ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIROMENTAL ENGINEERING

Effects of Masonry Infills on Reinforced Concrete frame Buildings

A thesis submitted to the School of Graduate Studies of Addis Ababa Institute of Technology in partial fulfillment of the requirements for the Degree of Master of Science in Civil Engineering (Structural Engineering)

By

Liyouwork Bekele

Advisor: Adil Zekaria (Dr.–Ing.)

March 7, 2016

Addis Ababa

ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIROMENTAL ENGINEERING

Effects of Masonry Infills on Reinforced Concrete frame Buildings

By

LiyouworkBekele

March 6, 2016

Approved by Board of Examiners

Adil Zekaria (Dr. Ing.)	____________	___________________
Advisor	Signature	Date
Bedilu Habte (Dr. Ing)	_________________	__________________
External Examiner	Signature	Date
Girma Zerayohannes(Dr. Ing)	_________________	_______________
Internal Examiner	Signature	Date
Dr. Agezew Negusse	_________________	__________________
Chairman	Signature	Date

ACKNOWLEDGEMENT

This research would not have been successful without the help, support and guidance rendered by many people who were directly or indirectly involved in this work. I would like to express my sincere gratitude and appreciation to my thesis advisor Dr. Adil Zekaria for his invaluable guidance, encouragement and insight provided throughout the research period. I am sincere thankful to lecturer Mesgun Samuel, in the department of civil and environmental engineering, for his guidance in development of the model. Thanks are due to Alemayehu Bekele for helping me during searching the data used in thesis work done.

I would like to express my gratitude to the Addis Ababa University for the scholarship offer and for selecting me to this study. I am also greatly indebted to Beles Consulting P.L.C for their support by letting me use office tools unlimitedly.

I am indebted to Mulugeta Abera who reviewed and edited my paper. His comments improved my work.

I would like to thank my family for their unwavering patience, and understanding and also for having applauded my success, supported my goals, and accepted my failures, making me a better person and supporting me continuously. I would like to take this opportunity to thank my friends Meseret Feleke, Yonas Alemu, Fethanegest Kasaye, and Selamawit Dege for encouraging me during tough times.

TABLE OF CONTENT

List of table

List of Figures

ABSTRACT

In Ethiopia, most of the reinforced concrete buildings use hollow masonry infill walls as non-structural partition walls. Since they are used as a non-structural member, during design stage, their contribution to overall building behavior is not well known. Observations made after the earthquakes revealed that these non-structural elements had beneficial effects on the lateral capacity of the building.

In this study, the contribution of the hollow masonry infill walls to the lateral behavior of reinforced concrete buildings was investigated. For this purpose, G+4, G+7 and G+10 buildings were chosen as case studies. These buildings are modeled as bare frame, base infilled frame, Soft ground story frame, frame with halfof the wall reduced from base infilledframe and frame with 75% of the wall reduced from base infilled frame. The parameters that were considered are Strut width and arrangement of masonry infill walls throughout the buildings. To determine the effect of each parameter, fundamental period, base shear, lateral displacement, story shear and member forces are computed and are compared for each case.

KEYWORDS:EARTHQUAKE, MASONRY INFILL WALL, STRUT WIDTH

CHAPTER ONE

1 INTRODUCTION

1.1 Back Ground

The recent trend of building construction in urban and semi-urban area of the world is reinforced concrete frames. The vertical space created by reinforced concrete (RC) beams and columns are usually filled in by walls referred to as Masonry infill wall or panels. The walls are usually of burnt clay bricks or hollow concrete block (HCB) in cement mortar. These walls are built after the frame is constructed and used as cladding or as partition. Typically, 10, 15 and 20 cm thick infill are used. Due to functional demand, openings for doors, windows etc. are rather a norm than an exception in these walls.

One of the main reasons in using masonry infill is economy and ease of construction, because it uses locally available material and labor skill. Moreover, it has a good sound and heat insulation and waterproofing properties, resulting in greater comfort for the occupants. This type of construction is frequently used in earthquake prone areas.

1.2 STATEMENT OF THE PROBLEM

The present practice of structural analysis is to treat the masonry infill as non-structural element and the analysis as well as design is carried out by only using the mass but neglecting the strength and stiffness contribution of infill. Therefore, the entire lateral load is assumed to be resisted by the frame only. One of the disadvantages of neglecting the effect of infill is that, the building can have both horizontal as well as vertical irregularities due to uncertain position of infill. Also, the infill walls are sometimes rearranged to suit the changing functional needs of occupants. The changes are carried out without considering their adverse effects on the overall structural behavior. The conventional finite element modeling of RC structures without considering the effect of infill in the analytical model renders the structures more flexible than they actually are. Since infills are not considered in conventional modeling in seismic design, their contributions to the lateral stiffness and strength may invalidate the analysis and proportioning of structural members for seismic resistance on the basis of its results. In reality, the additional stiffness contributed by these secondary components increases the overall stiffness of the buildings, which eventually leads to shorter time periods, as they are observed during earthquakes; and hence attracts larger seismic force to the structure. Since early 50's there have been numerous experimental as well as analytical researches to understand the influence of infill on the lateral strength and stiffness of frame structure. Past earthquakes have shown that buildings with regular masonry infill have a better response than with the irregular ones. Also, masonry infills have a very high initial stiffness and low deformability [Moghaddam and Dowling 1987][28]thus, making infill wall a constituent part of a structural system. This changes the lateral load transfer mechanism of the framed structure form predominant frame action to predominant truss action [Murthy and Jain 2000][29], which is responsible for reduction in bending moments and increase in axial forces in the frame members. The presence of infill also increases damping of the structures due to the propagation of cracks with increasing lateral drift. However, behavior of masonry infill is difficult to predict because of significant variations in material properties and failure modes that are brittle in nature. If not carefully placed, during seismic excitation, the infills also have some adverse effects. One of the major ill effects is the soft story effect. This is due to absence of infill wall in a particular story.

The absence of infill in some portion of a building plan will induce torsional moment. Also, the partially infilled wall, if not properly placed may induce short column effect thus creating localizedstress concentration. In generally the designer tends to ignore the stiffness and strength of infill in the design process and treat the infill as non-structural elements. This is mainly due to lack of generally accepted seismic design methodology in the Building Codes that incorporates structural effects of infill. In fact very few codes in the world currently provide specifications for the same. Hence, there is a clear need to develop a strong design methodology for seismic design of masonry infill Reinforced Concrete structure.

1.3 OBJECTIVES

Generally, this study aims to investigate the effect of masonry infill wall on a reinforced concrete moment resisting frame conventionally designed as a bare frame, using available macro-model proposed by Pauley and Priestley (1992).

The specific objectives of the study are:

1. To study the effects of infilled wall on reinforced concrete frame subjected to earthquake induced lateral load.

2. To investigate the effects of different schemes for the infilled wall arrangement on the response of the building.

3. To show the soundness of reduced natural period caused by the addition of infills given by the code

1.4 RESEARCH METHODOLOGY

1.4.1 LITERATURE REVIEW

Journals and articles on the effect of masonry or concrete infill on steel or reinforced concrete moment resisting frame were reviewed to familiarize with the theoretical part. In addition; books and relevant design codes were studied. The purpose of literature review was to gain firsthand knowledge on the methods of studies adopted, which could be used as a guideline for this study. The review of past studies would also provide some idea of the modeling techniques and parameters to be used for different materials like reinforced concrete, hollow concrete and brick masonry.

1.4.2 DATA COLLECTION

The study was done with the prevalent construction materials being used in Ethiopia. Thus, the required material data necessary to make the analytical model of the hollow concrete (HCB) masonry infill were collected from the building code of Ethiopia (EBCS).

1.4.3 METHODOLOGY ADOPTED

As discussed earlier, the present practice of structural analysis is to treat the masonry infill as non-structural element and the analysis as well as design is carried out by using only the mass but neglecting the strength and stiffness contribution of infill. Thus, the structure is modeled as bare frame, and usually considered fixed at base. In Ethiopia, structure is analyzed for seismic loading as per EBCS: 1995 Seismic Design of Buildings but in this paper Eurocode: 2004 is adopted. The frame structure has moment resisting joints. The beams and columns are modeled as a frame element which has the capability to deform axially, in shear, in bending and in torsion. The weight of RC slab is distributed as rectangular load to the surrounding beams as per EBCS 2: 1995. Asemi-rigid joint diaphragm action to resist lateral force is taken into account.

For the present study, a 5, 8 and 11 story office type buildings with a floor plan as shown in Figure 1.1 was considered.

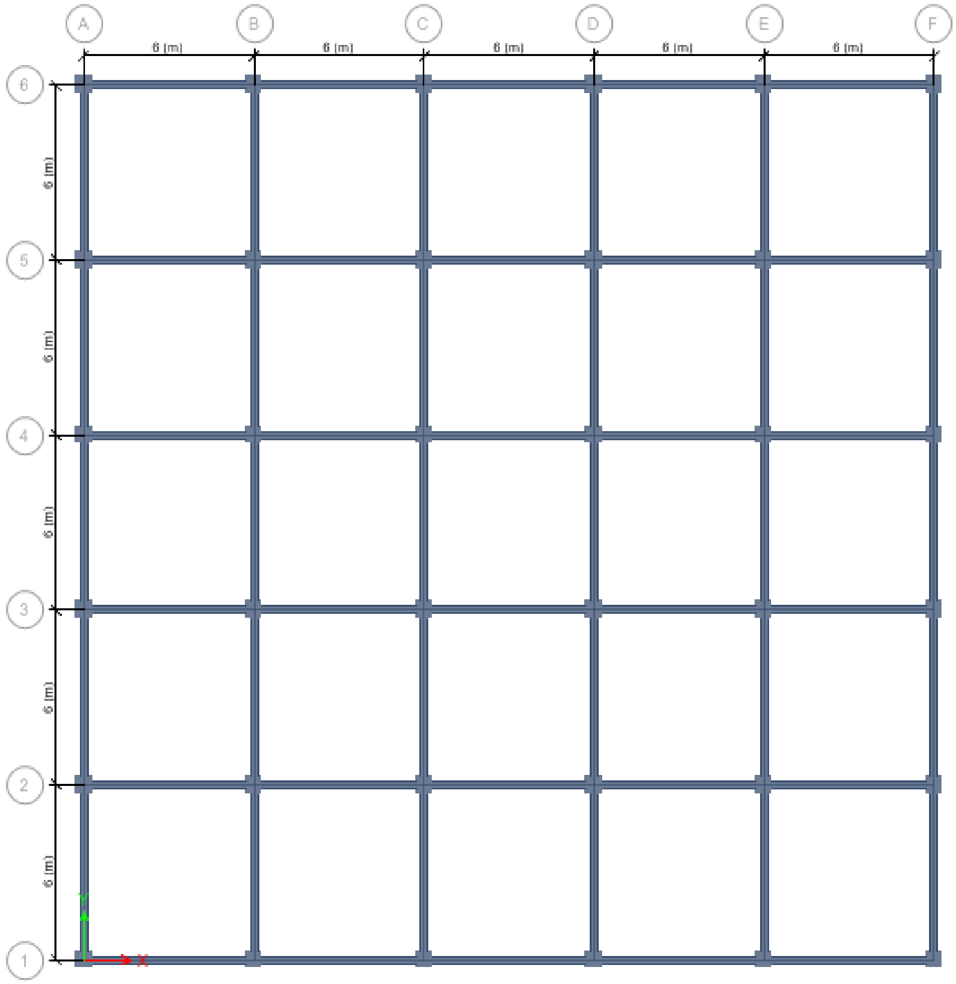


Fig 1.1. Building Floor plan

1.5 SCOPE

The thesis work is based on the Eurocode. Hence, the new Ethiopian Building Code Standard (EBCS) is similar with it. The present study is concerned only with the macro models of infill panels because these models are convenient for practicing engineers due to their simplicity.

1. This study only deals with the reinforced concrete moment resisting frame with unreinforcedhollow concrete block HCB masonry infill wall which is neither integral nor bonding with the surrounding frame.

2. The study is based on a 5, 8 and 11story office type building frames with typical floor loading and infill thickness of 20cm and 15cm in cement sand mortar ratio 1:3.

3. Linear staticanalysis is carried out. The comparisons are made forfundamental period, base shear, story displacement, story shear, and member forces.

4. And this study only deals with the in-plane stiffness of masonry infill

5. All models that are developed to determine the effect of masonry infill wall on seismic performance of the building were created in commercial programs ETABS.

1.6 APPLICATION OF RESULTS

This thesis is going to present how amasonry infill affect the lateral load resistance of reinforced concrete buildings in particular for G+4, G+7 and G+10 buildings. Furthermore, this thesis will serve as a reference guide for practicing Civil Engineers and Researchers that practice in the area of study. And will initiate the designers to consider the effect of masonry infills in designing buildings. This is useful in the sense that, it will add our knowledge on the effect of masonry infills in RC buildings in the area covered in this paper.

1.7 ORGANIZATION OF THESIS

This introductory chapter (Chapter 1) gives a brief introduction to the importance of the seismic evaluation of masonry infilled buildings and the reason why theytreat the masonry infill as non-structural element and the analysis as well as design is carried out by using only the mass but neglecting the strength and stiffness contribution of infill. The objectives, statement of the problem, scope and application of the result of the proposed research work are identified along with the methodology that is followed to carry out the work. Chapter 2 provides detailed review ofa literature survey on the effect of masonry infilled buildings during earthquake, have been presented. Chapter 3 presents the description of the selected building and the structural modelling parameters and modelling of infill walls. Chapter 4 presents results and discussion obtained from linear analyses of the buildings model considering various cases. Finally, in Chapter 5, the conclusions and Recommendation are given.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 GENERAL

Reinforced concrete frames with Masonry infills are a popular form of construction of high-rise buildings in urban and semi urban areas around the world. The term infilled frame is used to denote a composite structure formed by the combination of a moment resisting plane frame and infill walls. The masonry can be of brick, concrete units, or stones. Usually the RC frame is filled with hollow concrete block (HCB) as non-structural wall for partition of rooms. Social and functional needs for vehicle parking, shops, reception etc… are compelling to provide an open first story in high rise building. Parking floor has become an unavoidable feature for most of urban multistoried buildings. Though multistoried buildings with parking floor (soft story) are vulnerable to collapse due to earthquake loads, their construction is still widespread. These buildings are generally designed as framed structures without regard to structural action of masonry infill walls. They are considered as non-structural elements. Due to this in seismic action, RC frames purely acts as moment resisting frames leading to variation in expected structural response. In reality the presence of infill wall changes the behavior of frame action into truss action thus changing the lateral load transfer mechanism [Mulgund G. V.1][20].

Various construction forms include from un-mortared stacked stone blocks, where resistance to lateral forces is provided solely by gravitational load and friction, to carefully mortared and reinforced masonry walls designed for ductile response under seismic attack [Mekonnen Degefa][19].

2.2 PERFORMANCE OF MASONRY INFILL IN RC FRAME STRUCTURES

The weakness in structures is exposed by earthquake event. An earthquake force is a very peculiar forceandbehaves quite differently than other types of loads, such as gravity and wind loads. It strikes the weakest spot in the whole three dimensional building. This should be an eye opener for designers and builders. Due to ignorance in design and poor quality of constructions, results many weaknesses in the structure that cause serious damages to life and property. Masonry infill are used to fill the gap between the vertical and horizontal resisting elements of building frames, assuming that these infill will not take part in resisting any kind of either axial or lateral load. Hence, its significance in the analysis of frames is generally neglected. In fact, an infill wall considerably enhances the rigidity and strength of the frame structure. It has been observed through various researches, that the frame considering no infill has comparatively lesser stiffness and strength than the infill frame and therefore their ignorance cause failure of many multi-story buildings when subjected to seismic loads.

As recent studies have shown a properly designed infilled frames can be superior to a bare frame in terms of stiffness, strength and energy dissipation. From structural point of view, the composite action between infill panels and frames give more lateral resistance and in-plane stiffness. As a result, total and inter story drift is reduced. In non-linear range, infill acts as a good damper by dissipating energy through cracking. Subsequent to cracking of infill, the center of stiffness gets shifted towards the stiffer portion of the building and the eccentricity between the center of stiffness and the center of mass get increased, thus, torsion dominates the structural behavior of the building and extra shear stress get induced in frame elements. It is also been observed that structure below plinth are normally assumed to perform like a soft story with

loose soil material filled after excavation. To lay down the column foundation for the structure the material adjoining the column and footing is excavated and refilled after completion of foundation work. The frame thus formed above the footing level and up to the ground level is infilled with loosely filled material and fails to give similar effect of infill masonry and acts like a soft basement. The removal of in-fill leads to more ductility demand in the open ground story and structure below plinth. All the inelasticity gets concentrated in the open ground story and structure below plinth and it can damage severely [Mohd Danish. et.al][3].
Past studies also carried out on the behavior of R.C frame with in-fills and the modeling & analysis of the R.C frame with and without in-fills.

Smith used an elastic theory to propose the effective width of the equivalent strut and concluded that this width should be a function of the stiffness of the in-fill with respect to that of bounding frame. By analogy to a beam on elastic foundation, he defined the dimensionless relative parameters to determinethe degree of frame in-fill interaction and thereby, the effective width of the strut.

Singh found in his research that in the dynamic analysis of a complete building system, the inclusion of the effect of in-fill is essential for a realistic prediction of the behavior; he further concluded that there is very limited literature available on dynamic response of 3-D in-filled reinforcement concrete frames.

Bell and Davidson found that a review of international research and guidelines indicate that in-fill panels, where present in a regular arrangement, have a significant beneficial influence on the behavior of RC buildings. These can give an impression that in-fill masonry panels have a detrimental influence on the behavior of buildings due to soft story effects. The reviewed sources indicate that due to stiffness, strength, and damping effects of in-fill panels, deformations are below that required for a soft story mechanism.

Das and Murty carried out non-linear pushover analysis on five RC frame buildings with brick masonry in-fills, designed for the same seismic hazard as perEurocode,Nepal Building Code and Indian and the equivalent braced frame method. In-fills are found to increase the strength and stiffness of the structure, and reduce the drift capacity and structural damage. In-fills reduce the overall structure ductility, but increase the overall strength. Building designed by the equivalent braced frame method showed better overall performance.

Amato et al.discussed the mechanical behavior of single story-single bay in-filled framesand generalized analytical proceduresavailable in the literature for the identification of a pin- jointed strut equivalent to the in-fill to take the influence of vertical loads into account. Detailed numerical investigation on in-filled meshes has proved that in the presence of vertical loads it is possible that a strong correlation between the dimension of the equivalent diagonal strut model and a single parameter, which depends on the characteristics of the system. A family of curves has obtained for different values of vertical load.

Baran and Sevi have found through various analytical and experimental studies that hollow brick in-fills could not only increased both strength and stiffness of RC frames but also adequately be modeled by diagonal compression struts.

Asteris et al. conducted quasi-static experiments on frames with masonry in-fill panels with openings that reveal important insights regarding the global as well as the local response of the tested in-fill frames. In particular, the experimental results indicate that the failure modes of the in-filled frames classified into distinct modes. Such a classification of the failure modes (crack patterns) enhances considerably the understanding of the earthquake resistant behavior of in-filled frames and leads to improved comprehension of their modeling, analysis and design.

Mohan and Prabha concluded that Equivalent Static Method can be used effectively forsymmetric buildings up to 35m height. For higher and unsymmetrical buildings, response spectrum method shall use. For important structures, time history analysis shall performed as it predicts the structural response more accurately in comparison with other two methods since it incorporates P-Δ effects and material non-linearity, which is true in real structures. Therefore, the presence of in-fill influence the behavior of moment resisting frame and the characteristic configuration of the in-fill panels can alter the predominant mode of structural action particularly when the frames subjected to lateral loads [Mohd Danish. et.al][3].

2.3 INFLUENCE OF MASONRY INFILL ON SEISMIC BEHAVIOR OF FRAMES

It is a common misconception that masonry infill in structural steel or reinforced concrete frames can only increase the overall lateral load capacity, and therefore must always be beneficial to seismic performance. In fact there are numerous examples of earthquake damage, some of which are can be traced to structural modification of the basic frame by so called non-structural masonry partitions and infill panels. Even if they are relatively weak, masonry infill can drastically alter the intended structural response, attracting forces to parts of the structure that have not been designed to resist them. Two examples are illustrated below to examine this behavior.

Consider the floor plan of a symmetrical multistory reinforced concrete frame building with masonry infill panels on two boundary walls, as shown in Fig 2.1.

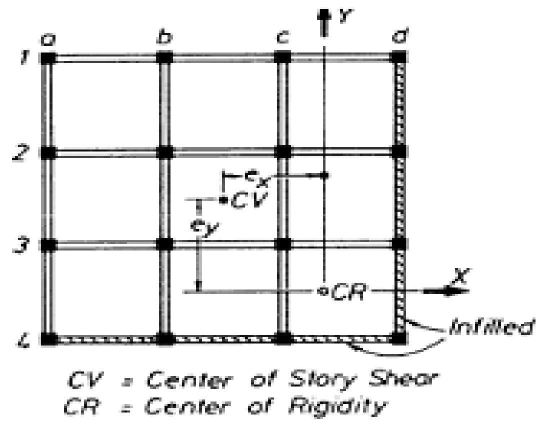


Figure 2.1. Floor plan of multistory reinforced concrete frame building with infill of two boundary frames. [Pauley and Priestley][1]

If the masonry infill is ignored in the design phase, it may be assumed that each frame in each direction (i.e., frames1, 2, 3 and 4 in the x direction, and frames a, b, c and d in the y direction) is subjected to very similar seismic lateral forces, because of the structural symmetry. The true influence of the infill on frames 4 and d will be to stiffen these frames relative to the other frames. The consequence will be that the natural period of the structure will decrease, and seismic force will correspondingly increase. Further, the proportion of the total seismic shear transmitted by the infilled frames will increase because of the increased stiffness of these frames relative to the other frames. The structure will also be subjected to seismic torsional response because of the shift in the center of rigidity. Thus for seismic response along the x and y axes, respectively, the torsional moments will be proportional to $M_{tx}=V_je_y$ and $M_{ty}=V_je_x$, respectively, where V_j is the total horizontal story shear and e_x and e_y are the eccentricities shown in fig 2.1.

The high shear forces generated in the infilled frames are transmitted primarily by shear stresses in the panels. Shear failure commonly results, with shedding of masonry into streets below, or into stairwells, with great hazard to life.

A second example is illustrated infig.2.2, which shows masonry infill that extends for only part of the story height, to allow for windows. Again the infill will stiffen the frame, reducing the natural period and increasing seismic forces. If the frame is designed for ductile response to the design-level earthquake, without consideration of the effect of the infill, plastic hinges might be expected at the top and bottom of columns, or, preferably, in beams at the column faces. These hinges could develop at a fraction of the full design-level earthquake. The influence of the infill will be to inhibit beam hinges and stiffen the center and right column (for the direction of lateral load shown), causing plastic hinges to form at top of the column and top of the infill, as shown in fig.2.2. The consequence will be a dramatic increase in column shears.

