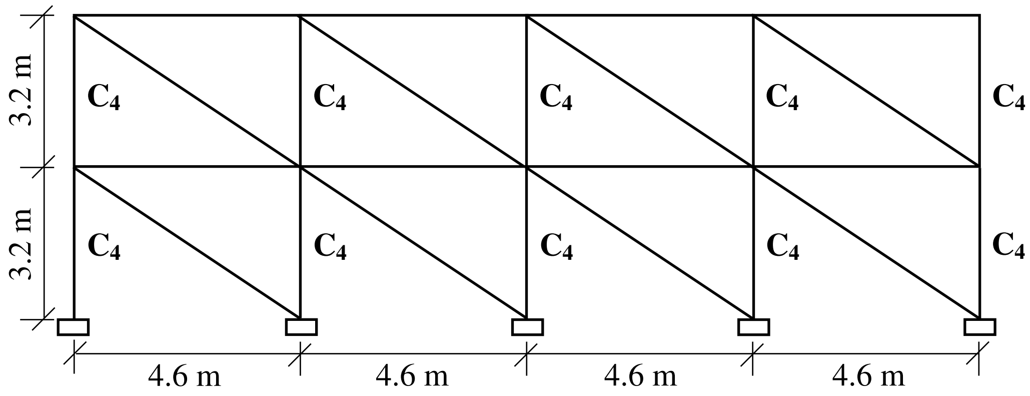
3.2 Description of Full Infill Wall Frames

In macro-modeling, the infill walls are modeled as equivalent diagonal strut. The concept of diagonal strut was investigated and adopted by many researchers. The mechanical properties of the infill walls must be known in order to determine the cross-section of the equivalent diagonal compression strut representing the infill wall and the axial load-bearing properties.

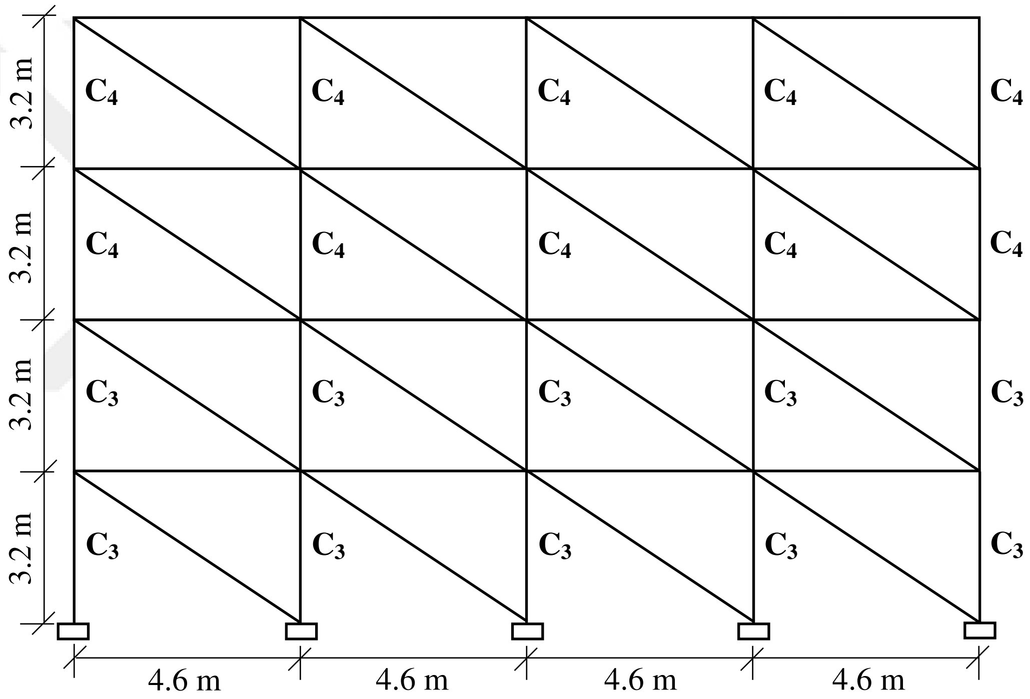
In this study, the experimental results of Porco et al. (2004) and Uva et al. (2012) studies were adopted as properties for the infill wall materials. Their study included diagonal compression tests on the main masonry wall types used in Calabrias in 1970. Figure 3.4 shows the test specimen of the diagonal compression test. According to that study, the tensile strength of masonry wall was taken as f_t = 0.36 MPa and the diagonal elastic modulus of the infill walls was taken as E_w = 1495 MPa. The transversal elastic modulus (shear modulus) of the infill walls was taken as G_w = 598 MPa.

In this study two, four, six and eight storey RC frames with full infill walls were analyzed. Each infill wall was modeled as one diagonal compression strut to show the contribution of infill walls in resisting lateral loads. Figure 3.5 shows the RC full infill walled frames with four bays which considered in the present study.

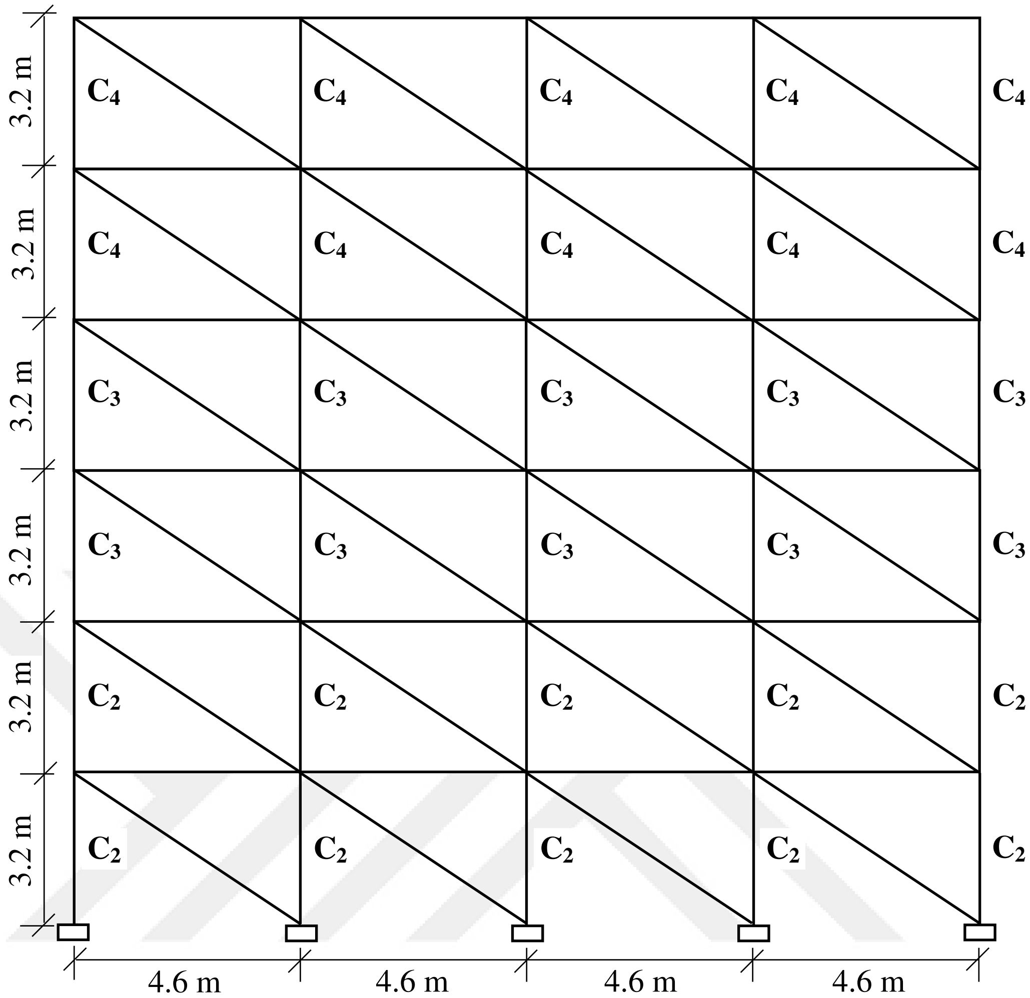
Figure 3.4 The diagonal compression test on a masonry infill sample (Uva et al., 2012)



(a) 2-storey

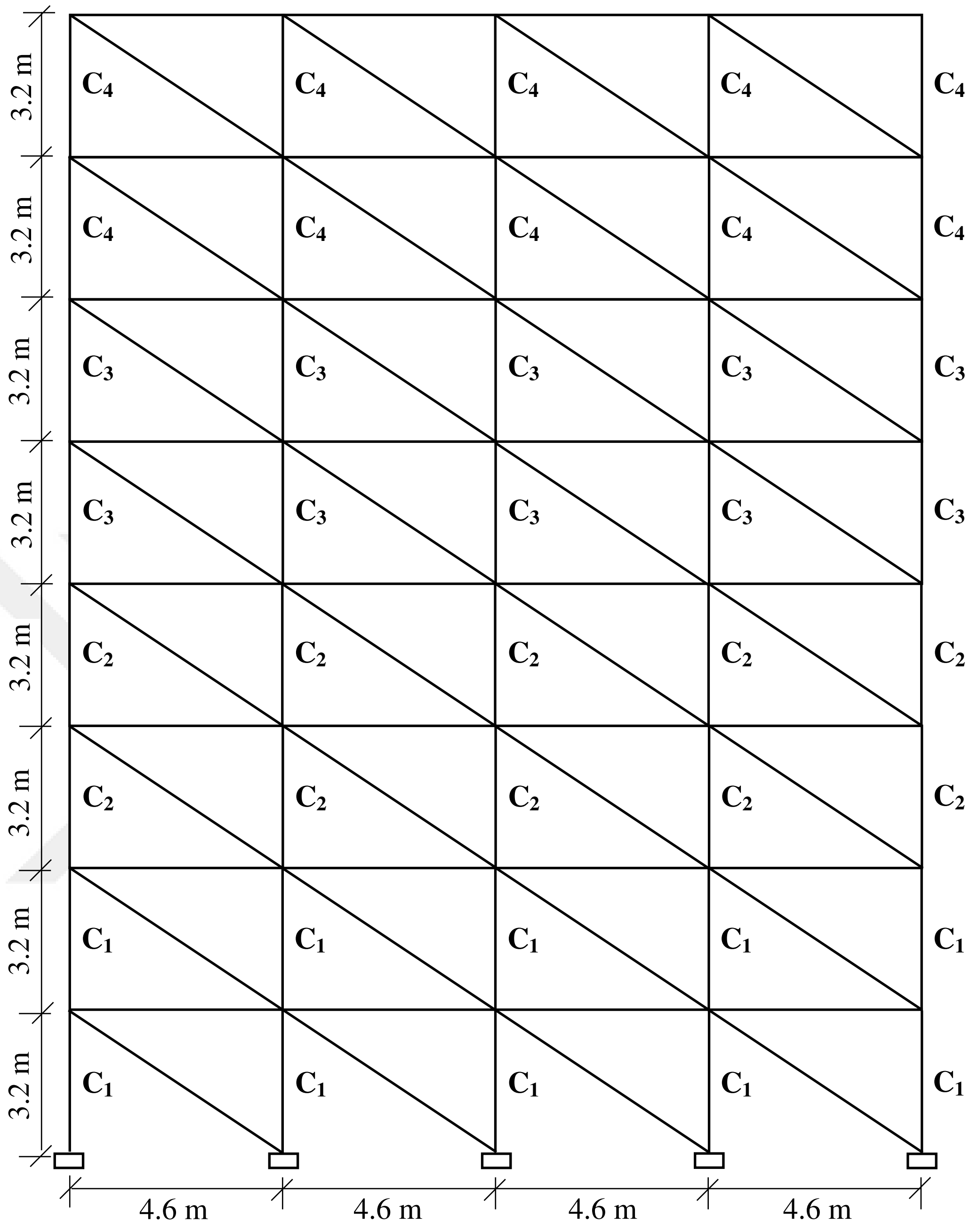


(b) 4-storey



3.2 m
3.2 m
3.2 m
3.2 m
3.2 m
3.2 m
C4 C4 C4 C4 C4
C4 C4 C4 C4 C4
C3 C3 C3 C3 C3
C3 C3 C3 C3 C3
C2 C2 C2 C2 C2
C2 C2 C2 C2 C2
4.6 m
4.6 m
4.6 m
4.6 m

(c) 6-storey

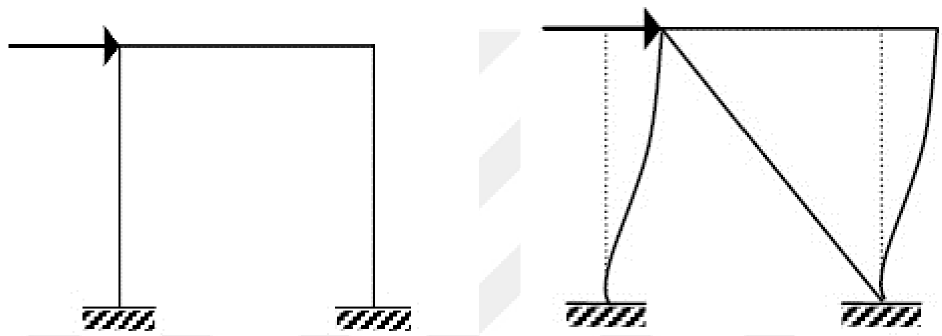


(d) 8-storey

Figure 3.5 Elevation view of RC full infill walled frames

3.3 Infill Walls Modeling

In the analysis of buildings, infill walls are included only as vertical loads in the calculations and are not considered as supporting elements. However, studies have shown that damage to past earthquakes has a significant effect on structural behavior of infill walls. For this reason, it is very important to take into account the positive or negative effects of the infill walls on the structural system in the structural analysis, so that the actual behavior can be reflected on the model. However, the modeling of buildings with infill walls and the study of the behavior under earthquake is not a simple and easy process. This is due to the complexity of the design and the inability to develop a suitable theory. There are two different methods used in the modeling of the infill walls with the frame. The first one is called a micro-modeling method which based on finite element analysis. The second one is called a macro-modeling method which based on the theory of diagonal compression strut (Albayrak et al., 2017). The second method (macro-modeling) has been chosen to perform the present analytical study. Figure 3.6 shows the modeling method of diagonal compression strut (Akyürek, 2014).



(a) (b)

Figure 3.6 Modeling a frame subjected to horizontal load (a) without infill wall and (b) with infill wall (Akyürek, 2014)

3.3.1 Modeling of Infill Walls without Openings

As mentioned above, the macro-modeling has adopted in this study. In this modeling method, one or multi diagonal strut can be used to simulate infill wall. In this study, one equivalent diagonal compression strut has considered to simulate each infill wall. One of the critical parameters is width of the equivalent diagonal strut (b_w). The width of the equivalent strut can be calculated by using a number of expressions in the literature which given by different researchers. Some of them are summarized in Table 3.2.

All expressions in Table 3.2 which proposed by different researchers have used to verify the variations between them. These expressions have applied to a panel sample of building as shown in Figure 3.7. The length of the strut is given by the diagonal distance (d_w) of the infill wall. The height of the infill wall is considered as 2.6 m and the length of the infill wall is considered as 4 m. Therefore, the diagonal distance (d_w) is taken as 4.77 m and the angle made by the strut with horizontal (θ) is taken as 33°. The thickness of the equivalent strut is usually assumed to be the same thickness of the infill wall. In this study, a thickness of 25 cm is proposed to be a fixed thickness for all diagonal strut calculated by different researchers.

Table 3.2 Different formulations proposed by the researchers and design code to calculate the equivalent strut width ($\mathbf{b_w}$)

Eq. No.	**Researchers/code**	**Equations**
1	Abdul-Kadir (1974)	$b_w = \frac{\pi}{2}\sqrt{\left(\frac{1}{\lambda_t}\right)^2 + \left(\frac{1}{2\lambda_p}\right)^2}$ $\lambda_p = \sqrt[4]{\frac{E_w\, t_w \sin(2\theta)}{4\, E_c\, I_c\, H_w}}$ $\lambda_T = \sqrt[4]{\frac{E_w\, t_w \sin(2\theta)}{4\, E_c\, I_T\, H_w}}$
2	Smith and Carter (1969)	$w = 0.58 \left(\frac{1}{H}\right)^{-0.445} (\lambda_h H_w)^{0.335 d_z \left(\frac{1}{H}\right)^{0.064}}$ $\lambda_p = \sqrt[4]{\frac{E_w\, t_w \sin(2\theta)}{4\, E_c\, I_c\, H_w}}$
3	Durrani and Luo (1994)	$b_w = \gamma \sqrt{L^2 + H^2}\, sin2\theta$ $\gamma = 0.32 \sqrt{\sin 2\theta} \left(\frac{E_w\, t_w\, H^4}{m\, I_c\, E_c\, H_w}\right)^{-0.1}$ $m = 6\left(1 + \frac{6\, E_c\, I_b\, H}{\pi\, E_c\, I_c\, L}\right)$
4	Paulay and Priestley (1992)	$b_w = 0.25\, d$
5	Holmes (1961)	$\frac{w}{d} = \frac{1}{3}$
6	FEMA (1998)	$b_w = 0.175 d\, (\lambda_h H_w)^{-0.4}$

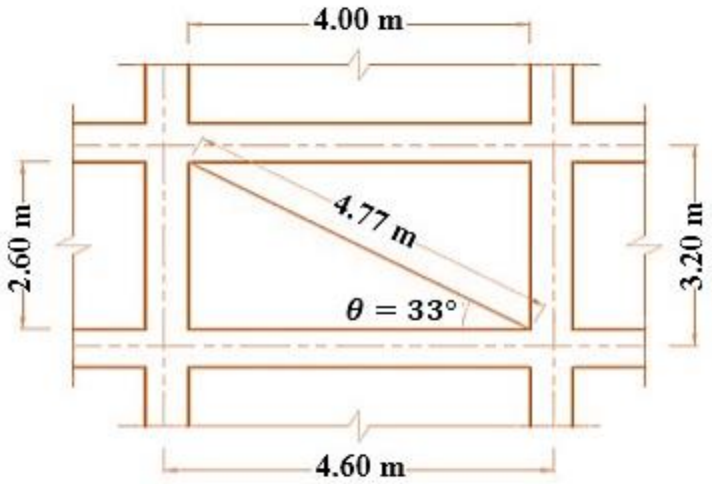


Figure 3.7 Equivalent compression strut used in this study

Figure 3.8 shows the equivalent strut width calculated by the proposed formulations listed in Table 3.2. The highest values of the equivalent strut width have provided by using formulations proposed by Abdul-kadir (1974) and Smith and Carter (1969). The formulation proposed by Holmes (1961) has provided the average value of equivalent strut width. The proposed formulation of FEMA (1998) has produced the lowest value of strut width and the formulations proposed by Durrani and Luo (1994) and Paulay and Priestley (1992) has just produced a value between the average one and the lowest one.

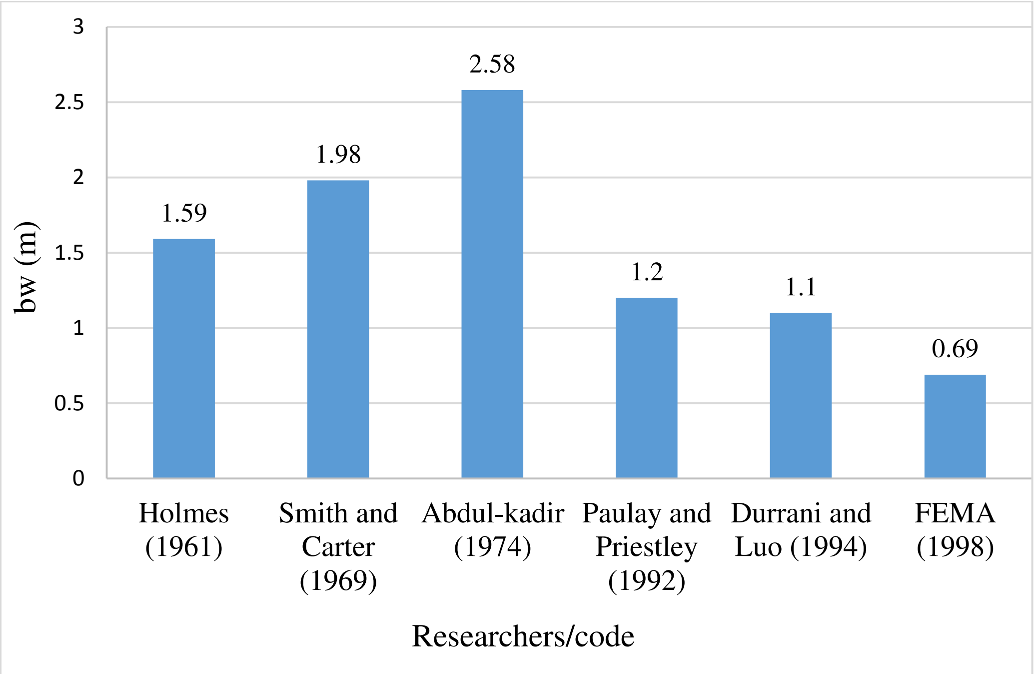


Figure 3.8 Equivalent strut width based on formulations proposed by different researchers

3.3.2 Modeling of Infill Walls with Openings

In practice, openings are often left in the infill walls for architectural reasons such as doors and windows. These openings are likely to reduce the contribution of the infill wall to the frame. Different studies have been carried out in order to investigate the effect of openings on infill walls behavior. In these studies, in general, the effect of openings was determined by reducing the rigidity of the infill wall to a certain extent (Asteris, 2003).

Although infill walls usually have oversized openings, recent research has mainly focused on the simple case of infill wall without openings. Research on infill walls with openings is mostly analytical, restricted to special cases, and as such cannot provide rigorous comparison to actual cases because of its focus on specific materials used and specific types of openings (Asteris et al., 2012).

For the modeling of infill walls with openings, the majority of methods proposed in the literature are evaluated theoretically. Among these proposed methods, the method proposed by Asteris et al. (2012) has been adopted cause it provides a stiffness reduction factor formula (Eqn. (3.2)) which is compatible to opening ratio (area of opening to the area of infill wall) up to 1 (100% opening which represent a bare frame) for openings upon the diagonal (central openings). The effect of openings in the middle region are more critical than the effect of openings up or down the diagonal due to producing the higher reduction in stiffness value of the frame (Asteris, 2003). In this study, 0, 20, 40, 60, 80 and 100% of opening percentage have considered to verify the effect of central openings on the behavior of infill walled frames. The infill wall modeling was performed as mentioned in section 3.3.1 and the effect of openings was taken into account. Table 3.3 shows infill walled panel samples with different opening percentages and the calculated stiffness reduction factor corresponded to these opening percentages.

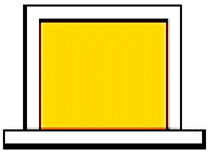
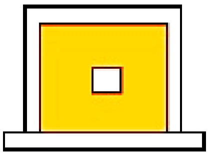
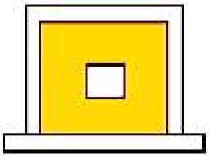
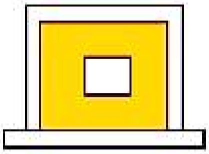
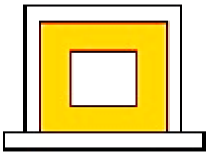
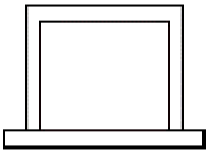
$$\lambda = 1 - 2\alpha_w^{0.54} + \alpha_w^{1.14} \tag{3.2}$$

Where;

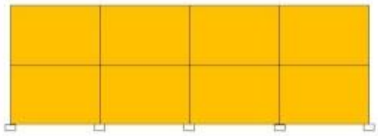
α_w is the opening percentage of infill wall (Area of opening/Area of infill).

The above coefficient is not yet calibrated with other experimental data. However, it can now be used to find the equivalent width of a strut for the case of an infill with opening by multiplying the equivalent strut width calculated by different formulations (Table 3.2) with this coefficient (Asteris et al., 2012).

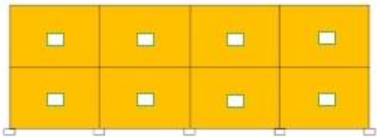
Table 3.3 Infill walled panel samples with different opening percentages and the corresponding stiffness reduction factor

Panel sample	**Opening percentage**	**Stiffness reduction factor**
	0%	1
	20%	0.321
	40%	0.132
	60%	0.04
	80%	0.002
	100%	0

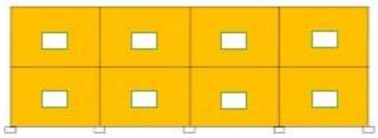
The two-dimensional frames used in the analysis made within the scope of the study are designed as two, four, six and eight storey. The 2-storey infill walled frame models are shown in Figure 3.9, the 4-storey infill walled frame models are shown in Figure 3.10, the 6-storey infill walled frame models are shown in Figure 3.11, and the 8-storey infill walled frame models are shown in Figure 3.12.



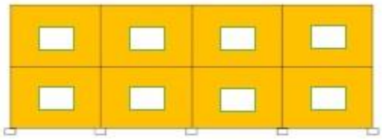
(a) 0% opening



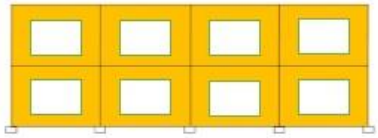
(b) 20% opening



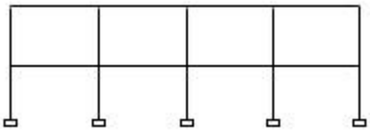
(c) 40% opening



(d) 60% opening

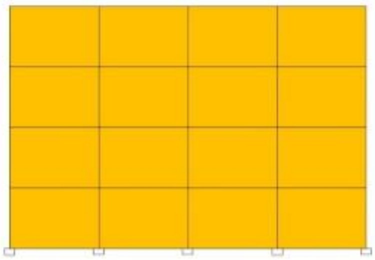


(e) 80% opening

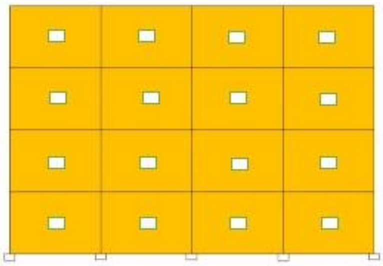


(f) 100% opening

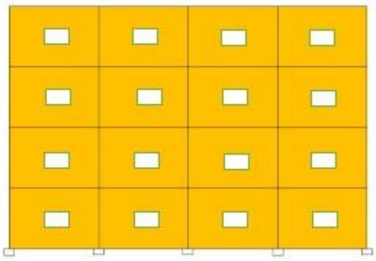
Figure 3.9 2-storey infill walled frame models with different opening percentages



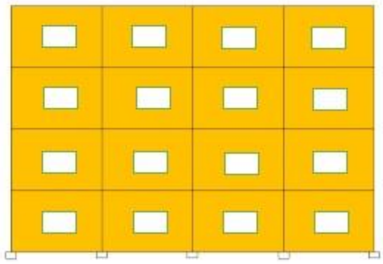
(a) 0% opening



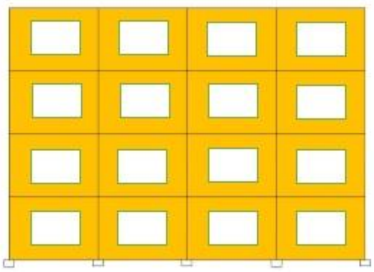
(b) 20% opening



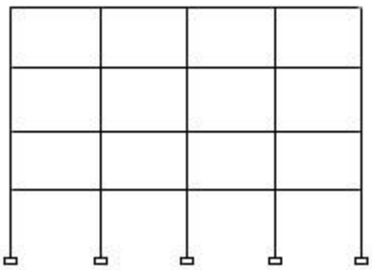
(c) 40% opening



(d) 60% opening

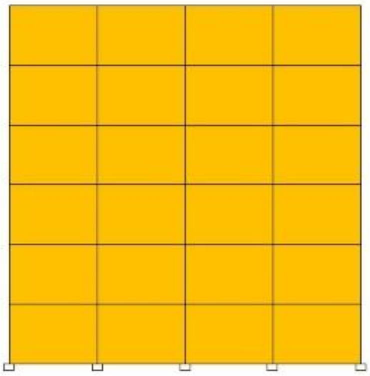


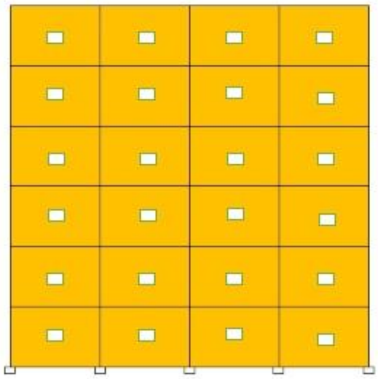
(e) 80% opening



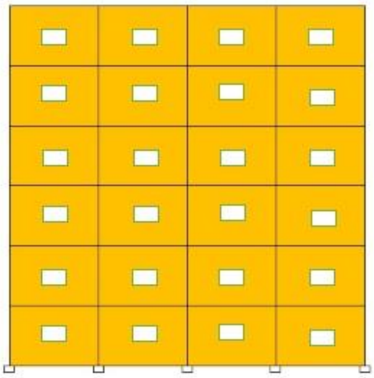
(f) 100% opening

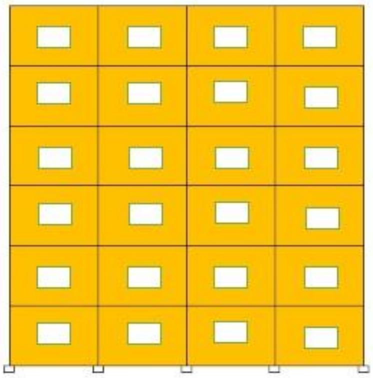
Figure 3.10 4-storey infill walled frame models with different opening percentages



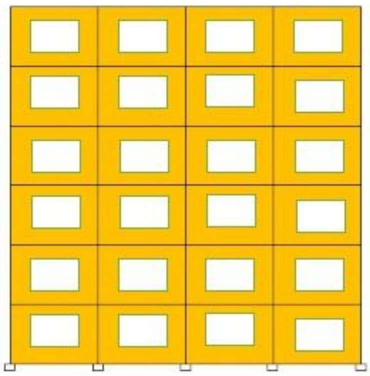


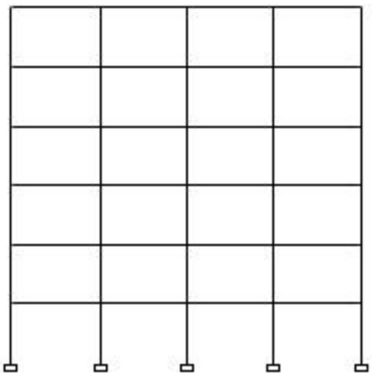
(a) 0% opening (b) 20% opening





(c) 40% opening (d) 60% opening





(e) 80% opening (f) 100% opening

Figure 3.11 6-storey infill walled frame models with different opening percentages

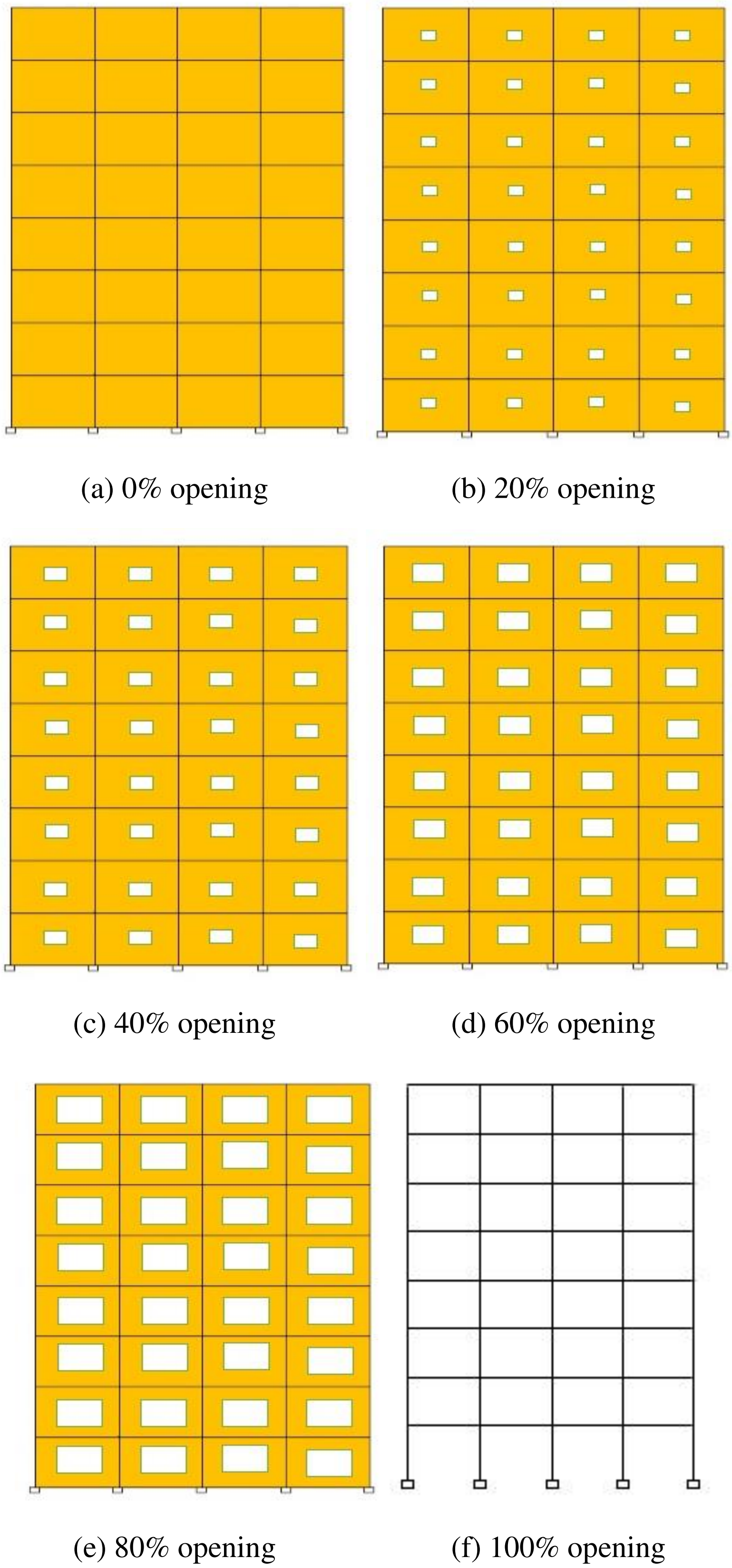


Figure 3.12 8-storey infill walled frame models with different opening percentages

3.4 Stiffness and Strength of Infill Walls

In the macro-modeling of infill walls, the stiffness and strength contribution of infill walls are considered by modeling the infill walls as an equivalent strut. Many researchers have proposed the equivalent strut method because of its simplicity (Abdelkareem et al., 2013; Shobha et al., 2016; Oinam et al., 2017). In this study, a frame model with a diagonal strut located in a lateral load direction is considered. This model should not neglect the bending moment in columns and beams. Thus, rigid joints were used to connect the columns and beams and pin joints were used to connect the equivalent strut to the column-beam junctions. A plastic hinge of type (PM2M3) is assigned at each end of the columns while a plastic hinge of type (M3) is assigned at each end of the beams. Axial plastic hinge of type (P) is assigned at the mid-point of the diagonal strut length. Figure 3.13 shows the types and distribution of plastic hinges within a frame model.

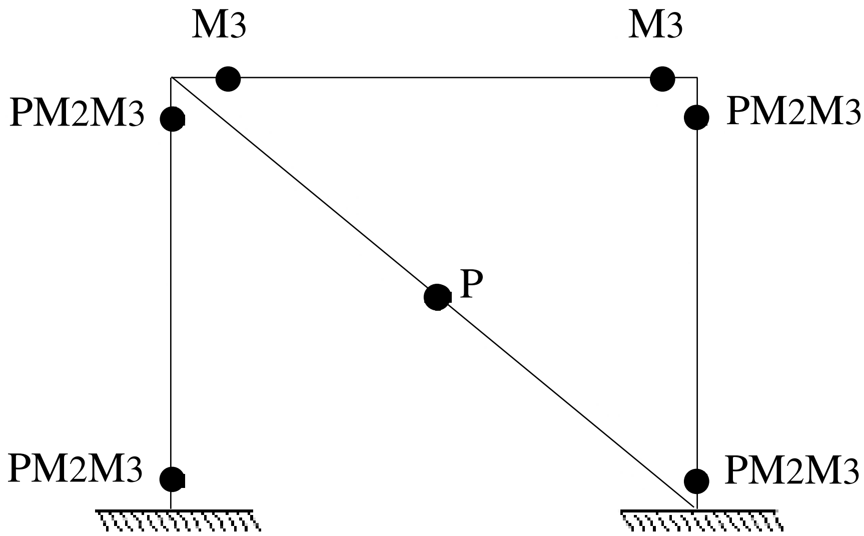


Figure 3.13 Types of plastic hinges used to engage the frame elements

There are several proposals in the literature to determine the strength and stiffness of infill walls. In the present work, the study of Panagiotakos and Fardis (1996) has adopted. They proposed a model based on the equivalent diagonal strut concept to obtain the curve shown in Figure 3.14. This curve describes a constitutive relationship validated experimentally by cyclic tests on scaled samples of a frame with brick infill panels. The curve consists of four parts (if no residual resistance is assumed, the segments are reduced to 3). The first part shows the initial shear behavior of the un-cracked infill panel. The second part represents the formation of the equivalent strut in the infill panel, after the separation of the infill panel from the surrounding frame. The third part describes the softening behavior of the infill panel after the critical displacement S_m. This part is characterized by K3 slope. The last part (horizontal line in the curve) defines the final state of the infill panel. Some authors (Dolšek and Fajfar, 2005) decided to neglect the last part by assuming a line that reaches a zero residual strength S_u(dashed line in Figure 3.14).

Initial shear stiffness K1 of the un-cracked panel:

$$K_1 = \frac{G_w \, t_w \, L_w}{H_w} \tag{3.3}$$

Where G_w is the tangential elastic modulus of the masonry infill; L_w, H_w and t_w respectively are the length, the height, and the thickness of the panel.

Yielding force F_y corresponding to the first cracking of the panel:

$$F_y = f_{tp} \, t_w \, L_w \tag{3.4}$$

Where f_{tp} is the tensile strength of the panel (evaluated by the diagonal compression test).

Displacement at the yielding point, S_y

$$S_y = \frac{F_y}{K_1} \tag{3.5}$$

Axial stiffness K_2 of the equivalent strut:

$$K_2 = \frac{E_m \, b_w \, t_w}{d} \tag{3.6}$$

Displacement S_m corresponding to the maximum force:

$$S_m = S_y + \frac{F_m - F_y}{K_2} \tag{3.7}$$

Due to the lack of data and uncertainties in the modeling, it was necessary to arbitrarily assume several parameters for the mathematical model. Some authors

(Dolšek and Fajfar, 2008; Dolšek and Fajfar, 2001) have simplified some parameters and assumed the following:

The ratio between the yield force and the maximum force:

$$F_y/F_m = 0.6 \tag{3.8}$$

The residual force:

$$F_r = 0$$

The ratio between the ultimate displacement and the displacement corresponding to the maximum force:

$$S_r/S_m = 5 \tag{3.9}$$

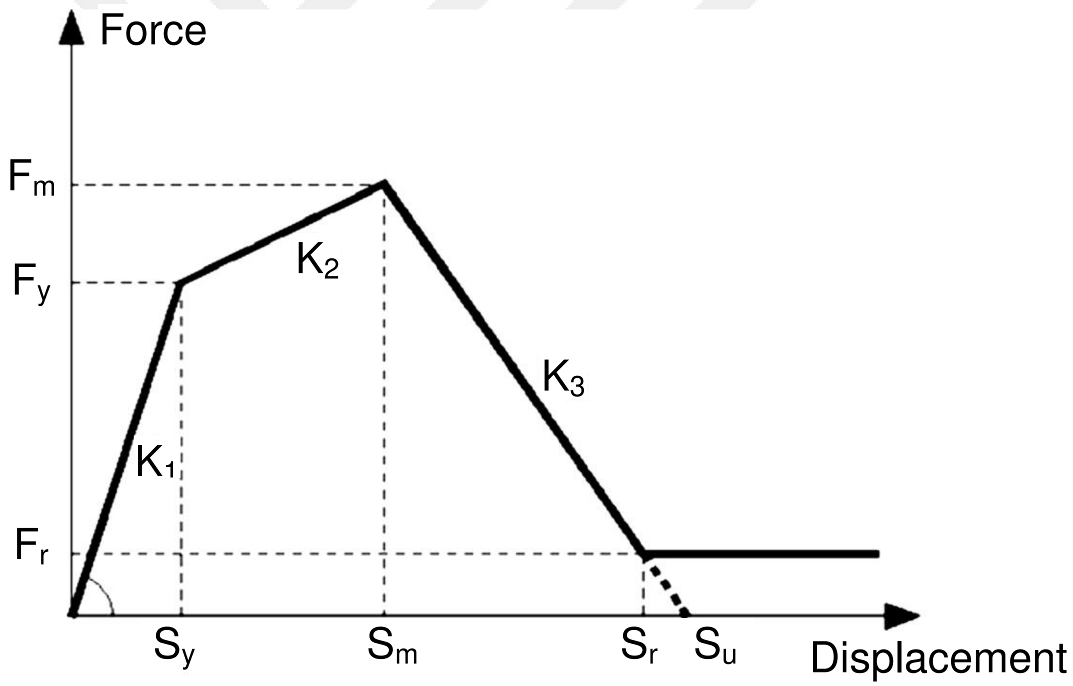


Figure 3.14 Force–displacement relationship for the equivalent strut model proposed by Panagiotakos and Fardis (1996)

Table 3.4 shows the parameters of the constitutive relationship for the force-displacement curve (Figure 3.14) which have calculated based on the equivalent diagonal strut width proposed by different researchers.

Table 3.4 Parameters of the force-displacement constitutive relationship calculated in this study

Researcher/code	b_w (mm)	F_y (kN)	F_m (kN)	S_y (mm)	S_m (mm)	S_r (mm)
Holmes (1961)	1590	414	690	1.92	4.13	20.65
Smith and Carter (1969)	1980	414	690	1.92	3.7	18.5
Abdul-Kadir (1974)	2580	414	690	1.92	3.28	16.4
Paulay and Priestley (1992)	1200	414	690	1.92	4.85	24.25
Durrani and Luo (1994)	1100	414	690	1.92	5.12	25.6
FEMA (1998)	690	414	690	1.92	7.02	35.1

3.5 Nonlinear Analysis

By using SAP 2000 design program, it is possible to design, model, optimize and solve 2D and 3D symmetric and non-symmetric structures quickly. In this program, beams and columns are modeled as rod elements and rigid diaphragm can be accepted in the horizontal plane for floor pavers. Time-domain calculation, combining mode, and non-linear static pushover methods can be used for steel and RC structures. The nonlinear static pushover analysis became a common method in seismic performance assessment of new and existing structures considerably. The pushover analysis method provided sufficient information about seismic demands which imposed by the designing ground motions on the structural members. The pushover method appearance in structural designing for a specific level of seismic performance, like immediate occupancy, has produced in guidelines like FEMA-356, ATC-40 and standards like ASCE- 41. The pushover analysis is one of the best methods for analysis compared to other methods because of its efficiency, easiness of use, and optimal accuracy. A pushover analysis carried out by subjecting a structure to a monotonically increasing pattern of lateral loads, including the inertial forces that could be expert factors within the structure when subjected to ground shaking. Increasing loads of different structural members might yield sequentially under gradual effect. As a result, in each case, the structure subjected to loss and lack in stiffness. By using pushover analysis, a special force-deformation relationship can be obtained. The standards and guidelines for the pushover analysis include acceptance criteria, design, and analysis proceedings. The purpose of these documents is to define force-deformation standard for possible places of lumped inelastic behavior, specified as plastic hinges used in pushover analysis. As demonstrated in Figure 3.15 below, five points can be specified as A, B, C, D, and E, these points are used to define the force-deformation relationship of the plastic hinge also, in this figure, three specified points can be demonstrated including LS (Life Safety), IO (Immediate Occupancy) and CP (Collapse Prevention). These three points are used in defining the acceptance standard for the hinges. In addition, if all the structural members provide the acceptance standard for a specified performance level, like Life Safety, thereafter, full structure expects to realize the life safety level of structural performance. The value assigned to every point is differ according to the kind of structural member also according to other parameters such as the expected kind of failure, code compliance, or the level of stresses with respect to the strength (Govind et al., 2014).

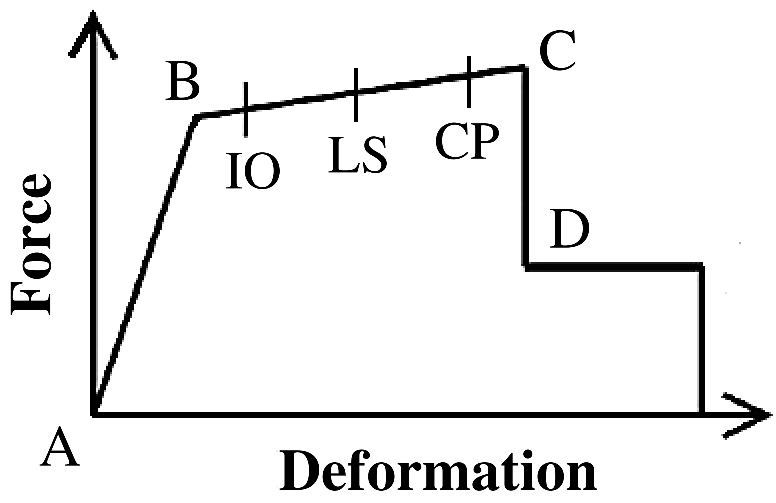


Figure 3.15 Force-deformation relation for plastic hinge in pushover analysis (Govind et al., 2014)

In this study, nonlinear static pushover calculations of model frames were made using SAP 2000 18.2.0 version. In the pushover analysis, a nonlinear static analysis was first carried out under mass loads compatible to vertical loads, and the results of this analysis were taken as the initial conditions for the horizontal load and the incremental pushover analysis. As a result of pushover analysis, the capacity curves have plotted between base shear and roof displacement for all frame models to verify the effect of the variable parameters proposed in the present study.

CHAPTER 4

RESULTS AND DISCUSSION

This chapter includes the results obtained from the nonlinear static pushover analysis work on the reinforced concrete (RC) frame models described in chapter three. The aim of this kind of analysis was to evaluate the response of buildings infilled with masonry walls with and without openings. As a result, the force-displacement plots of these buildings have obtained, and the basic parameters of the equivalent strut width (b_w) and the opening ratio (opening area to the total area of infill) in the infill panels have discussed comparatively in this chapter.

4.1 Comparison of Equivalent Strut Width of Different Formulations

Different formulations as mentioned in Table 3.2 were used to model the masonry infill walls as a single equivalent diagonal compression strut to show the variations between them. The corresponding values of the equivalent strut width to these formulations have calculated and illustrated in Figure 3.8. It is obvious that each formulation has given different values of strut width which reflects on the behavior of the infilled frame.

Figures 4.1 to 4.4 show the capacity curves of 2, 4, 6 and 8 storey infilled frames based on different equivalent strut widths according to the formulations. Variations between different formulations are quite apparent and the highest value of base shear is achieved by the strut of the smallest width. Infill walls have shown a significant improvement in the capacity curve of bare frames. The maximum base shear of the bare frame was about 643, 707, 815, and 902 kN for two, four, six, and eight storey frames, respectively. Whereas, the maximum base shear of the infilled frame with thinner strut was about 3423, 3311, 3177, and 3040 kN for two, four, six, and eight storey frames, respectively. This implied lateral load carrying capacities about 5.3, 4.7, 3.9, and 3.4 times higher than that of the bare frame for 2, 4, 6, and 8 storey buildings, respectively. Also, according to the findings obtained from the nonlinear static analysis, the infilled frames have significantly lower roof displacements than the bare frame. It was found that the roof displacement is proportional to the building height in both cases of frame with infill walls and frame without infill walls.

It was observed that thinner strut (according to FEMA (1998)) produces a higher strength and lower stiffness than the thicker one (according to Abdul-kadir (1974)). This high strength has distinguished with a ductile behavior. This means that the frame with the thinner strut deform with a higher deflection and recorded a higher roof displacement than the frame with the thicker strut. Whereas, the thicker strut has a brittle behavior and produced a lower strength and a higher stiffness.

From the above, it can be said that the base shear and roof displacement obtained from the pushover analysis are functions of building stiffness. As the building stiffness increased (due to presence of infill walls), the base shear and roof displacement decreased and vice versa.

It is obvious that the presence of infill walls decreased the ductility of the bare frame because infill walls have high strength and stiffness. However, it was observed that the stiffness and peak strength of the infilled frames decrease with the increase of frame height. The different strut widths according to the formulations have showed similar behavior in case of increasing number of storeys.

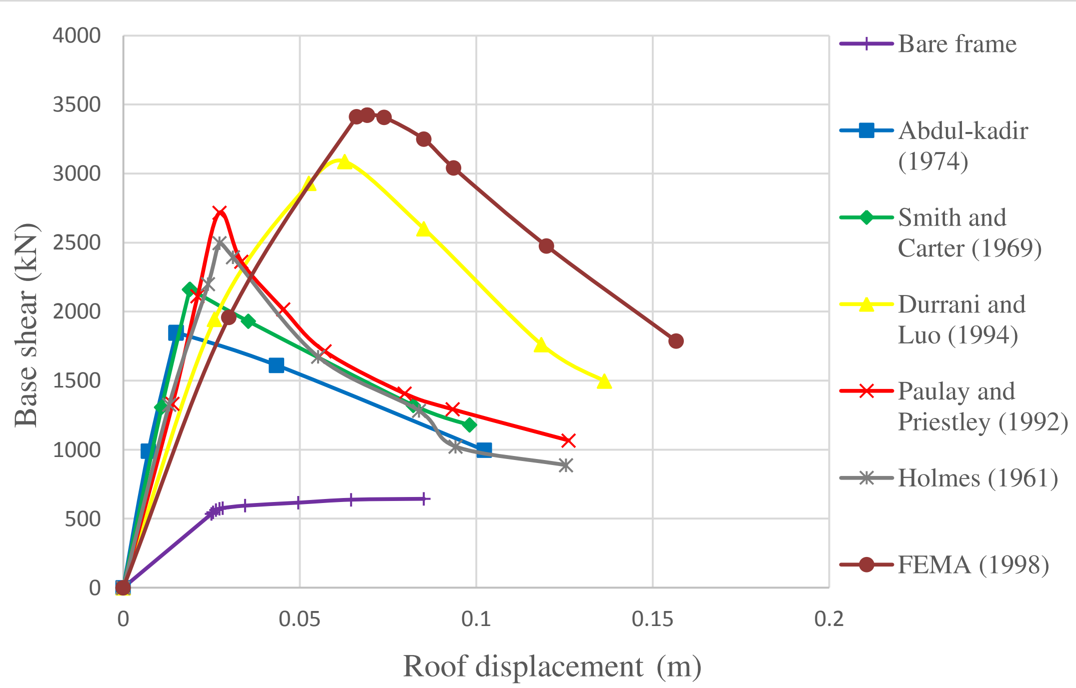


Figure 4.1 Comparison of the capacity curves of 2-storey bare and infill walled frames according to the formulation by different researchers

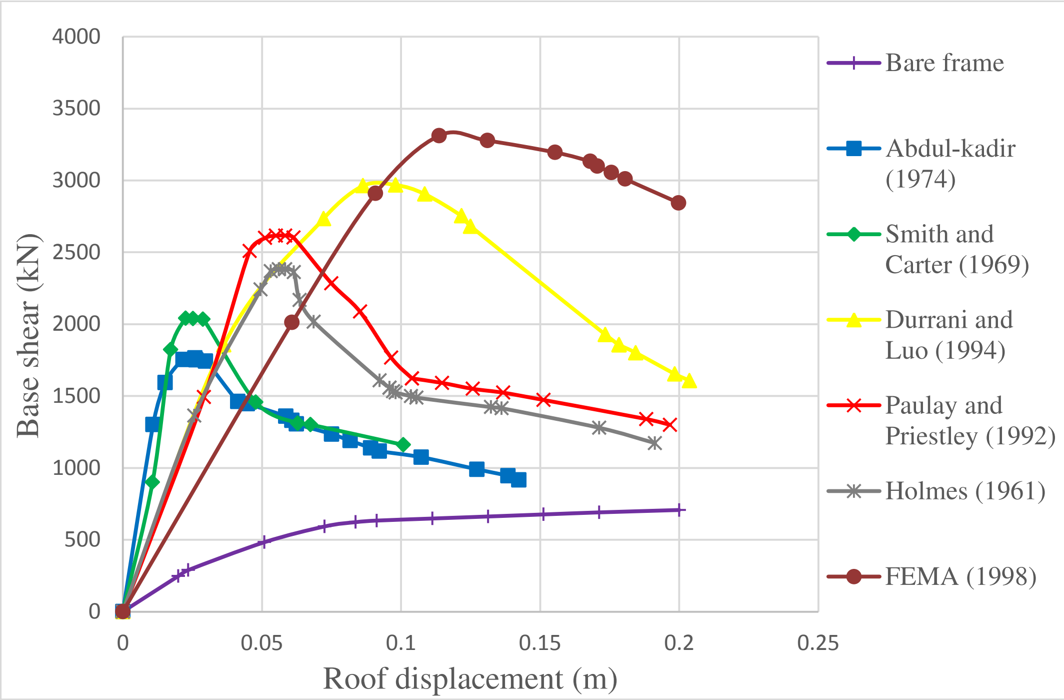


Figure 4.2 Comparison of the capacity curves of 4-storey bare and infill walled frames according to the formulation by different researchers