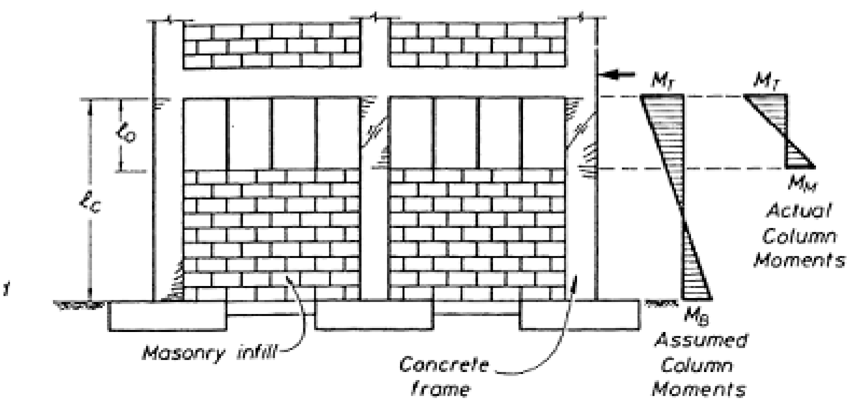


Figure 2.2. Partial masonry infill in concrete frame[Pauley and Priestley][1]

The design level of shear force in the column will be

$$V_d = \frac{M_T + M_B}{l_c} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..2.1$$

Where $l_c$is the clear story height, and $M_T$and $M_B$are moments at the top and bottom of the first-story columns. These moments should be based on capacity design principles with the base moment M_B at flexural over strength and the top moment $M_T$corresponding to flexural over strength of the beam plastic hinges, with dynamic amplification effects taken into account. However, in reality, structure incorporating partial infill, such as that shown in fig.2.2, is unlikely to have been subjected to the sophistication of capacity design, and it is more probable that M_T and $M_B$will be moments directly derived from elastic analysis under the code distribution of lateral forces.

Regardless of the design philosophy adopted, Eq.2.1 will underestimate the likely shear force, which, with the notation in fig.2.2 will be

$$V_d^* = \frac{M_T + M_M}{l_O} \dots\dots\dots\dots\dots\dots\dots 2.2$$

Where $l_O$is the height of the window opening. Eq. 1.2 corresponds to development of plastic hinges at the top of the column and at the top of the infill. If the column is not designed for the higher shear force of Eq. 2.2, shear failure can be expected. It should be noted that this higher shear force, corresponding to formation of plastic hinges, as shown, can develop because the original design was based on large ductility capacity. Hence the higher shear force will be developed, but at lower ductility demands.

When the masonry infill of the type implied in fig.2.1 is to be used, there are two design alternatives. The designer may effectively isolate the panel from frame deformations by providing a flexible strip between the frame and the panel, filled with a highly deformable material such as polystyrene. Alternatively, the designer may allow the panel and frame to be in full contact, and design both for the seismic force to which they may be subjected. The first option, of isolation, is not very effective, as it is neither possible nor desirable to provide flexibility at the base of the panel. Moreover, it is difficult to provide support against out-of-plane seismic forces.

Isolated panels must be fully reinforced to carry the out-of-plane forces, because compression membrane action, which can assist in resisting in-plane loads, as will subsequently be established, is eliminated by the flexibility of the strip between the frame and the panel. Shear connection between frame and panel through the flexible layer will need to be designed for flexibility in the plane of the infill panel, while remaining stiff and strong enough to carry the out-of-plane reactions from inertia response back into the frame.

The most effective way of providing this behavior is to lay up the infill panel before the upper beam is poured, separating the top of the panel from the beam with a layer of flexible material. Shear connection to the beam can be provided by extending the panel vertical reinforcement into the beam and taping layers of flexible material (e.g., polystyrene) to the sides of the reinforcement in the in- plane direction, up to the beam mid-height. After the beam concrete is placed, the flexible material will allow relative in-plane movement of panel and frame, while restricting out-of-plane relative movements [Pauley and Priestley][1].

2.4STRENGTH AND STIFFNESS OF MASONRY INFILL

a. IN-PLANE STIFFNESS**: -** At low levels of in-plane lateral force, the frame and infill will act in fully composite fashion, as structural wall with boundary elements. As lateral deformations increase, the behavior becomes more complex as a result of the frame attempting to deform in a flexural mode while the panel attempts to deform in a shear mode, as shown in fig. 2.3, the result is separation between frame and panel at the corners on the tension diagonal, and the development of a diagonal compression strut on the compression diagonal. Contact between frame and panel occurs for a length z, shown in fig. 2.3.

   The separation may occur at 50 to 70% of the ideal lateral shear capacity of the infill for reinforced concrete frames. After separation, the effective width of the diagonal strut, w, shown in fig.2.3, is less than that of the full panel [Paulay and Priestley][1].

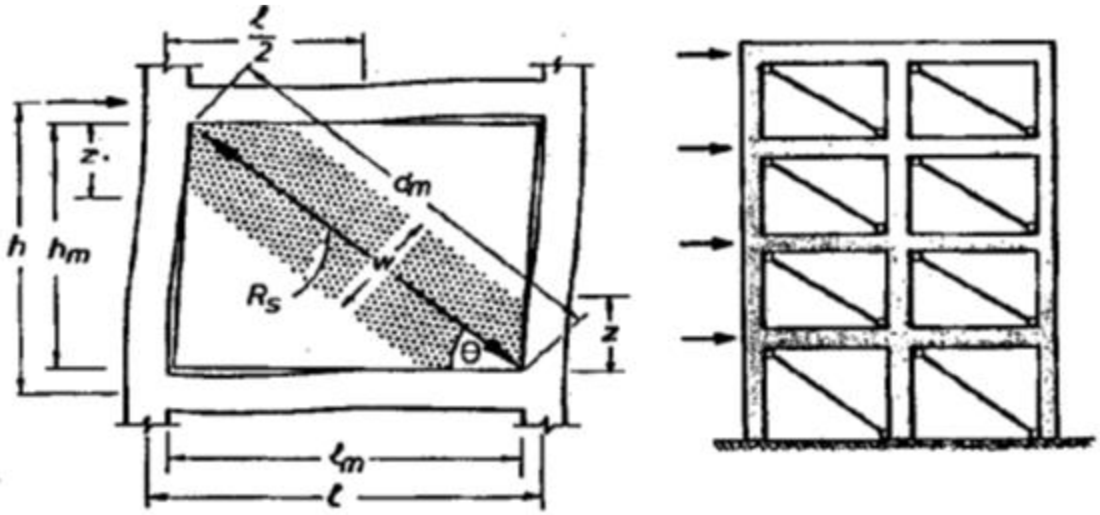


Figure.2.3 and 2.4, Equivalent bracing action of masonry infill[Paulay and Priestley][1]

Natural-period calculations should be based on the structural stiffness after separation occurs. This may be found by considering the structure as an equivalent diagonally braced frame, where the diagonal compression strut is connected by pins to the frame corners. fig.2.4 shows the equivalent system for a two-bay, four-story frame. Analytical expressions have been developed based on a beam-on-elastic foundations analogy modified by experimental results which show that the effective width w of the diagonal strut depends on the relative stiffness of frame and panel, the stress-strain curves of the materials, and the load level. However, since a high value of w will result in stiffer structure, and therefore potentially higher seismic response, it is reasonable to take a conservatively high value of

$$w = 0.25d_m \qquad 2.3$$

Where, d_m is the diagonal length. This agrees reasonably well with published charts, assuming typical masonry-infill properties and a lateral force level of 50% of the ultimate capacity of the infilled frame [Paulay and Priestley][1].

b. THE IN-PLANE STRENGTH**:-** There are several different possible failure modes for masonry infilled frame buildings:

1. TENSION FAILURE MODE**:-** For infilled frames of high aspect ratio, the critical failure mode may be flexural, involving tensile yield of the steel in the tension column, acting as a flange of the composite wall, and of any vertical steel in the tension zone of the infill panel. Under this condition the frame is acting as a cantilever wall, and a reasonably ductile failure mode can be expected [Paulay and Priestley][1].
2. CORNER CRUSHING MODE (CC MODE):- Represents crushing of the infill in at least one of its loaded corners, as shown in fig. 2.5a. This mode is usually associated with infill of weak masonry blocks surrounded by a frame with weak joints and strong members [Wael W. El-Dakhakhni1,et.al][4].
3. SLIDING SHEAR MODE (SS MODE):-Represents horizontal sliding shear failure through bed joints of a masonry infill, as shown in fig. 2.5b. Bed-joint sliding is likely to occur when the bounding frame is strong and flexible (such as steel frames) [Wael W. El-Dakhakhni1,et.al]. If the mortar beds are relatively weak compared to the adjacent masonry units (especially bricks), a plane of weakness forms, usually near the mid-height level of the infill panel. There is really no limit to the displacement capacity of this behavior mode. This mode is associated with infill of weak mortar joints and strong frame. If sliding shear failure of the masonry infill occurs, the equivalent structural mechanism from the diagonally braced pin-jointed frame of fig.2.5, to the knee-braced frame shown in fig 2.5c. the support provided by the masonry panel forces column hinges to form at approximately mid-height and top or bottom of the columns or may result in column shear failure. Initially, the entire shear will be carried by the infill panel, but as the sliding shear failure

develops, the increased displacement will induce moments and shears in the columns [Wael W. El-Dakhakhni1,et.al][4].

4. DIAGONAL COMPRESSION MODE (DC MODE):-Represents crushing of the infill within its central region, as shown in fig. 2.5c. This mode is associated with a relatively slender infill, where failure results from out-of-plane buckling instability of the infill[Wael W. El-Dakhakhni1,et.al][4].
5. DIAGONAL CRACKING MODE (DK MODE):-The form of a crack connects the two loaded corners, as shown in fig. 2.5d. This mode is associated with weak frame or frame with weak joints and strong members infilled with a rather strong infill[Wael W. El-Dakhakhni1,et.al][4].
6. FRAME FAILURE MODE (FF MODE):- The form of plastic hinges in the columns or the beam-column connection, as shown in fig. 2.5e. This mode is also associated with weak frame or frame with weak joints and strong members infilled with a rather strong infill [Wael W. El-Dakhakhni1,et.al][4].

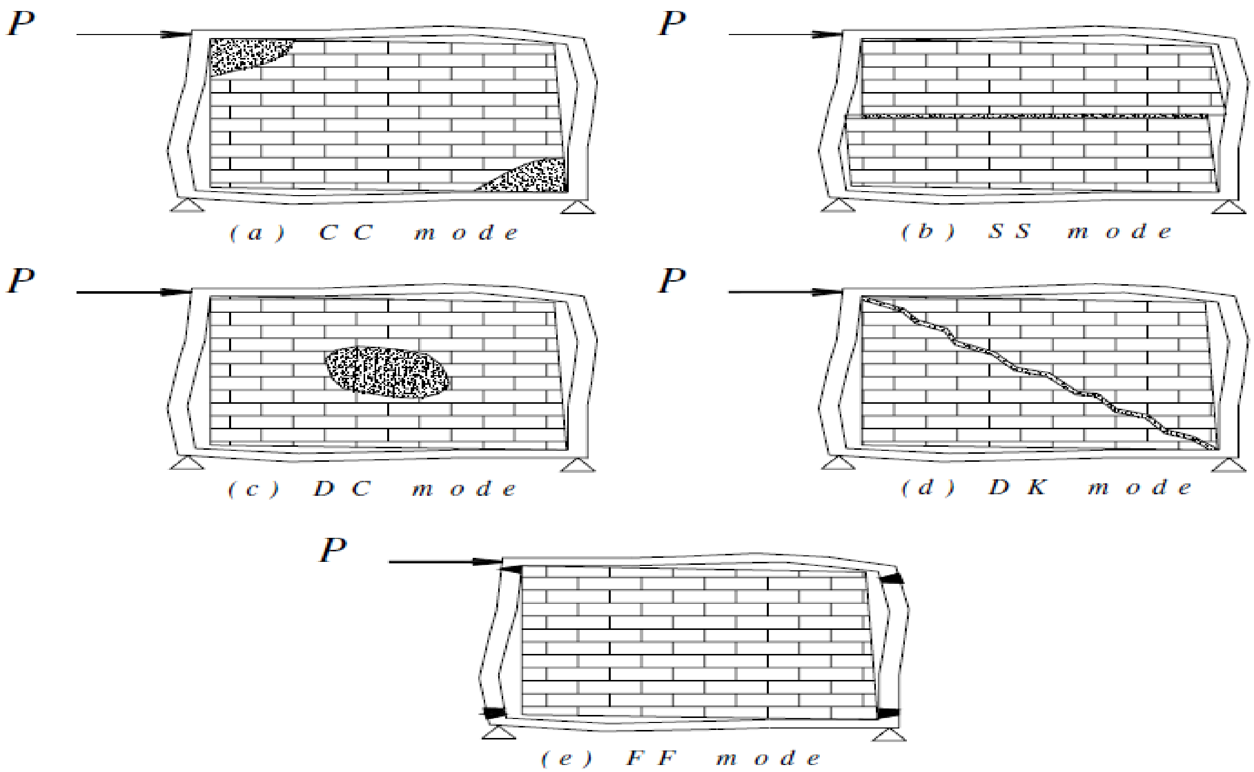


Figure. 2.5, Different failure modes of masonry infilled frames: a) Corner Crushing mode; b) Sliding Shear Mode; c) Diagonal Compression Mode; d) Diagonal Cracking Mode; and e) Frame Failure Mode.[Wael W. El-Dakhakhni1,et.al][4].

It is worth mentioning that only the first two modes, the CC and the SS modes, are of practical importance since the third mode is very rare to occur and requires a high slenderness ratio of the infill to result in out-of-plane buckling of the infill under in-plane loading. This is hardly the case when practical panel dimensions are used, and the panel thickness is designed to satisfy the acoustic isolation and fire protection requirements.

The fourth mode should not be considered a failure mode, due to the fact that the wall still carries more loads after it cracks. The fifth mode, although might be worth considering in the case of reinforced concrete frames, yet when it comes to steel frames infilled with unreinforced hollow masonry blocks, this mode hardly occurs. The study conducted herein models the CC mode only, which is the most common mode of failure. In order to determine the governing failure mode, the capacity of the infill panels obtained by the proposed method should be compared to the capacity under SS mode which may be estimated using the method suggested by Paulay and Priestley.

Subjecting a bare masonry panel to a diagonal loading usually results in a sudden failure initiated by a stepped crack along the loaded diagonal, dividing the panel into two separate parts and immediately leading to the collapse of the specimen due to lack of confinement. Unlike the unconfined panel, as soon as a diagonal crack develops within an infilled panel (usually at a much lower load and deflection levels than ultimate) the panel finds itself confined within the surrounding frame and bearing against it over contact lengths, as shown in fig. 2.6a. The contact lengths provide enough confinement to prevent failure and allowing the panel to carry more load until the existing diagonal crack continues to widen and new cracks appear leading, eventually, to ultimate failure.

To model this behavior it is rational to consider the panel to be composed of two diagonal regions, as shown in fig. 2.6. One region connects the top beam to the leeward column and the other connects the windward column to the lower beam. As reported by many researchers, (Reflak and Fajfar, Saneinejad and Hobbs, Mosalam et al, and Buonopane and White, the bending moments and shearing forces in the frame members cannot be replicated using a single diagonal strut (although has been used frequently) connecting the two loaded corners. Based on the above, it is suggested that, at least two additional off-diagonal struts located at the points of maximum field moments in the beams and the columns are required to reproduce these moments as shown in fig. 2.6b. Furthermore, since the load transfer from the frame members to the infill depends on the contact length which, in turn, is affected by the stiffness and the deflected shape of the frame members, the use of a multi-strut model will allow for the interaction between different panels in multi-story buildings. This is due to the fact that some beams (and/or columns) will be loaded from the upper and lower panels (or left and right panels) at different locations within the span (or height), which will affect their deflected shape and hence the panel's strains, and consequently changing the failure load.

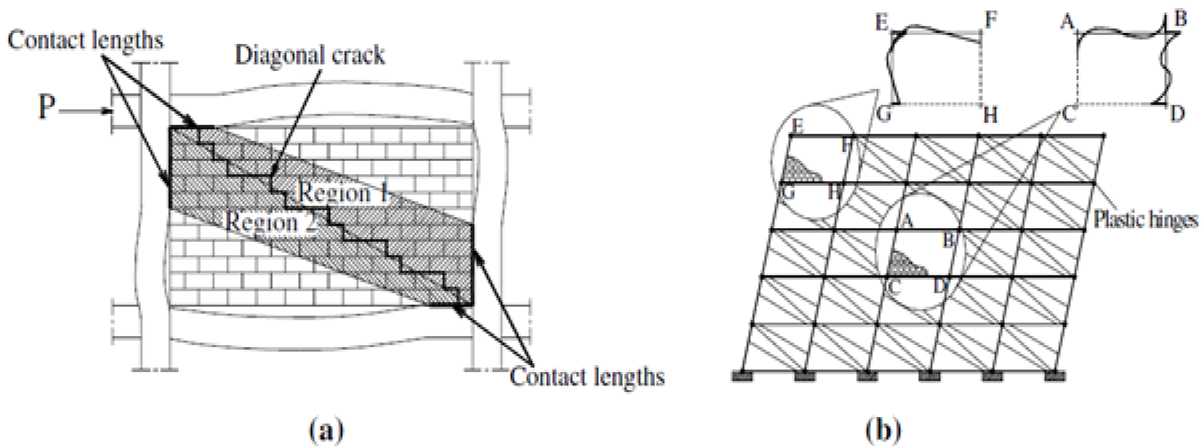


Figure 2.6, The infill panels behavior; a) Separation into two diagonal regions; and b) Resulting bending moment diagrams for a different Bays in multi-story infilled Frame Building. [Wael W. El-Dakhakhni1,et.al][4].

2.5MODELING AND ANALYSIS OF INFILLED FRAME STRUCTURES UNDER SEISMIC LOADS

Different types of analytical models based on the physical understanding of the overall behavior of an infill panels were developed over the years to mimic the behavior of infilled frames. The available infill analytical models can be broadly categorized as i) Macro Model and ii) Micro models. The Macro models are a single-strut model and the three-strut model; the Micro model is the finite element models. By analyzing the resulting forces in the beam and columns both as values and distribution, it has been observed that the three-strut model can estimate local effects more precisely due to frame infill interaction [Diana M. Samoila][24]. The single strut model is the most widely used as it is simple and evidently most suitable for large structures [Das and Murthy, 2004][27].

The main advantages of macro-modelling are computational simplicity and the use of structural mechanical properties obtained from masonry tests, since the masonry is a very heterogeneous material and the distribution of material properties of its constituent elements is difficult to predict.

Holmes was the first in replacing the infill by an equivalent pin-jointed diagonal strut. Stafford Smith proposed a theoretical relation for the width of the diagonal strut based on the relative stiffness of infill and frame. Alternative proposals were given by Mainstone, Liaw and Kwan, Decanini and Fantin, and more recently by Paulay and Priestley, Durrani and Luo, Cavaleri and Papia[Diana M. Samoila][24].

In the last decades it has become clear that one single strut is not sufficient to model the complex behavior of the infilled frame. This is because the local effects resulting from the interaction of the infill with the surrounding frame are not apparent if only the two loaded corners of the frame are connected through a single strut. As a result, bending moments and shear forces in the frame members are not modelled realistically and the location of potential plastic hinges cannot be adequately predicted. More complex macro-models were then proposed by many researchers (Crisafulli and Carr, Chrysostomu, Syrmakezis and Vartsnou, Andreaus) based on two, three or multiple diagonal struts. Despite of increasing complexity, the main advantage of these models is the ability to reflect the actions in the frame more accurately. Micro-modelling is a more complex method based on dividing the masonry panel and the concrete frame into several elements. This modelling can provide an accurate computational representation of both material and geometrical aspects, but is too time-consuming to be used in large and practical-oriented analysis.

From the first approach developed by Mallick and Severn using the finite element method for the analysis of 2D infilled frames, different alternatives have been proposed by using a micro-model. Among these, Riddington and Stafford Smith, Liaw and Kwan, Dhanaskar and Page or Asteris [Diana M. Samoila][24].

Suitability of a model is judged depending on several factors, namely,

1. The time required and the effort involved in modeling,
2. The ability to model lateral stiffness and the strength of infilled frame, and
3. The ability to model failure modes in not only infills but also in frame members [Diana M. Samoila][24].

2.6DETERMINATION OF THE EQUIVALENT STRUT WIDTH

The width of the equivalent diagonal strut (w) can be found out by using a number of expressions given by different researcher

1. Holmes (1961)

$$w = \frac{d_z}{3} \qquad 2.4$$

2. Smith and Carter (1969)

$$w = o.58(\frac{1}{H})^{-0.445}(\lambda_h H')^{0.335 d_z}(\frac{1}{H})^{0.064} \qquad 2.5$$

$$\lambda_h = \sqrt[4]{\frac{E_m t \sin 2\theta}{4 E_b I_s H}} \qquad 2.6$$

3. Mainstone (1971)

$$w = 0.175 d_z(\lambda_h H')^{-0.4} \qquad 2.7$$

4. Liaw and Kwan (1984)

$$w = \frac{0.95 H \cos\theta}{\sqrt{\lambda_h H'}} \qquad 2.8$$

5. Decanini and Fantin (1986)
    1. Uncracked Masonry

$$w = \left(\frac{0.748}{\lambda_h} + 0.085\right) d_z \qquad 2.9$$

    2. Cracked Masonry

$$w = \left(\frac{0.707}{\lambda_h} + 0.01\right) d_z \qquad 2.10$$

6. Paulay and Priestley (1992)

$$w = \frac{d_z}{4} \qquad 2.11$$

7. Durrani and Luo (1994)

$$w = \gamma\sqrt{L'^2 + H'^2}\sin 2\theta \qquad 2.12$$

Where:

$$m = 6\left[1 + \frac{6E_b I_g H'}{\pi E_b I_s L'}\right] \qquad 2.13$$

$$\gamma = 0.32\sqrt{\sin 2\theta}\ \left[\frac{H'^4 E_m t}{m E_b I_s H}\right]^{-0.1} \qquad 2.14$$

8. Cavaleri and Papia (2003)

$$w = \frac{d_z . k . c}{z} \frac{1}{(\lambda^*)^{\beta}} \qquad 2.15$$

$$\lambda^* = \frac{E_m}{E_b}\frac{t.H'}{A_s}\left[\frac{H'^2}{L'^2} + \frac{1}{4}\frac{A_s}{A_g}\frac{L'}{H'}\right] \quad 2.16$$

$$c = 0.249 - 0.0116\gamma + 0.567\,\gamma^2 \quad 2.17$$

$$\beta = 0.146 + 0.0073\gamma + 0.126\gamma^2 \quad 2.18$$

$$z = 1 + 0.25\left(\frac{L}{H} - 1\right) \quad 2.19$$

$$F_v(vertical\ load)$$

$$\varepsilon_v = \frac{F_v}{2A_sE_b} \quad 2.20$$

$$k = 1 + (18\lambda^* + 200)\varepsilon_v \quad 2.21$$

9. $w = \frac{d_z}{10}$ 2.22

According to FEMA306, 1997 the strut area A_e is given by following expression.

$$A_e = W_t \quad 2.23$$

$$w = 0.175(\lambda h)^{-0.4}d_z \quad 2.24$$

Where: -

D_z= Length of the diagonal strut (The diagonal length of infill panel)

L and L' are length of the infill and center to center distance between columns respectively

H and H' are height of the infill and center to center distance between Beams respectively

E_m = Modulus of elasticity of the infill material

E_b = Modulus of elasticity of the frame material

I_c = Moment of inertia of column

t = Thickness of infill

θ = Slope of infill diagonal to the horizontal.