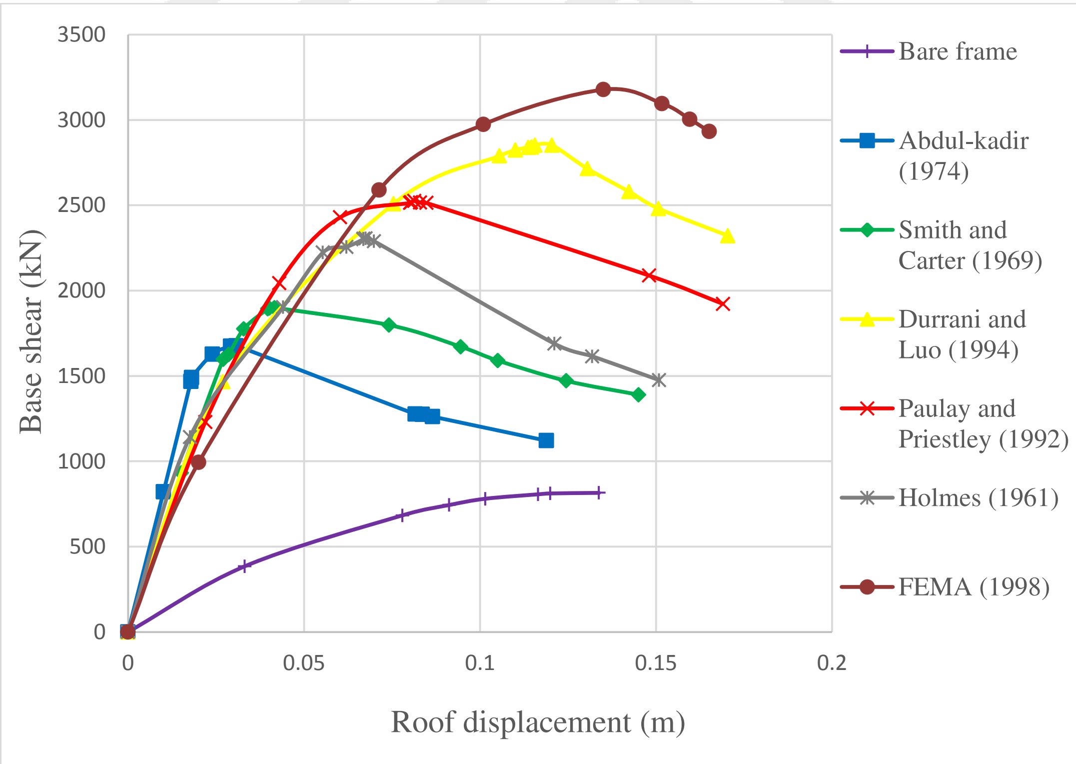


Figure 4.3 Comparison of the capacity curves of 6-storey bare and infill walled frames according to the formulation by different researchers

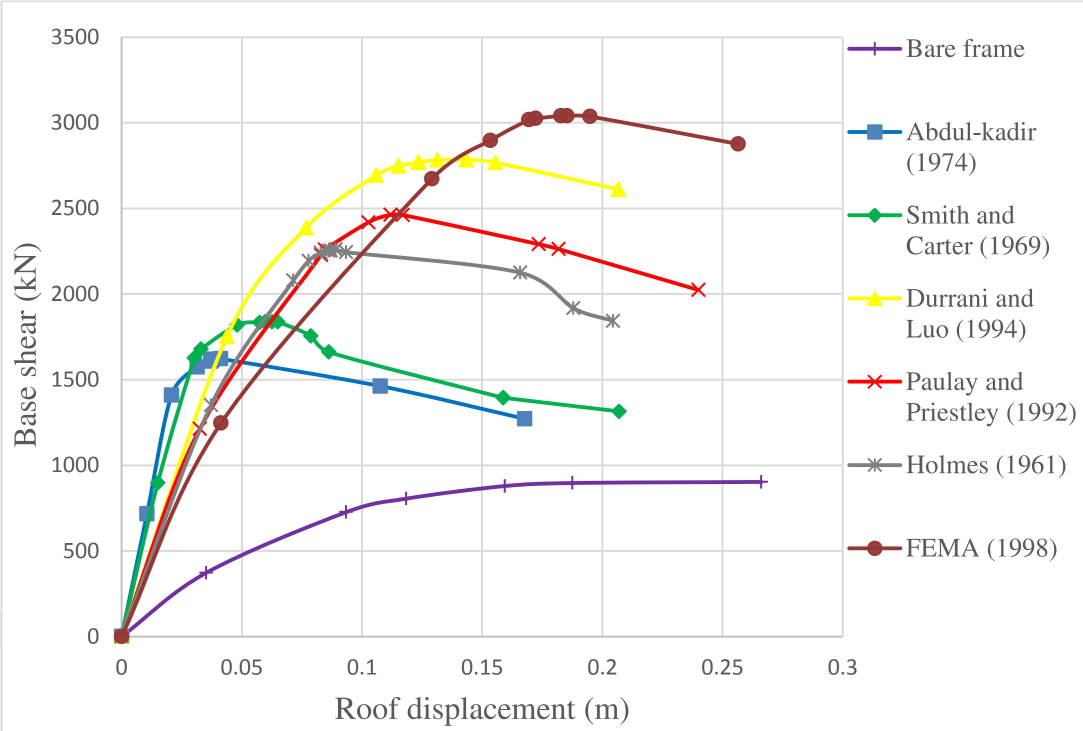


Figure 4.4 Comparison of the capacity curves of 8-storey bare and infill walled frames according to the formulation by different researchers

4.2 Comparison of Plastic Hinge Formations at Mechanism of Building Frames

Hinges are points within a frame where one expects yielding and cracking to take place in greater intensity so as they show high shear (or flexural) deformation, when it becomes close to its extreme strength under cyclic loading. These are positions where one expects to observe cross diagonal cracks in an real building frame after an earthquake, and they are found to be at the either ends of beams and columns, the ‘cross’ of the cracks being at a short distance from the joints – that is where one is expected to insert the hinges in the columns and beams of the corresponding computer analysis model. Hinges come in different sorts namely; shear hinges, flexural hinges, and axial hinges. The first two types are inserted into the ends of beams and columns. Since the existence of masonry infill walls have significant influence on the seismic behavior of the frame, modeling them using equivalent diagonal struts is common in pushover analysis, unlike in the conventional analysis, where its inclusion is a rarity (Kumar et al., 2016). The axial hinges are assigned at the center of the diagonal struts to simulate cracking of infills during analysis (Adukadukam and Sengupta, 2013).

Pushover nonlinear analysis requires the evolution of the force-displacement curve (see Figure 3.15) for the sensitive section of columns and beams by utilizing the guideline FEMA-356. Point A refers to the unloaded state. Force-displacement relation is described by the linear response from point A to an efficient yield point B. After that the stiffness decrease from B to C. Point C possess a resistance equal to the ultimate strength then sudden reduction in resisting the lateral load to point D, thereafter the latest loss of resisting is described by the response from D to E. The inclination of line BC is usually taken as 0-10% of the initial inclination. Line CD refers to primary failure of the member. Line DE refers to the residual strength of the member. These points are assigned according to FEMA-356 to estimate hinge rotation conduct of RC members. The line BC refers to the accepted criteria for the hinge, which is Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) (Hakim et al., 2014).

Plastic deformation will be exhibited only by the hinge after point B. Point C refers to the maximum capability for pushover nonlinear analysis. Although, point D refers to a residual strength of the member. However, you may positively determine a slope from C to D or D to E for another purpose. Point E refers to final failure (Pambhar, 2012).

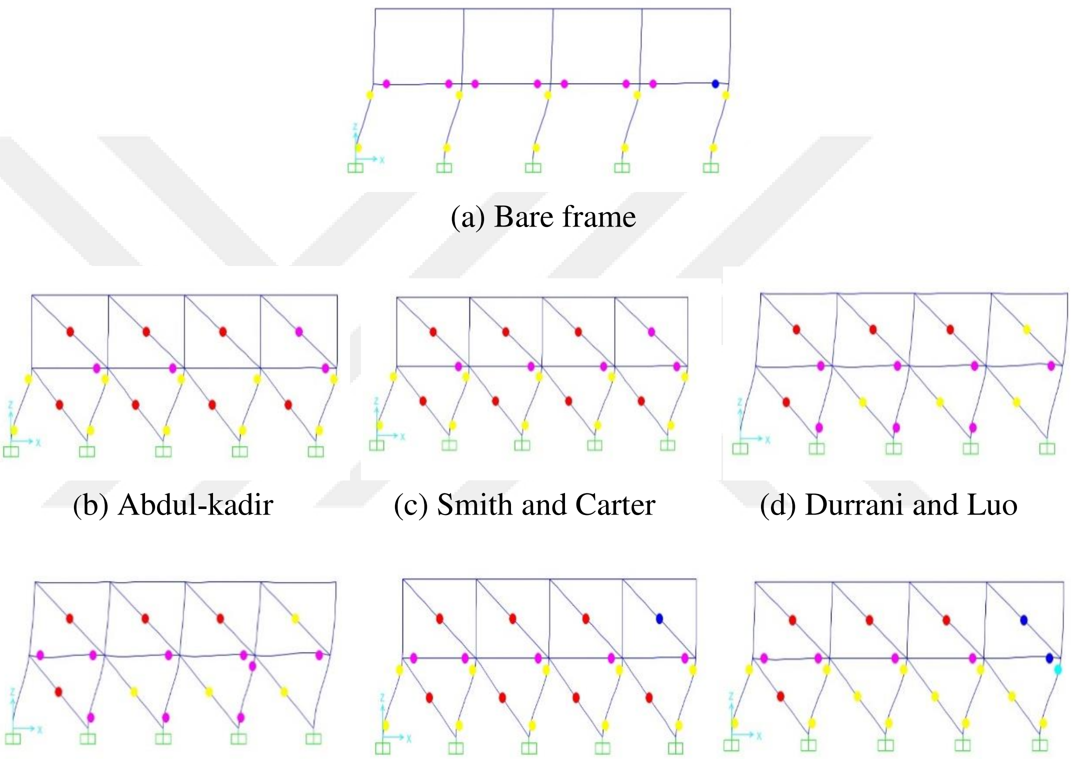
The formation of plastic hinges based on FEMA-356 rules is introduced as input into the SAP 2000 program. At every step of deformation in the pushover analysis, the program can do the following (Giannopoulos, 2009):

a) Determine the position and plastic rotation of hinges in beams and columns.
b) Determine which hinges have reached one of the three FEMA-356 limit states: IO, LS and CP using suitable colors for their identification.

Figure 4.5 shows the colors for plastic hinges limit state identification.

Figure 4.5 Plastic hinges limit state colors identification (Hakim et al., 2014)

Figure 4.6 shows variations of plastic hinge formation at mechanism of two-storey frame with and without infills. From this figure, it can be seen that beams were in safe state in both cases of bare frame and infilled frame. Columns of only the first storey have reached collapse point except in case of infill walls modeled according to Durrani and Luo (1994) and Paulay and Priestley (1992). This means that equivalent strut with geometry proposed by those authors could carry most loads to keep the structure safe. No plastic hinges had formed in beams and columns of the second storey. In case of infill walled frames, infill walls have reached the state of final loss of resistance (point E) for wide strut while for small strut 50% of the infills have just reached collapse point.



(a) Bare frame

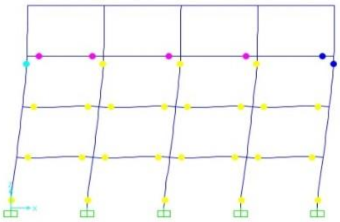
(b) Abdul-kadir (c) Smith and Carter (d) Durrani and Luo

(e) Paulay and Priestley (f) Holmes (g) FEMA

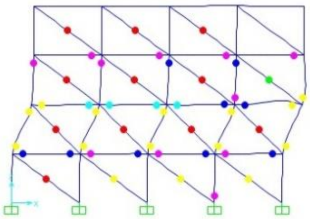
Figure 4.6 Variation of plastic hinge formations at mechanism of 2-storey frame according to the formulation by different researchers

According to Figure 4.7, at last floor (fourth floor), no plastic hinges have appeared in both beams and columns in case of bare and infilled frame. For bare frame, beams reached collapse in first and second floors while for infilled frame beams collapsed in second floor only. Only the interior columns in first and third storey have collapsed in case of bare frame. For infilled frame, there was different distribution of plastic hinges in columns. According to Abdul-kadir (1974) and Smith and Carter (1969), the collapse appeared in the second floor, according to Holmes (1961) and

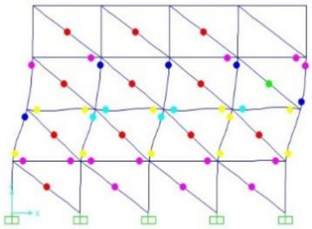
FEMA (1998) collapse appeared in the third floor while collapse appeared in both second and third floor according to Durrani and Luo (1994). The whole structure (beams and columns) was in safe when infill walls were modeled according to Paulay and Priestley (1992) noting that most of infill walls failed under pushover loads and a few of them collapsed only.



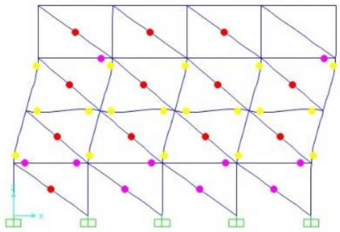
(a) Bare frame



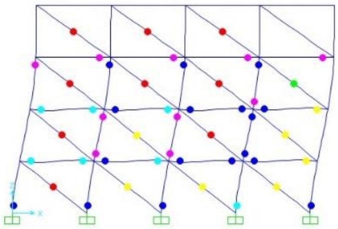
(b) Abdul-kadir



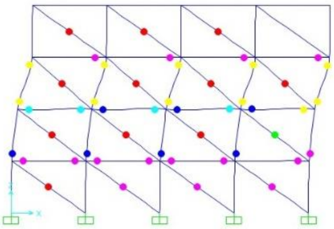
(c) Smith and Carter



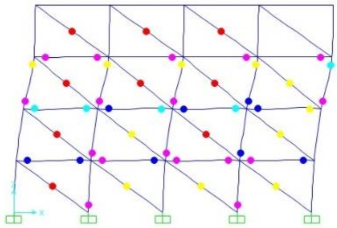
(d) Durrani and Luo



(e) Paulay and Priestley



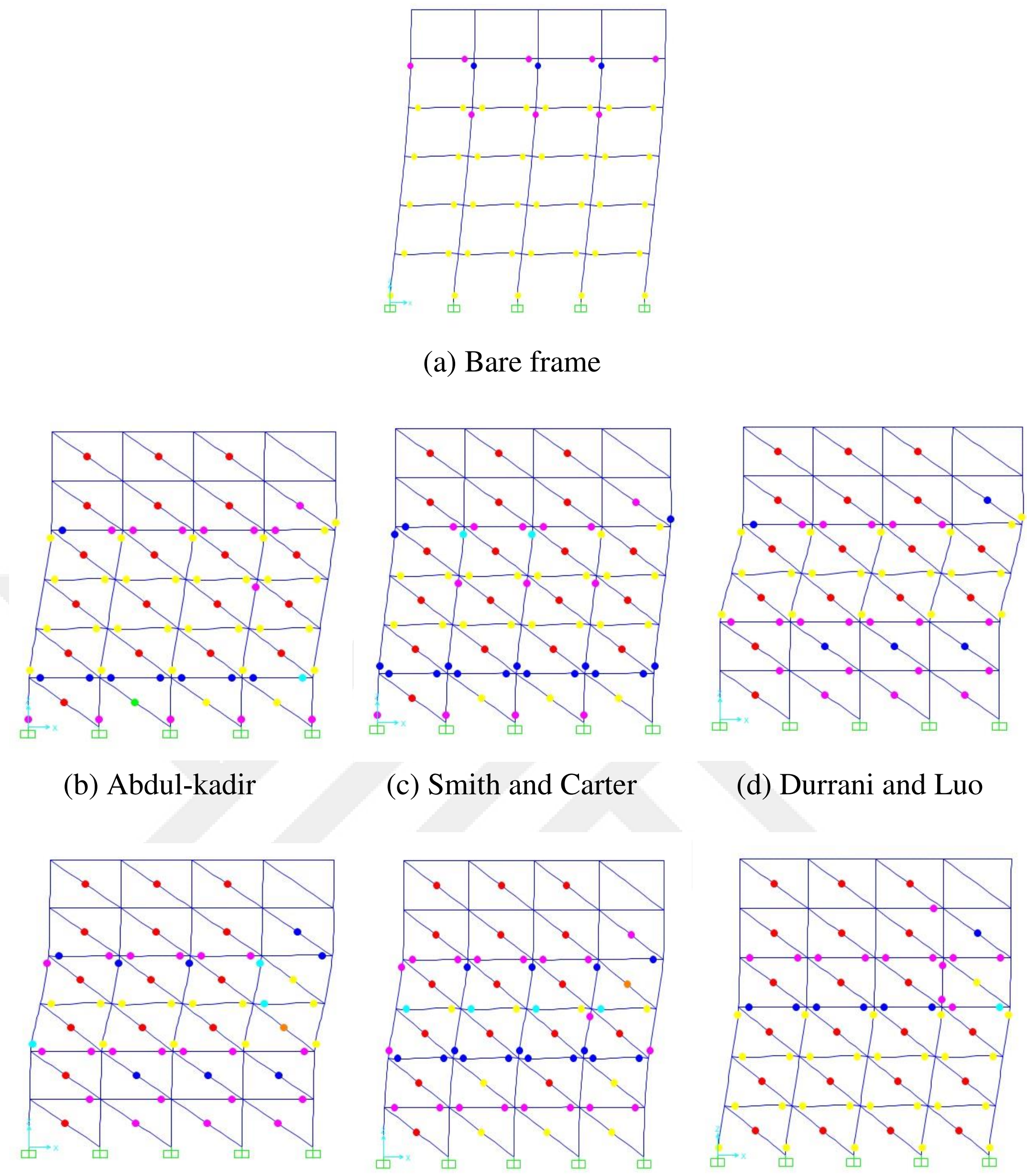
(f) Holmes



(g) FEMA

Figure 4.7 Variation of plastic hinge formations at mechanism of 4-storey frame according to the formulation by different researchers

Based on the results shown in Figure 4.8, for a bare frame of six storeys, beams reached collapse point at 1^{st}, 2^{nd}, 3^{th}, and 4^{th} floor while columns were in safe state except those in the 1^{st} floor. For infilled frames, no frame model was found in safe mode under nonlinear analysis. Collapse can be observed in second, third, and fourth floor in case of wide strut and in first, second, and third floor in case of thin strut while collapse can be observed only in third and fourth floor in case of models with the average value of strut width. Collapse and failure can be seen in all infill panels except in two upper right infill panels.



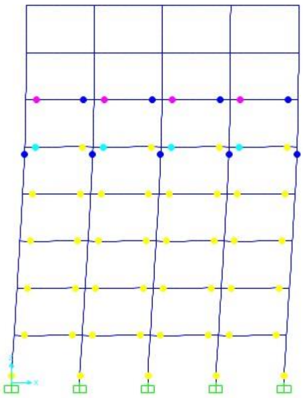
(a) Bare frame

(b) Abdul-kadir (c) Smith and Carter (d) Durrani and Luo

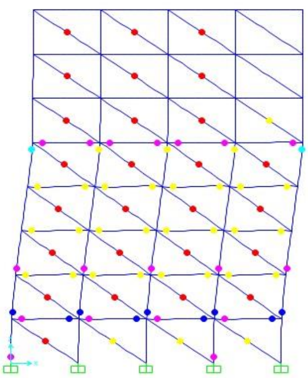
(e) Paulay and Priestley (f) Holmes (g) FEMA

Figure 4.8 Variation of plastic hinge formations at mechanism of 6-storey frame according to the formulation by different researchers

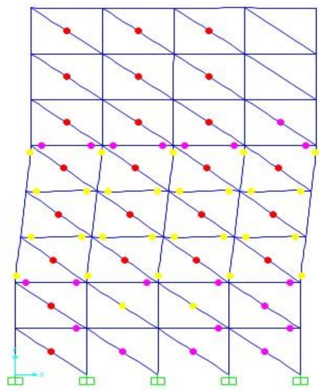
The distribution of plastic hinges in 8-storey building can be shown in Figure 4.9. Beams collapsed at the first five storey while columns collapsed only at the first storey in case of bare frame. In all infilled frame models, beams reached collapse at third and fourth floor except frame model with wider strut (according to Abdul-kadir (1974)) where beams collapsed at second, third, and fourth floor. The presence of infill walls has changed the distribution of plastic hinges in columns where columns have reached collapse state at third and fifth storey for models with wide and thin strut. Whereas, for models with moderate value of the strut width, obtained from formulations of Durrani and Luo (1994) and Paulay and Priestley (1992), columns in only third floor have collapsed. It is worth mentioning that most of the infill walls in 8-storey frames failed under pushover loading.



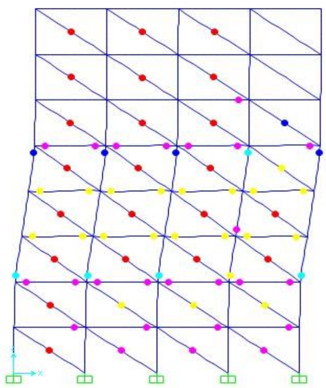
(a) Bare frame



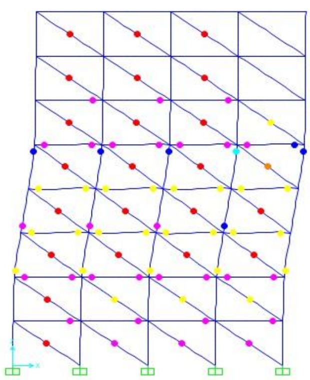
(b) Abdul-kadir



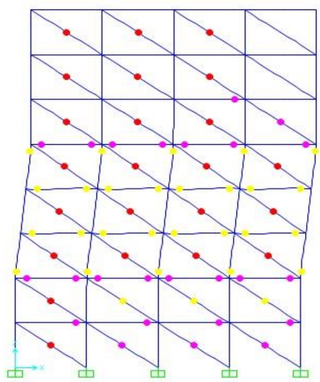
(c) Smith and Carter



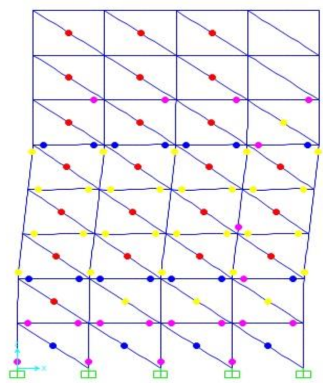
(d) Durrani and Luo



(e) Paulay and Priestley



(f) Holmes



(g) FEMA

Figure 4.9 Variation of plastic hinge formations at mechanism of 8-storey frame according to the formulation by different researchers

4.3 Effect of Opening Size on the Capacity Curve

In equivalent diagonal compression strut method, full-infill wall can be modeled, in this study, as a single strut. On the other hand, openings are usually left in buildings such as doors and windows within the infill walls. The effect of these openings must be taken in consideration through design process.

Generally, the existence of openings reduces strength and stiffness of infilled structures. A survey of design and analysis rules related to masonry walls infilled RC structures in the codes of seismic design of different countries offers a few codes which have considered the influence of infills in the design and analysis of masonry walls infilled RC structures. Moreover, the strength and stiffness of the infilled structures having openings are not taken a considerable attention by most of the design codes. Thereby, the behavior of infilled structures having openings is required to be studied vastly so as to evolve rational guidelines for design (Mondal and Jain, 2008).

The infilled structure stiffness is affected by the dimensions of the opening. Analytical equation of a reduction factor have proposed, which represents the ratio of the effective diagonal strut width of a panel with openings over that of a solid panel, so as to be able to determine the initial lateral stiffness of RC structures infilled with panels having openings (Asteris, 2012).

This reduction factor can be used to reduce the width of the diagonal strut in case of an infill panel with openings by multiplying the reduction factor with the diagonal strut width (see section 3.3.2).

After obtaining the equivalent width of the diagonal strut by using the reduction factor, the new width, corresponds to a specified opening size, takes place in modeling analysis instead of the strut width of an infill without openings in order to verify the effect of each size of central openings on the capacity curve of the infilled frame.

Figures 4.10 to 4.15 show the capacity curves according to different formulations of two-storey infilled frames with different percentages of opening. As shown in these figures, the existence of openings in infill walls reduced the strength. By increasing the opening percentage the strength of the building decreased. A specified opening size results in different reduction in the capacity curve of the infilled frame modeled with equivalent strut according to different formulations. However, the results showed that masonry infill walls with opening size exceed 40% almost lead to cancel the contribution of infill walls to the stiffness and strength of the building. Opening of size 20% of the infill wall reduced the base shear of the solid infill by about 40-50% with respect to wider strut (according to Abdul-kadir (1974)) and smaller strut (according to FEMA (1998)), respectively. Whereas, infill walls with opening size of 40% decreased the base shear capacity of the solid wall by about 50-60% with respect to wider and smaller strut, respectively. Infill walled frames with opening size of 60-80% are behaved just like a bare frame with a bit rise in strength. Thus, infill walls with big opening sizes do not produce a contribution to the structural response of the building frame under the nonlinear static loading. It is worth to mention that the existence of openings in infill walls increase the ductility of the structural system.

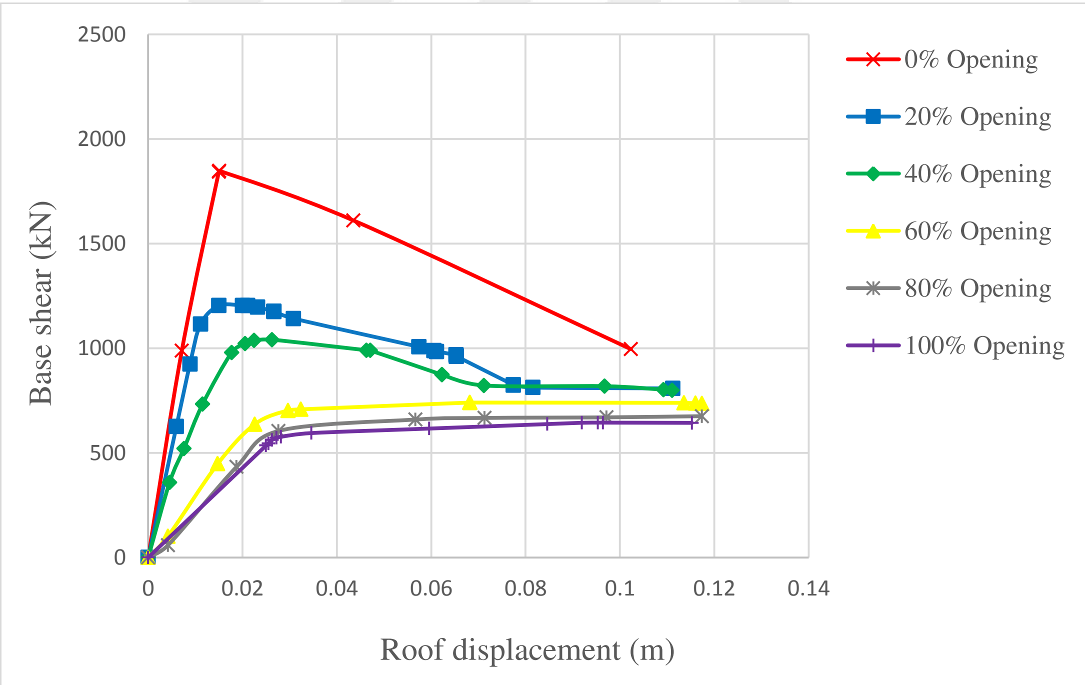


Figure 4.10 Effects of openings in infill walls on the capacity curve of 2-storey frame according to Abdul-kadir (1974)

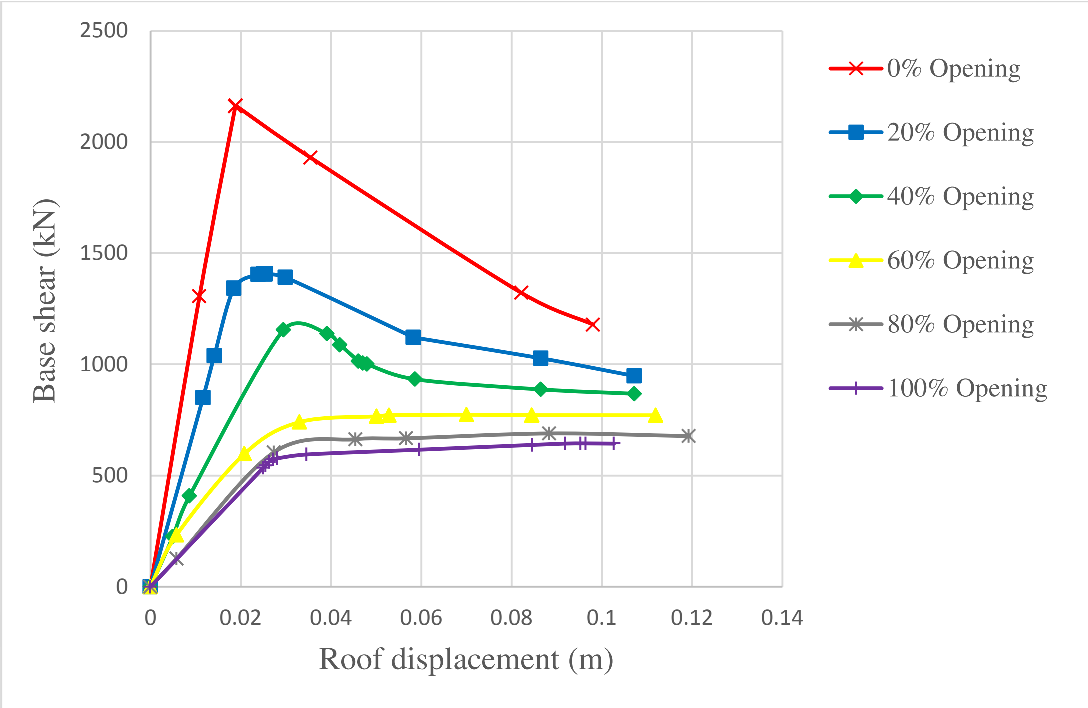


Figure 4.11 Effects of openings in infill walls on the capacity curve of 2-storey frame according to Smith and Carter (1969)

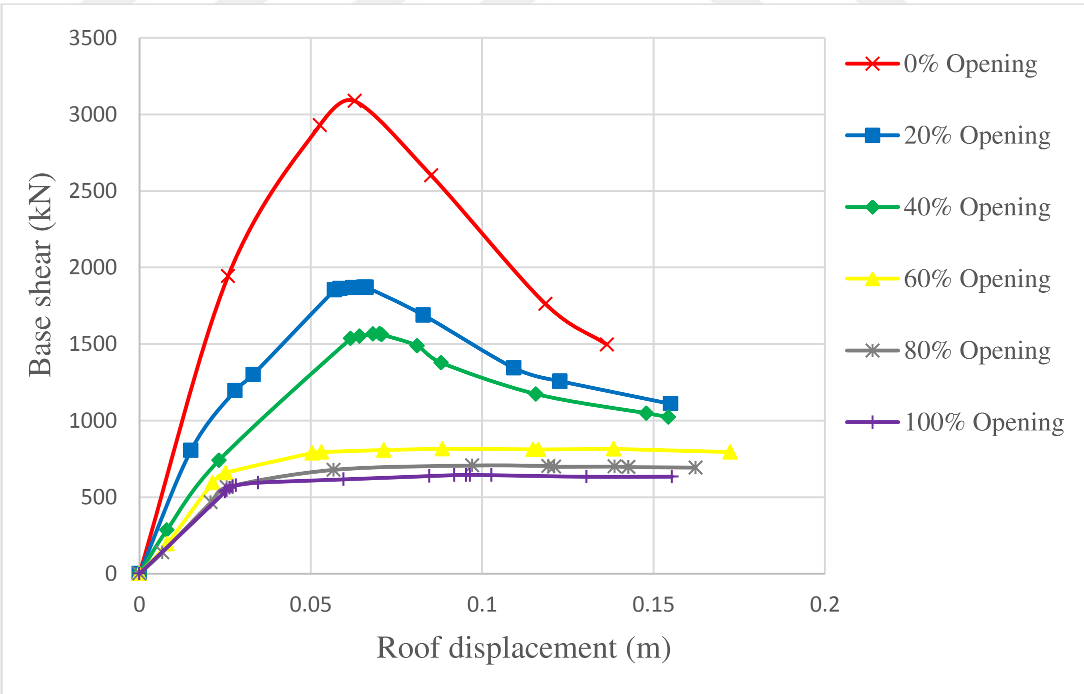


Figure 4.12 Effects of openings in infill walls on the capacity curve of 2-storey frame according to Durrani and Luo (1994)

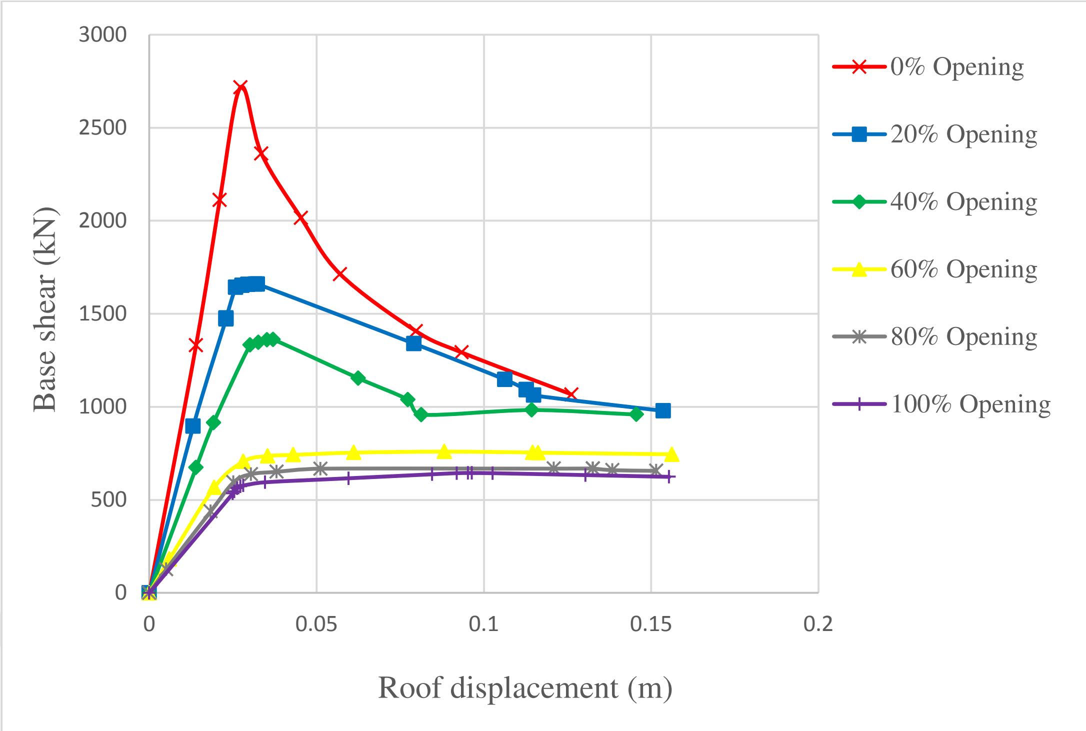


Figure 4.13 Effects of openings in infill walls on the capacity curve of 2-storey frame according to Paulay and Priestley (1992)

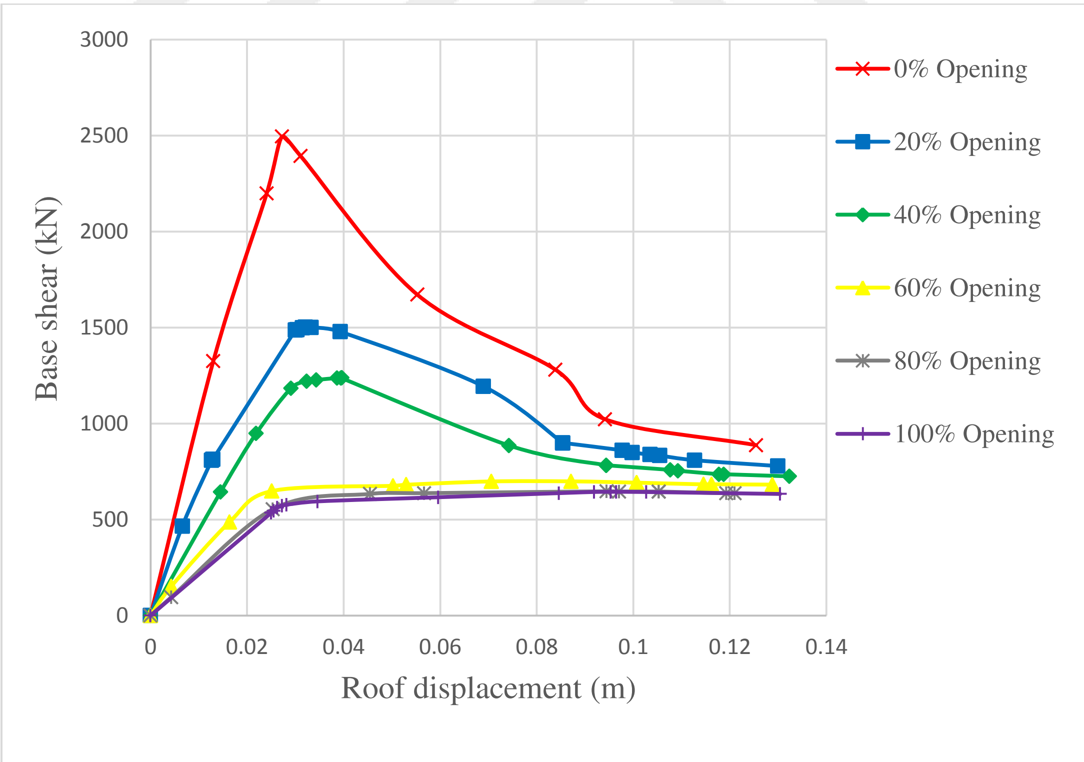


Figure 4.14 Effects of openings in infill walls on the capacity curve of 2-storey frame according to Holmes (1961)

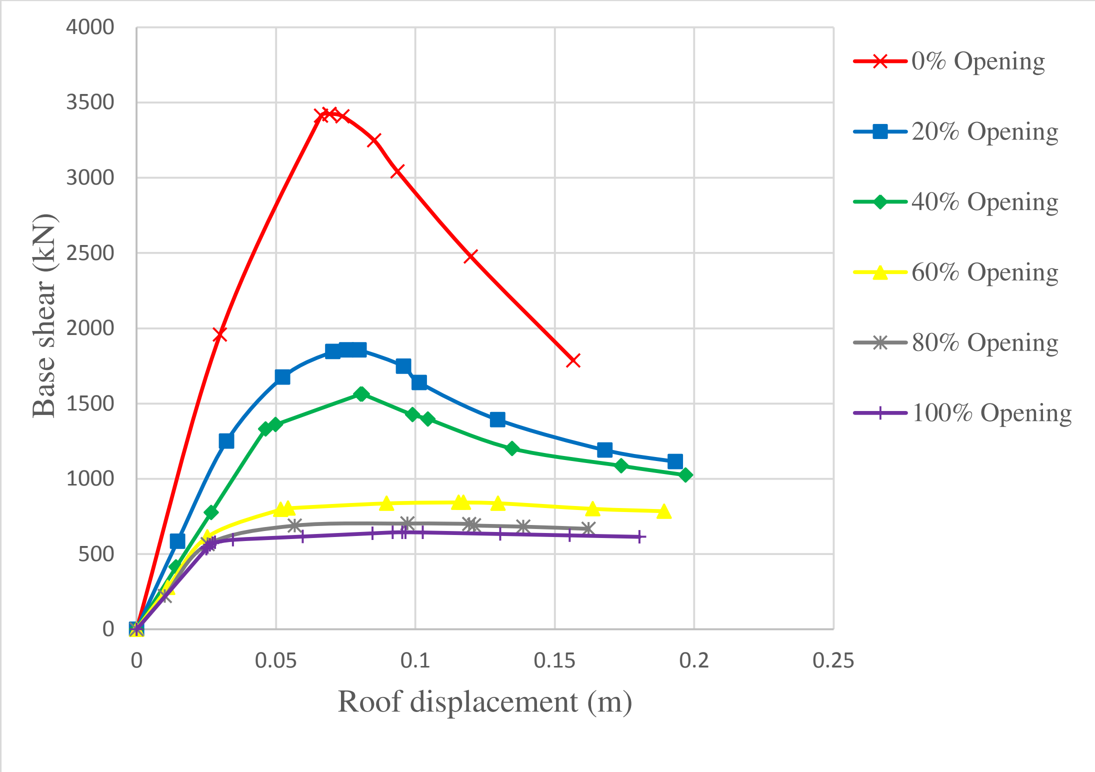


Figure 4.15 Effects of openings in infill walls on the capacity curve of 2-storey frame according to FEMA (1998)

Figures 4.16 to 4.21 show the capacity curves according to different formulations of four-storey infilled frames with different percentages of opening. As seen in these figures, the peak strength capacity of infilled frames decrease as the opening size increase. The reduction amount in peak strength with respect to different opening percentages was found very close to that of two-storey frames. On the other hand, the roof displacement was found to be higher than that of two-storey frame models. This means more ductile behavior can be obtained if the number of storeys increases to four.

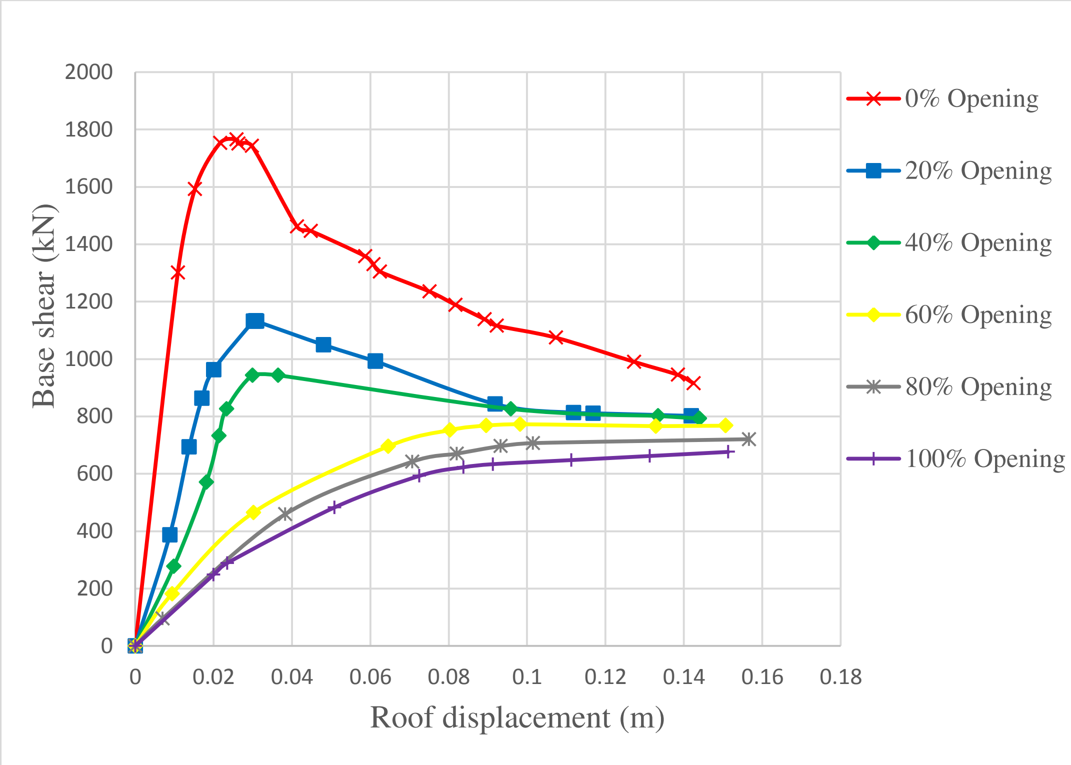


Figure 4.16 Effects of openings in infill walls on the capacity curve of 4-storey frame according to Abdul-kadir (1974)

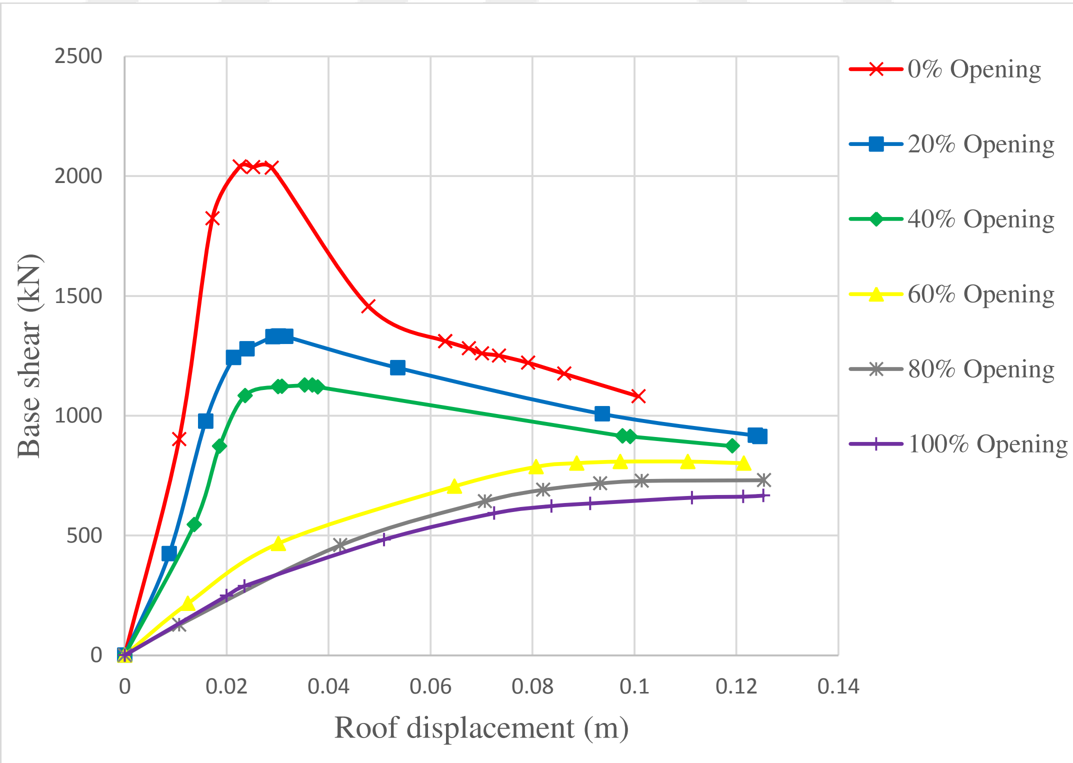


Figure 4.17 Effects of openings in infill walls on the capacity curve of 4-storey frame according to Smith and Carter (1969)

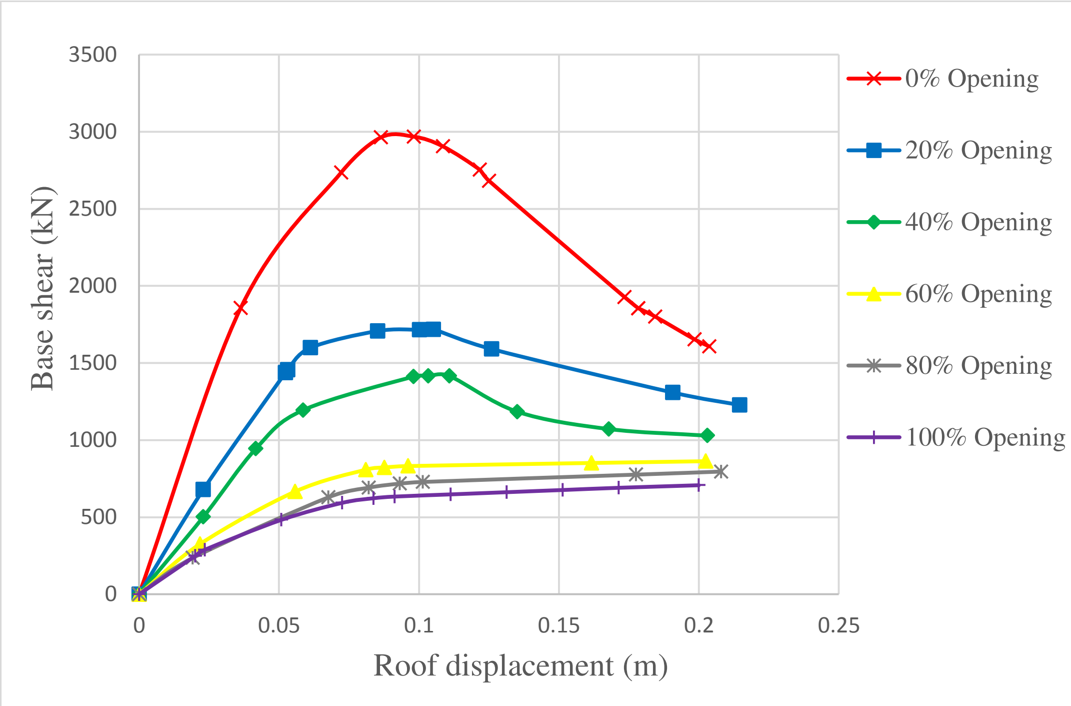


Figure 4.18 Effects of openings in infill walls on the capacity curve of 4-storey frame according to Durrani and Luo (1994)

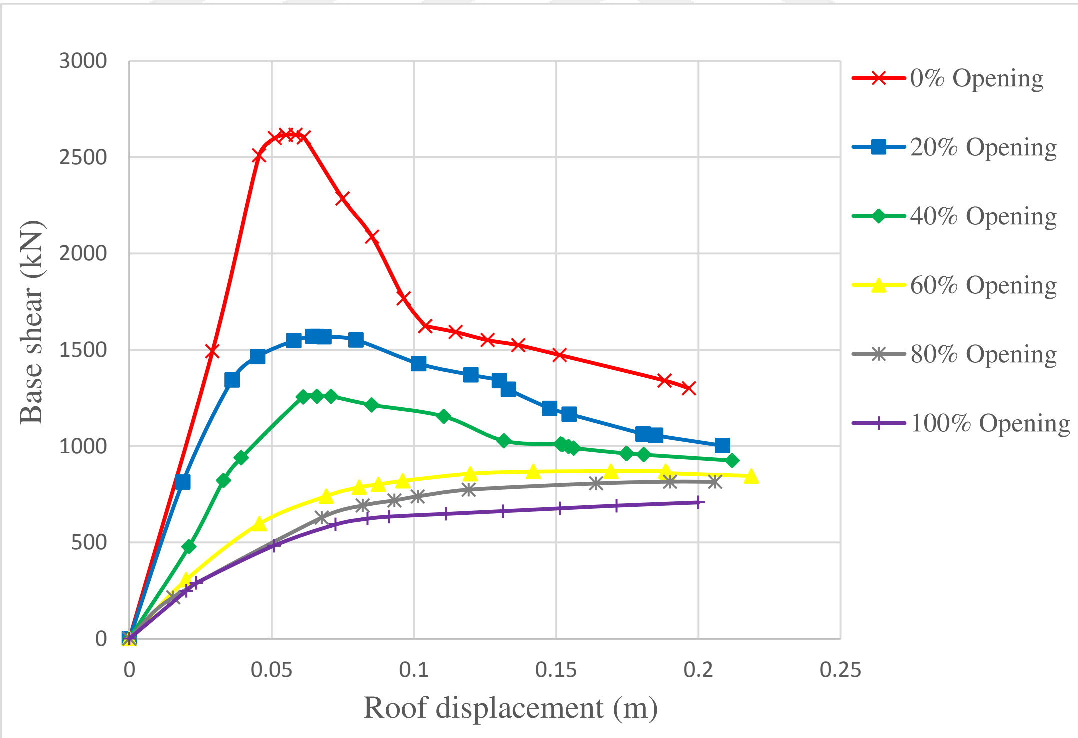


Figure 4.19 Effects of openings in infill walls on the capacity curve of 4-storey frame according to Paulay and Priestley (1992)