2.7COMPARING THE STRUT WIDTH DETERMINED BY DIFFERENT RESEARCHERS

1. Smith and Carter and Decanini and Fantin equations generate large values for the diagonal strut width.
2. Mainstone relation is very close to that proposed by the Romanian code, both of them being at the inferior limit.
3. The other expressions (Holmes, Liaw and Kwan, Paulay and Priestley, Durrani and Luo, Cavaleri and Papia) are comparable.
4. Paulay and Priestley relation is recommended to be used in design analysis because it gives an average value and because of its simplicity.

2.8EFFECT OF OPENING

Presence of openings in masonry infill walls changes the actual behavior of RC frames because of reduction in lateral strength and stiffness. Such infills pose the hazard of out-of-plane collapse. Hence, it is best to avoid situations that lead to infill panels of large width or height. Unfortunately, there is little information on the effects of openings on the strength and stiffness of masonry infill reinforced concrete frames in seismic codes.

The effect of opening in the infill wall is to reduce the lateral stiffness and strength of the frame. This can be represented by a diagonal strut of reduced width. The reduction factor is defined as ratio of reduced strut width to strut-width corresponding to fully infilled frame[Sachin R Patel, Sumant B patel][22]. Using the simplified equation for the reduction factor ρ_w is given as:

$$\rho_w = 1 - 2.6\alpha_{co} \tag{2.25}$$

Where,ρ_w- is strut width reduction factor

α_{co} -is the opening area ratio.

$$\alpha_{co} = \frac{Area\ of\ opening\ (A_{op})}{Area\ of\ infill\ (A_{infill})} \tag{2.26}$$

2.9EFFECT OF INFILL THICKNESS

The effect of thickness on the fundamental period of vibration is insignificant. The fundamental period only slightly increases as the infill wall thickness increases, since the increase in thickness only increases the mass of the structure rather than its stiffness. Both the roof displacement and the inter-story drift ratio increase with the increase in thickness and the percentage of increase in roof displacement and inter-story drift ratio were 4.69% and 4.45% respectively. Thus, it is evident that there is no improvement in the lateral stiffness of the infill wall by increasing its thickness. Since the influence of infill thickness on the global responses, particularly the natural periods, roof displacement and the inter-story drift ratios, were not significant; the stresses in the infill walls were not affected by varying the thickness [Sachin R Patel, Sumant B patel][22].

2.10 EFFECT OF PLAN IRREGULARITY

A building may have a symmetrical geometric shape without re-entrant corners or wings but still be classified as irregular in plan because of distribution of mass (i.e., asymmetric placement of masonry infill walls) or vertical, seismic-force-resisting elements. According to EC8 and EBCS 8, slight plan irregularities may be taken into account by doubling the accidental eccentricity. In case of severe plan irregularities, due to excessive unsymmetrical placement of masonry infill walls, three-dimensional analysis is required considering stiffness distribution related to the uncertain position of masonry walls.

2.11 EFFECT OF VERTICAL IRREGULARITY

Vertical irregularities are introduced into masonry infill reinforced concrete frames due to reduction or absence of masonry infill walls in a particular story compared to adjacent stories, e.g., buildings with parking space in the first story and masonry infill walls on upper stories. In general, this gives rise to mass, stiffness, and strength irregularities along the height of buildings. Vertical irregularities in the bottom stories make the beams and columns of those stories more susceptible to damage or failure. Open ground story buildings have consistently shown poor performance during past earthquakes across the world.

According to EC 8 and EBCS 8; 1995, recommends an increase in the resistance of columns of soft stories by a factorα that is given by:

$$\alpha = \left(1 + \frac{\Delta V_{RW}}{\sum V_{sd}}\right) < q \qquad 2.27$$

Where:- q is the response reduction factor

ΔV_{RW} Is the total reduction in lateral resistance of masonry infill walls in a story compared to the story above.

$\sum V_{sd}$ is the sum of seismic shear forces acting on all structural vertical elements of the story concerned.

The design forces are not required to be increased if the factor α is less than 1.1

2.12 VERIFICATION OF STRUT MODEL

Experiments are very important to observe the behavior of complex structures. Many a times, analytical models have been developed on the basis of experimental results, and sometimes, experimental studies have been carried out to verify the analytically developed model. Though, numerous experimental studies have been reported on RC frames with unreinforced HCB masonry infill, only a few published studies provide detailed data about the specimens and the experimental results.

EXPERIMENTAL STUDY OF SPECIMEN

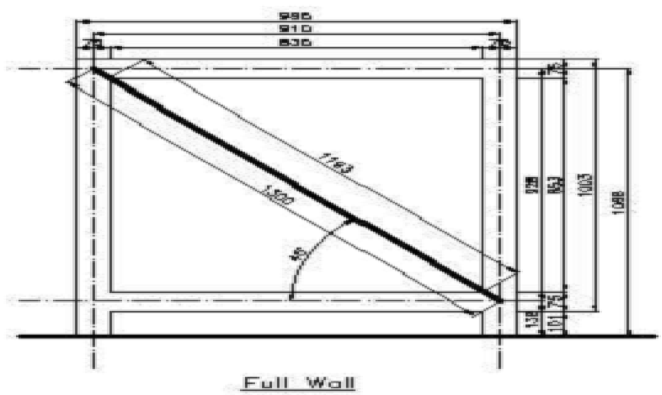
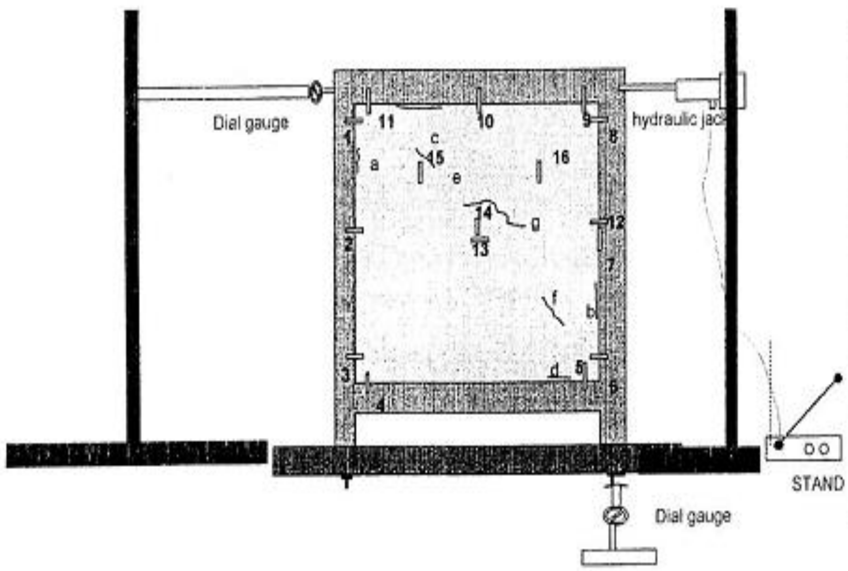


Figure 2.7. Geometry of test spacemen [Sumat Shrestha (2005)][31]

Sumat Shrestha (2005)[31] prepared 4 models in 1:3 reduced scale single bay single story model of RC frame with unreinforced full infill panel. The outer dimension were, 985 mm between column and floor height 1003 mm. Infill panel was built with 75 mm x 35 mm x 10 mm block in 1:4 cement sand mortar. The sizes of both beam and columns were 75 mm x 75 mm. the specimens were tested under monotonic static loading applied at roof level. The model with test setup for no is shown in fig. 2.8.



Strain Guage No.	Coordinate x(mm)	Coordinate y(mm)
1	75	850
2	75	500
3	75	150
4	150	75
5	830	75
6	910	150
7	910	440
8	910	850
9	830	930
10	500	930
11	150	930
12	910	500
13	500	450
14	500	505
15	260	670
16	725	670

Figure.2.8. Test setup for infill RC frame [Sumat Shrestha (2005)][31]

PROPERTIES OF SPECIMEN

For modeling of the specimens, geometric properties and properties of material used in these specimens are required. The geometry of the test specimen is shown in fig.2.8, and properties of materials are listed in Table 2.1. During the analytical analysis, loads on the models are applied in the same way as those were applied on the specimens in the experimental studies.

Table 2.1. Properties of Material

Section	Cross Section (mm*mm)	center line dimension (mm)	Comp. Strength f'_c (Mpa)	Young's Modulus (Mpa)	Poisons' Ratio	Long Reinf.(f_y = 248 MPa
Beam	75*75	928	7.93	12500	0.15	4-4.74mm
Column	75*75	910	7.93	12500	0.15	4-4.75mm
Infill	832*853	1300	-	225	0.17	-

ANALYTICAL STUDY OF SPECIMENS

The specimen for infill frame was modeled using equivalent diagonal strut as shown in fig. 2.9 using three different strut widths as proposed by Holmes, Pauley & Priestley and FEMA273. The experimental as well as analytical results are shown in fig 2.10.

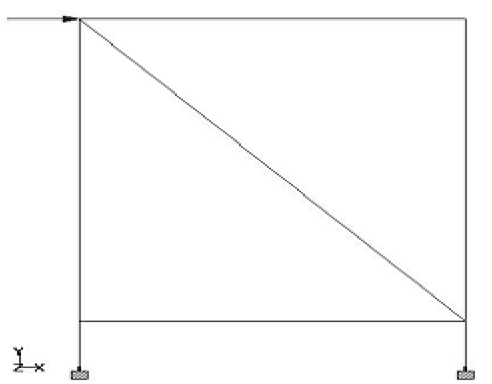


Figure. 2.9 Analytical model for full wall[Sumat Shrestha (2005)][31]

As seen from the fig. 2.10, though initial stiffness as predicted by all the analytical models are less than the experimental values, the overall stiffness from Holmes model is higher than the experimental value, whereas; FEMA model predicts considerably lesser value. The Pauley and Priestley model however seems to predict stiffness which reasonably matched with the experimental one.

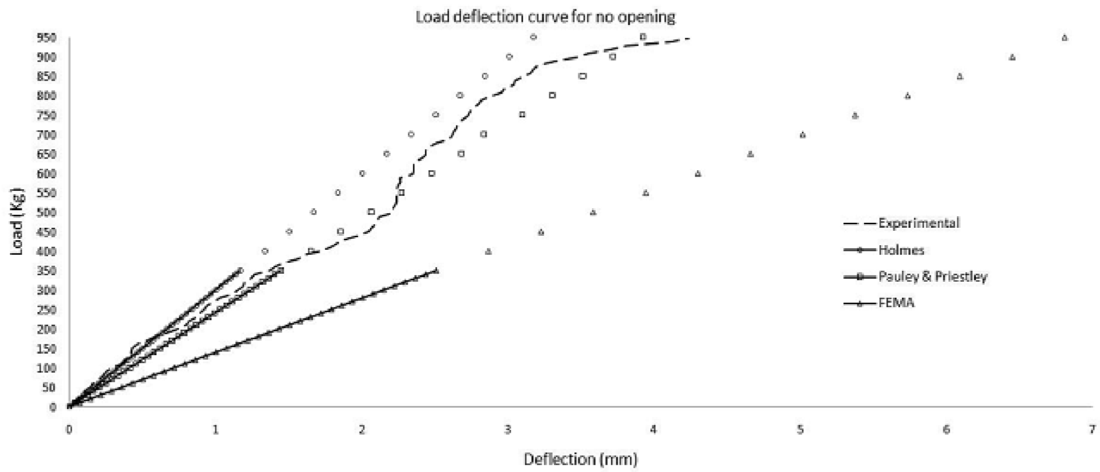


Figure.2.10. Load Deflection curve for full wall[Sumat Shrestha (2005)][31]

2.13 VERIFICATION OF THE PROPOSED REDUCTION FACTOR FOR INFILLS HAVING OPENINGS

In the parametric study, all the parameters except the number of stories and the size of openings are kept constant for different models. Based on this, a reduction factor for infilled frames with openings is proposed. To demonstrate the applicability of the pro-posed reduction factor, some of the parameters of the single-bay single-story model are changed one at a time while all others are kept constant. In addition, results of experimental specimens available in the literature are considered to verify the proposed strut-width reduction factor. Moreover, the proposed strut-width reduction factor is compared with that proposed by Durrani and Luo (1994), and by Al-Chaar (2002) [Goutam Mondal and Sudhir K. Jain][30].

The effects of the sizes of beam and column, compressive strength of concrete, thickness and area of infill panel on strut-width reduction factor are shown in Table 2.2. It is seen that a 37.5% reduction in size of beam results in 8% error in the strut-width reduction factor. Similarly, 70% reduction in size of column leads to 8% error in reduction factor. A 7% error in reduction factor was observed due to a 20% increase in the compressive strength of concrete. Table 2.2 also displays that reduction factor is weakly dependent on thickness and area of infill panel. All these strut-width reduction factors are also plotted in fig 2.11 along with the proposed strut-width reduction factor as per Equation 2.28, which shows that there is not much variation in the reduction factor in relation to the changes made in the parameters [Goutam Mondal and Sudhir K. Jain][30].

$$\rho_w = 1 - 2.6\alpha_{co} \qquad 2.28$$

Where:- ρ_w - Strut − width reduction factor

α_{co} -Opening- Area- Ratio

$$\alpha_{co} = \frac{Area\ of\ opening\ A_{op}}{Area\ of\ infill\ A_{infill}}$$

Table. 2.2 Effect of change of parameters on strut-width reduction factor

			Error in reduction Factor (%) for different Opening size (mm^2)		
Parameters	In Parameric Study	In Verification	1000*1000	1500*1500	2500*1500
Size of beam ((mm^2)	250*250	250*250	7.7	8.3	0
Size of column (mm^2)	400*400	250*250	8.1	3.2	1.2
f_{ck} (Mpa)	25	30	7	4.7	6.9
Thickness of infill (mm)	225	112.5	12	1.7	0
Area of infill panel (mm^2)	3000*5000	4000*5000	4.8	9	2

Single equivalent diagonal strut analysis of these three specimens are performed in SAP 2000 where the infills are modelled as diagonal strut, and the width of struts is varied to find the strut-width which gives stiffness same as the experimental initial stiffness value of these specimens. The corresponding strut-width reduction factor for the three specimens are also plotted in fig.2.11 which shows a good match with proposed strut width reduction factor.

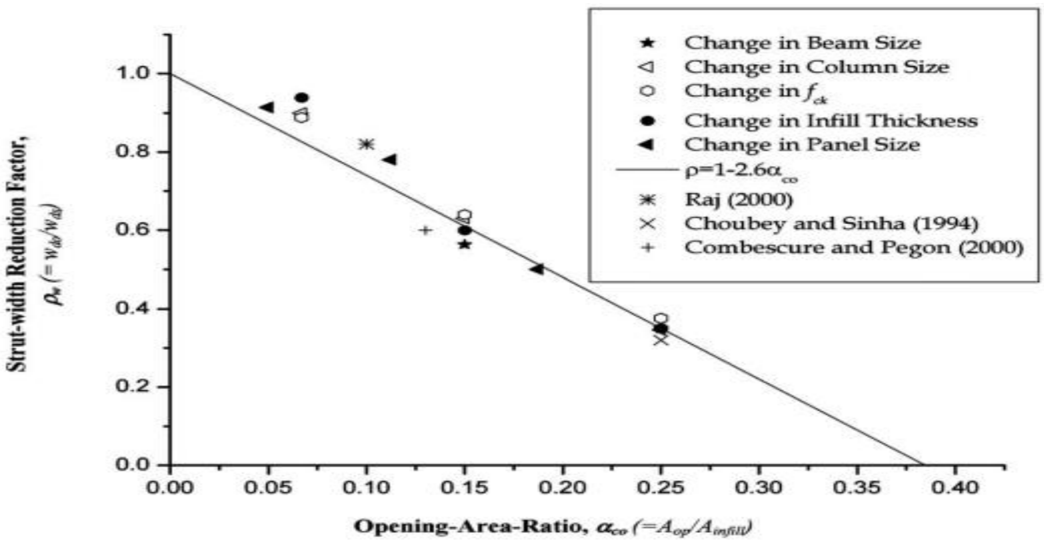


Figure.2.11 Verification of the proposed strut-width reduction factor. [Durrani and Luo (1994)].

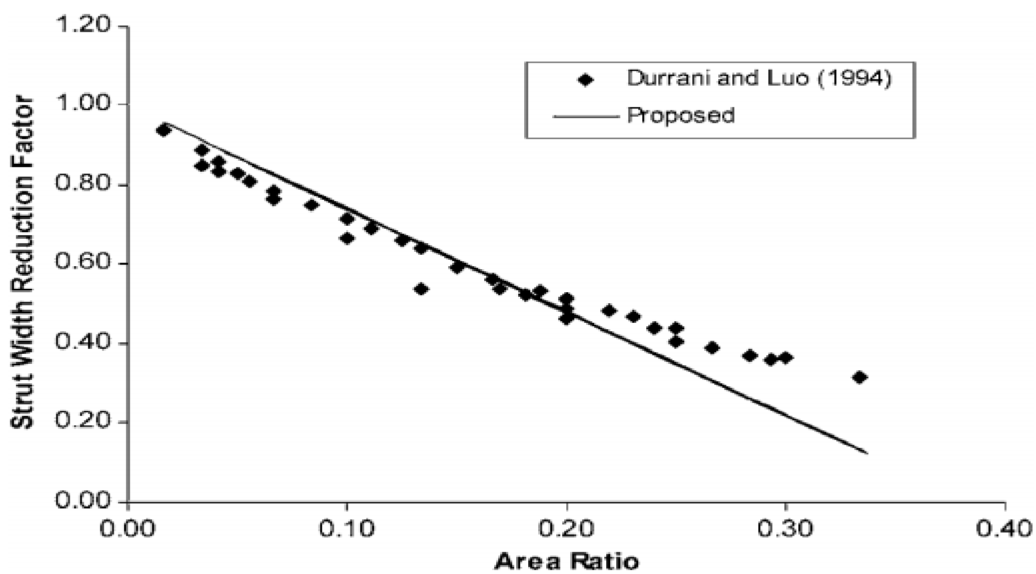


Figure.2.12. Comparison of proposed strut width reduction factor with that proposed by [Durrani and Luo (1994)].

The proposed strut width reduction factor is also compared with that proposed by Durrani and Luo (1994). For this comparison, infilled frames of aspect ratios 0.6 to 1.3and central opening of aspect ratios 0.5 to 4.0 are considered. fig. 2.12 shows that the proposed reduction factor matches well with that proposed by Durrani and Luo (1994) up to area ratio 0.25 and beyond that Durrani and Luo (1994) give substantially higher value of reduction factor. The expression developed in the present study is far simpler than that proposed by Durrani and Luo (1994) and is very easy to implement in the design office [Goutam Mondal and Sudhir K. Jain][30].

The proposed reduction factor is also compared indirectly with that proposed by Al-Chaar (2002). The reduction factor proposed by Al-Chaar estimates the in-plane capacity (strength) of the infilled frame satisfactorily. However, it does not predict the initial lateral stiffness with sufficient accuracy. The initial lateral stiffness of infilled frame obtained from Al-Chaar's experiment is almost three times larger (for aspect ratio of panel between 0.67 and 1.5) than that estimated from the bilinear pushover curve of infilled frame obtained by using strut-width proposed by Al-Chaar (2002). Moreover, reduction factor for strength to account for openings proposed by Al-Chaar should be applied for eccentric single equivalent strut where strut element is connected to the columns rather than to the beam-column joints. Therefore, it is not possible to directly compare the strut width reduction factor. Instead, lateral stiffness reduction due to presence of openings obtained by using strut width proposed by Al-Chaar (2002) and that proposed in the present study are compared. For this purpose, specimen of Choubey and Sinha (1994) with varying opening size is considered. fig. 2.13 shows that Al-Chaar's reduction factor somewhat overestimates the stiffness reduction. This difference becomes quite significant beyond the area ratio of 0.25 [Goutam Mondal and Sudhir K. Jain][30].

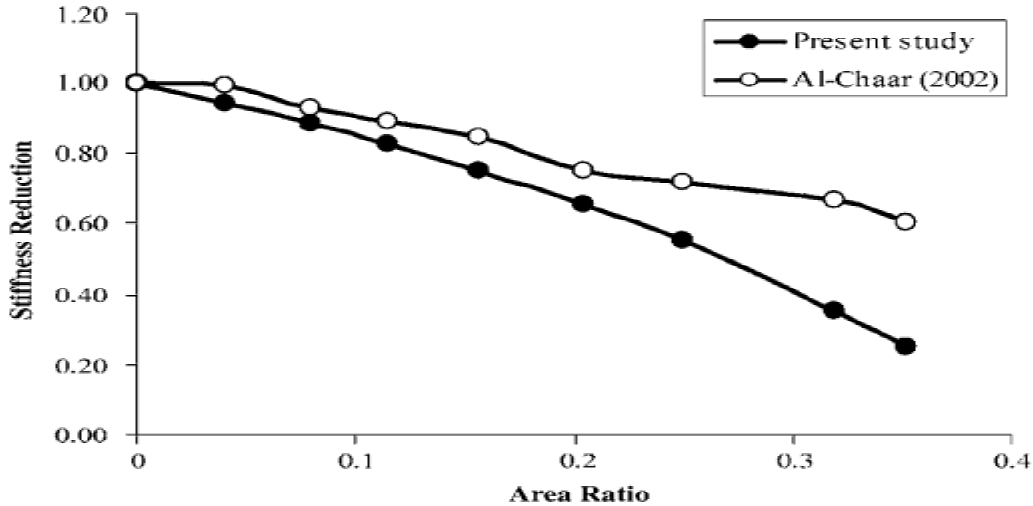


Figure.2.13 Comparison of stiffness reduction obtained by using proposed strut width reduction factor [Al-Chaar (2002)].