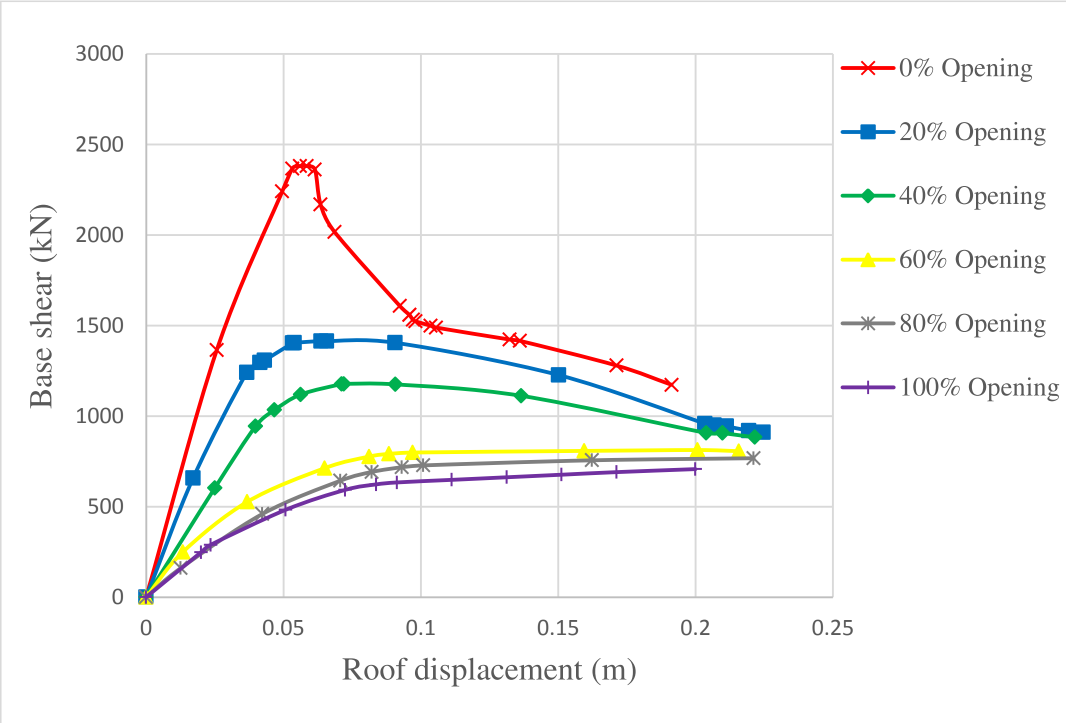


Figure 4.20 Effects of openings in infill walls on the capacity curve of 4-storey frame according to Holmes (1961)

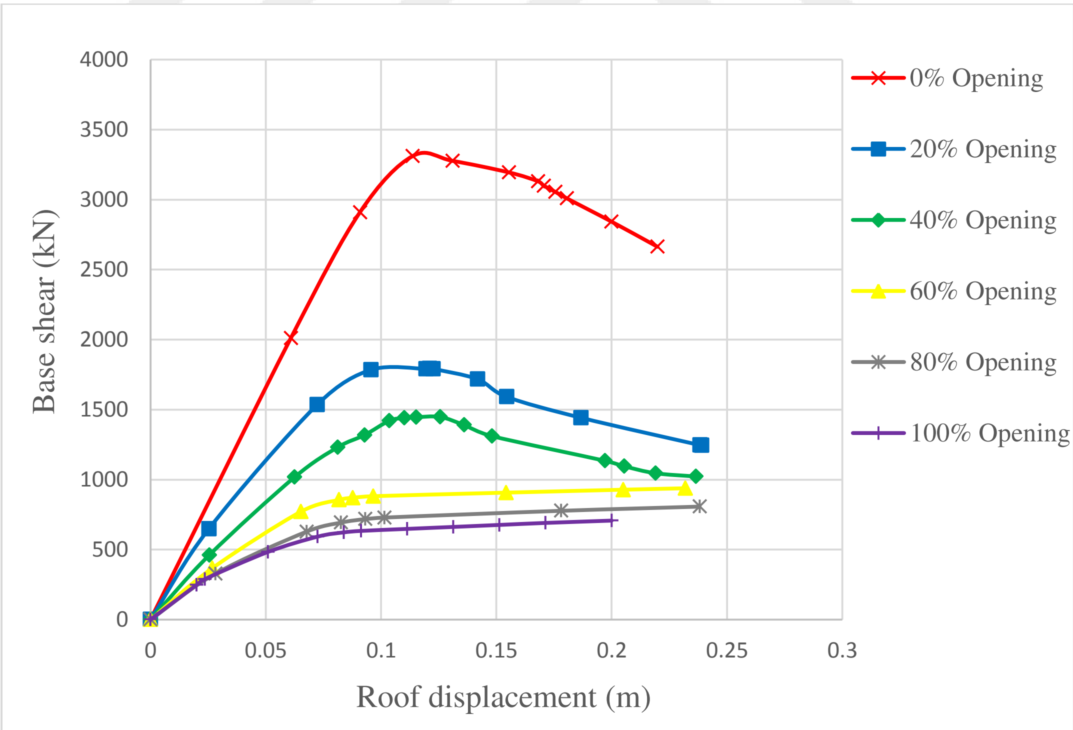


Figure 4.21 Effects of openings in infill walls on the capacity curve of 4-storey frame according to FEMA (1998)

From analysis results, it can be concluded that at a particular opening size the base shear capacity decreased with the increase of building height. By increasing the height of building to six and eight storeys, it was quite obvious that the ductility of infilled frame models increases too, irrespective of opening percentage. This behavior was characterized by lower peak strength. Thus, all building frames with different number of storeys had conducted similar to each other, in case of solid infill walls and infill walls with openings, with respect to strength and ductility. Figures 4.22 to 4.27 show the capacity curves according to different formulations of six-storey infilled frames with different percentages of opening and Figures 4.28 to 4.33 show the same capacity curves obtained by analyzing eight-storey frame models.

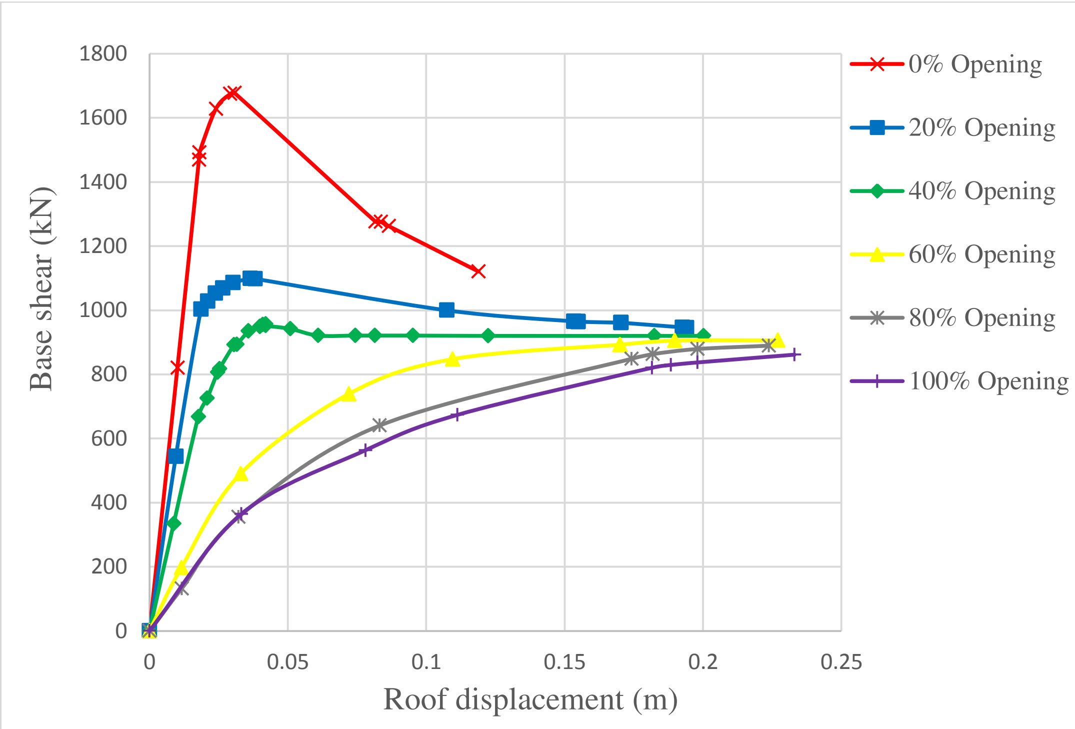


Figure 4.22 Effects of openings in infill walls on the capacity curve of 6-storey frame according to Abdul-kadir (1974)

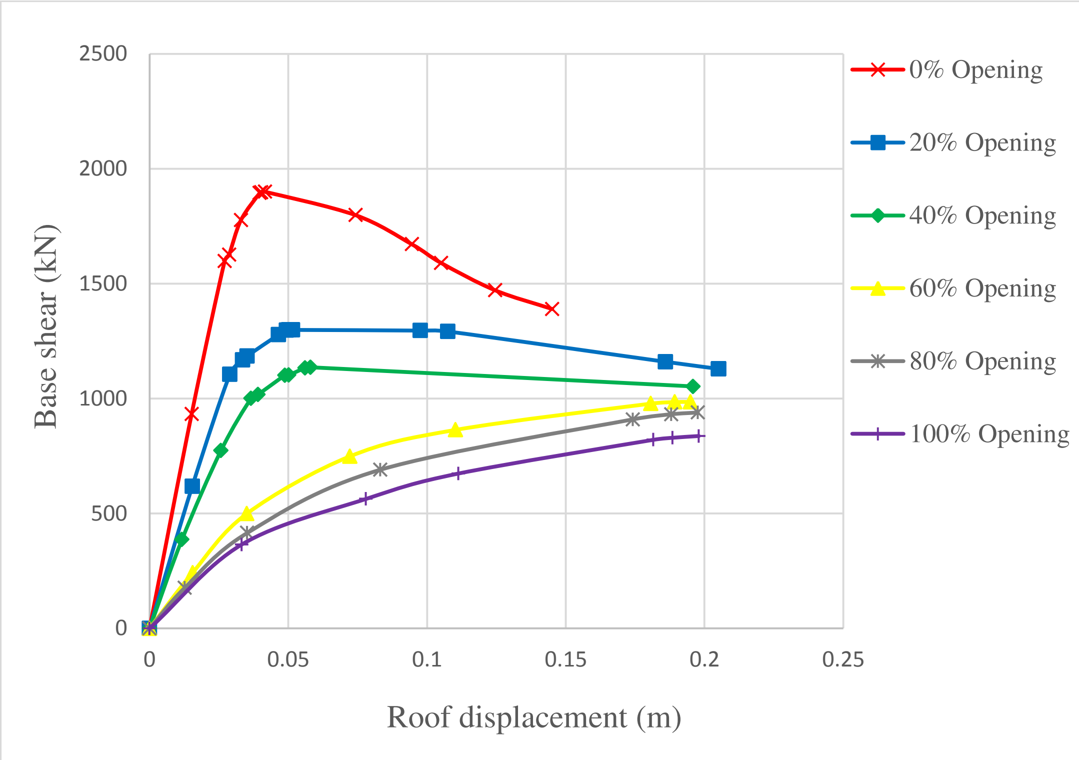


Figure 4.23 Effects of openings in infill walls on the capacity curve of 6-storey frame according to Smith and Carter (1969)

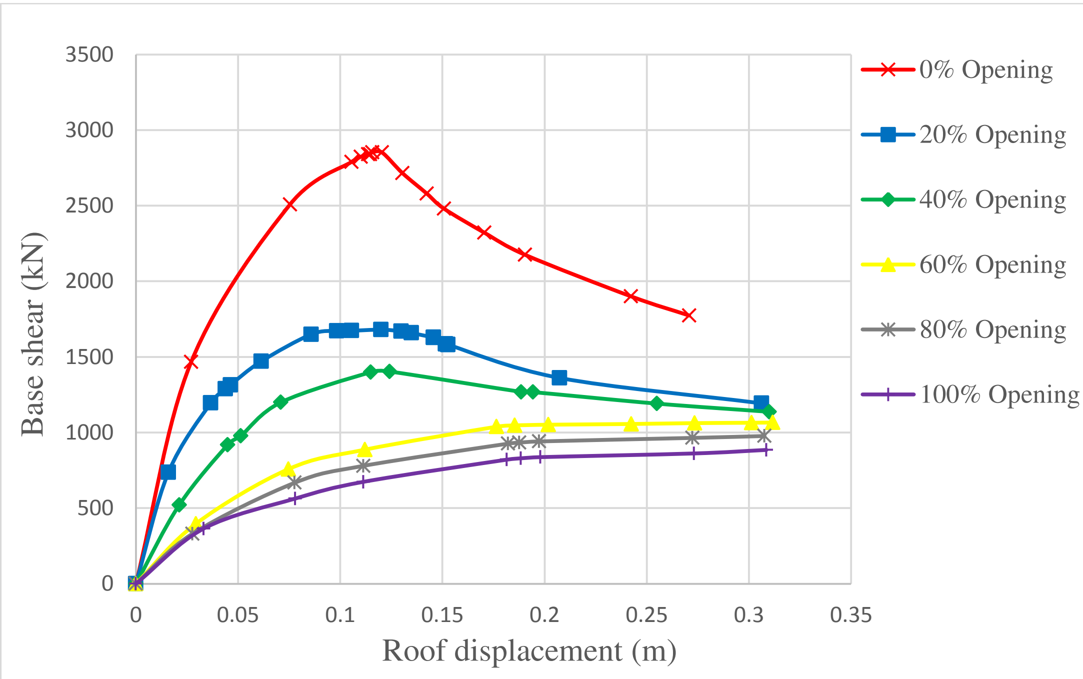


Figure 4.24 Effects of openings in infill walls on the capacity curve of 6-storey frame according to Durrani and Luo (1994)

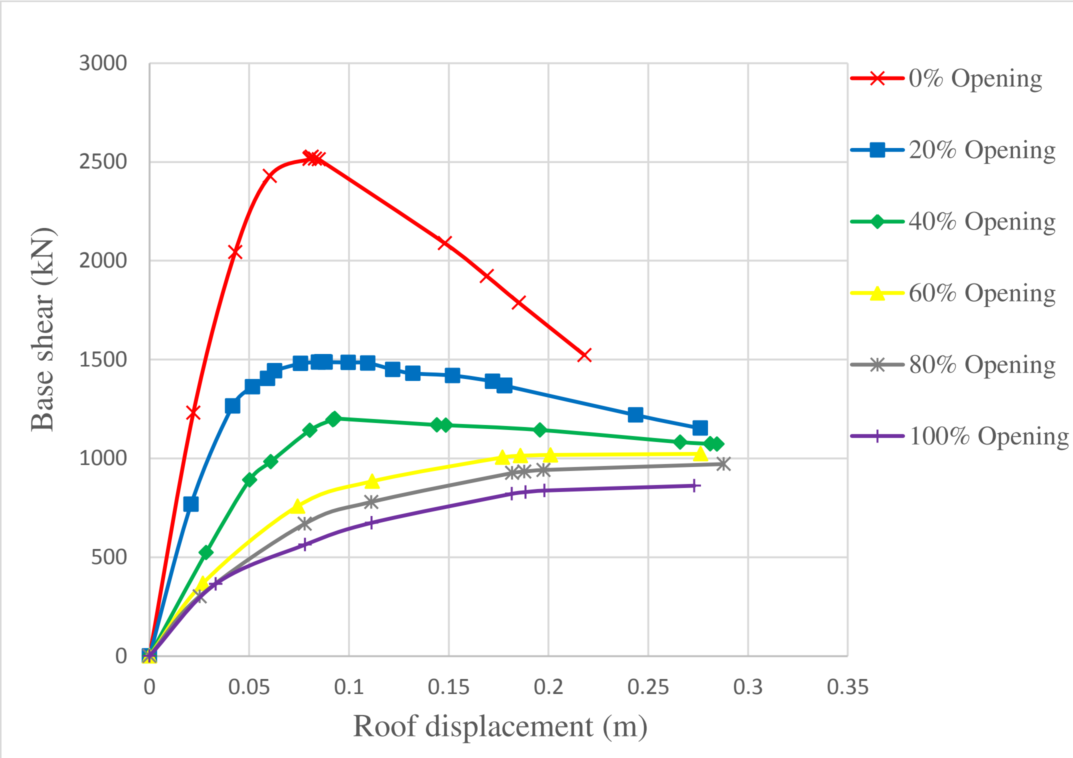


Figure 4.25 Effects of openings in infill walls on the capacity curve of 6-storey frame according to Paulay and Priestley (1992)

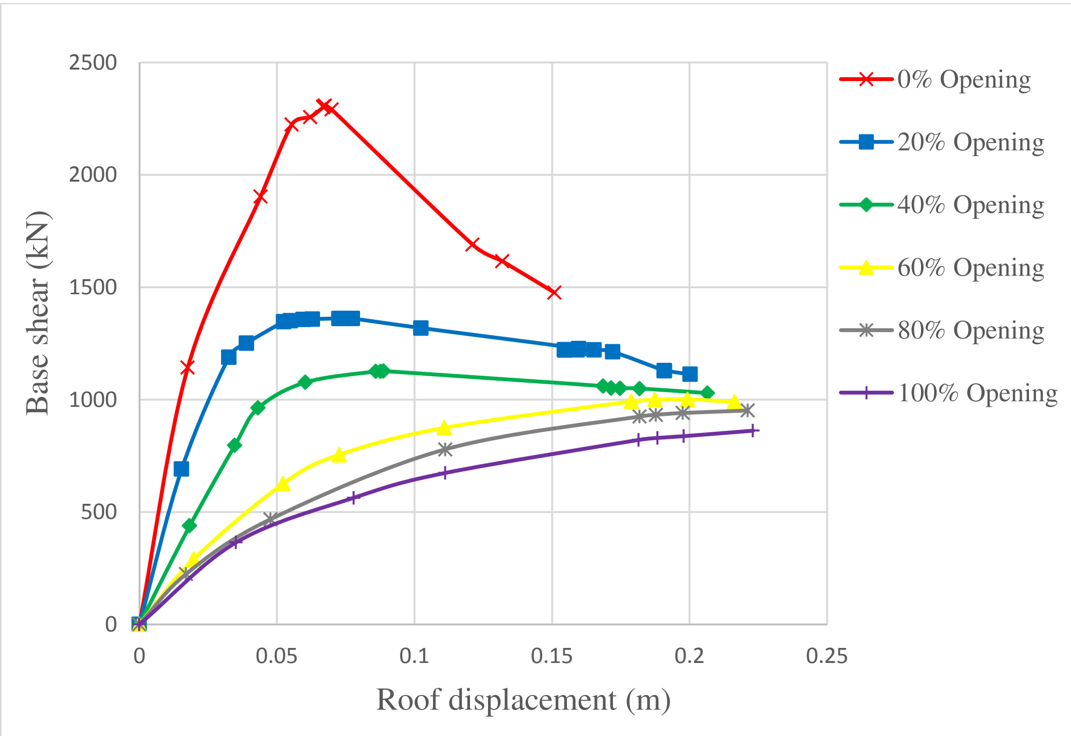


Figure 4.26 Effects of openings in infill walls on the capacity curve of 6-storey frame according to Holmes (1961)

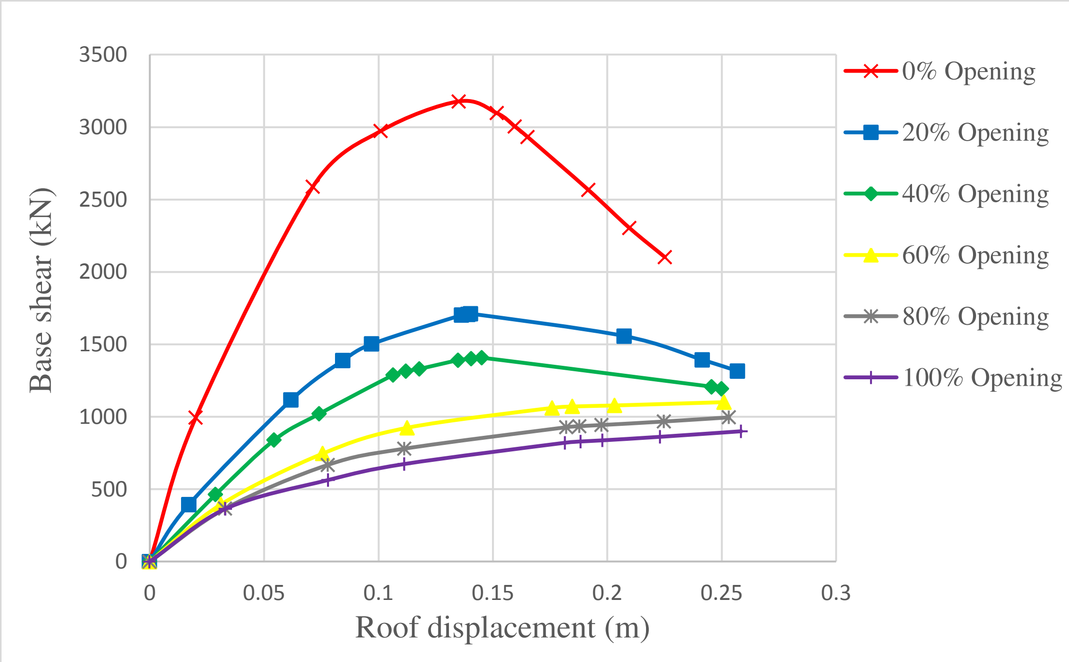


Figure 4.27 Effects of openings in infill walls on the capacity curve of 6-storey frame according to FEMA (1998)

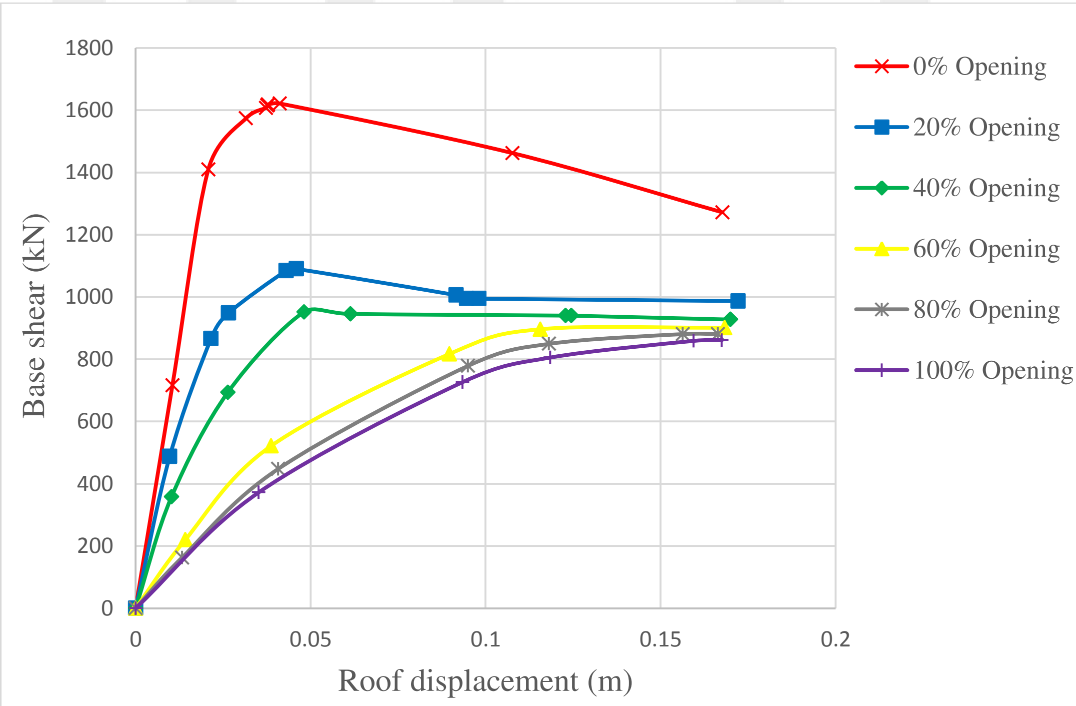


Figure 4.28 Effects of openings in infill walls on the capacity curve of 8-storey frame according to Abdul-kadir (1974)

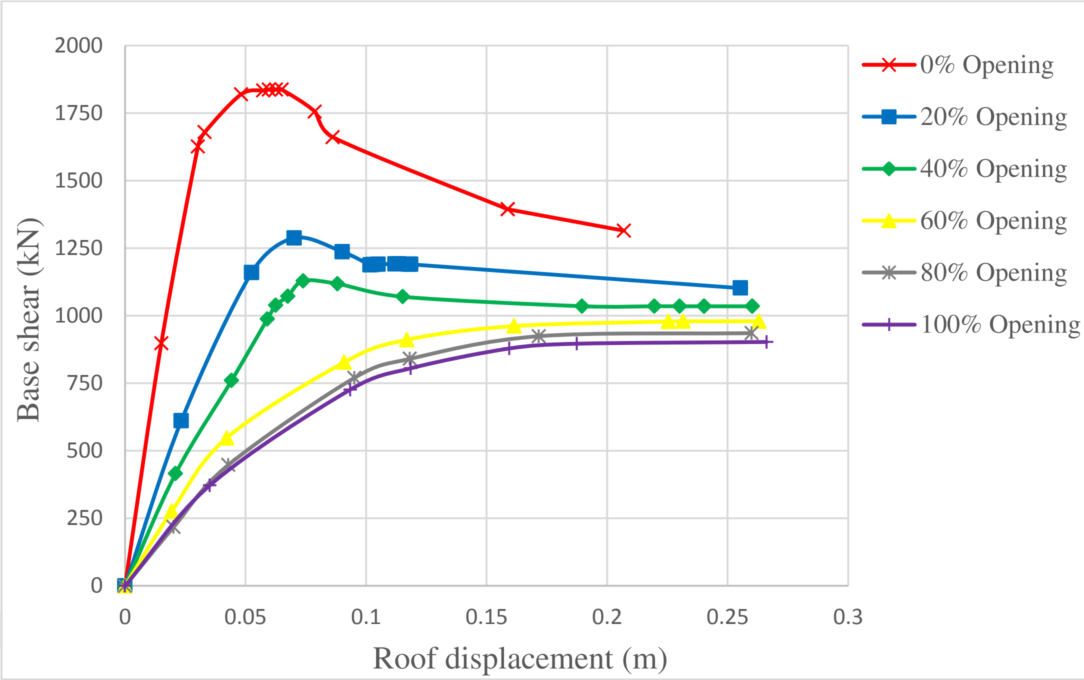


Figure 4.29 Effects of openings in infill walls on the capacity curve of 8-storey frame according to Smith and Carter (1969)

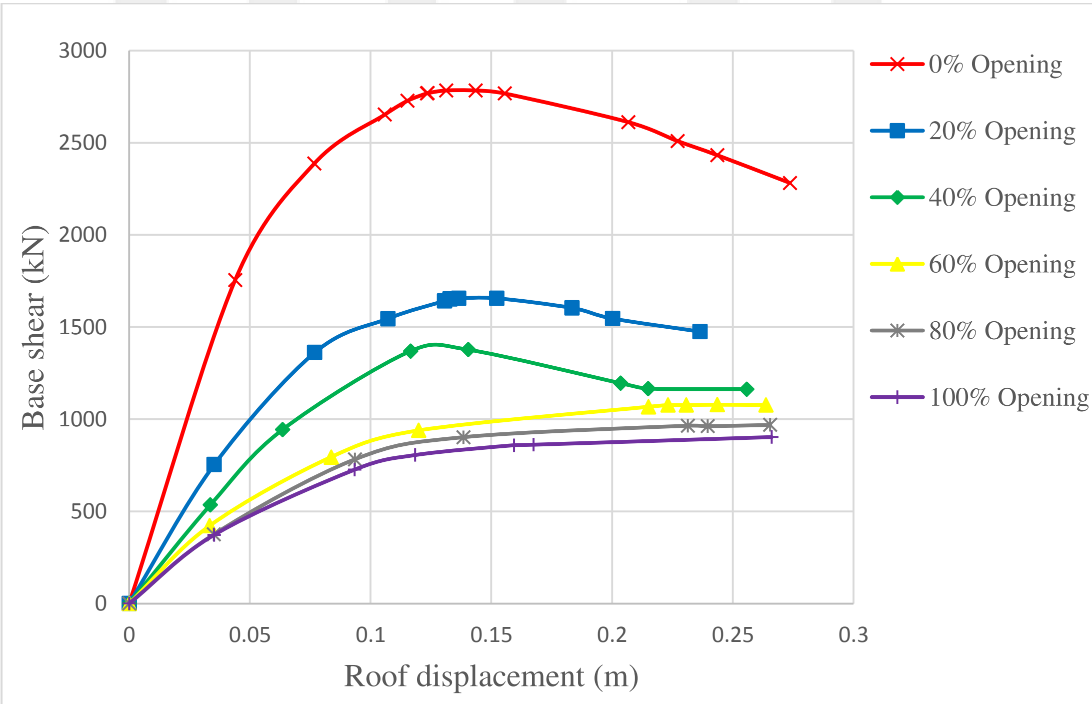


Figure 4.30 Effects of openings in infill walls on the capacity curve of 8-storey frame according to Durrani and Luo (1994)