CHAPTER THREE

3 STRUCTURAL MODELING

3.1 OVERVIEW

It is very important to develop a computational model on which linearanalysis is performed. The first part of this chapter presents a summary of various parameters defining the computational models, the basic assumptions and the geometry of the selected building considered for this study.

Infill walls are modelled as equivalent diagonal strut elements. The last part of the chapter deals with the computational model of the equivalent strut.

3.2 BUILDING DESCRIPTION

The buildings areG+4, G+7 and G+10and they are symmetric in plan and in elevation. It is because, in order to show the effect of infill panels only. The plan dimension of the buildings are 30m x 30mandthe height of the buildings are 15m, 24m and 33mwith typical story height of 3mand is made of Reinforced Concrete (RC) Ordinary Moment Resisting Frames (OMRF). The concrete slab is 150mm thick at each floor level.

Imposed load is taken as 4 kN/m^2 for all floors. fig. 3.1 presents typical floor plan showing different column and beam locations. The cross sections of the structural members are shown in Table3.1 .

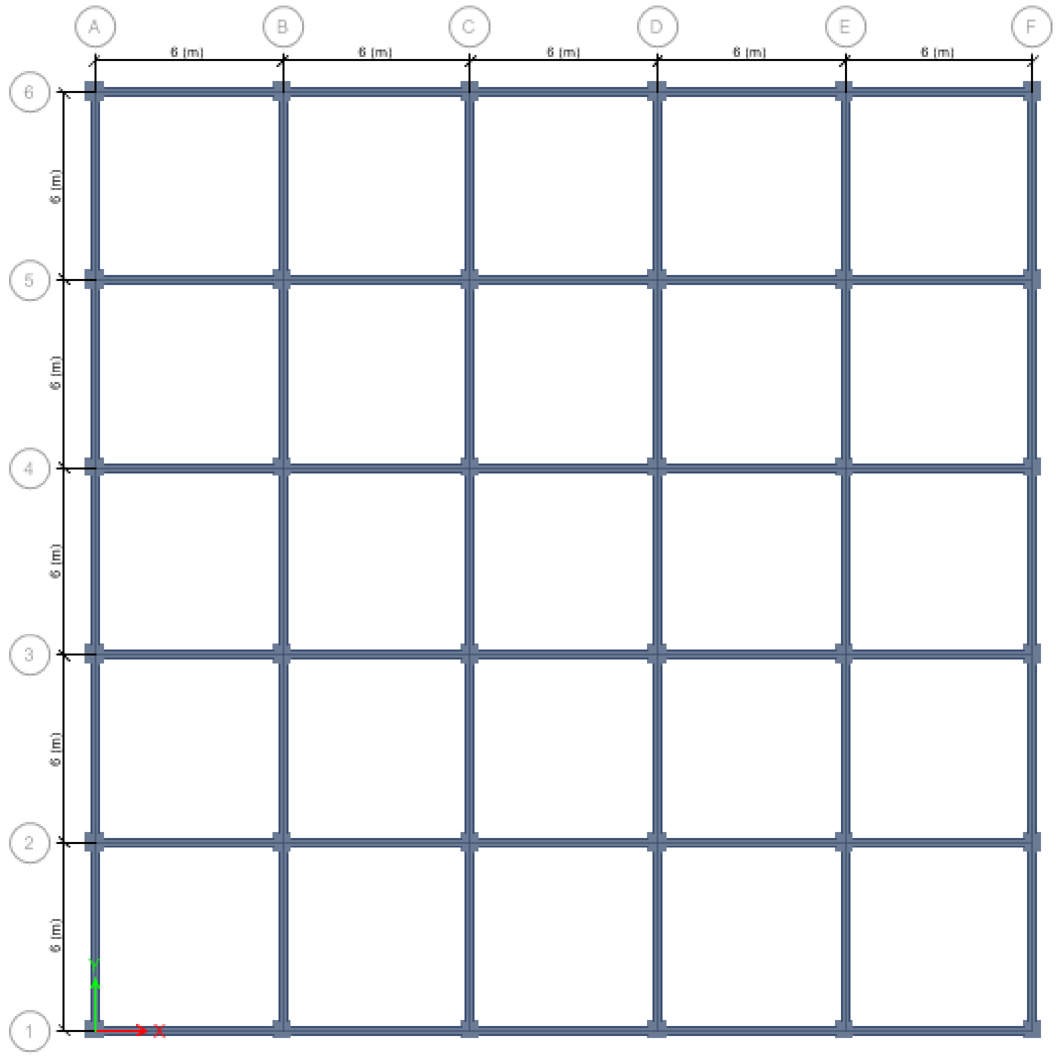


Figure. 3.1 Typical floor plan/ typical floor plan for bare frame

The hollow concrete block wall thickness is 200mm and 150 mm.Only the masonry surrounded by beams and columns in five different arrangements are considered as infill. Minor details that are less likely to significantly affect the analysis are deliberately left out from the models. The main purpose is to compare the overall behavior of the structure, but notthe behavior of infill panel or on the behavioral effect due to minute details.

Table 3.1. Member Size

	Column Crossection					
	G+10		G+7		G+4	
	Depth	width	Depth	width	Depth	width
Footing Column	0.9m	0.9m	0.8m	0.8m	0.8m	0.8m
Ground floor column	0.9m	0.9m	0.8m	0.8m	0.8m	0.8m
First floor column	0.9m	0.9m	0.8m	0.8m	0.7m	0.7m
second floor column	0.8m	0.8m	0.7m	0.7m	0.7m	0.7m
Third floor column	0.8m	0.8m	0.7m	0.7m	0.6m	0.6m
Forth floor column	0.8m	0.8m	0.7m	0.7m	0.6m	0.6m
Fivth floor column	0.7m	0.7m	0.6m	0.6m		
Sixth floor column	0.7m	0.7m	0.6m	0.6m		
Seventh floor column	0.7m	0.7m	0.6m	0.6m		
Eighth	0.6m	0.6m				
Nineth	0.6m	0.6m				
Tenth	0.6m	0.6m				
	Beam Crossection					
	Total Depth	Total width	Flange Thickness	Web Thickness		
Grade Beam(Rectangular	0.45	0.25				
Floor Beam T-Section	0.4	1.5	0.15	0.3		
Floor Beam L-Section	0.4	0.6	0.15	0.3		
Top tie Beam T-section	0.35	1.5	0.15	0.2		
Top tie Beam L-section	0.35	0.6	0.15	0.2		
	Strut Crossection					
	Length	Thickeness	Width			
WS1 (with out opening)	6.708204	0.15	1.68			
WS2(with window opening of 1.5m*2m)	6.708204	0.2	0.9521			
WS3 (with door opening of 2m*25m)	6.708204	0.15	0.4666			
WS4 (with door opening of 1.2m*2.5m)	6.708204	0.15	0.9521			
WS5 (with door opening of 1m*2.5m)	6.708204	0.15	1.073			

Initial dimensioning of the beams and columns were made on the basis of bare frame.The same sections were used for the cases of infilled frames analysis with earthquake load as per Eurocode: 2004 suchthat the structure met the strength and ductility requirements of the new Ethiopian building code of standard (EBCS).Further, it was assumed that the infill panels were neither integral nor bonding with the frame. Five different models with and without infill were developed to analyze and to investigate the effect of infill wall on seismic response of the typical structures.

ARRAGEMENT OF INFILL WALL

Four different models which have different wall arrangements are prepared in addition to that of the bare frame model, its floor plan was as shown in fig.3.1.

1. BASE STRUCTURE OFINFILLED FRAME

As shown in fig. 3.2 infill panels are arranged symmetrically throughout the building with and without openings. There are windows with the dimension of 1m*2m on outer periphery of infill panels. Whereas infills on axis C between axis 2 and 5 and on axis 2 and 5 between axis B and C and D and E and on axis 5 between axis C and D have doors with dimension of 1.2m*2.5m, 2m*2.5m and 1m*2.5m respectively. On the other hand infills on axis B and E between axis 3 and 4, on axis C and D between axis 1 and 2 and 5 and 6, on axis 2 and 5 between axis A and B and E and F and on axis 3 and 4 between axis B and C and D and E have no opening.

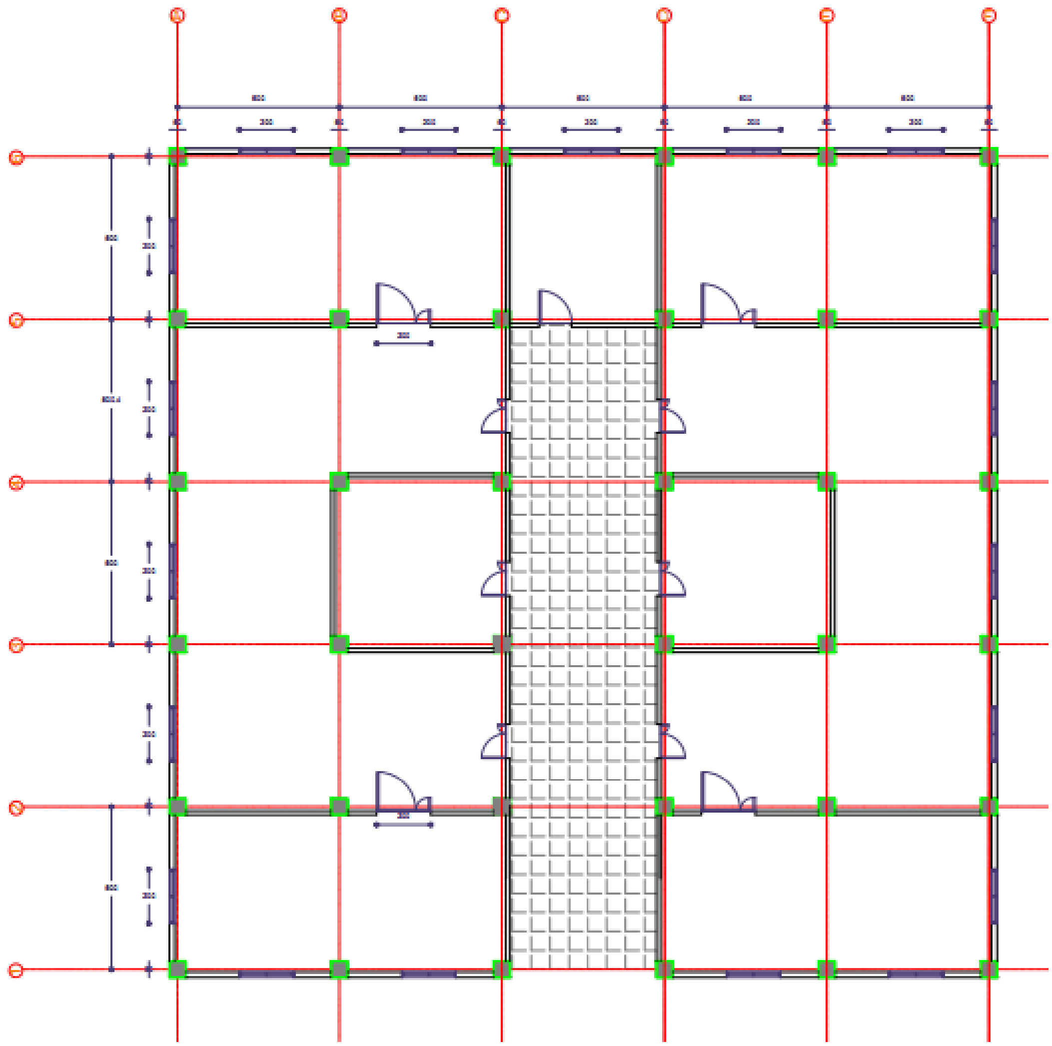
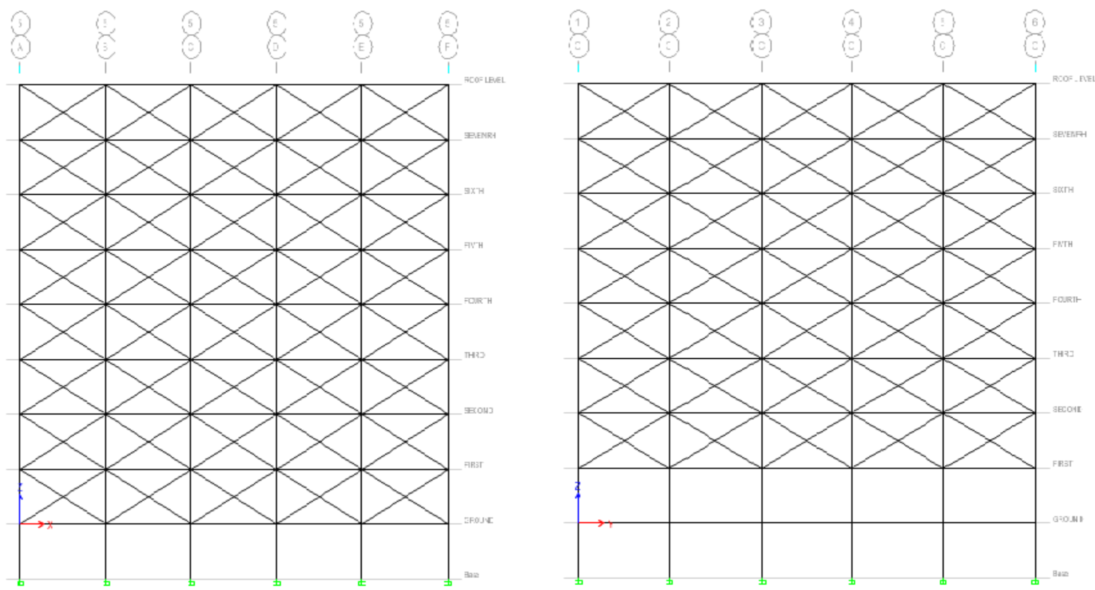


Figure. 3.2 Typical floor plan for basestructure of infilled frame model

2. SOFT GROUND STORY FRAME

Soft ground storys are introduced into masonry infilled reinforced concrete frames due to reduction or absence of masonry infill walls in a particular story compared to adjacent stories, e.g., buildings with parking space in the first story and masonry infill walls on upper stories. In this thesis soft ground story due to the absence of infill on the ground floor has been studied. Modification on basestructure of infilled frame has been done by removing all the infill panels from the ground floor as shown in fig. 3.3.



a b

Figure. 3.3. Elevation View. a) For base infilled frame b) For soft ground story frame

3. FRAME WITH HALF OF THE WALL REMOVED FROM BASE STRUCTURE OF INFILLED FRAME

Comparing with the base structure of infilled frame, this one is where half of the masonry infill panels were removed. In order to maintain comparison with other frames; the infill arrangement were kept symmetrical. Besides that the opening size and infill location are similar to the fully infilled frame.The arrangement used in the analysis is shown in fig. 3.4.

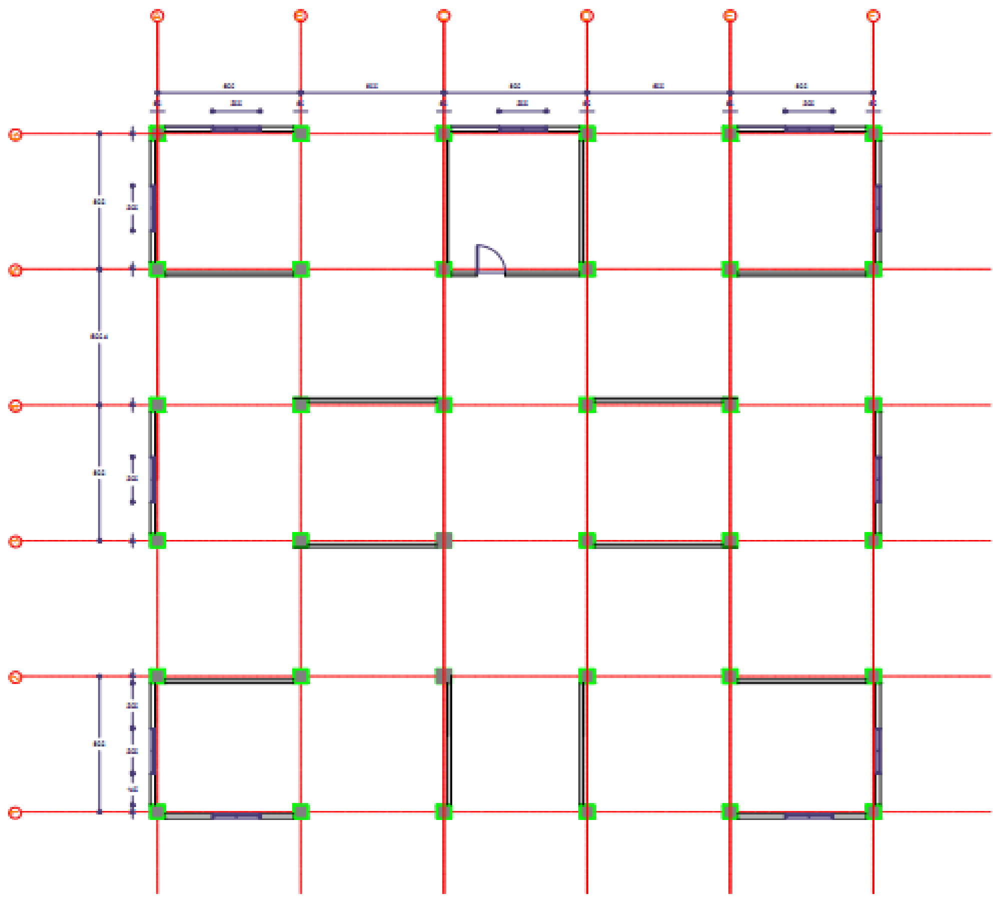
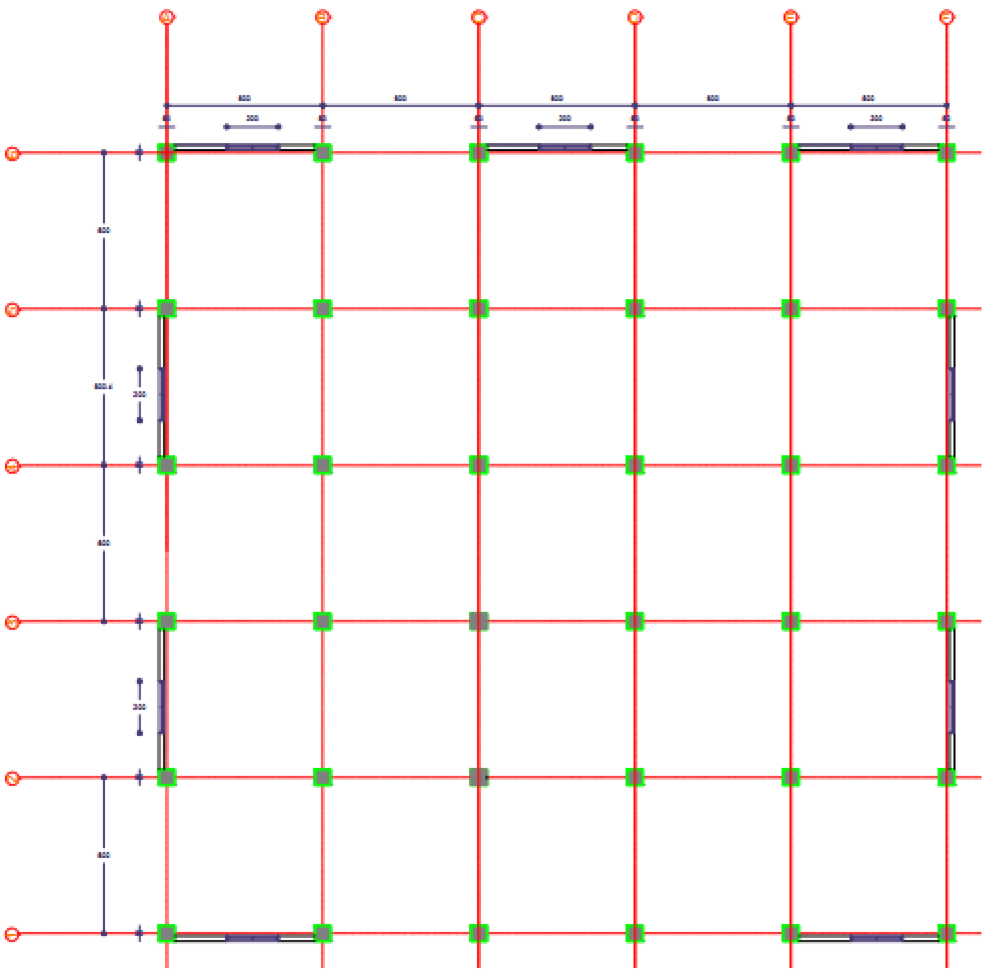


Figure 3.4. Typical floor plan offrame with half wall removed from basestructure infilled frame

4. FRAME WITH 75% OF THE WALL REMOVED FROM BASE STRUCTURE OF INFILLED FRAME

As shown in fig 3.5 some section of the outer periphery of the building are infilled. In order to indicate the impact of infill on structural integrity in this case 75% of the infill were removed from the basestructure ofinfilled frame model. The opening size and location of the existing infill are similar to base and half infilled frames.Infills with opening are modeled as discussed in chapter 2 verification section.



i

Figure 3.4. Typical floor plan ofwith 75% of wall removed from baseinfilled frame

3.3 STRUCTURAL MODELLING

Modelling a building involves the modelling and assemblage of its various load-carrying elements. The model must ideally represent the mass distribution, strength, stiffness and deformability. Modelling of the material properties and structural elements used in the present study is discussed below.

3.3.1 MATERIAL PROPERTIES TO BE USED

For this study, the material property for concrete, reinforcing bar and hollow concrete block (HCB) masonry panels are as follows:

For Reinforcing Bar:

Yield strength of reinforcing bar $f_y = 500MPa\ (Fe\ 500)$

For Concrete:

Unit weight (weight per unit volume) = $25\ KN/m^3$

Characteristic compressive strength, $f_{ck} = 30/37MPa$

Young's modulus of elasticity, $E_c = 32000MPa$

Poisson's ratio, $v_c = 0$for cracked concrete

Shear Modulus, $G_c = \frac{E_c}{2(1+v_c)} = 16000Mpa$ 3.1

For HCB Masonry Panel:

Size of HCB = 15 cmx 20cm x 40cm and 20 cmx 20cm x 40cm

Horizontal mortar thickness = 2cm

Mortar ratio = 1:3

Unit weight (weight per unit volume) = $12\ KN/m^3$

Characteristic compressive strength, $f_{cm} = 3.5MPa$

Young's modulus of elasticity of the masonry, can be calculated by the relation given by Paulay and Priestley (1992);

$$E_m = 750\sqrt{f_{cm}} = 750\sqrt{3.5} = 1403\ MPa \qquad 3.2$$

Poisson's ratio, $v_m = 0$

Shear Modulus, $G_c = \frac{E_m}{2(1+v_m)} 701.5Mpa$ 3.3

3.3.2 LOAD CASES USED:

Dead load: The Unit Weights of Materials used in this study is based on EBCS 1: 1995.

Imposed Load: the imposed load used in this study is based on EBCS 1: 1995

Earthquake Load: Eurocode 8: 2004 Criteria for Earthquake Resistant design of Structure was used.

3.3.3 STRUCTURAL ELEMENTS

The reference structures studied areG+4, G+7 and G+10 RC frames, it analyzed according to the Eurocode 8 ductility class M ('medium') provisions for bare frames for a design ground acceleration of 0.1g. The structures has been assessed both as a bare frame and as an infilled one. The beams and columns are modeled as a frame element which has the capability to deform axially, in shear, in bending and in torsion. The beam-column joints are assumed to be rigid. The weight of the slab is distributed as rectangular load to the surrounding beams as per EBCS 2: 1995. A semi-rigid joint diaphragm action were assumed, which ensure integral action of all the vertical andlateral load-resisting elements. A Finite Element software ETABS 2015were used for the modeling.

3.3.4 MODELLING INFILL WALLS

Infill walls are two dimensional elements that can be modelled with orthotropic plate element for linear analysis of buildings with infill wall. But the nonlinear modelling of a two dimensional plate element is not understood well. Therefore infill wall has to be modelled with a one-dimensional line element for nonlinear analysis of the buildings. Same building model with infill walls modelled as one-dimensional line element.[Wael W. El-Dakhakhni, et al][4].Also used in the present study for linear analyses. Infill walls are modelled here as equivalent diagonal strut elements. Section 3.3.5 explains the modelling of infill was as diagonal strut in detail.

3.3.5 MODELLING OF EQUIVALENT STRUT

For an infill wall located in a lateral load-resisting frame, the stiffness and strength contribution of the infill has to be considered. Non-integral infill walls subjected to lateral load behave like diagonal struts. Thus an infill wall can be modelled as an equivalent diagonal compression strut in the building model. Thus in order to have only the diagonal compression strut, the diagonal tension struts were set to tension limit zero in four different models. Rigid joints connect the beams and columns, but pin joints connect the equivalent struts to the beam-to-column junctions as shown in fig. 3.3. This section explains the procedure based on Paulay and Priestley (1992);) to calculate the modelling parameters (effective width, elastic modulus and strength) of an equivalent strut. The length of the strut is given by the diagonal distance (d) of the panel and its thickness is equal to the thickness of the infill wall. The elastic modulus of the strut is equated to the elastic modulus of masonry (Em). For the estimation of width (w) of the strut, a simple expression as given in Eq. 2.16to (Chapter 2) is adopted.

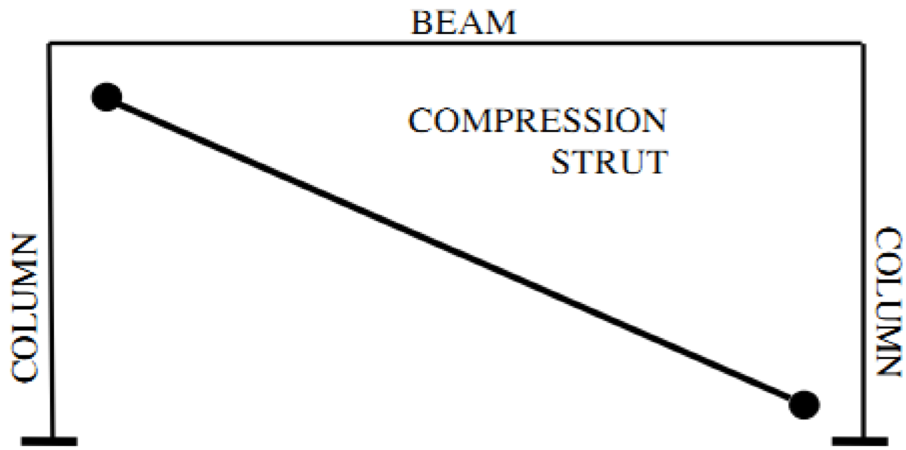


Figure. 3.5 Compression Strut Connection.

3.3.6 ALGORITHM FOR GENERATING THE EQUIVALENT STRUT MODEL

The algorithm for calculating the strut width as per Paulay and Priestley (1992);) is as given below and the detail procedure is shown in appendix A.

Step 1. Specify material properties

Step 2. Specify geometric properties

Step 3. Calculate the diagonal length of the infill panel

Step 4. Divide the diagonal length by 4

Step 5. Then multiply the strut width by the reduction factor if the infill has an opening. Computational model of the building can be analyzed using the obtained values of w and Em for the struts.

CHAPTER FOUR

4.1 INTERPRETATION OF RESULTS

The interpretation of results is based on the global behavior of the structure and not on the micro level behavior of infill panels. The major behavioral studies considered are the fundamental period, base shear, story displacement,story shear, and member forces.

Based on these behaviors, the results of the analysis such as fundamental period, base shear, story displacement, story shear, and member forces due to the effect of masonry infill are presented and discussed in the Results and Discussion section.

4.2 RESULTS AND DISCUSSIONS

The bare frame and infill frames with different arrangement were studied analytically. Based on the results obtained from the numerical analysis, the behavior of different structural systems in terms offundamental period, base shear, story displacement, story shear, and member forces are compared in the following pages. The results of analytical studies are presented in thissections. Only the findings of the effects of infill based on Pauley & Priestley ware studied and compared with bare frame model.Although from the verification the Pauley & Priestley model with effective width of one-fourth the diagonal seems most appropriate strut model. Thus, five different models, a bare frame,base structure infilled frame, soft ground story frame, frame with half of the walls removed from baseinfilled frame and frame with 75% of the wall removed from base infilled frame were considered.

The comparison of seismic excitation in terms of fundamental periodbetween bare frame,basestructure of infilled frame, building with soft ground story, frame with half of the walls removed from base structure of infilled frame and frame with 75% of the wall removed from basestructure ofinfilled frame respective are presented. This is followed by the presentation of comparative study of, base shear and Story displacement of bare frame and the four infill frame models. Next, the structural responses of different bare and infill models in terms ofstory shear iscompared. Lastly, the member forces of structural member due to combined effect of gravity and seismic loading for both the bare and infill frame are studied and discussed.

4.3. EFFECT OF INFILL WALL PANEL ON RC FRAME STRUCTURE

4.3.1 FUNDAMENTAL TIME PERIOD

In the seismic analysis of a building structure, the fundamental time period is one of the most important and unique properties, as the base shear, design lateral load, story shear, story moments, etc. depends on this property.

Almost all building codes impose an upper limit on the natural period determined from a rational numerical analysis by way of empirical equation and the Ethiopian Building Code of Standard EBCS 1995 is not an exception to this. But, since the bare frame models does not takes in to account the stiffness rendered by the infill panel, it gives significantly longer time period than predicted by the code equations as shown in Table 4.1, and fig. 4.1and hence smaller lateral forces. However, when the effect of infill is included, the time periods determined were found usingthe code formulas. Due to the fact that the fundamental time period of a structure depends not only on the mass of a structure but also on the stiffness of thestructure. And when the infill is modeled, the structure becomes much stiffer than the bare frame model.

EC8 and EBCS 8 specifies that these actions should correspond to a period T_1' equal to the average of that of the bare frame and the infilled frame. A series of empirical formulae are included in EC8 Part 1-3 and EBCS 8 section 3.9.4for the calculation of T1i, the period of the infilled frame. Table 4.1 listsshowsthe periods calculated according to the code procedures and analytical result. Note that T_{1b} refers to the fundamental period of a bare frame, T_{1i} to that of an infilled frame, while the code value:-

$$T_1' = ((T_{1b} + T_{1i}))/2 \quad 4.1$$

Where:- $T_{1b} = 0.075H^{3/4}$ 4.2

$$T_{1i} = \frac{T_{1b}}{[1 + \frac{{T_{1b}}^2 A_w G g}{16HW}]^{1/2}} \quad 4.3$$

Where:- A_w-- average horizontal cross sectional area of infill walls per story in the relevant direction,

G-- shear modulus of infill walls,

g--acceleration of gravity,

H --Height of the building,

W --weight of the building

Table 4.1 Comparison of Fundamental Time Period for different model types

A. *Fundamental period of G+4 building*

Types of Models	*H(m)*	*Emperical results T1'=(T1b+T1i)/2*	*Analytical results (T1')*	*Period Reduction in % for emperical result*	*Period Reduction in % for Analytical result*
Bare Frame	*15*	*0.572*	*0.82338*	*0*	*0*
Frame with 75% of the wall reduced	*15*	*0.476*	*0.68556*	*16.70*	*16.70*
Frame with half of the wall reduced	*15*	*0.429*	*0.61718*	*25.03*	*25.04*
Building With Soft Story	*15*	*0.410*	*0.58766*	*28.32*	*28.63*
Base infilled Frame	*15*	*0.395*	*0.56807*	*30.85*	*31.00*

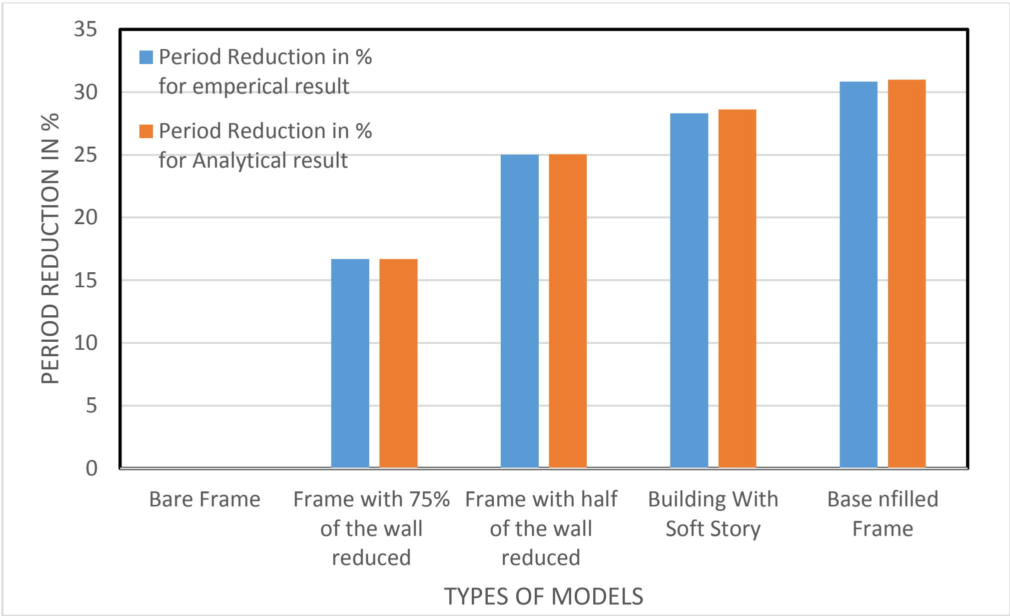


Figure. 4.1A. Fundamental period reduction in % (G+4).

Introducing infill panels in the RC frame reduces the time period of bare frames and also enhances the stiffness of the structure. Bare frame idealization leads to overestimation of natural periods and under estimation of the design lateral forces. The trend in the analysis for both analytical and empirical taken to be bare, frame with 75% of wall removed from base structure of infilled frame, frame with half of wall removed from base structureof infilled frame, soft ground story and basestructure of infilled frame.

The fundamental natural period of vibration from the empirical expression of the EBCS 8 1995 is compared with the analytical time period. As shown in table 4.1 analytical time period do not tally with empirical time period (caudal). The analytical natural period depends on the mass and stiffness, but empirical time period only depends on the height of the building. Thus, results obtained for fundamental natural period are shown in Table 4.1,and fig 4.1 indicate that analytical natural period is greater than the natural periods obtained from the empirical expression of the code in all the corresponding trends. The natural period for empirical analysis decrease in the trend for G+4, G+7 and G+10, similar sequence is also observed in the analytical analysis for G+4, G+7 and G+10.

The results on the table and figure 4.1A show the fundamental period of G+4 for frame with 75% of the wall reduced, half reduced, soft story and base infill in comparison with the bare frame. For both empirical and analytical analysis the period reduction increases for the stated sequence. The percentage of period reduction among the bare frame and respective models in percentages are 16.70%, 25.03%, 28.32%, and 30.85% for empirical and 16.70%, 25.04%, 28.63% and 31% for analytical respectively. With this empirical and analytical results show similar reduction for 75% of wall reduced and half reduced infill, whereas there are tangible difference on soft story and base infill.

B. Fundamental period of G+7 building

Types of Models	*H(m)*	*Emperical results T1'=(T1b+T1i)/2*	*Analytical results (T1')*	*Period Reduction in % for emperical result*	*Period Reduction in % for Analytical result*
Bare Frame	*24*	*0.813*	*1.12226*	*0*	*0*
Frame with 75% of the wall reduced	*24*	*0.648*	*0.99251*	*20.29*	*23.78*
Frame with half of the wall reduced	*24*	*0.578*	*0.79494*	*28.91*	*29.20*
Building With Soft Story	*24*	*0.544*	*0.72761*	*33.10*	*35.16*
Base infilled Frame	*24*	*0.535*	*0.69853*	*34.19*	*37.75*

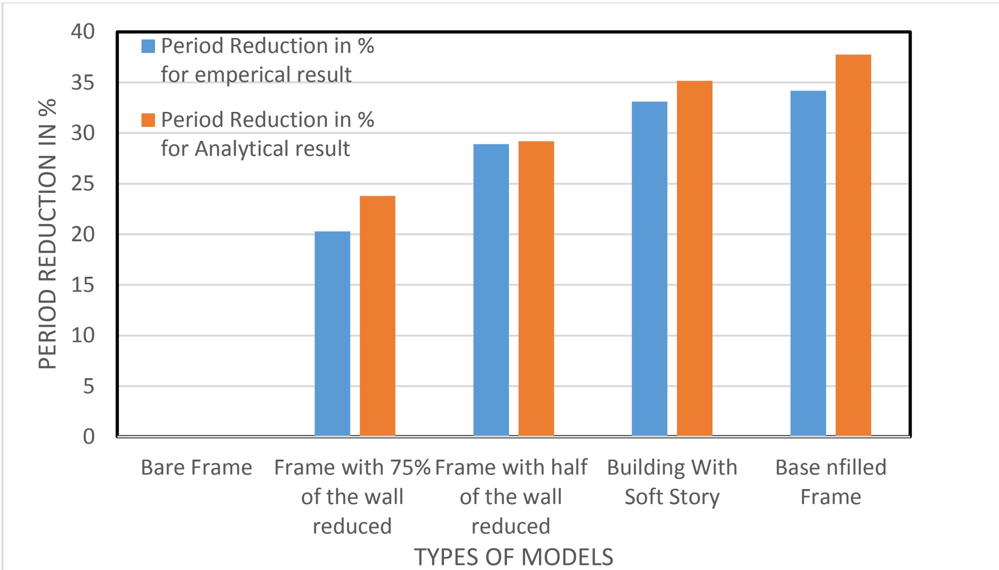


Figure. 4.1B. Fundamental period reduction in % (G+7).

For the G+7 building fundamental period reduction has been increased with percentage of 20.29%, 28.91%, 33.10% and 34.19% for empirical and 23.78%, 29.20%, 35.16% and 37.75% for analytical analysis of sequenced models. Remember that period reduction has increased with the comparison of bare frame for both results as it is indicated on table and figure 4.1B.

C. Fundamental period of G+10 building

Types of Models	*H(m)*	*Emperical results T1'=(T1b+T1i)/2*	*Analytical results (T1')*	*Period Reduction in % for emperical result*	*Period Reduction in % for Analytical result*
Bare Frame	*33*	*1.033*	*1.46265*	*0*	*0*
Frame with 75% of the wall reduced	*33*	*0.840*	*1.15647*	*19.00*	*20.93*
Frame with half of the wall reduced	*33*	*0.734*	*1.00871*	*28.90*	*31.03*
Building With Soft Story	*33*	*0.687*	*0.91269*	*33.50*	*37.60*
Base infilled Frame	*33*	*0.677*	*0.89776*	*34.50*	*38.62*

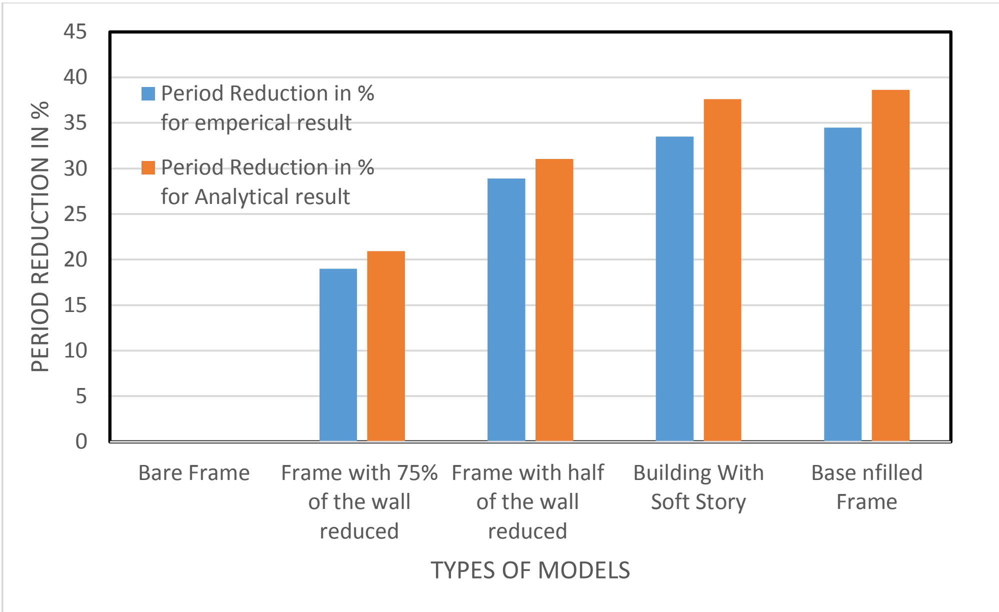


Figure. 4.1C. Fundamental period reduction in % (G+10).

As shown in table and figure 4.1C, relate to bare frame G+10 period reduction is also increases with the same pattern as for G+4 and G+7. Comparing with respective models the increase percentage period reduction is 19.00%, 28.90%, 33.5% and 34.50% for empirical and 20.93%, 31.03%, 37.60% and 38.62% for analytical results respectively.As the number of storys increased from fifth story to tenth story, the influence of the infill panels increased.

Therefore, verification of EBCS 8 section 3.9.4 for themodificationof natural period caused by addition of infillsgives almost a close result with that of analytical result. This shows that the empirical formula used for the modification of the fundamental period can be effectively used in the analysis. The reduction of the fundamental period usually implies higher base shear as shown in the next section.

Generally, period of the building decreases when the effect of masonry infill wall is considered. This clearly indicates that the masonry infill panel has structural implications and should not be ignored in the analysis. And it is known that, when the period is greater than two seconds a dynamic analysis has to be applied that is recommended for analysis. Therefore, the presence of infill may also avoid the requirement of dynamic analysis.

4.3.2 BASE SHEAR

Base shear is the total horizontal seismic shear force at the base of structure. EC8 and EBCS 8 bases the calculation of seismic actions for infilled frames on the average periods (T_1' in last columns of Table 4.1).

$$F_b = S_d(T_1')\,W \qquad 4.4$$

Where:-$F_b = Base\ shear\ force$

$W = Total\ Weight\ of\ the\ Building$

$$S_d(T_1') = Design\ Spectrum = \alpha\beta\Upsilon$$

Table 4.2 and fig 4.2 showsbase shear values of five different models inwhich the base shear in basestructure of infilled frame is about 43.78%, 47.00% and 44.56%and 43.87%, 49.17% and 47.22% greater for empirical and Analytical resultsrespectively, while, the bare framehaving less value of base shear for G+4, G+7 and G+10 buildings respectively. And as it is observed, base shear obtained from empirical result has larger values than that of analytical result, but when it comes to comparison with bare frame,percentage increase of analytical results of infilled frames shows slightly larger base shear than empirical one.

Table. 4.2 Comparison of Base Shear for different Models

A. *Base shear of G+4 building*

Types of Models	*Total wight of the building W(KN)*	*Sd(T1') using emperical result*	*Sd(T1') using analytical result*	*Fb = Sd(T1') using emperical result*	*Fb = Sd(T1') using analytical result*
Bare Frame	15235.39	0.078852908	0.059974241	1201.3548	913.7309454
Frame with 75% of the wall	16322.11654	0.090395387	0.068806725	1475.444036	1123.071392
Frame with half of the wall	18133.3316	0.097874818	0.074448527	1774.796521	1349.999828
Building With Soft Story	19485.70551	0.1012197	0.07723611	1972.337262	1505.000102
Base nfilled Frame	20548.28501	0.10398528	0.079225247	2136.719172	1627.942955

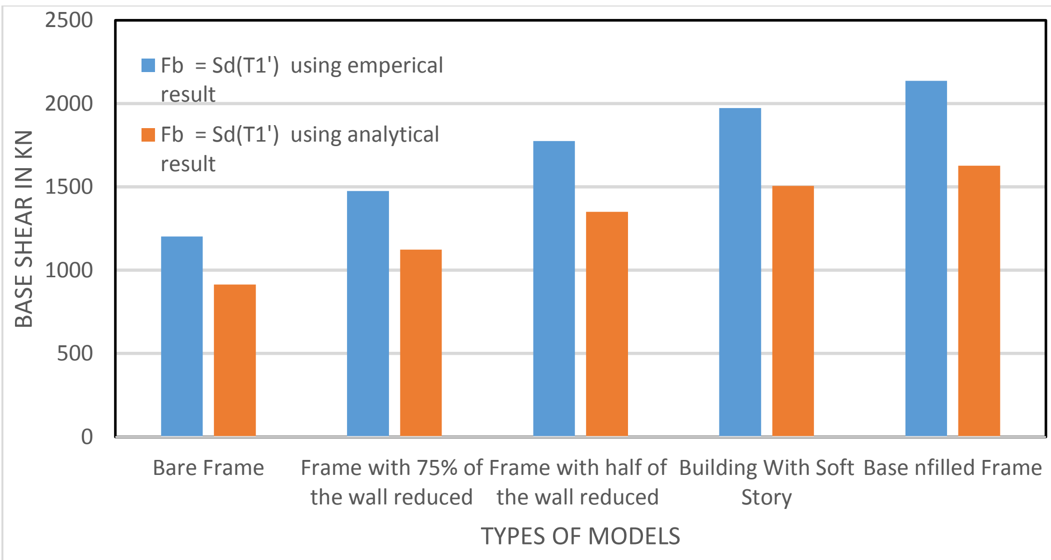


Figure. 4.2A. Base shear of G+4 building

This is because of bare frame is having larger value of fundamental natural time period as compared to other models due to absence of masonry infill walls. Frame with 75% of wall removed from basestructure infilled frame has lesser base shear value than the other infilled frames but, still greater than that of the bare frame and this is because of having large value of the fundamental natural period compared to the other infilled frames, due to the absence of masonry infills on most part of the building. Fundamental natural period get increased and therefore base shear get reduced. Base shear response of models more in magnitude when diagonal wall strut comes into action.