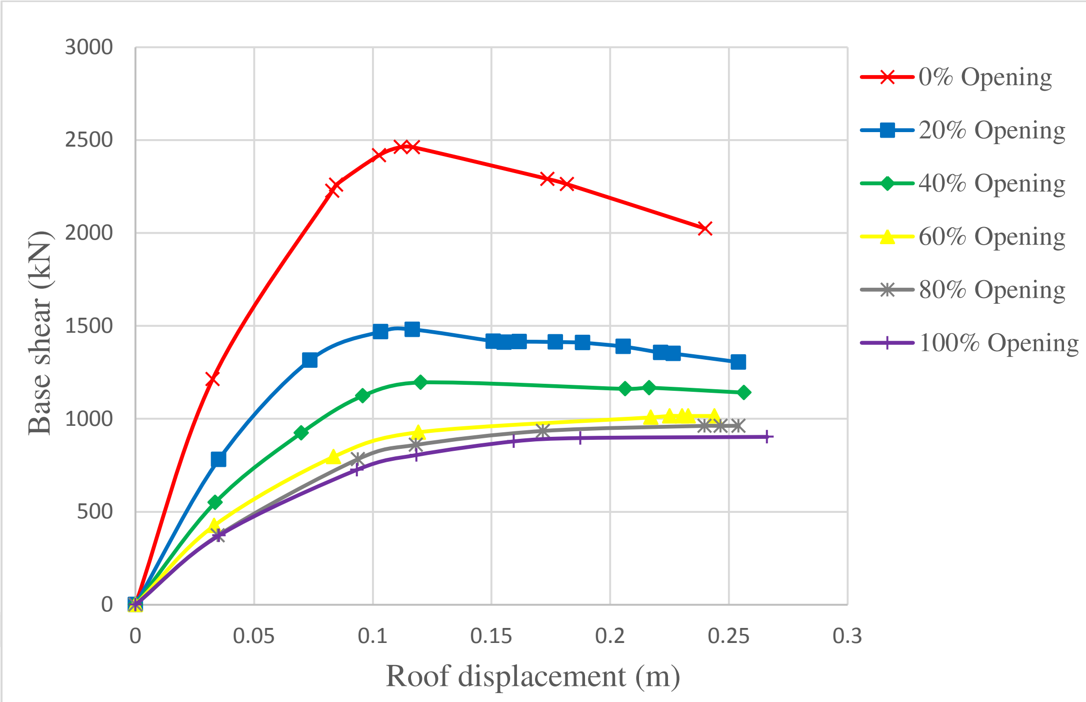


Figure 4.31 Effects of openings in infill walls on the capacity curve of 8-storey frame according to Paulay and Priestley (1992)

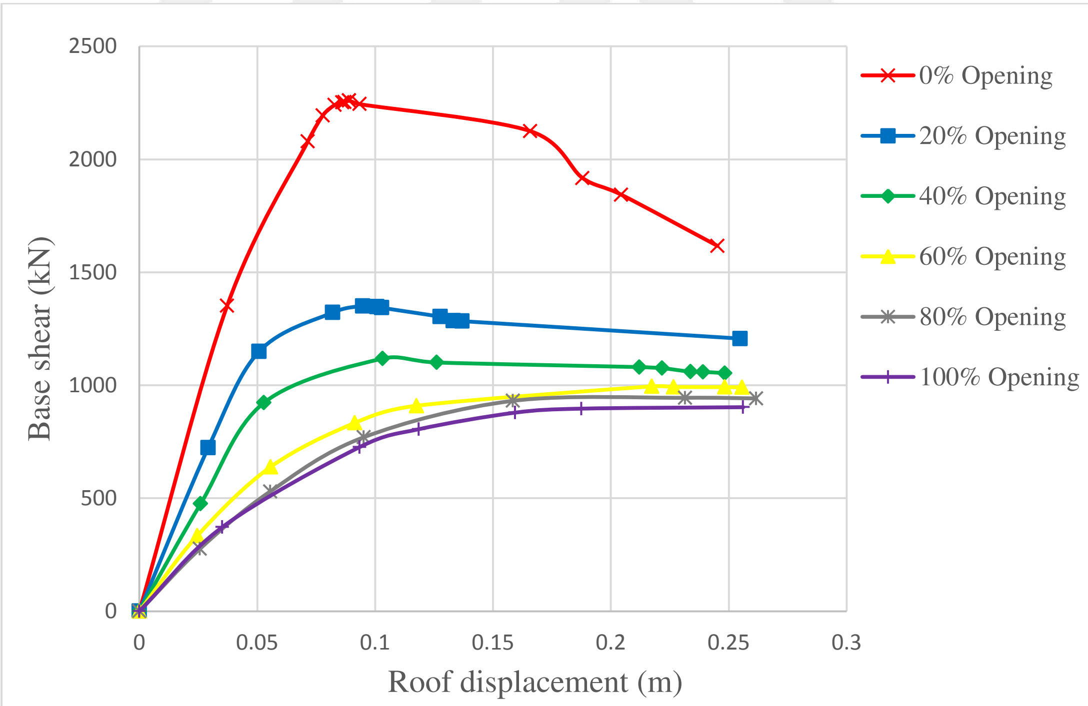


Figure 4.32 Effects of openings in infill walls on the capacity curve of 8-storey frame according to Holmes (1961)

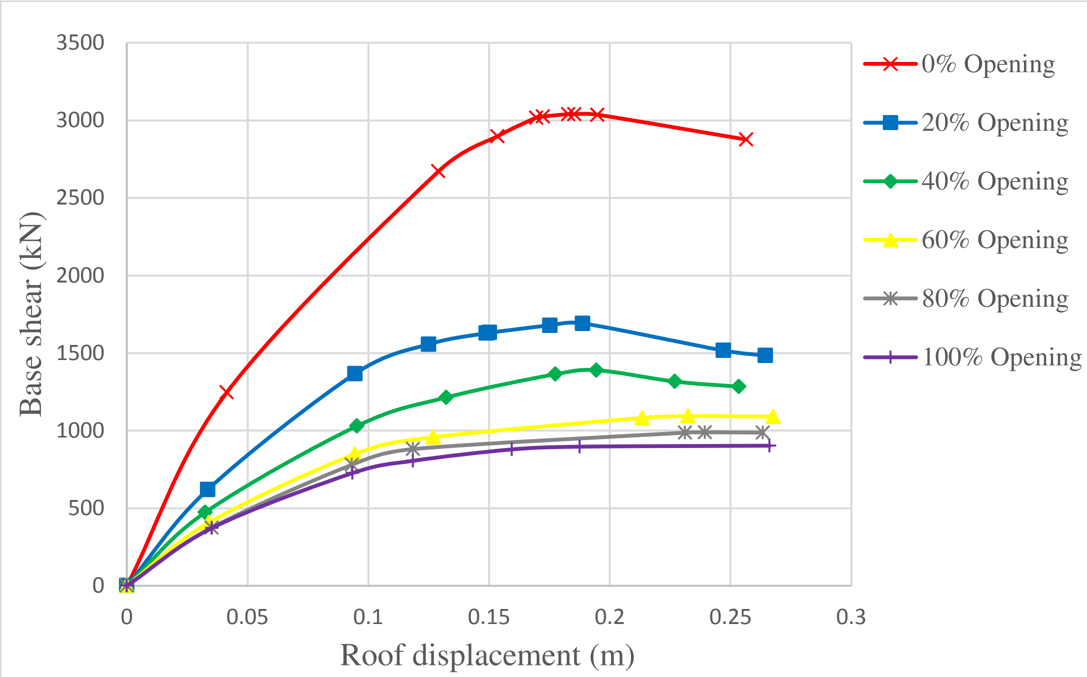


Figure 4.33 Effects of openings in infill walls on the capacity curve of 8-storey frame according to FEMA (1998)

4.4 Comparison of Maximum Base Shear

The maximum base shear capacity of frame buildings with and without infill walls was studies. Infill walled frames with equivalent strut based on different formulations have produced different values of maximum base shear. The wider strut given by Abdul-kadir (1974) formula has characterized by the lowest base shear while the thinner strut given by FEMA (1998) formula had the highest value of base shear. Figure 4.34 shows the maximum base shear provided by infilled frames according to different formulations. From this figure, it can be said that the maximum base shear is a function of width of equivalent strut, i.e. as the strut width increased the maximum base shear decreased. Another parameter (openings in infill walls) influencing the maximum base shear was studied. The effect of openings on the maximum base shear is quite obvious in Figure 4.34. The presence of openings decreased the base shear capacity significantly. It was found that the infilled frame with opening percentage exceed 40%, lost most contribution of infill walls to the shear capacity. Also, an infill wall with opening percentage of 20-40% could decrease the maximum base shear by 40-60%. Infilled frames with opening percentage of 60% and 80% behaved almost like a bare frame and their maximum base shear values were close to that one of the bare frame.

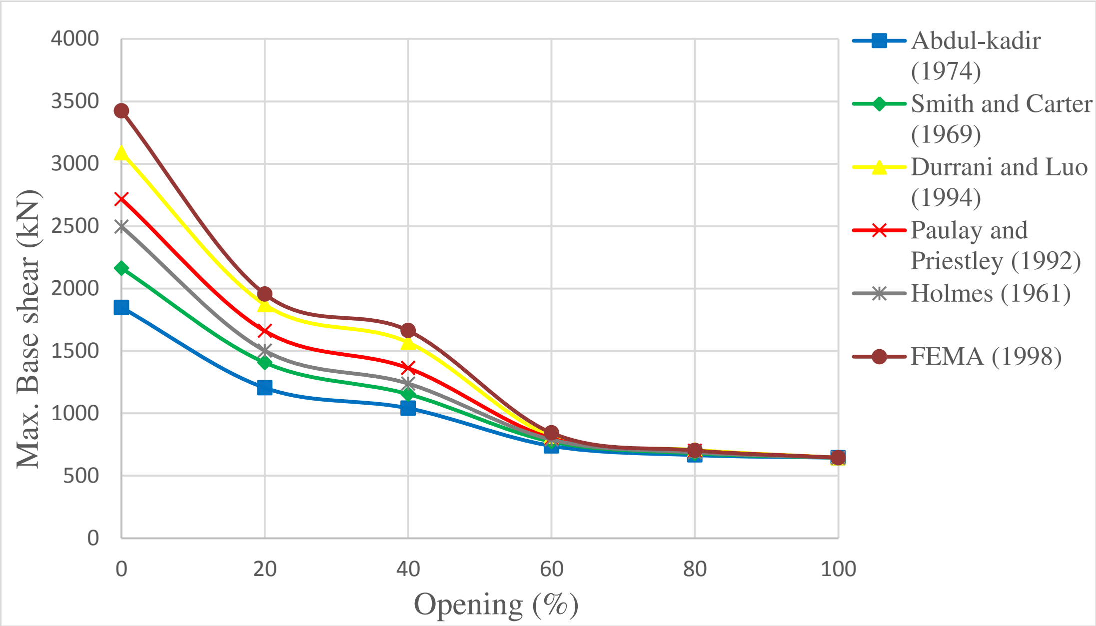


Figure 4.34 Variation of max. base shear with opening of 2-storey frame according to the formulation by different researchers

The height of frame has a considerable effect on the base shear capacity. As seen in Figures 4.35 to 4.37, the maximum base shear of infilled frame with different opening sizes decreased as the height of building increased because the case study of tall building is more ductile than short building.

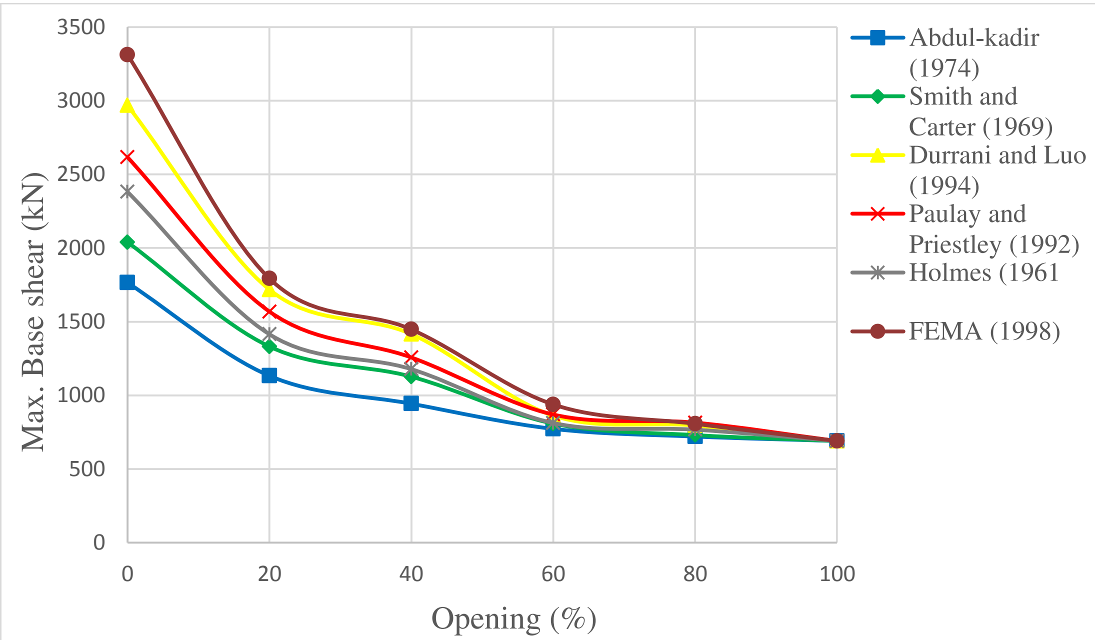


Figure 4.35 Variation of max. base shear with opening of 4-storey frame according to the formulation by different researchers

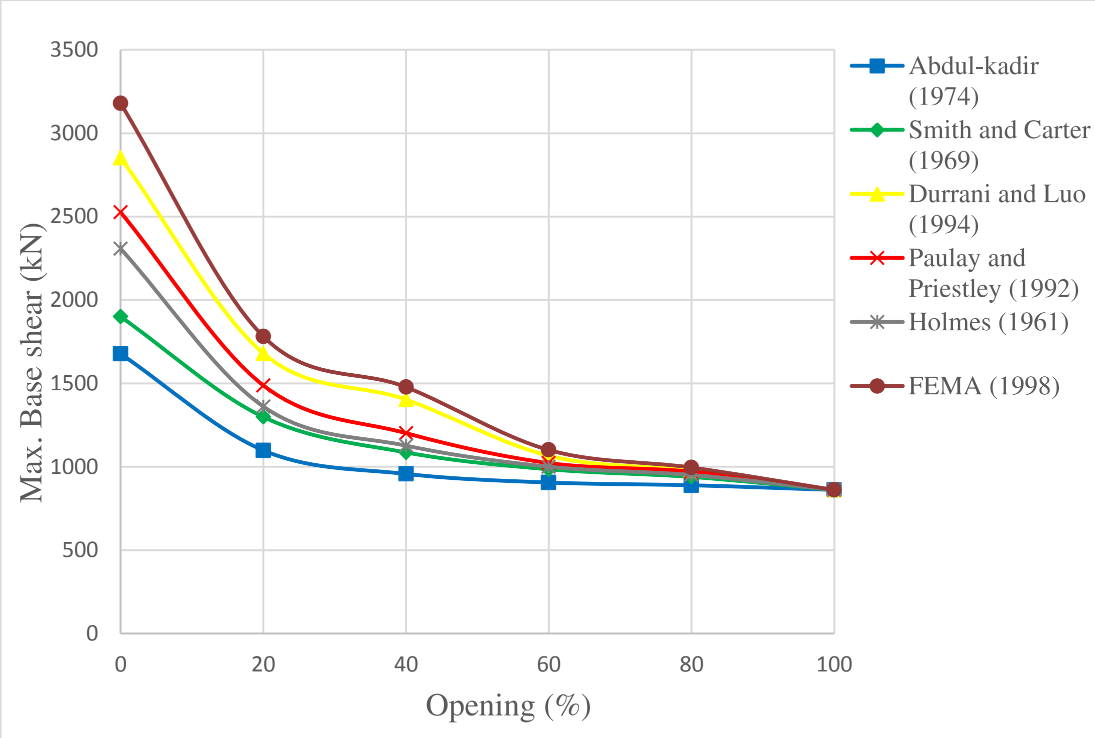


Figure 4.36 Variation of max. base shear with opening of 6-storey frame according to the formulation by different researchers

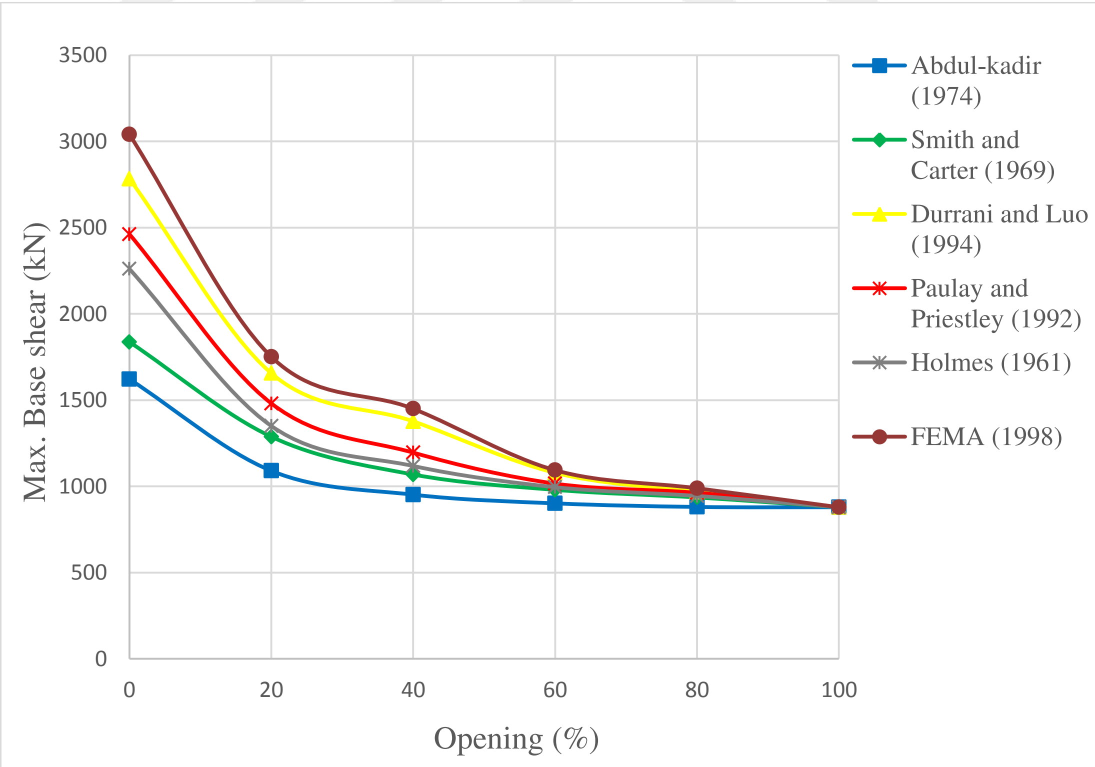


Figure 4.37 Variation of max. base shear with opening of 8-storey frame according to the formulation by different researchers

4.5 Comparison of Initial Stiffness

Stiffness is the rigidity of an object the extent to which it resists deformation from applied load. The initial stiffness is defined by the slope of the force–displacement curve for first 5 cycles (Essa et al., 2014). Although the infill walls considerably improve both the strength and stiffness of the building, their contribution is usually not taken in consideration due to the deficiency of knowledge of the composite behavior of the structure and the infill walls (Asteris, 2003). It is well known since long time that masonry walls affect the stiffness and strength of infilled structure buildings. In seismic regions, neglecting the interaction between infill wall and frame is not always secure, since, underneath lateral loads, the infill walls greatly enhance the stiffness by acting as a compressed diagonal strut, thereby, potential alteration in seismic demand because of the considerable decrease in the natural period of the composite behavior of buildings (Asteris, 2012).

Window and door openings estimated both the maximum strength and initial stiffness of the infill walled frames (Al-chaar et al., 2003). Thus, a reduction factor, λ, is proposed (Eqn. (3.2)) in order to take in consideration the influence of central openings on the rigidity of infilled frames in design considerations (Asteris, 2012).

The influence of opening on the initial lateral stiffness of infilled structures must be ignored if the opening area is less than 5% of the total area of the infill wall, and the reduction factor of strut width have to be set equal to one i.e. the structure can be analyzed as a solid infilled structure. The influence of infills on the initial lateral stiffness of infilled structure may be neglected if the opening area exceeds 40% of the total area of the infill wall, and the reduction factor of strut width have to be set to zero, i.e. the structure can be analyzed as a bare structure (Mondal and Jain, 2008).

The initial stiffness of frame buildings with and without infill walls was studies. Infill walled frames with equivalent strut based on different formulations have produced different values of initial stiffness. The thinner strut given by FEMA (1998) formula has characterized by the lowest initial stiffness while the wider strut given by Abdul-kadir (1974) formula had the highest value of initial stiffness. Figure 4.38 shows the variations of initial stiffness of 2-storey infilled frames according to different formulations. From this figure, it can be said that the initial stiffness is proportional to the width of equivalent strut, i.e. as the strut width increase the initial stiffness increase too. In order to investigate the effect of central opening of various sizes on the initial stiffness of infilled frames, multi-storey infilled frames have analyzed under nonlinear static pushover analysis. In Figure 4.38, the reduction of initial stiffness versus the opening percentages of 20, 40, 60, 80, and 100% has been plotted. From this figure, it can be directly concluded that masonry infill walls have a great contribution to the initial stiffness of RC frames. However, the presence of openings could negatively change that contribution. When the opening percentage exceeded 40%, the initial stiffness value of an infilled frame was found to be very close to that of the bare frame. Also, set openings with percentage of 20-40% in an infill panel resulted in decreasing the initial stiffness by 50-70%.

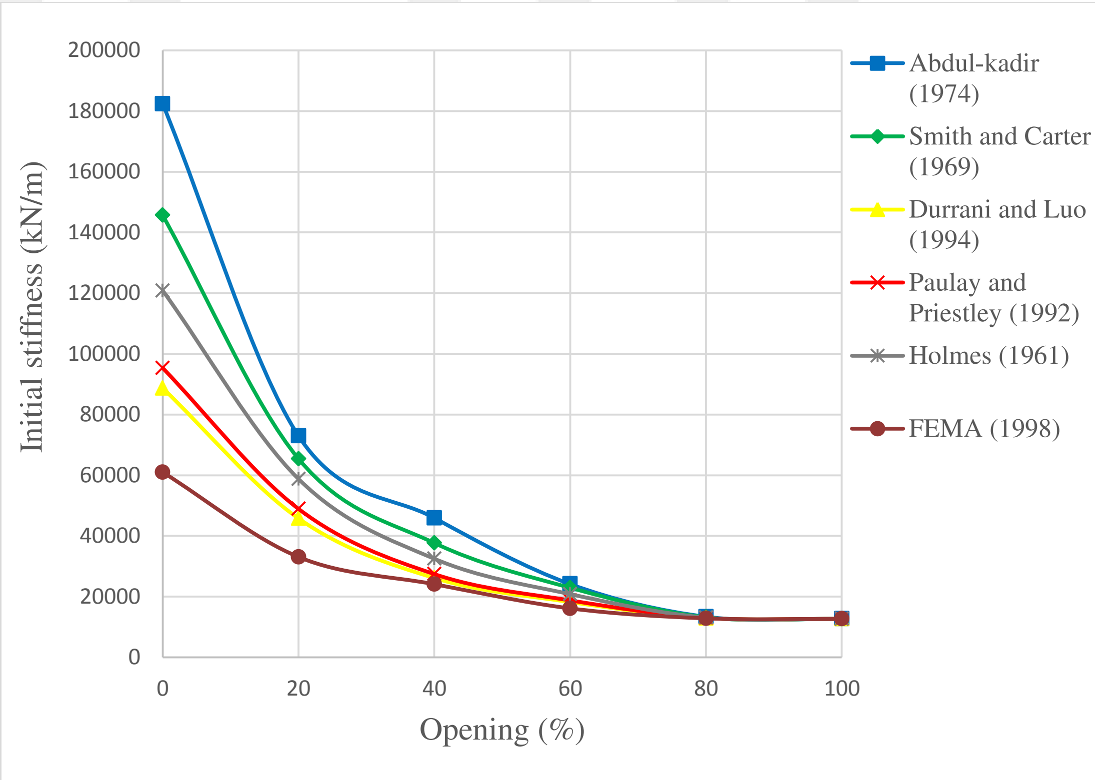


Figure 4.38 Variation of initial stiffness with opening of 2-storey frame according to the formulation by different researchers

The height of building can also affect the initial stiffness of an infilled frame with and without openings. The initial stiffness decreased with the increase of the building height. As seen in Figures 4.39 to 4.41, the initial stiffness of infilled frames with different opening sizes decreased as the height of building increased because the case study of tall building is more ductile than short building.

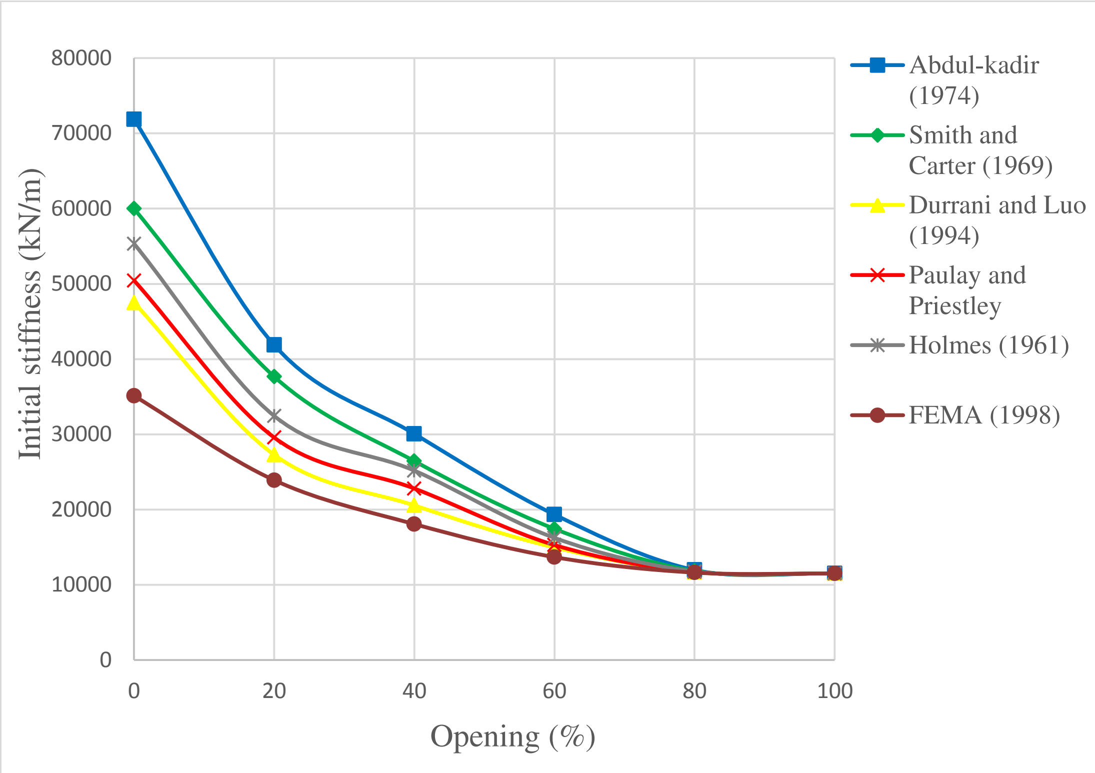


Figure 4.39 Variation of initial stiffness with opening of 4-storey frame according to the formulation by different researchers

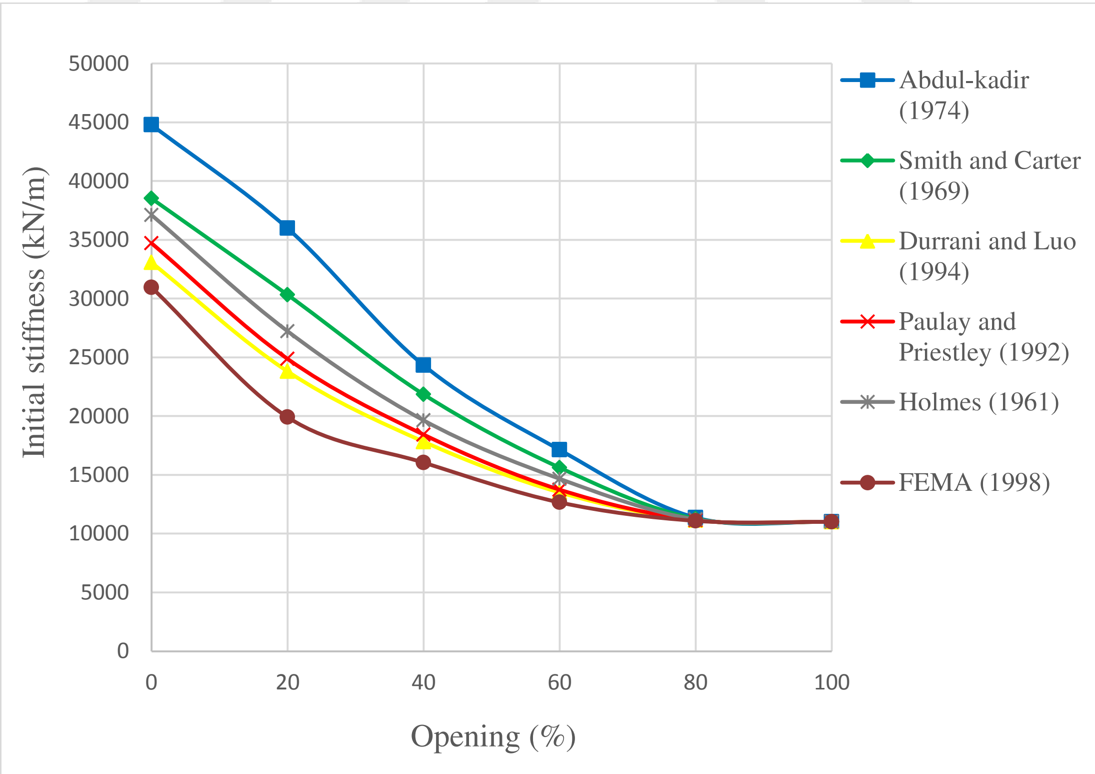


Figure 4.40 Variation of initial stiffness with opening of 6-storey frame according to the formulation by different researchers

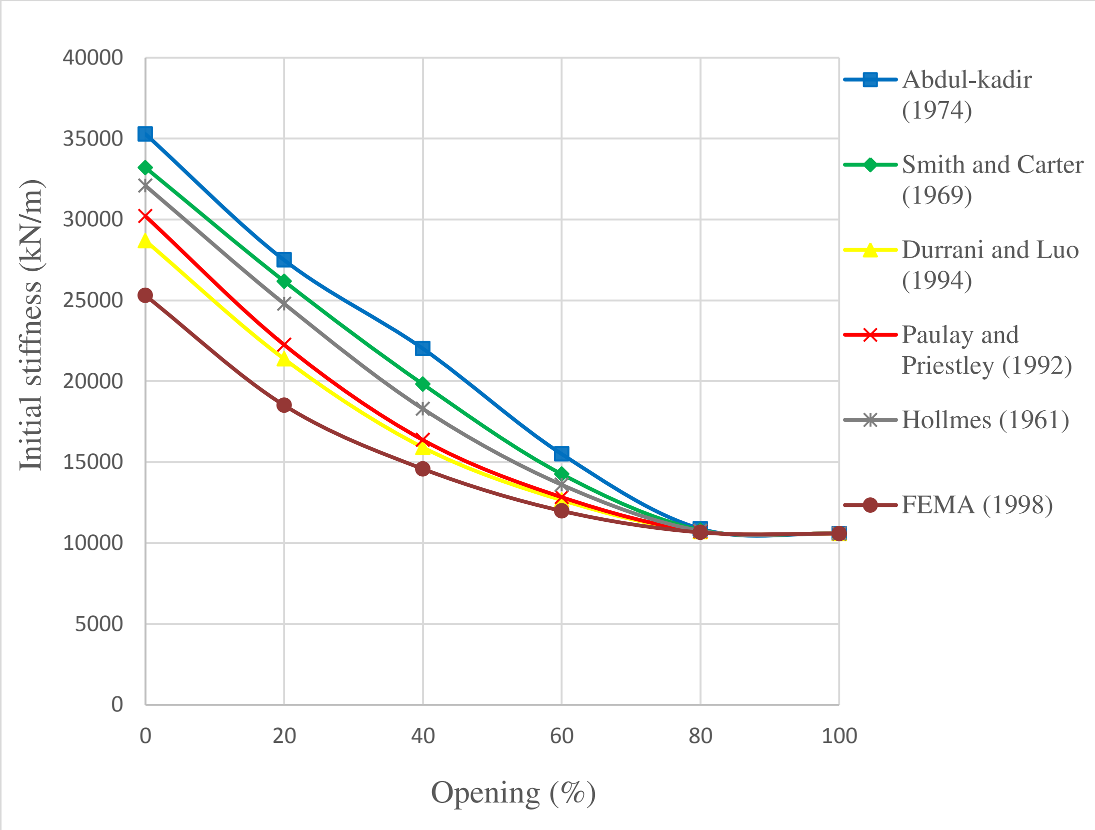


Figure 4.41 Variation of initial stiffness with opening of 8-storey frame according to the formulation by different researchers

4.6 Comparison of Energy Dissipation Capacity

A pushover curve represents the relation between base shear and roof displacement as it is achieved by pushover analysis, step by step, and is used to characterize the ability of a structure to dissipate energy, when subjected to an earthquake (Kotanidis and Doudoumis, 2008). Instead of depending on the displacement at any position, absorbed energy (or external work) is employed to characterize the structure resistance to lateral forces (Hernandez-Montes et al., 2004). Energy-based nonlinear static analysis has arose because of some troubles noticed on the performance of displacement-controlled or force-controlled nonlinear static pushover analysis. For example, it has been noticed that a force-based approach ignores the energy dissipation of the structure and the re-distribution of forces in the structure whilst displacement-control approach may disregard any strain locations at weak zones in the structure (Coleman and Spacone, 2001; Albanesi et al., 2002).

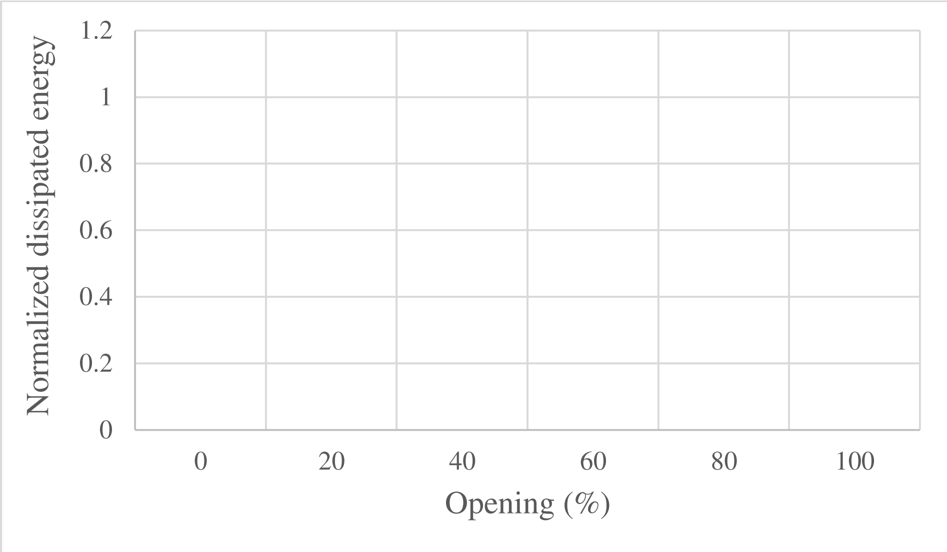
The energy dissipation is considered one of the most important aspects in studying the behavior of frame under seismic loads. A ductility behavior is preferable than the rigid one because it implies the ability of a structure to sustain large deformation without failure. Energy dissipation during loading is the area enclosed by the hysteresis loops of the load–displacement relationship (Essa et al., 2014).

Infills, if exist in all storeys, produce a considerable contribution to the energy dissipation capability by reducing considerably the ultimate displacements. Thus, the contribution of masonry infill walls is of great significance, even though highly depending on the properties of the ground shake, particularly for structures which have been designed without accounting the seismic forces (Rajesh et al., 2014).

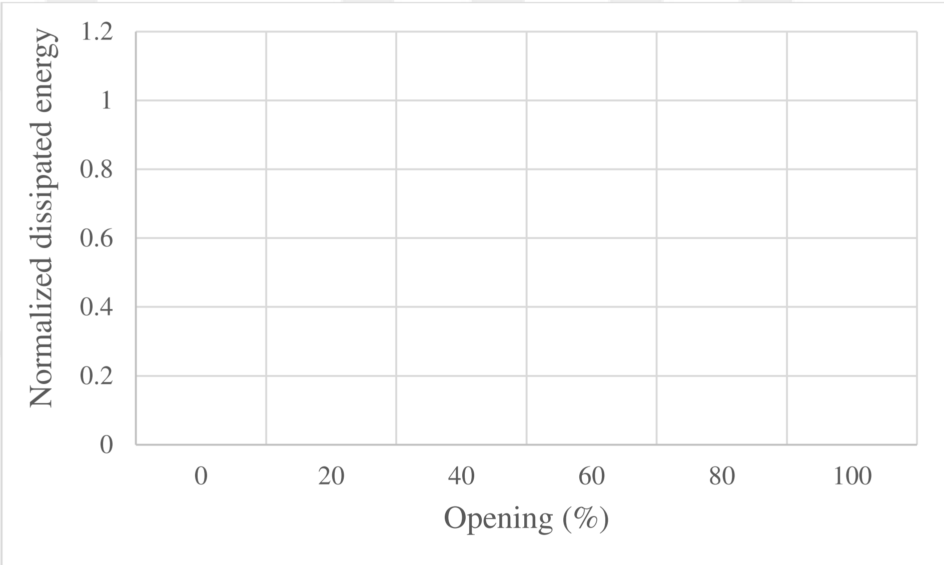
The energy dissipated capacity values of the test frames are defined by calculating the area under the lateral load-displacement curve. Thus, the highest energy dissipation is obtained by an infilled frame modeled with the thinner strut because it provided the maximum base shear capacity. Whereas the lowest energy dissipation is obtained by an infilled frame modeled with the wider strut, because the wider strut had provided the minimum base shear capacity. Being the base shear capacity decreased as the building height increased; therefore, the energy dissipated by a frame was tended to decrease as the number of frame storeys increase. In the current study, for the comparison purposes, normalized dissipated energy (NDE) index was proposed in order to plot energy dissipated ratios versus the opening percentages. NDE is defined as:

$$\text{NDE} = \frac{\text{energy dissipated by a frame infilled with walls having openings}}{\text{energy dissipated by a frame infilled with solid walls}}$$

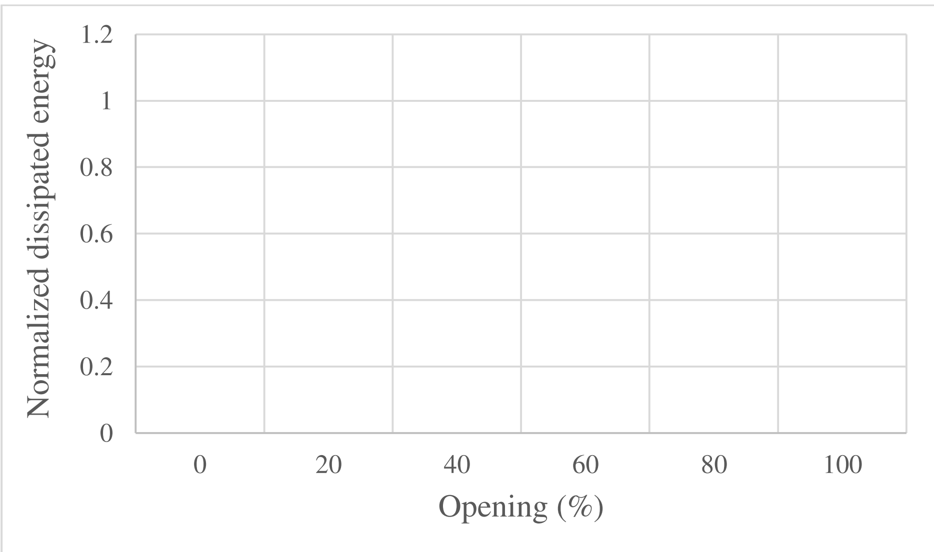
Figures 4.42 to 4.45 show the dissipated energy ratios versus the opening percentages in masonry infills for all frame models. From these figures, it can be concluded that the energy dissipated by an infilled frame decrease gradually with the increase of opening size. The minimum energy dissipated can be shown in case of a bare frame and the maximum energy dissipated is provided by a frame infilled with solid masonry walls because the masonry infill walls possess high strength and stiffness and hence can absorb high energy before failure.



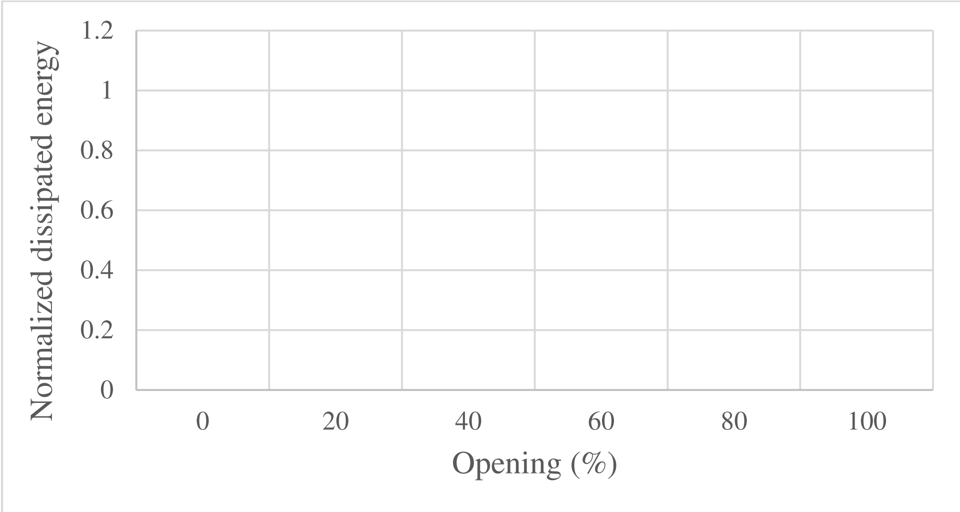
(a) Abdul-kadir (1974)



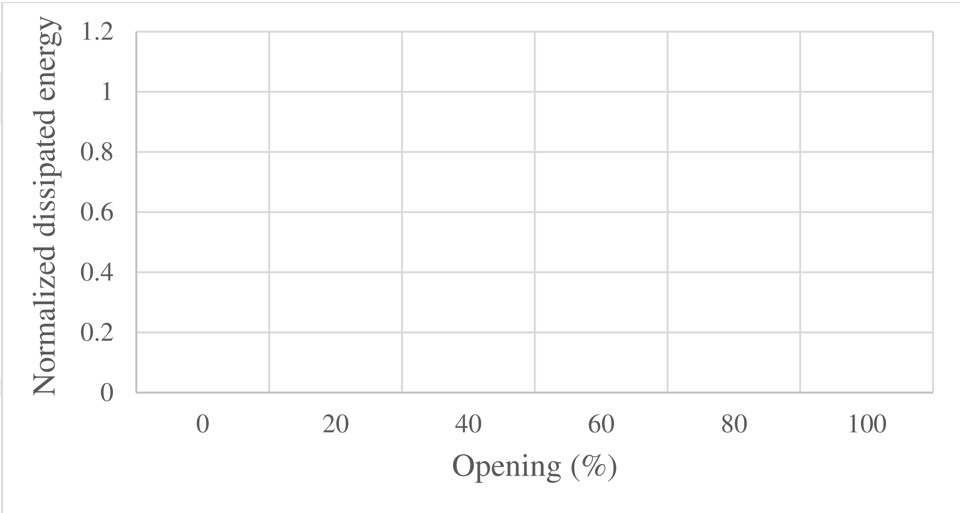
(b) Smith and Carter (1969)



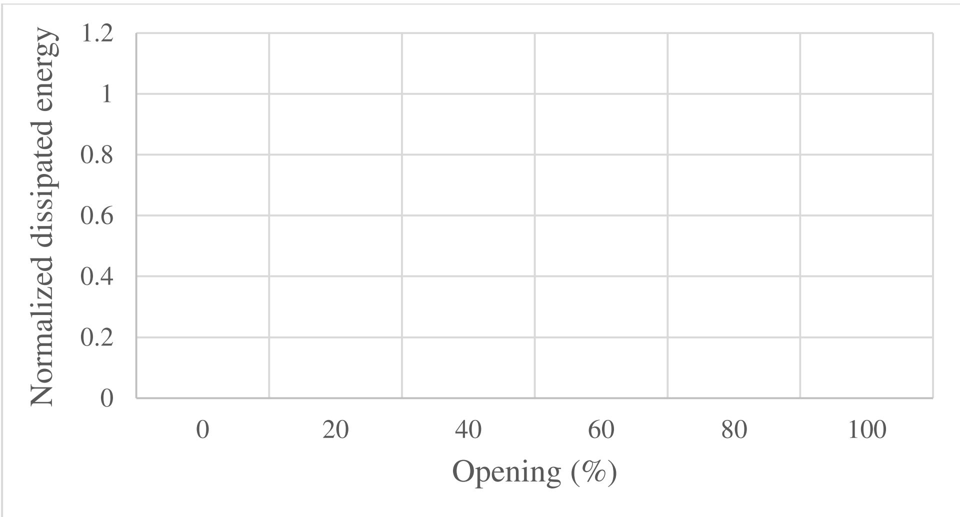
(c) Durrani and Luo (1994)



(d) Paulay and Priestley (1992)

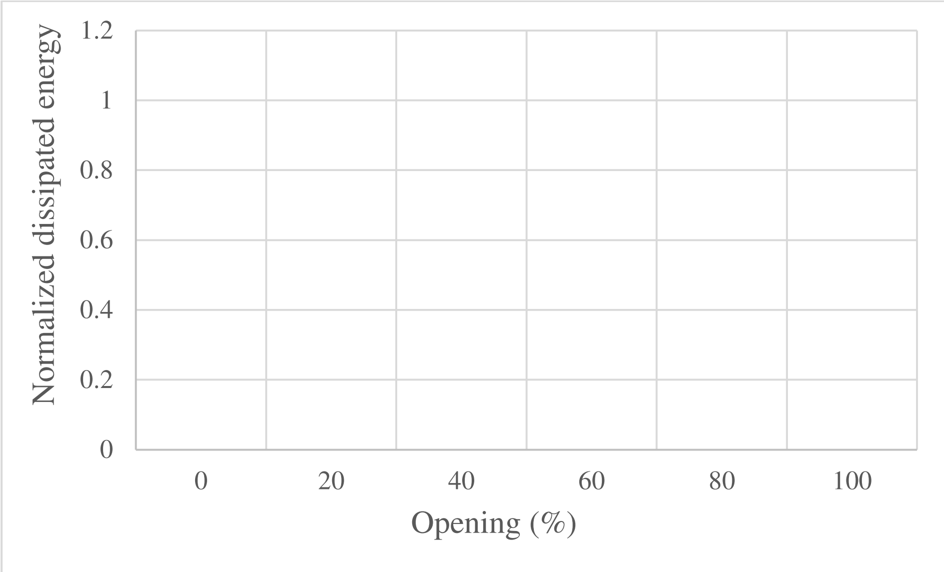


(e) Holmes (1961)

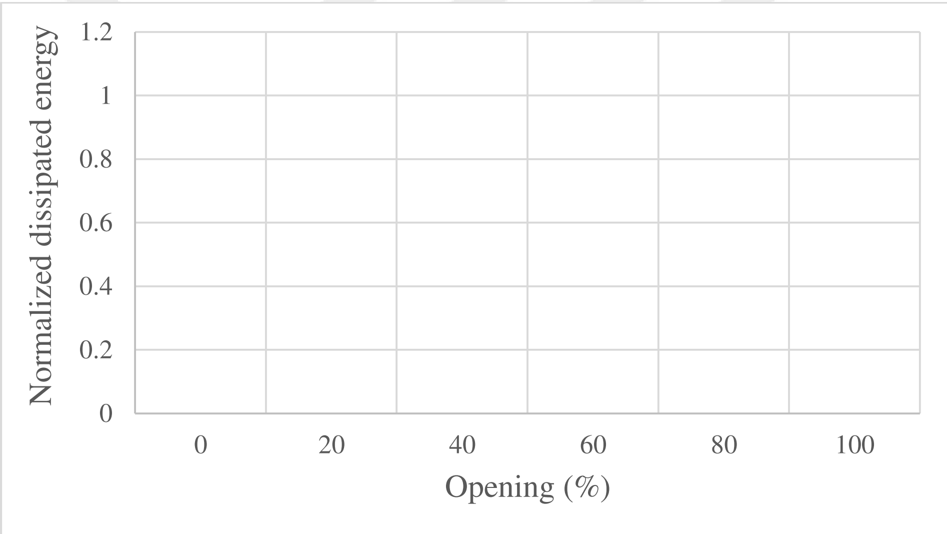


(f) FEMA (1998)

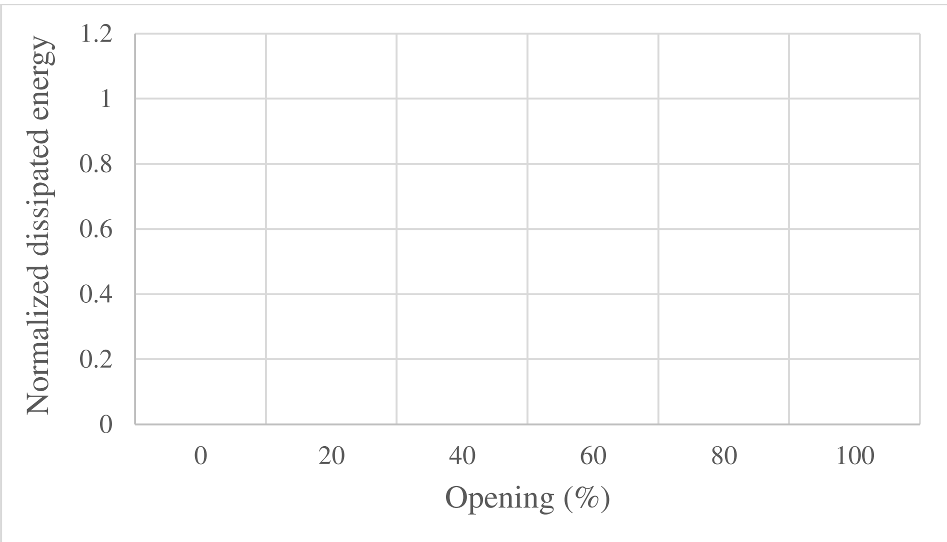
Figure 4.42 Variation of energy dissipated by the building with opening percentage of infill wall for 2-storey cases



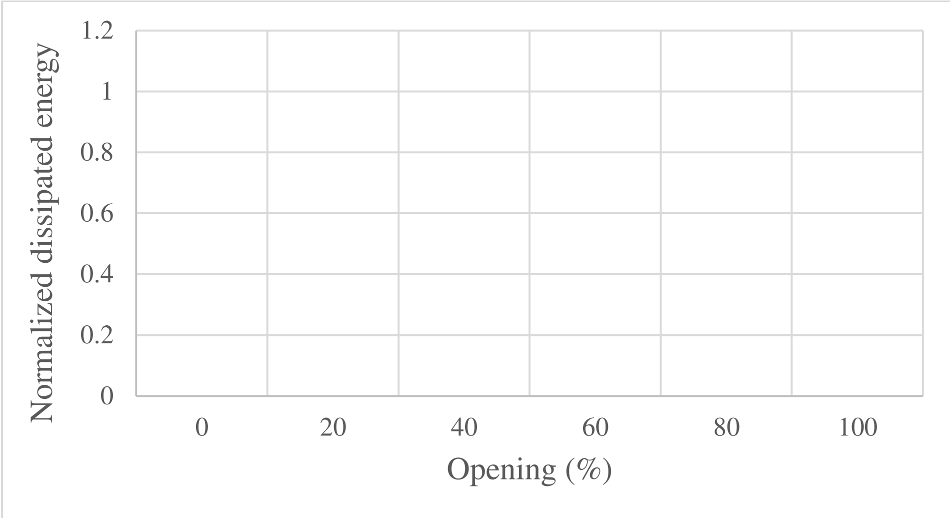
(a) Abdul-kadir (1974)