B. Base shear of G+7 building

Types of Models	*Total wight of the building W(KN)*	*Sd(T1') using emperical result*	*Sd(T1') using analytical result*	*Fb = Sd(T1') using emperical result*	*Fb = Sd(T1') using analytical result*
Bare Frame	22451.1375	0.060547621	0.047543908	1359.362964	1067.414823
75% of the wall reduced	24189.90396	0.071776741	0.052133133	1736.272483	1261.095479
of the wall reduced	26894.652	0.078202226	0.061576391	2103.221648	1656.075603
Building With Soft Story	29889.19	0.081840037	0.065802289	2446.132421	1966.777106
Base nfilled Frame	30951.7735	0.082870442	0.067846304	2564.987157	2099.963443

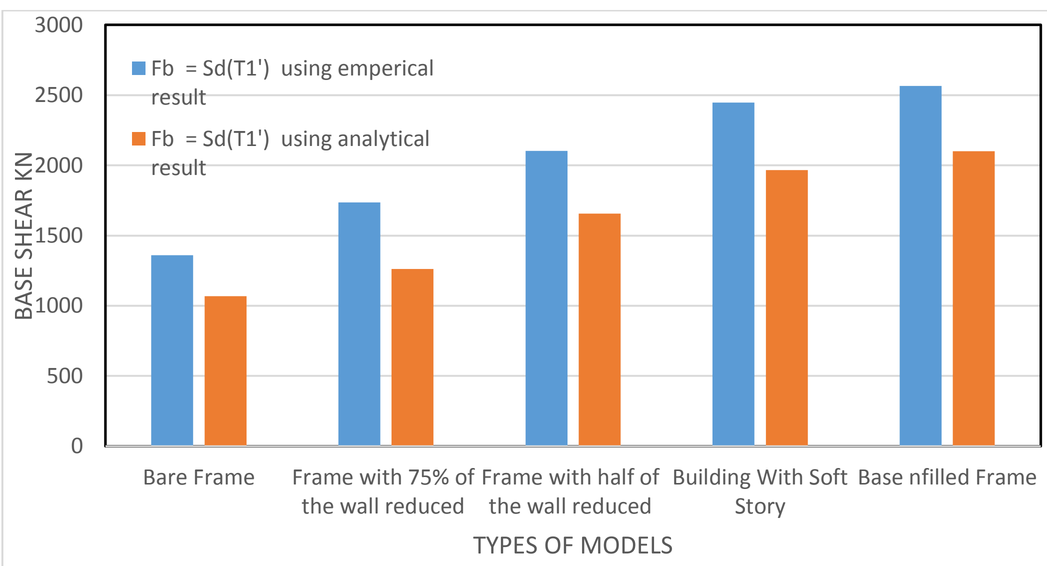


Figure. 4.2B. Base shear of G+7 building

Since the bare frame models does not takes in to account the stiffness rendered by the infill panel, it gives significantly longer time period. And hence smaller lateral forces. This is due to the fact that the fundamental time period of a structure depends not only on the mass of a structure but also on the stiffness of the structure. And when the infill is modeled, the structure becomes much stiffer than the bare frame model.

C. Base shear of G+10 building

Types of Models	*Total wight of the building W(KN)*	*Sd(T1') using emperical result*	*Sd(T1') using analytical result*	*Fb = Sd(T1') using emperical result*	*Fb = Sd(T1') using analytical result*
Bare Frame	37234.05	0.050592921	0.038977114	1883.779339	1451.275818
75% of the wall reduced	39624.8538	0.059082046	0.046485148	2341.117445	1841.967178
Frame with half of the wall reduced	43343.88214	0.065372177	0.051503916	2833.483935	2232.37965
Building With Soft Story	47859.84503	0.068698529	0.055516557	3287.900968	2657.013811
Base nfilled Frame	48922.4245	0.069458195	0.056207569	3398.063287	2749.810564

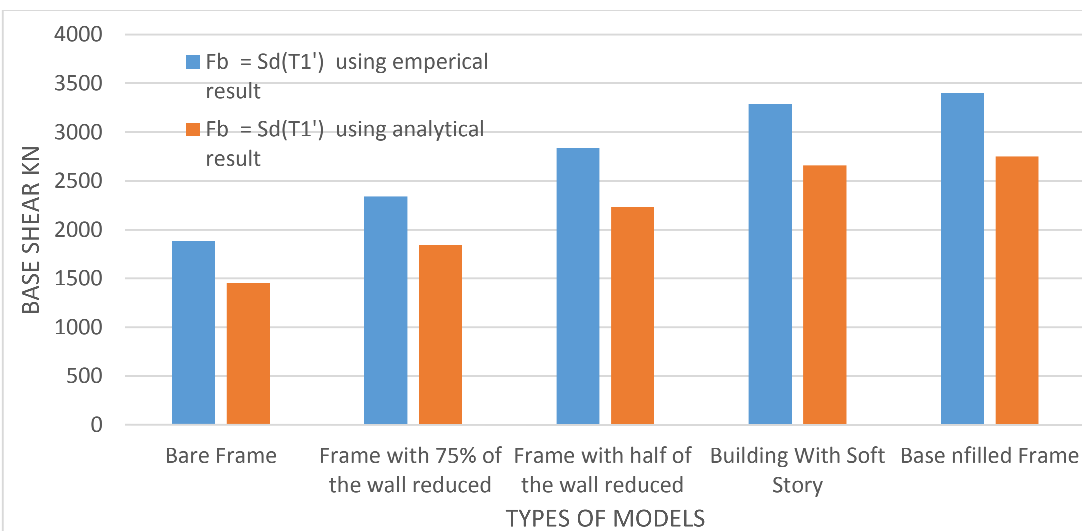


Figure. 4.2C. Base shear of G+10 building

It has been found that calculation of earthquake forces by treating RC frames as ordinary frames without regards to infill leads to underestimation of base shear. The configuration of infill and number of stories in the parking frame changes the behavior of the frame therefore it is essential for the structural systems selected, to be thoroughly investigated and well understood for catering to soft ground floor.

4.3.3 STORY DISPLACEMENT

As shown in table 4.3, and fig. 4.1, introduction of infill panels in the RC frame reduces the lateral story displacement considerably.

Table 4.3. Comparisons of story displacement for different models

A. Story displacement of G+4 building

		Bare frame		Frame with 75% of the wall reduced		Frame with half of the wall reduced		Soft ground Story		Base infilled frame	
Story	Elevation	X-dir	Y- dir	X-dir	Y- dir	X-dir	Y- dir	X-dir	Y- dir	X-dir	Y- dir
	m	m	m	m	m	m	m	m	m	m	m
ROOF LEVEL	15	0.02191	0.02296	0.01443	0.01567	0.00797	0.00981	0.00902	0.00901	0.0062	0.00607
FOURTH	12	0.01812	0.01899	0.01237	0.01338	0.00723	0.00885	0.00861	0.00859	0.00579	0.00564
THIRD	9	0.01494	0.01532	0.01038	0.011	0.00625	0.00755	0.00789	0.00795	0.00508	0.00499
SECOND	6	0.01097	0.01132	0.00775	0.00825	0.00491	0.00592	0.00683	0.00705	0.00409	0.00409
FIRST	3	0.00673	0.00693	0.00485	0.00512	0.00345	0.00407	0.00546	0.0057	0.00294	0.00309
GROUND	0	0.00244	0.00251	0.00222	0.00227	0.00166	0.00195	0.00227	0.00237	0.00164	0.00177
Base	-3	0	0	0	0	0	0	0	0	0	0

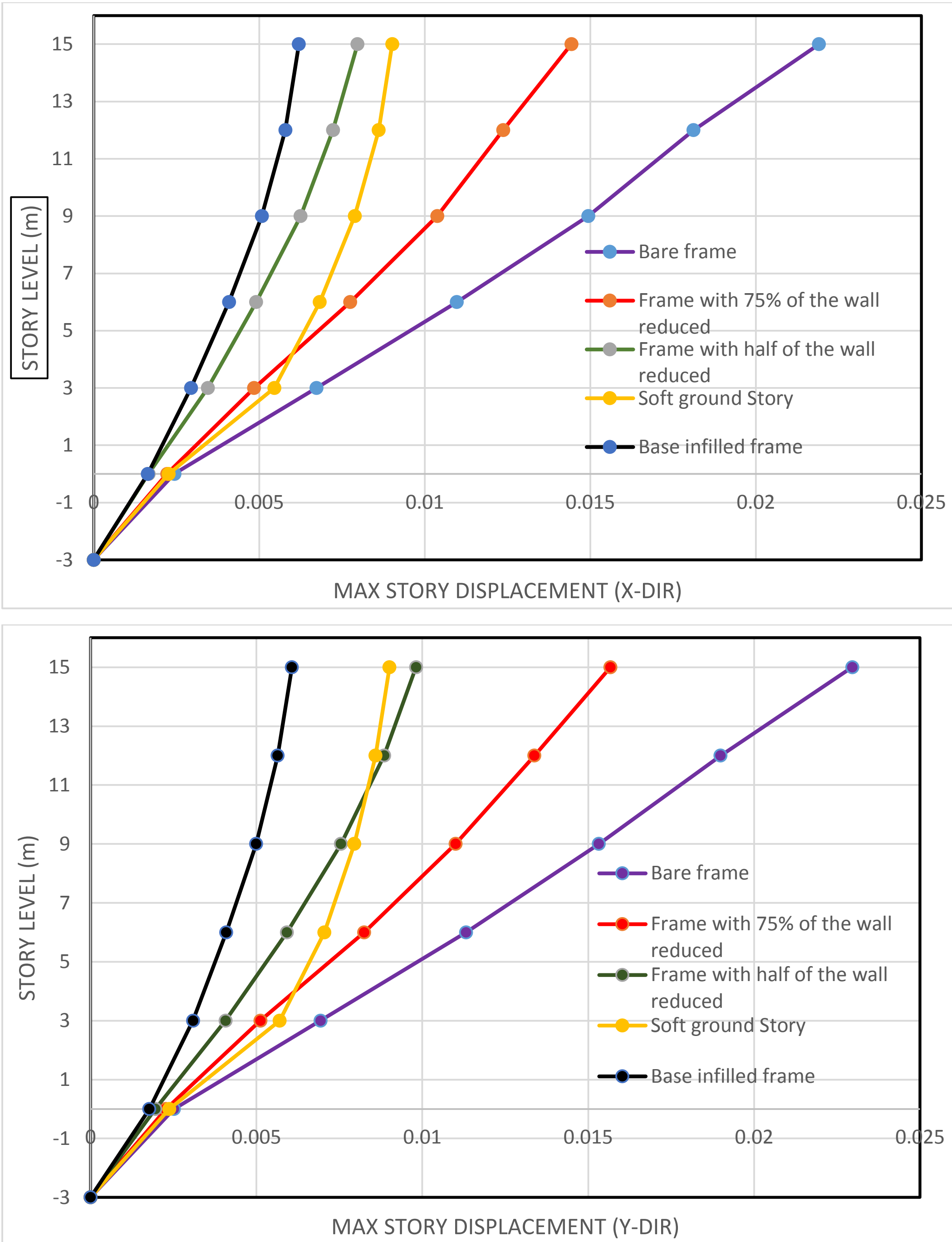


Figure.4.3A. Max story displacement along X-and Y-direction of G+4 buildings

Table 4.3 and fig. 4.3 show the comparative study of seismic demand in terms of lateral story displacement amongst all the four types of infilled model and the bare frame model of different building types. The lateral displacement obtained from the bare frame model is the maximum which is about71.7%, 76.94%, and 67.93% greater from basestructureof infilled frame for G+4, G+7 and G+10 buildings respectively; While the displacement reduction between the bare frame and the vertically soft ground story frame is 58.83%, 69.18% and 63.88% .And which is about 63.62%, 65.42% and 52.34% smaller in frame with half of the wall removed than that of the bare frame and nearly 34.14%, 40.31% and 28.22% smaller in frame with 75% of the wall removed model for G+4, G+7 and G+10 buildings respectively in the X-direction and by 73.56%, 78.57%, and 71.55% the displacement of the bare frame was greater than basestructure of infilled frame; While the displacement reduction between the bare frame and the soft ground story frame was 60.76%, 71.21% and 67.72% .And which is about 57.27%, 62.16% and 52.03% smaller in frame with half of the wall removed than that of the bare frame and nearly 31.75%, 40.14% and 32.50% smaller in frame with 75% of the wall removed model for G+4, G+7 and G+10 buildings respectively in the Y-direction.

B. Story displacement of G+7 building

		Bare frame		Frame with 75% of the wall reduced		Frame with half of the wall reduced		Soft ground Story		Base infilled frame	
Story	Elevation	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir
	m	m	m	m	m	m	m	m	m	m	m
ROOF LEVEL	24	0.04393	0.04728	0.02622	0.0283	0.01519	0.01789	0.01354	0.01361	0.01013	0.01013
SEVENRH	21	0.04096	0.04414	0.0247	0.02676	0.01422	0.01683	0.01324	0.01324	0.00983	0.00966
SIXTH	18	0.03539	0.03843	0.0219	0.02388	0.01296	0.01534	0.01259	0.0126	0.00918	0.00904
FIVTH	15	0.03027	0.03183	0.0191	0.02041	0.01151	0.01351	0.01174	0.01176	0.00833	0.00818
FOURTH	12	0.02463	0.0259	0.01591	0.01699	0.00979	0.01148	0.01063	0.01075	0.00723	0.0072
THIRD	9	0.01918	0.01968	0.01267	0.01325	0.008	0.00927	0.00942	0.0096	0.00603	0.00601
SECOND	6	0.01352	0.01397	0.00919	0.00964	0.00606	0.00699	0.00807	0.00835	0.00469	0.0048
FIRST	3	0.00808	0.00833	0.00599	0.00624	0.00415	0.00467	0.0065	0.00667	0.00342	0.00356
GROUND	0	0.00288	0.00296	0.00272	0.00278	0.00194	0.00226	0.00269	0.00275	0.00192	0.00203
Base	-3	0	0	0	0	0	0	0	0	0	0

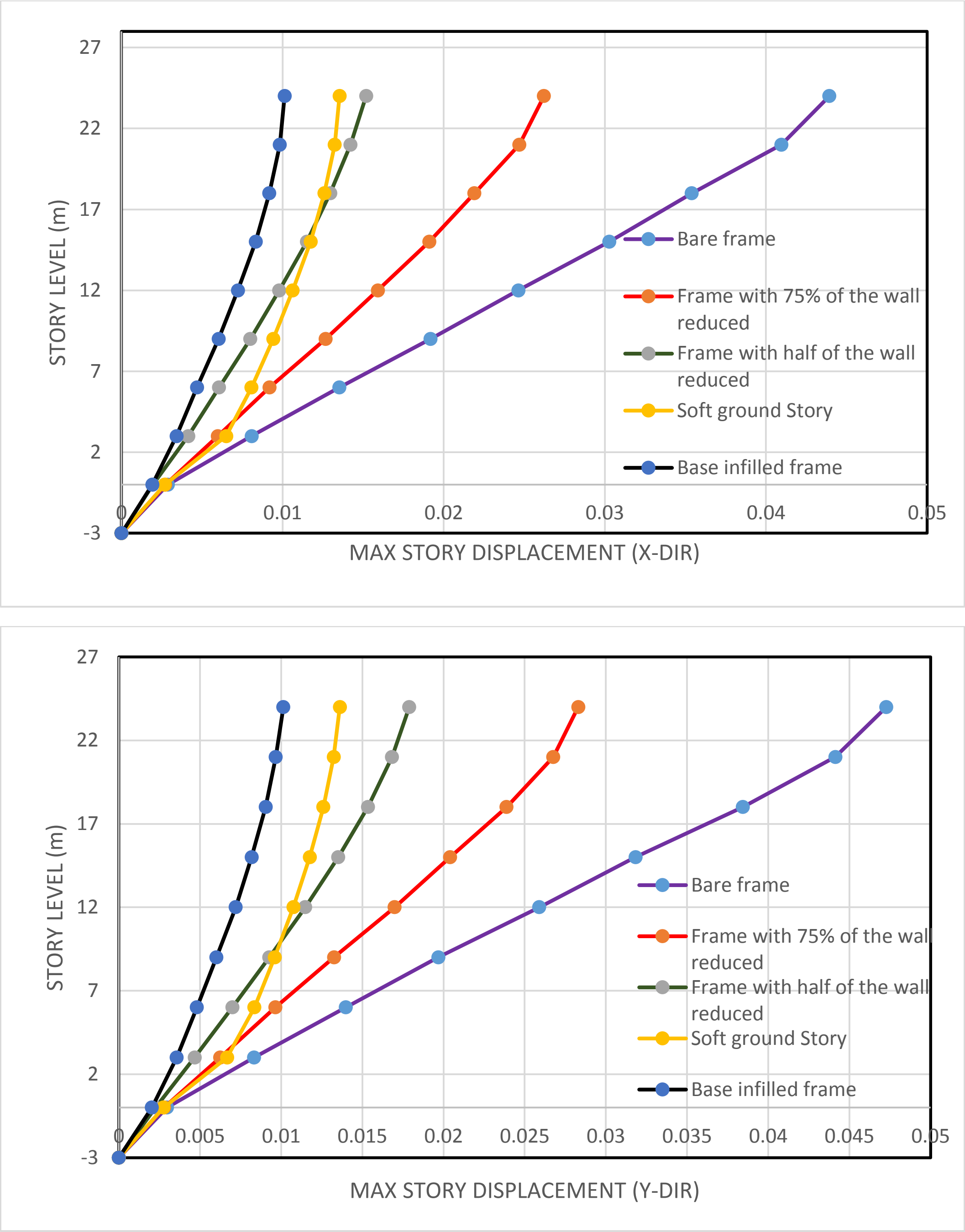
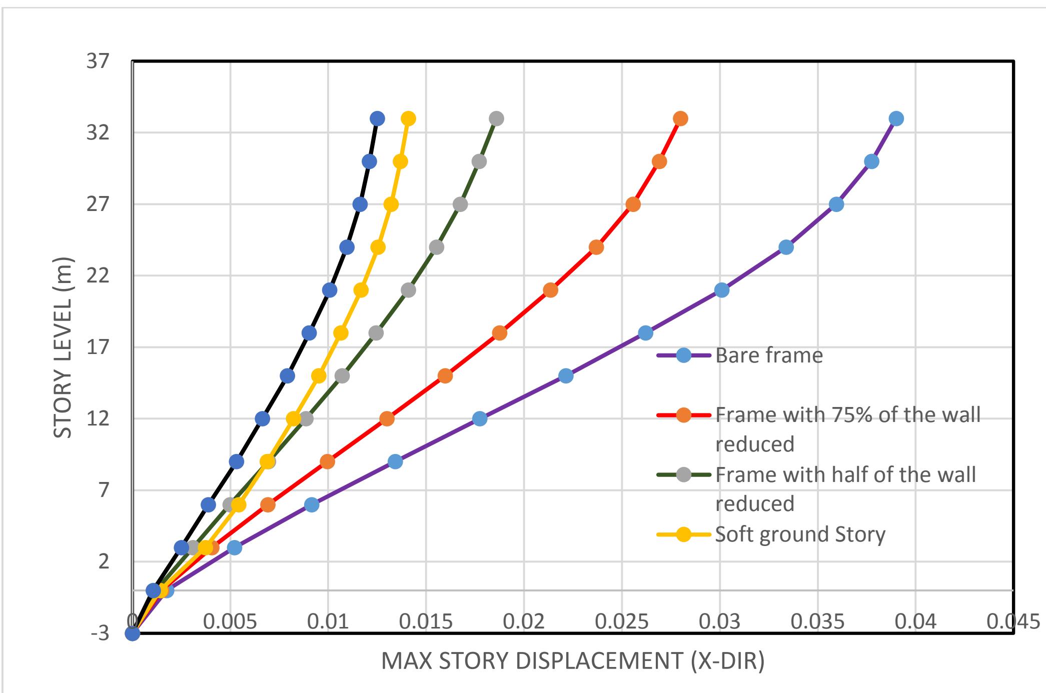


Figure.4.3B. Max story displacement along X-and Y-direction of G+7 buildings

C. *Story displacement of G+10 building*

		Bare frame		Frame with 75% of the wall reduced		Frame with half of the wall reduced		Soft ground Story		Base infilled frame	
Story	Elevatio	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir
	m	m	m	m	m	m	m	m	m	m	m
ROOF LEVEL	33	0.039	0.044	0.028	0.02982	0.01859	0.02119	0.01409	0.01426	0.01251	0.0126
TENTH	30	0.038	0.043	0.02691	0.02888	0.01771	0.02034	0.01369	0.01382	0.01211	0.0121
NINETH	27	0.036	0.041	0.02557	0.02754	0.01675	0.01933	0.01322	0.01334	0.01163	0.0116
EIGHTTH	24	0.033	0.038	0.0237	0.0256	0.01553	0.01799	0.01255	0.01266	0.01095	0.011
SEVENRH	21	0.03	0.034	0.02135	0.02308	0.01409	0.01631	0.01168	0.01179	0.01008	0.0101
SIXTH	18	0.026	0.029	0.01876	0.02036	0.01245	0.01446	0.01064	0.01079	0.00903	0.0091
FIVTH	15	0.022	0.025	0.01597	0.01731	0.01072	0.01243	0.00952	0.00966	0.00792	0.0079
FOURTH	12	0.018	0.02	0.013	0.01403	0.00886	0.01021	0.00823	0.0084	0.00664	0.0067
THIRD	9	0.013	0.015	0.00996	0.01065	0.00695	0.00794	0.00689	0.00704	0.00532	0.0054
SECOND	6	0.009	0.01	0.00692	0.0073	0.00499	0.00562	0.00544	0.00559	0.00388	0.004
FIRST	3	0.005	0.005	0.00406	0.00425	0.0031	0.0034	0.00374	0.00388	0.0025	0.0026
GROUND	0	0.002	0.002	0.00149	0.00154	0.0012	0.00131	0.00142	0.00146	0.00106	0.0011
Base	-3	0	0	0	0	0	0	0	0	0	0



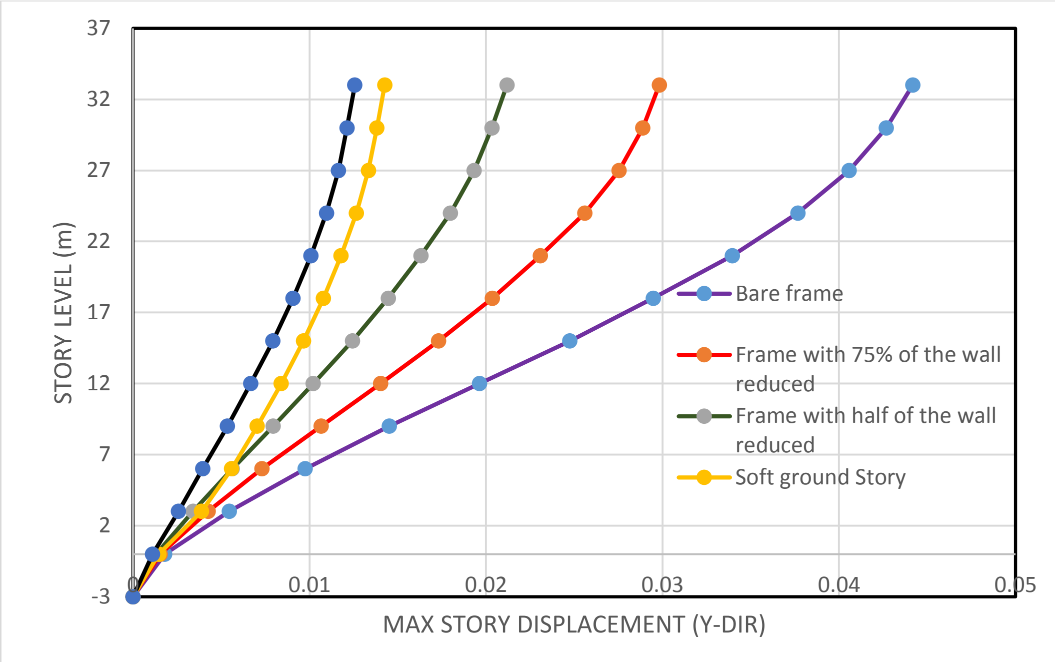


Figure.4.3C. Max story displacement along X-and Y-direction of G+10 buildings

From this observation, it is evident that masonry infills panel reduces the story displacement of RC moment resisting frame structure. However, frame with 75% of the wall reduced shows small displacement reduction compared to other infilled model and this is due to the absence of infills in the internal part and some from periphery of the building. And also, the absence of infills in a lower story, usually the bottom and most critical one (soft ground story), actually it has a significant influence in reducing fundamental period and story displacement. However, it has an adverse effect on the building, which is, damage is concentrated in the columns of the open story, at design acceleration levels this concentration is not much higher than in the bare structure, despite the fact that these columns had not been designed against the associated soft-story effect according to Sect. 2.9.3.2 of EC8, Part 1-3; only at excitation intensities much beyond the design level, some of the open-story columns do approach failure; The higher the strength and stiffness of infills in the upper storys, the more severe the concentration of deformations and damage in the open ground story

Thus, the infill panel reduces the seismic demand of a RC moment resisting frame structure. The lateral story displacement is dramatically reduced due to introduction of infill. This probably is the cause of building designed in conventional way behaving near elastically even during strong earthquake.

4.3.4 STORY SHEAR

Since, from the discussion or comparison of fundamental time period and base shear it is clear that the bare frame model is flexible structure than other models.

Table.4.4 Comparison of Story Shear for different Models

A. *Story shear for G+4 building*

G+4						
Story Shear						
Story	Elevation m	Bare Frame	Frame with 75% of the wall reduced	Frame with half of the wall	soft ground story	Base infilled frame
ROOF LEV	15	-342.876	-348.27277	-357.25302	-364.8438	-369.20077
		-342.876	-348.27277	-357.25302	-364.8438	-369.20077
FOURTH	12	-698.981	-713.02907	-736.4334	-758.56371	-767.62248
		-698.981	-713.02907	-736.4334	-758.56371	-767.62248
THIRD	9	-991.562	-1012.54366	-1047.50666	-1081.2011	-1094.11281
		-991.562	-1012.54366	-1047.50666	-1081.2011	-1094.11281
SECOND	6	-1214.9	-1241.08737	-1284.72922	-1327.0625	-1342.91033
		-1214.9	-1241.08737	-1284.72922	-1327.0625	-1342.91033
FIRST	3	-1364.96	-1394.62134	-1444.05203	-1484.2801	-1509.95343
		-1364.96	-1394.62134	-1444.05203	-1484.2801	-1509.95343
GROUND	0	-1378.95	-1409.43738	-1460.25595	-1498.2005	-1528.01405
		-1378.95	-1409.43738	-1460.25595	-1498.2005	-1528.01405
Base	-3	0	0	0	0	0
		0	0	0	0	0

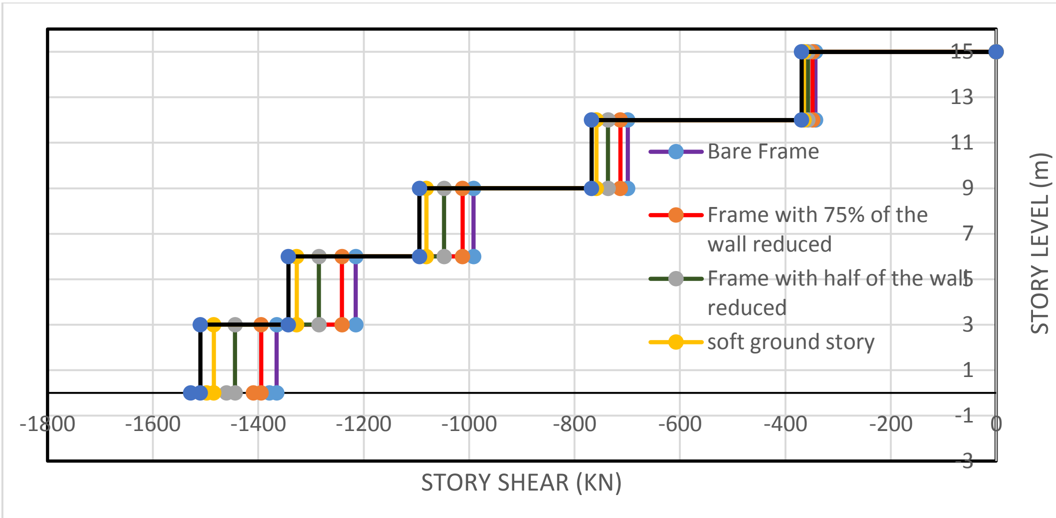


Figure.4.4A. Story Shear for G+4 Building

B. *Story shear for G+7 building*

G+7						
Story Shear						
Story	Elevation		Frame with	Frame with	soft ground	Base
	m	Bare Frame	75% of the	half of the	story	infilled
ROOF LEV	24	-272.64302	-276.79065	-283.23464	-290.30271	-292.88432
		-272.64302	-276.79065	-283.23464	-290.30271	-292.88432
SEVENRH	21	-571.1746	-582.49561	-600.10022	-620.97282	-626.49504
		-571.1746	-582.49561	-600.10022	-620.97282	-626.49504
SIXTH	18	-833.06143	-850.65987	-878.0311	-910.9782	-919.07939
		-833.06143	-850.65987	-878.0311	-910.9782	-919.07939
FIVTH	15	-1059.5155	-1082.4968	-1118.2425	-1161.526	-1171.8553
		-1059.5155	-1082.4968	-1118.2425	-1161.526	-1171.8553
FOURTH	12	-1250.1981	-1277.6671	-1320.3947	-1372.2788	-1384.4823
		-1250.1981	-1277.6671	-1320.3947	-1372.2788	-1384.4823
THIRD	9	-1405.2184	-1436.2802	-1484.5973	-1543.3453	-1557.0701
		-1405.2184	-1436.2802	-1484.5973	-1543.3453	-1557.0701
SECOND	6	-1523.5518	-1557.3106	-1609.823	-1673.7053	-1688.5893
		-1523.5518	-1557.3106	-1609.823	-1673.7053	-1688.5893
FIRST	3	-1603.0608	-1638.6182	-1693.9285	-1757.0631	-1776.8918
		-1603.0608	-1638.6182	-1693.9285	-1757.0631	-1776.8918
GROUND	0	-1610.4746	-1646.4682	-1702.4583	-1764.4473	-1786.4433
		-1610.4746	-1646.4682	-1702.4583	-1764.4473	-1786.4433
Base	-3	0	0	0	0	0

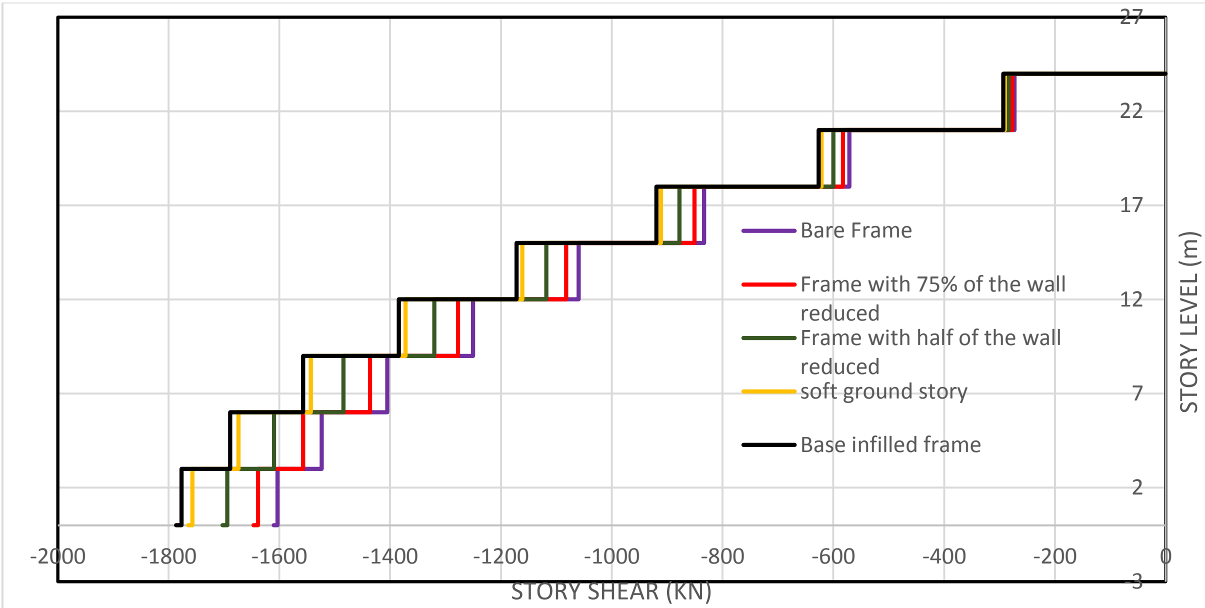


Figure.4.4B. Story Shear for G+7 Building

C. *Story shear for G+10 building*

G+10						
Story Shear						
Story	Elevation		Frame with	Frame with	soft ground	Base infilled
	m	Bare Frame	75% of the	half of the	story	frame
ROOF LEV	33	-220.37334	-223.80964	-229.15298	-235.59663	-237.16384
		-220.37334	-223.80964	-229.15298	-235.59663	-237.16384
TENTH	30	-501.70799	-511.39107	-526.45211	-545.41237	-549.0405
		-501.70799	-511.39107	-526.45211	-545.41237	-549.0405
NINETH	27	-757.46675	-772.82873	-796.72404	-827.06305	-832.56473
		-757.46675	-772.82873	-796.72404	-827.06305	-832.56473
EIGHTTH	24	-987.64964	-1008.12263	-1039.96878	-1080.54866	-1087.73655
		-987.64964	-1008.12263	-1039.96878	-1080.54866	-1087.73655
SEVENRH	21	-1192.8651	-1217.88147	-1256.79543	-1306.47481	-1315.16558
		-1192.8651	-1217.88147	-1256.79543	-1306.47481	-1315.16558
SIXTH	18	-1374.74306	-1403.73588	-1448.83565	-1506.46383	-1516.48494
		-1374.74306	-1403.73588	-1448.83565	-1506.46383	-1516.48494
FIVTH	15	-1532.60553	-1565.0076	-1615.41075	-1679.84091	-1691.01534
		-1532.60553	-1565.0076	-1615.41075	-1679.84091	-1691.01534
FOURTH	12	-1666.09308	-1701.33703	-1756.16089	-1826.24828	-1838.39662
		-1666.09308	-1701.33703	-1756.16089	-1826.24828	-1838.39662
THIRD	9	-1774.55713	-1812.07529	-1870.43681	-1945.04037	-1957.97893
		-1774.55713	-1812.07529	-1870.43681	-1945.04037	-1957.97893
SECOND	6	-1857.2612	-1896.48562	-1957.50124	-2035.48417	-2049.02436
		-1857.2612	-1896.48562	-1957.50124	-2035.48417	-2049.02436
FIRST	3	-1913.3007	-1953.66302	-2016.44861	-2093.97089	-2110.62654
		-1913.3007	-1953.66302	-2016.44861	-2093.97089	-2110.62654
GROUND	0	-1920.09773	-1960.74139	-2023.96487	-2100.73624	-2118.80009
		-1920.09773	-1960.74139	-2023.96487	-2100.73624	-2118.80009
Base	-3	0	0	0	0	0

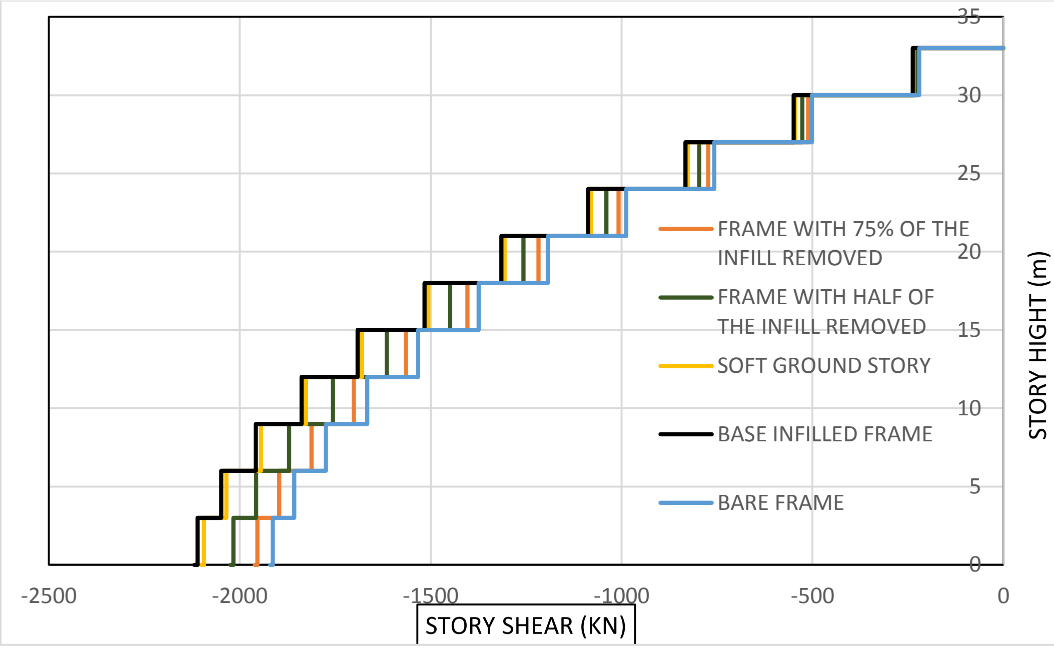


Figure.4.4C. Story Shear for G+10 Building

The story shear calculated on the basis of bare frame model gave a lesser value than the other infilled frames; It was observed that the story shear in base infilled Frame is nearly 7.13% greater compared to bare frame model and was nearly 6.46%, 3.83% and 1.53 % in soft ground story frame, frame with half of the wall removed and frame with 75% of the wall removed compared to bare frame.

4.3.5MEMBER FORCES

Next, the effect of infill on the member forces in beams and columns were studied. In general compared to bare frame model, the infill models predicted higher axial forces in columns but lower shear forces and bending moments in both beams and columns. Thus, the effect of infill panel is to change the predominantly a frame action of a moment resisting frame system towards truss action. Generally, for the bottom floors where the axial force is large, base structure of infilled frames showed around 30%

increase in axial force. The other infill models showed a lesser increase. The effect of infill on frame is to reduce the shear force and bending moments.

Similarly in the case of beam, the effect of infill is to reduce the shear force as well as bending moment when subjected to seismic loading. The base structure of infilled frame predicted about 35 % of the bare frame model whereas; the other infill models showed a lesser increase.

The Pauley & Priestley model gives the effective width. The larger effective strut width yield more rigid frame, less time period and thus more lateral force from earthquake analysis. However, this large force, when applied to structure, still produce less lateral displacement and member forces since the increased stiffness has larger effect than corresponding increased forces. So it is not always safe to assume larger value of strut width. However, as seen from the verification model and the comparisons of fundamental time period, base shear, lateral displacement and story shear Pauley & Priestley model is the most realistic one. In general, the effect of infill panel is to reduce the seismic demand of a building structure in terms of lateral displacement as well.

As expected, the infill has a better response during earthquake excitation. In the case of column, bare frame model predicts the maximum moment of about 426.7kN-m at the bottom floor which is reduced by about 17%, 22%, 27% and 40% kN-m in the case of infill model in the corresponding trend. However, with the increase in the story level, the bare frame model predicts gradual decrease in bending moment, so does the infill model. The percentage reduction in bending moment predicted by infill model to that by bare frame model reduces gradually to about 35% at the upper floors. For the top two floors there is almost no reduction in the bending moment. This is true for all the cases of openings and position of columns in the building. In the case of both these columns, there is a large reduction in bending moment predicted by the infill model; but the reduction remains almost the same throughout the floor. Even in these columns, there is no reduction in bending moments at the top floor. This discussion is true even for the shear force. The pattern of reduction of shear force in all columns as predicted by infill model to that by the bare frame model is same as that of bending moment.

Thus, from above discussion, it is quite clear that the effect of infill on frame is to reduce the shear force and bending moments. In general for all columns, both shear force and bending moments are reduced. At the lower floors the reduction is more than 40%, which decrease to about 35% in the case of corner columns but remains about the same for edge and middle column even at the upper floors. One typical fact is that at the top most floors the member force does not decrease. This might be the case of further research to verify the effect of infill on taller structure.

CHAPTER FIVE

5.1 CONCLUSIONS AND RECOMMENDATIONS

5.1.1 Conclusion

The HCBmasonry infill is modeled as a diagonal strut member whose thickness is same as that of the masonry and the length is equal to the diagonal length between compression corners of the frame. The effective width of the diagonal strut depends on diagonal length of the infill. Various researchers had proposed different strut width, however in the present study the effective width as suggested by Pauley and Priestleywere considered initially. Since, Pauley &Priestley suggested effective width seems to agree closer to the experimental case considered in the study, this was used.

This thesis work is towards the understanding of the effect of infill wall. Introducing infill panels in the RC frame reduces the time period of bare frames and also enhances the stiffness of the structure. Bare frame idealization leads to overestimation of natural periods and under estimation of the design lateral forces. The trend in the analysis taken to be bare frame, frame with 75% of infill wall removed from base structure, frame with half of infill wall removed from base structure, soft ground story and base structureof infilled frame.The main conclusions are summarized below:

1. The results obtained for fundamental natural period indicate that analytical natural period is greater than the natural periods obtained from the empirical expression of the code in all the corresponding trends. The natural period for empirical analysis decrease in the trend for G+4, G+7 and G+10, similar sequence is also observed in the analytical analysis for G+4, G+7 and G+10.

2. The bare frame models does not takes in to account the stiffness rendered by the infill panel, it gives significantly longer time period. And hence smaller lateral forces. This is due to the fact that the fundamental time period of a structure depends not only on the mass of a structure but also on the stiffness of the structure. And when the infill is modeled, the structure becomes much stiffer than the bare frame model. EC8 and EBCS 8 bases the calculation of seismic actions for infilled frames on the average periods T1'.

3. It is evident that masonry infills panel reduces the story displacement of RC moment resisting frame structure. The absence of infills in a lower story, usually the bottom has a significant influence in reducing fundamental period and story displacement by forming soft story as it has been observed in number of earthquakes.

4. Verification of EBCS 8 section 3.9.4 for themodificationof natural period caused by addition of infillsgives almost a close result with that of analytical result. This shows that the empirical formula used for the modification of the fundamental period can be effectively used in the analysis. The reduction of the fundamental period usually implies higher base shear as shown in the next section.

5. The effect of infill wall is to change the predominantly a frame action of a moment resisting frame structure towards a truss action and changing the lateral load transfer mechanism. Thus, axial forces in columns are increased in infill frame model compared to a bare frame model. And it is quite clear that the effect of infill on frame is to reduce the shear force and bending moments. In general for all columns, both shear force and bending moments are reduced. At the lower floors the reduction is more than 40%, which decrease to about 35% in the case of corner columns but remains about the same for edge and middle column even at the upper floors. One typical fact is that at the top most floors the member force does not decrease.

5.1.2 Recommendations

1. The additional stiffness contributed by these infill increases the overall stiffness of the building, which eventually leads to shorter time period. With further study this may lead to a practical way to determine the fundamental period of RC frames using rational approaches like modal analysis, and eliminate the necessity of imposing code limits.
2. Since, codes give an empirical value to compute the natural period which depends upon height and width only, further study could be done to find the effect of span length, number of span, stiffness of beam and columns etc.
3. The present study was carried out using linear elastic analysis method for the seismic analysis. This could be extended to nonlinear static and dynamic analysis to cater for the structure with horizontal aswell as vertical irregularity.
4. Further study on partial infill with openings at various locations could lead to valuable information regarding the practical aspect of design work.
5. The present study was done based on the strut width suggested by Pauley &Priestley. Many researchers had recommended different strut width to replace infill panel. The study could be extended to more strut width and compared with experimental result to find out the most suitable one.
6. The study can be extended to a building frame with greater number of story to see the effect of infill panels on tall structure during seismic excitation.
7. The macro modeling approach used here takes into account only the equivalent global behavior of the infill in the analysis. As a result, the approach doesnot permit study of local effects such as frame-infill interaction within the individual infilled frame subassemblies. More detailed micro-modeling approaches need to be used to capture the local conditions within

the infill. Thus, further studies should be conducted to develop design guidelines for engineered infill.