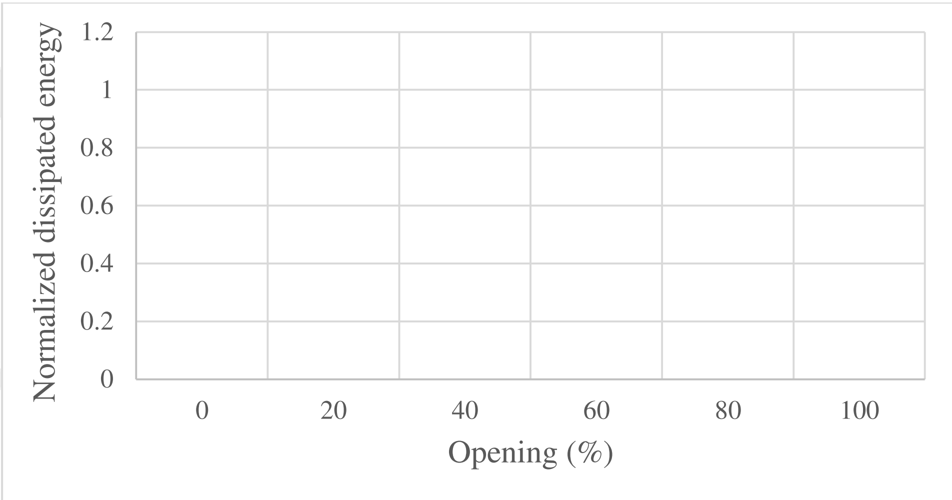
(b) Smith and Carter (1969)



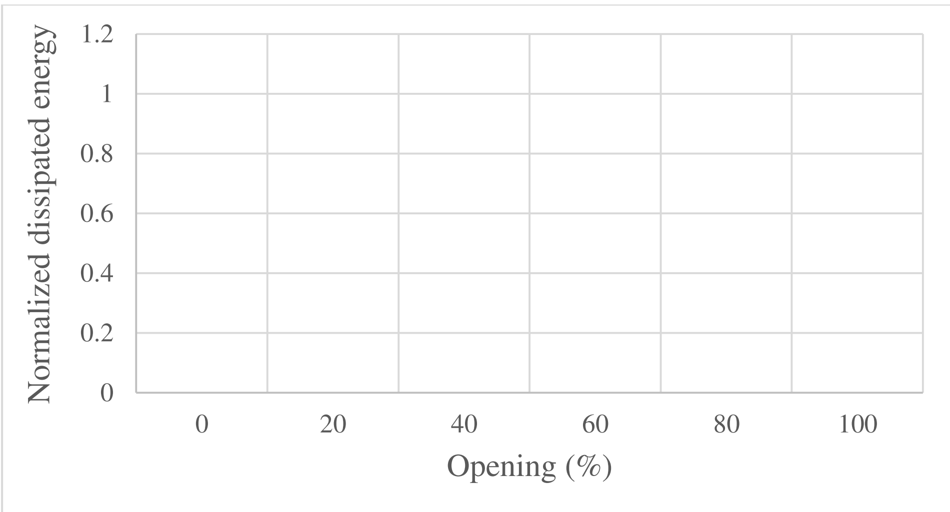
(c) Durrani and Luo (1994)



(d) Paulay and Priestley (1992)

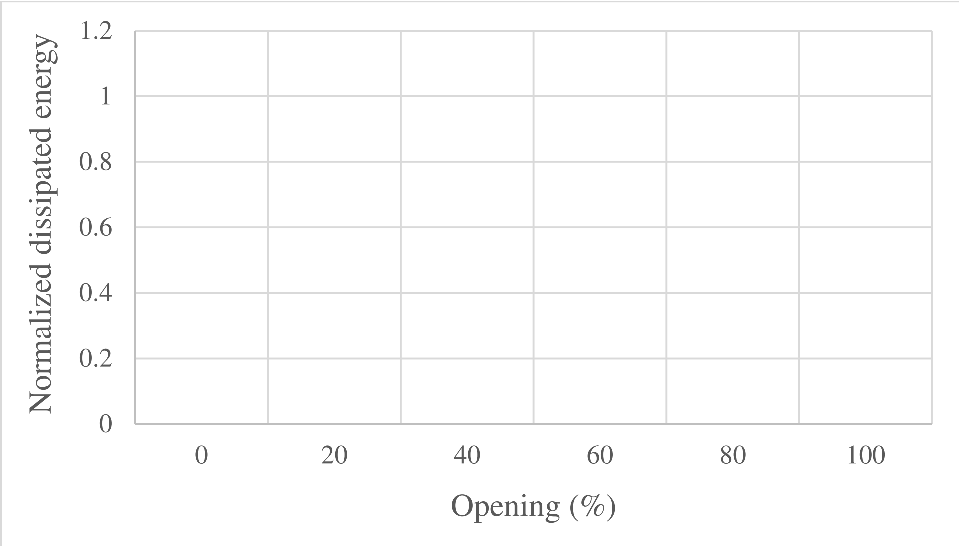


(e) Holmes (1961)

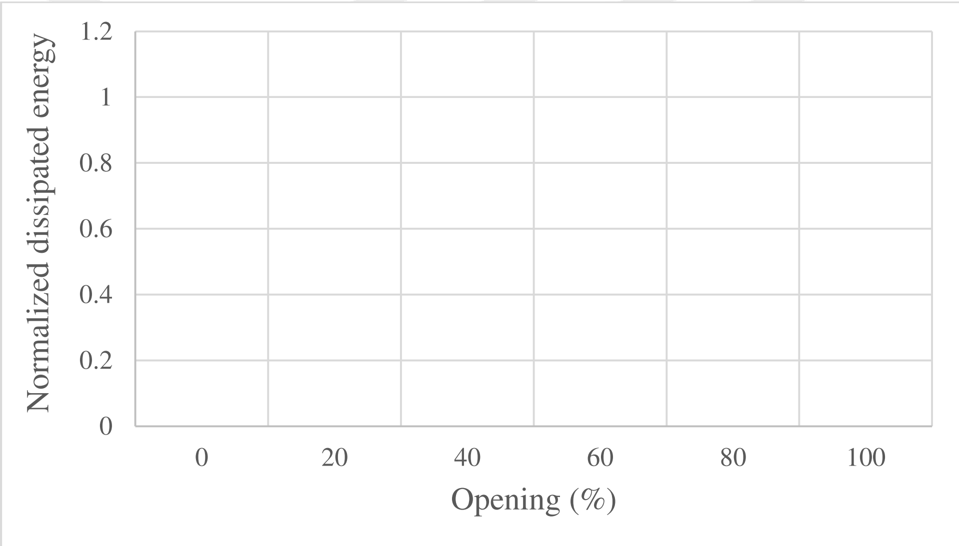


(f) FEMA (1998)

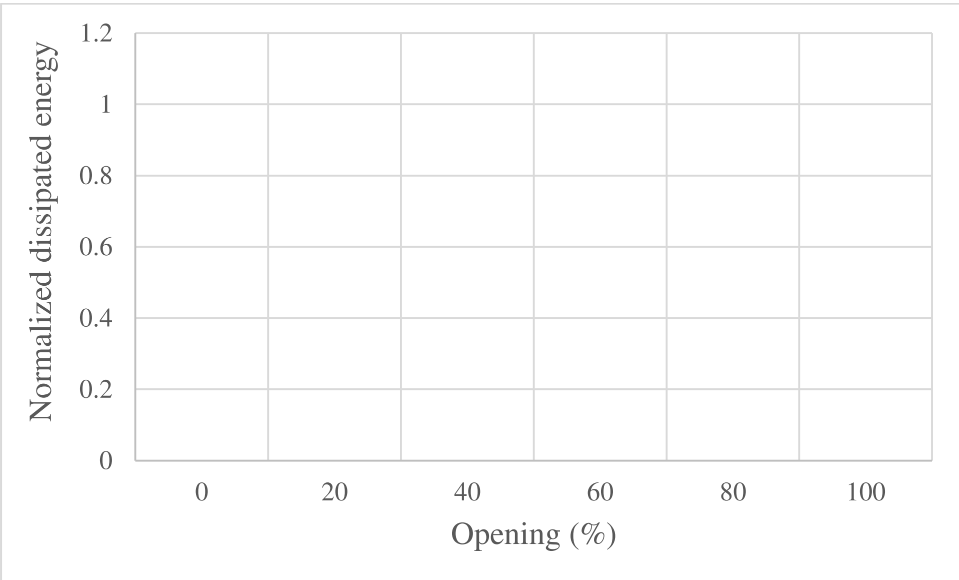
Figure 4.43 Variation of energy dissipated by the building with opening percentage of infill wall for 4-storey cases



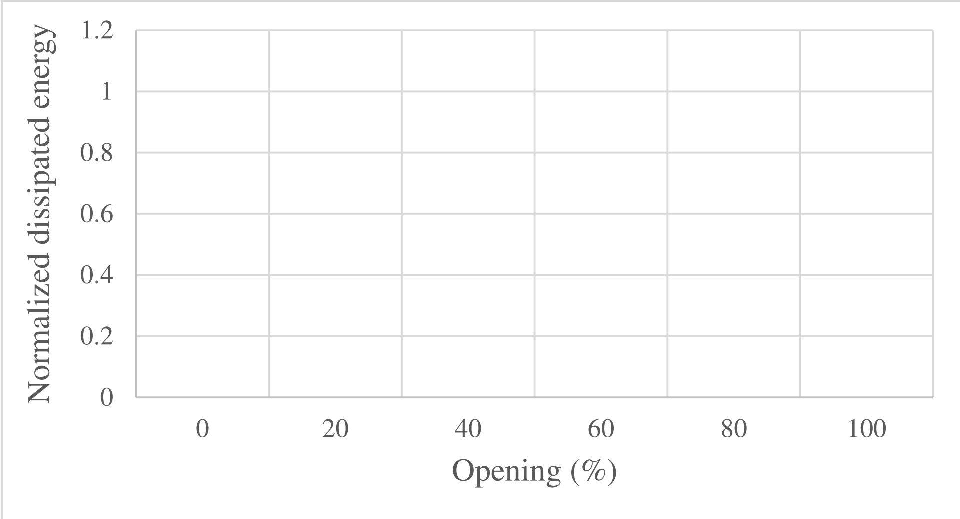
(a) Abdul-kadir (1974)



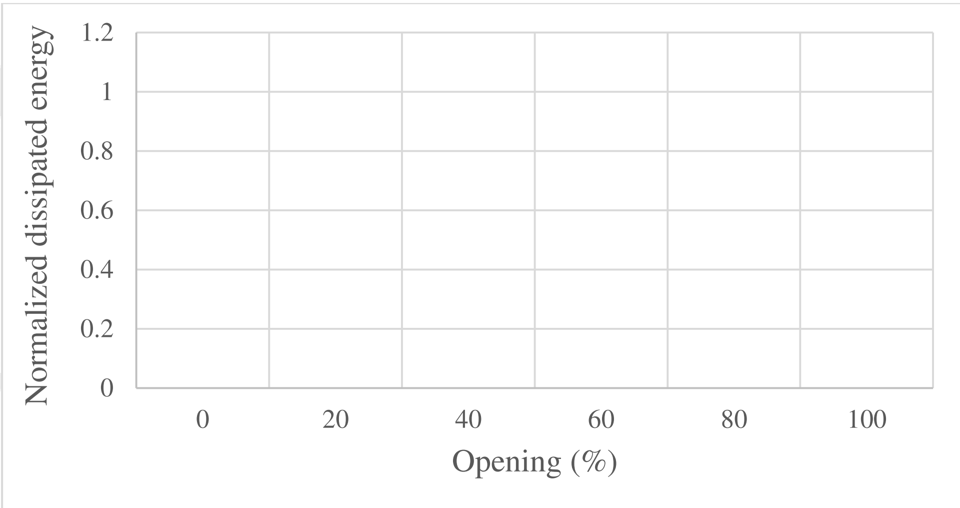
(b) Smith and Carter (1969)



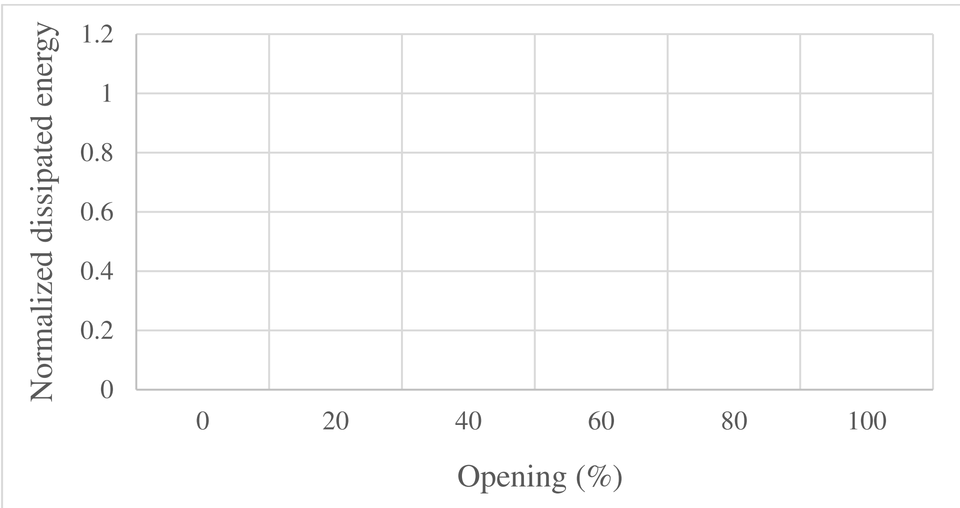
(c) Durrani and Luo (1994)



(d) Paulay and Priestley (1992)

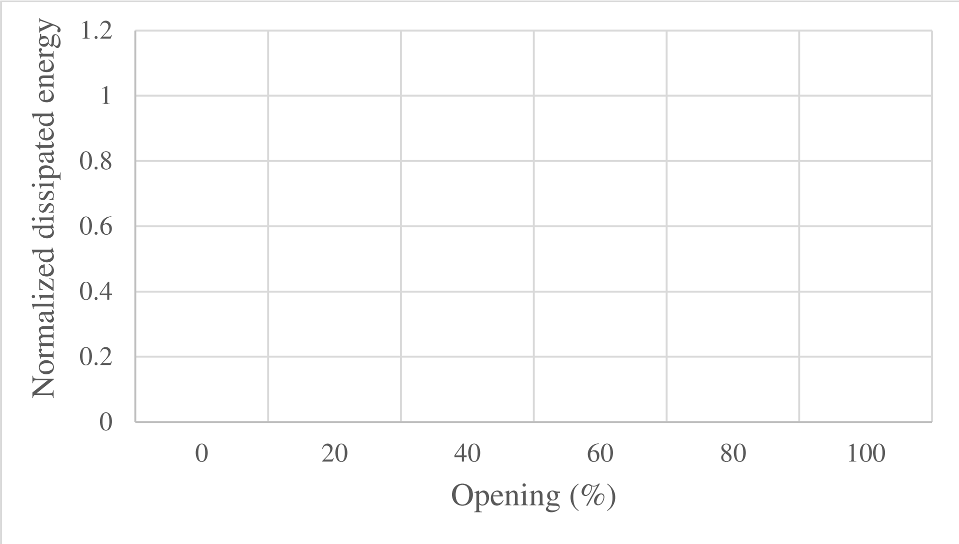


(e) Holmes (1961)

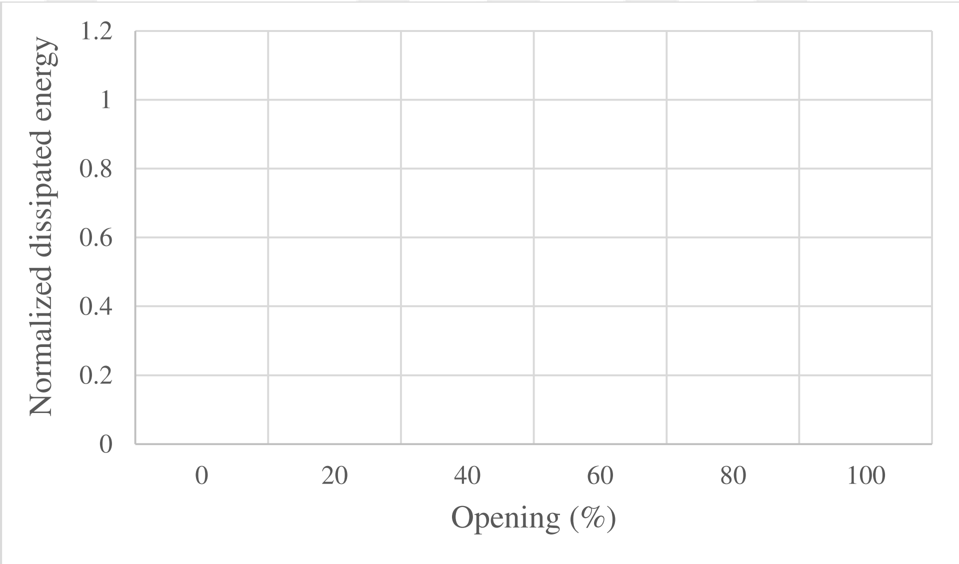


(f) FEMA (1998)

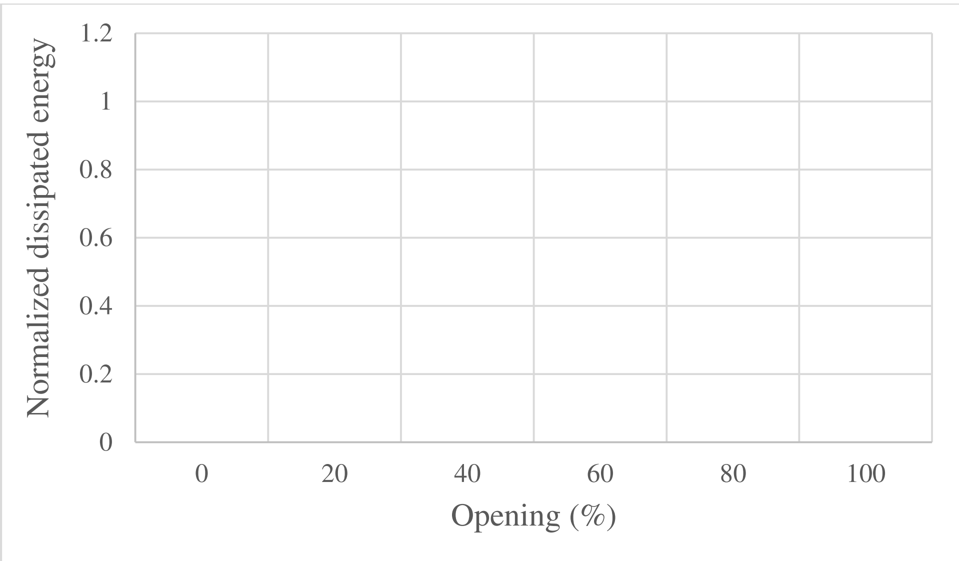
Figure 4.44 Variation of energy dissipated by the building with opening percentage of infill wall for 6-storey cases



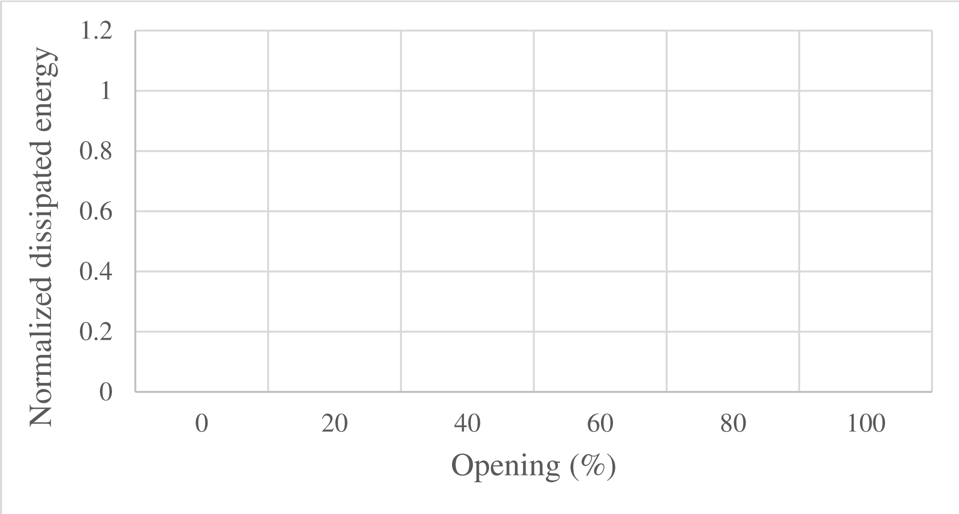
(a) Abdul-kadir (1974)



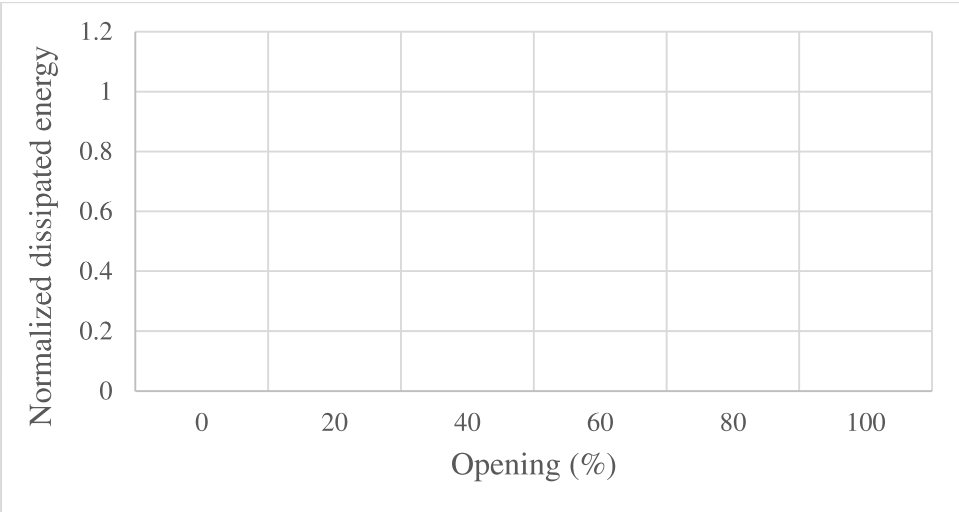
(b) Smith and Carter (1969)



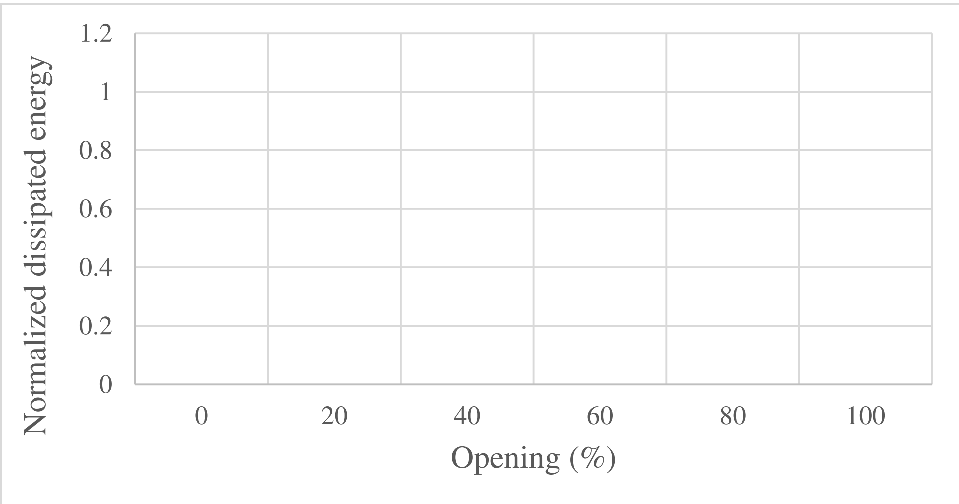
(c) Durrani and Luo (1994)



(d) Paulay and Priestley (1992)



(e) Holmes (1961)



(f) FEMA (1998)

Figure 4.45 Variation of energy dissipated by the building with opening percentage of infill wall for 8-storey cases

CHAPTER 5

CONCLUSIONS

In the present study, macro-modeling was adopted to simulate infill walls with specified mechanical properties by using SAP2000 program. As reportedly, the equivalent diagonal strut concept is a convenient way and used to model the infill panels. The crucial issue in this method is how to calculate the diagonal strut width. In literature, there are several formulations proposed to find out the strut width. Some of those formulations were implemented in this study. The variations between the formulations were investigated and discussed. Also, the effect of openings in infill walls on reinforced concrete (RC) frame behavior was studied in detail. For this, multi-storey with four-bay of RC frames with different opening percentages were studied. The central opening was formed at different percentages up to 100%. The nonlinear pushover analysis was performed for all case study structures. From the analysis results, it can be concluded the following:

- The inclusion of infill walls were observed to be significantly change the response of building structures to lateral forces. The infills also provided more resistance against the lateral deflection of the frame and thus, the structure globally became stronger.
- The results of the analysis showed that for a given dimension and mechanical property of infill wall, the wider diagonal compression strut had lower strength than the strut of smaller width. Also, the frame with smaller width of compression strut had more ductile behavior than the wider strut.
- For a given diagonal strut width, the strength and stiffness of the infilled structures increased in comparison to the bare structures, irrespective of number of frame storeys. The energy dissipated by the structure improved considerably with the contribution of infills.
- It was also pointed out that if infills were considered in the seismic analysis, the plastic hinges were observed at the different distribution and damage level of the system as compared to those of bare frame.

- The lateral load-carrying capacity of the RC frame was determined by the fact that the smallest one was obtained by a bare frame and the largest one was achieved by a full-infill walled frame.
- As the opening percentage in the infill wall increased, the lateral load-carrying capacity of the frame decreased with respect to the full-infill walled frame. The smallest value of the capacity was obtained in case of 100% opening (bare frame). When central opening occupied about 20-40% of the infill area, the maximum base shear was reduced by about 40-60%.
- As the opening percentage in the infill wall increased, the initial stiffness of the frame decreased with respect to the full-infill walled frame. The smallest value of the initial stiffness was obtained in case of 100% opening (bare frame) while the full-infill walled frame acquired the largest value. When central opening occupied about 20-40% of the infill area, initial lateral stiffness was reduced by about 50-70%.
- The largest energy dissipation capacity was obtained by a full-infill walled frame while the smallest one was measured for a bare frame. Generally, the energy dissipation capacity decreased as the opening percentage in the infill wall increased.
- It was noted that the openings in the infill walls are required to take into consideration in the building models in order to reflect the actual building behavior since the amount of the openings alters significantly the structural response.

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