REFERENCES

1. T. Paulay and M.I.N. Priestly, -- Seismic design of reinforced concrete and masonry Buildings
2. T. Elouali, --Effect of infill masonry panels on the seismic response of frame buildings.
3. Mohd Danish et.al, --Analysis of Masonry Infilled R.C.Frame with & without Opening Including Soft Storey by using "Equivalent Diagonal Strut Method" International Journal of Scientific and Research Publications, Volume 3, Issue 9, September 2013 ISSN 2250-3153
4. Wael W. El-Dakhakhni, et al, --Strength and stiffness prediction of masonry infill panels. World Conference on Earthquake Engineering
5. Md Irfanullah1,et al, -- Seismic evaluation of RC framed buildings with influence of masonry infill panel.International Journal of Research in Engineering and Technology eISSN: 2319-1163
6. EN 1998-1-1:2004, ―Eurocode 8: Design of structures for earthquake resistance -Part 1: General rules, seismic actions and rules for buildings.
7. EBCS 8, --Design of structures for earthquake resistance
8. EBCS 1, -- Basis of design and actions on structure
9. EN 1992-1-1:2004, ―Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings,. .
10. EBCS 2, -- Structural use of concrete
11. Mehmet selim öztürk, -- Effects of masonry infill walls on the seismiperformance of buildings
12. Goutam Mondal, -- Lateral Stiffness of Masonry Infilled Reinforced Concrete (RC) Frames with Central Opening
13. Michael N Fardis, -- Design provisions for masonry-infilled RC frames World Conference on Earthquake Engineering

14. Andreas J Kappos and Frederick Ellul, --seismic design and performance assessment of masonryinfilled r/c frames. World Conference on Earthquake Engineering
15. Robin Davis, et al --effect of infill stiffness on seismic performance of multi-story RC framed buildings in India. World Conference on Earthquake Engineering.
16. Hemant B. Kaushik, et al --A rational approach to analytical modeling of masonry infills in reinforced concrete frame buildings. World Conference on Earthquake Engineering
17. FEMA 306. 1998, -- Evaluation of Earthquake Damaged Concrete and Masonry.
18. Holmes, M. 1961, -- Steel Frames with Brickwork and Concrete Infilling. Proceedings of Institution. Of Civil Engineers London, Vol 19: 473-478.
19. Mekonnen Degefa, --Response of masonry infilled RC frame under horizontal seismic force
20. Mulgund G. V, -- Seismic assessment of RC frame buildings with brick masonry infills.International journal of advanced engineering sciences and technologies
21. Smith, B.S. and A. Coull. 1991, -- Tall Building Structures: Analysis and Design.
22. Sachin R Patel,Sumant B patel, Effect of Brick Infill Panel in Design of High Rise Building.National Conference on Recent Trends in Engineering & Technology
23. Salah El-Din Fahmy Taher*, et al, --Role of masonry infill in seismic resistance of rc structure.

24. Diana M. Samoilă*1, --Analytical Modelling of Masonry Infills Journal homepage: http://constructii.utcluj.ro/ActaCivilEngIOSR Journal of Engineering
25. Neelima Patnala VS, Pradeep Kumar Ramancharla, --Seismic Behaviour of RC Frame with URM Infill: A Case Study. International Journal of Education and applied research.
26. Kashif Mahmud et al, --Study the Reinforced Concrete Frame with Brick Masonry Infill due to Lateral Loads.International Journal of Civil & Environmental Engineering IJCEE-IJENS Vol:10 No:04
27. Madan, A., A.M. Reinhorn, J.B.Mander and R.E.Valles. 1997. Modeling of Infill Panels for Structural Analysis. Journal of Structural Engineering, ASCE, 123 (10): 1295-1302.
28. Moghaddam, H.A. and P.J. Dowling. 1987. The State of the Art in Infilled Frames. ESEE Research Report No. 87-2, Imperial College of Science and Technology, Civil Engineering Department, London, U.K
29. Murthy, C.V.R. and S.K. Jain. 2000. Beneficial influence of masonry infills on seismic performance of RC frame buildings. Proceedings of 12thWorld Conference on Earthquake Engineering, New Zealand, Paper No. 1790
30. Goutam Mondal and Sudhir K. Jain, M.EERI, Lateral Stiffness of Masonry InfilledReinforced Concrete (RC) Frameswith Central Opening. Department of Civil Engineering, Indian Institute of Technology Kanpur, India 208016
31. Sumat Shrestha (2005). Interaction between RC Frames & Brick Masonry Infill Wall. M.S Thesis, Tribhuvan University, Institute of Engineering, Pulchowk Campus.

APPENDICES

Appendix A

Verification of strut width

VERIFICATION

This is to verify that, an infill wall can be modelled as an equivalent diagonal compression strut in the building model. Thus in order to have only the diagonal compression strut, the diagonal tension struts were set to tension limit zero. The properties and parameters used in the verified model is the one verified by [Sumat Shrestha (2005)] [31], discussed in section 2.13 of chapter 2. Which is :-

Section	Cross Section (mm*mm)	center line dimension (mm)	Comp. Strength f'_c (Mpa)	Young's Modulus (Mpa)	Poisons' Ratio	Long Reinf.(f_y = 248 MPa
Beam	75*75	928	7.93	12500	0.15	4-4.74mm
Column	75*75	910	7.93	12500	0.15	4-4.75mm
Infill	832*853	1300	-	225	0.17	-

Comparison of the output of this study and Sumat Shresta are summarized in table A1.

Appendix Table A1. Verification of strut width in terms of defelection

Defelection (m) From Thesis (A)	Defelection (m) FromSumat Shrestha (B)	Ratio(A/B)
0.0123	0.0122	1.0082
0.01165	0.01155	1.0087
0.011	0.0109	1.0092
0.01035	0.01025	1.0098
0.0097	0.0096	1.0104
0.00905	0.00895	1.0112
0.00841	0.00831	1.0120
0.00776	0.00766	1.0131
0.00711	0.00701	1.0143
0.00646	0.00636	1.0157
0.00582	0.00572	1.0175
0.00517	0.00507	1.0197
0.00452	0.00442	1.0226
0.00388	0.00378	1.0265
0.00323	0.00313	1.0319
0.00258	0.00248	1.0403
0.00194	0.00184	1.0543
0.0013	0.0012	1.0833
0.00064	0.00054	1.1852
0	0	0.0000

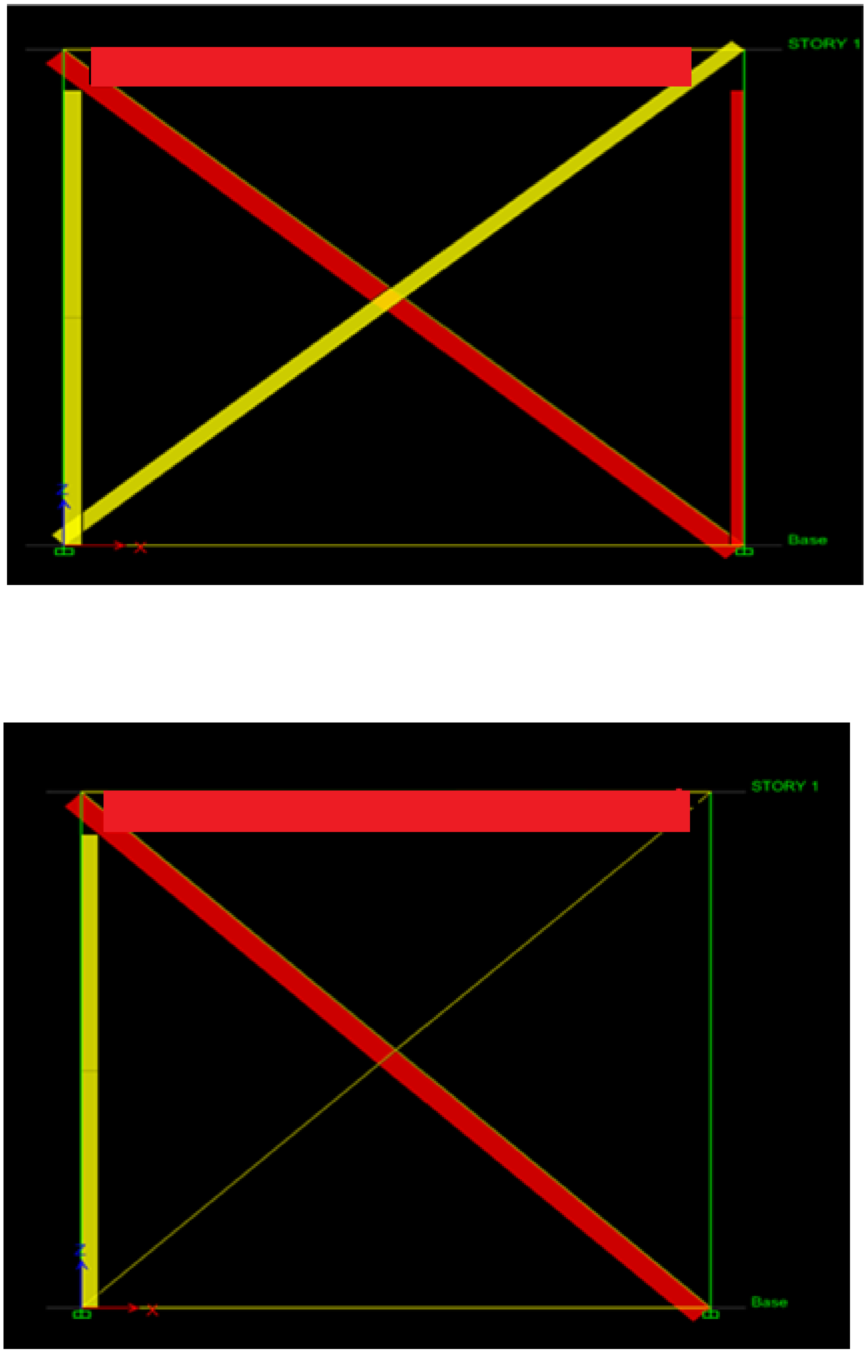


Figure. A1. Verification of tension limit validation

As shown in fig. A1 before the diagonal strut assigned to tension limit zero the diagonal member which goes from left of the frame base to the right of the frame first story were axially tensioned. But, after it has been assigned to tension limit zero, the member doesn't take tension force as shown in fig.

Appendix B

Effective width Calculation of diagonal strut

EFFECTIVE WIDTH OF DIAGONAL STRUT

It is usual practice to provide masonry infill in a moment resisting frame as exterior walls, partitions, and walls around stair, elevator and service shafts and hence treated as nonstructural elements. But it has been recognized by many studies that it also serve structurally to brace the frame against horizontal loading. It has been stated that the use of masonry infill is to brace a frame and combines some of the desirable structural characteristics of each, while overcoming some of their deficiencies. When the frame is subjected to lateral loading, the translation of the upper part of the column in each story and the shortening of the leading diagonal of the frame cause the column to lean against the wall as well as compress the wall along its diagonal. This is analogous to a diagonally braced frame. Thus to model an infilled frame, the masonry panel is replaced by an equivalent diagonal strut whose thickness is same as that of the masonry panel and the length is the diagonal length of the compression side of the panel.

Pauley and Priestley (1992) suggested that the effective width shall be one-fourth the diagonal length which relates the width w of infill to parameter d (length of diagonal strut) and given by equation (B1).

$$w = \frac{d}{4} \qquad B1$$

And the reduction factor for the infilled frame with opening can be simplified as

$$\rho_w = 1 - 2.6\alpha_{co} \qquad B2$$

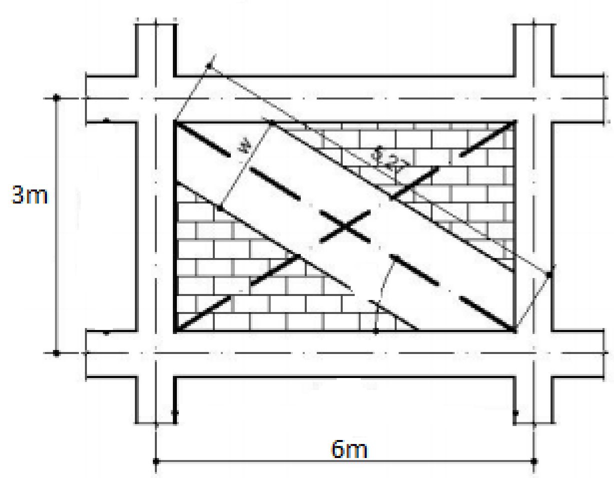
Where:- ρ_w - Strut − width reduction factor

α_{co} -Opening- Area- Ratio

$$\alpha_{co} = \frac{Area\ of\ opening\ A_{op}}{Area\ of\ infill\ A_{infill}}$$

Thus, the effective width as proposed by Pauley & Priestley can be found by just knowing the diagonal length whereas for. The effective widths calculated are shown in *Appendix Table B1. Calculation of strut width*

	Strut Crossection		
	Length	Thickeness	Width
WS1 (with out opening)	6.708204	0.15	1.68
WS2(with window opening of 1.5m*2m)	6.708204	0.2	0.9521
WS3 (with door opening of 2m*25m)	6.708204	0.15	0.4666
WS4 (with door opening of 1.2m*2.5m)	6.708204	0.15	0.9521
WS5 (with door opening of 1m*2.5m)	6.708204	0.15	1.073



Appendix Figure B1 Diagonal length and diagonal angle

Appendix C

Calculation of fundamental period and Base Shear

Appendix Table C1. Calculation of Fundamental Period

C1.1

G+4														
Types of Models	*Aw(m2)*	*G(Mpa)*	*g(m/sec 2)*	*H(m)*	*W(KN)*	*T1b (sec)*	*T1b2*	*T1b2*Aw*G*g*	*16*H*W*	*T1b2*Aw*G*g/16*H*W*	*1+(T1b2*Aw*G*g/16*H*W)*	*T1i(sec)*	*T1'=(T1b+T1i)/2*
Bare Frame	0.000	701.500	9.980	15.000	15235.390	0.572	0.327	0.000	3656493.600	0.000	1.000	0.572	0.572
Frame with 75% of the wall reduced	855.000	701.500	9.980	15.000	16322.117	0.572	0.327	1956067.092	3917307.970	0.499	1.499	0.381	0.476
Frame with half of the wall reduced	1907.500	701.500	9.980	15.000	18133.332	0.572	0.327	4363974.243	4351999.584	1.003	2.003	0.285	0.429
Building With Soft Story	2670.000	701.500	9.980	15.000	19485.706	0.572	0.327	6108420.041	4676569.322	1.306	2.306	0.248	0.410
Base infilled Frame	3472.500	701.500	9.980	15.000	20548.285	0.572	0.327	7944377.751	4931588.402	1.611	2.611	0.219	0.395

C1.2

G+7													
Types of Models	*Aw(m2)*	*G(Mpa)*	*g(m/sec 2)*	*H(m)*	*W(KN)*	*T1b (sec)*	*T1b2*	*T1b2*Aw*G*g*	*16*H*W*	*T1b2*Aw*G*g/16*H*W*	*1+(T1b2*Aw*G*g/16*H*W)*	*T1i(sec)*	*T1'=(T1b+T1i)/2*
Bare Frame	0.000	701.500	9.980	24.000	22451.138	0.813	0.661	0.000	8621236.800	0.000	1.000	0.813	0.813
Frame with 75% of the wall reduced	1368.000	701.500	9.980	24.000	24189.904	0.813	0.661	6334082.321	9288923.121	0.682	1.682	0.484	0.648
Frame with half of the wall reduced	3052.000	701.500	9.980	24.000	26894.652	0.813	0.661	14131300.617	10327546.368	1.368	2.368	0.343	0.578
Building With Soft Story	4861.500	701.500	9.980	24.000	29889.190	0.813	0.661	22509606.143	11477448.960	1.961	2.961	0.275	0.544
Base infilled Frame	5556.000	701.500	9.980	24.000	30951.774	0.813	0.661	25725264.163	11885481.024	2.164	3.164	0.257	0.535

C1.3

G+10													
Types of Models	*Aw(m2)*	*G(Mpa)*	*g(m/sec 2)*	*H(m)*	*W(KN)*	*T1b (sec)*	*T1b2*	*T1b2*Aw*G*g*	*16*H*W*	*T1b2*Aw*G*g/16*H*W*	*1+(T1b2*Aw*G*g/16*H*W)*	*T1i(sec)*	*T1'=(T1b+T1i)/2*
Bare Frame	0.000	701.500	9.980	33.000	37234.050	1.033	1.066	0.000	19659578.400	0.000	1.000	1.033	1.033
Frame with 75% of the wall reduced	1665.000	701.500	9.980	33.000	39624.854	1.033	1.066	12429850.096	20921922.806	0.594	1.594	0.648	0.840
Frame with half of the wall reduced	4196.500	701.500	9.980	33.000	43343.882	1.033	1.066	31328448.006	22885569.770	1.369	2.369	0.436	0.734
Building With Soft Story	6845.000	701.500	9.980	33.000	47859.845	1.033	1.066	51100494.841	25269998.176	2.022	3.022	0.342	0.687
Base infilled Frame	7639.500	701.500	9.980	33.000	48922.425	1.033	1.066	57031735.622	25831040.136	2.208	3.208	0.322	0.677

Appendix Table B2. Calculation of Base Shear

C2.1

G+4																						
Types of Models	Story Hight H(m)	Total wight of the building W(KN)	T1b (sec)	Emperical result T1'=(T1b+T1i)/2	Analytical result (T1')	3//4	T1'^3/4 for Emperical tesult	T1' ^3/4for analytical result	α	S	β for Emperical	β for Analytical	ϒo	KD	KR	KW	ϒ= ϒo kD kR kw<=0.7	Sd(T1')using emperical result	Sd(T1')using analytical result	Fb = Sd(T1') using emperical result	Fb = Sd(T1') using analytical result
Bare Frame	15	15235.39	0.571649342	0.571649	0.823380	0.750000	0.6574	0.8644	0.12	1.20	2.190359	1.665951	0.20	1.50	1.0	1.0	0.30	0.078852908	0.059974241	1201.3548	913.7309454
Frame with 75% of the wall reduced	15	16322.11654	0.571649342	0.476458	0.685560	0.750000	0.5735	0.7534	0.12	1.20	2.510983	1.911298	0.20	1.50	1.0	1.0	0.30	0.090395387	0.068806725	1475.444036	1123.071392
Frame with half of the wall reduced	15	18133.3316	0.571649342	0.428541	0.617180	0.750000	0.5297	0.6963	0.12	1.20	2.718745	2.068015	0.20	1.50	1.0	1.0	0.30	0.097874818	0.074448527	1774.796521	1349.999828
Building With Soft Story	15	19485.70551	0.571649342	0.409763	0.587660	0.750000	0.5122	0.6712	0.12	1.20	2.811658	2.145448	0.20	1.50	1.0	1.0	0.30	0.1012197	0.07723611	1972.337262	1505.000102
Base infilled Frame	15	20548.28501	0.571649342	0.395298	0.568070	0.750000	0.4985	0.6543	0.12	1.20	2.888480	2.200701	0.20	1.50	1.0	1.0	0.30	0.10398528	0.079225247	2136.719172	1627.942955

C2.2

G+7																						
Types of Models	*Story Hight H(m)*	*Total wight of the building W(KN)*	*T1b (sec)*	*Emperical result T1'=(T1b+T1i)/2*	*Analytical result (T1')*	*3//4*	*T1'^3/4 for Emperical tesult*	*T1' ^3/4for analytical result*	*α*	*S*	*β for Emperical*	*β for Analytical*	*ϒo*	*KD*	*KR*	*KW*	*ϒ= ϒo kD kR kw<=0.7*	*Sd(T1')using emperical result*	*Sd(T1')using analytical result*	*Fb = Sd(T1') using emperical result*	*Fb = Sd(T1') using analytical result*
Bare Frame	24	22451.1375	0.813241803	0.813000	1.122260	0.750000	0.8562	1.0904	0.12	1.20	1.681878	1.320664	0.20	1.50	1.0	1.0	0.30	0.060547621	0.047543908	1359.362964	1067.414823
Frame with 75% of the wall reduced	24	24189.90396	0.813241803	0.648000	0.992510	0.750000	0.7222	0.9944	0.12	1.20	1.993798	1.448143	0.20	1.50	1.0	1.0	0.30	0.071776741	0.052133133	1736.272483	1261.095479
Frame with half of the wall reduced	24	26894.652	0.813241803	0.578000	0.794940	0.750000	0.6629	0.8419	0.12	1.20	2.172284	1.710455	0.20	1.50	1.0	1.0	0.30	0.078202226	0.061576391	2103.221648	1656.075603
Building With Soft Story	24	29889.19	0.813241803	0.544000	0.727610	0.750000	0.6334	0.7878	0.12	1.20	2.273334	1.827841	0.20	1.50	1.0	1.0	0.30	0.081840037	0.065802289	2446.132421	1966.777106
Base infilled Frame	24	30951.7735	0.813241803	0.535000	0.698530	0.750000	0.6256	0.7641	0.12	1.20	2.301957	1.884620	0.20	1.50	1.0	1.0	0.30	0.082870442	0.067846304	2564.987157	2099.963443

C2.3

G+10																					
Types of Models	*Story Hight H(m)*	*Total wight of the building W(KN)*	*T1b (sec)*	*Emperical result T1'=(T1b+T1i)/2*	*Analytical result (T1')*	*3//4*	*T1'^3/4 for Emperical tesult*	*T1' ^3/4for analytical result*	*α*	*S*	*β for Emperical*	*β for Analytical*	*Υo*	*KD*	*KR*	*KW*	*Υ= Yo kD kR kw<=0.7*	*Sd(T1')using emperical result*	*Sd(T1')using analytical result*	*Fb = Sd(T1') using emperical result*	*Fb = Sd(T1') using analytical result*
Bare Frame	33	37234.05	1.032634709	1.033000	1.462650	0.750000	1.0246	1.3300	0.12	1.20	1.405359	1.082698	0.20	1.50	1.0	1.0	0.30	0.050592921	0.038977114	1883.779339	1451.275818
Frame with 75% of the wall reduced	33	39624.8538	1.032634709	0.840000	1.156470	0.750000	0.8774	1.1152	0.12	1.20	1.641168	1.291254	0.20	1.50	1.0	1.0	0.30	0.059082046	0.046485148	2341.117445	1841.967178
Frame with half of the wall reduced	33	43343.88214	1.032634709	0.734000	1.008710	0.750000	0.7930	1.0065	0.12	1.20	1.815894	1.430664	0.20	1.50	1.0	1.0	0.30	0.065372177	0.051503916	2833.483935	2232.37965
Building With Soft Story	33	47859.84503	1.032634709	0.687000	0.912690	0.750000	0.7546	0.9338	0.12	1.20	1.908292	1.542127	0.20	1.50	1.0	1.0	0.30	0.068698529	0.055516557	3287.900968	2657.013811
Base infilled Frame	33	48922.4245	1.032634709	0.677000	0.897760	0.750000	0.7463	0.9223	0.12	1.20	1.929394	1.561321	0.20	1.50	1.0	1.0	0.30	0.069458195	0.056207569	3398.063287	2749.810564