THESIS

EFFECT OF UNREINFORCED FULL AND PARTIAL INFILLED BRICK MASONRY WALL IN RC FRAME UNDER SEISMIC LOADING

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GRADUATE SCHOOL, KASETSART UNIVERSITY

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THESIS

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A Thesis Submitted in Partial Fulfillment of
the Requirements for the Degree of
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Masonry infill panels in framed structures affect the strength, stiffness, and ductility. Being a stiffer component, it attracts larger part of the lateral seismic shear force on the building and hence reduces the demand on the reinforced concrete frame members. However, the behavior of infill is not easy to predict because of its inherent brittle nature and variable material property and hence are treated as a non-structural component in analysis and design of a frame structure.

For seismic loading, ignoring the composite action is not always on the safe side, since the interaction between the panel and the frame under lateral loads dramatically changes the stiffness and the dynamic characteristics of the composite structure, and hence, its response to seismic loads. The influence of brick masonry infill panels on seismic performance of reinforced concrete (RC) frames that were designed in accordance with the current seismic code IS1893:2002 is studied. Equivalent diagonal strut is used to model the stiffness effect of the masonry panels. The response of a bare frame is compared with the full and partial infill with centrally located opening of 10%, 20%, 30%, 40% and 50%. In general, axial force in the column is increased whereas; the shear forces and bending moments in columns and beams are decreased by the presence of infill panels. When subjected to lateral loadings, the frame with full infill has better response whereas; infill with large openings has little effect.

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Binay Charan Shrestha

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LIST OF ABBREVIATIONS

ASCE	=	American Society of Civil Engineers
DL	=	Dead load
EQx	=	Earthquake load in X-direction
FEA	=	Finite element analysis
FEMA	=	Federal Emergency Management Agency
IS	=	Indian Standards
LL	=	Live load
NBC	=	Nepal Nation Building Code
RC	=	Reinforced Concrete

EFFECT OF UNREINFORCED FULL AND PARTIAL INFILLED BRICK MASONRY WALL IN RC FRAME UNDER SEISMIC LOADING

INTRODUCTION

The recent trend of building construction in urban and semi-urban area of Nepal, like several other countries around 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 in cement mortar. These walls are built after the frame is constructed and used as cladding or as partition. Typically, 230 mm and 115 mm thick infill are used. Due to functional demand, openings for doors, windows etc. are rather a norm than an exception in these walls (Figure 1).

Figure 1 A Building with Full and Partial Infill

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. Like Nepal, which lies in one of the earthquake prone area, this type of construction is frequently used in other regions of high seismic activities, such as Latin America, southern Europe, North Africa, Middle East, South Asia etc.

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 and opening in them. 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. For this reason building codes imposes an upper limit to the natural period of a structure by way of empirical relations. 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) 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), 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 judiciously 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 storey. 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 localized stress concentration.

In Nepal, 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 National Building Code of Nepal 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 robust design methodology for seismic design of masonry infill Reinforced Concrete structure.

OBJECTIVES

Generally, this study aims to investigate the effect of brick masonry infill wall on a reinforced concrete moment resisting frame conventionally designed as a bare frame, using available macro-model proposed by FEMA273 (1997), Pauley and Priestley (1992) and Holmes (1961).The specific objectives of the study are:

1. To study the effects of the full and partial infill wall on reinforced concrete frame subjected to earthquake induced lateral load.

2. To study the effects of opening sizes on the behavior of RC frame under earthquake induced lateral load.

Scope

The thesis work is based on the Code of Practice of Nepal and India. But, the ductility requirement and the seismic loading are based on the Indian Code of Practice. 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 full and partial unreinforced brick masonry infill wall which is neither integral nor bonding with the surrounding frame.

2. The study is based on a hypothetical 10 storey apartment type building frames with typical floor loading and infill thickness of 230 mm in cement sand mortar ratio 1:5. The openings are of centrally located square type. The opening sizes considered are 10%, 20%, 30%, 40%, and 50%.

3. Only linear elastic analysis is carried out and hence P-delta effect is not considered. The comparisons are made for fundamental period, base shear, displacement, story drift, shear force, and bending moment.

LITERATURE REVIEW

1. General structural modeling and analysis

Different literatures and past studies were studied to gain the knowledge about the modeling process in general and modeling of infill in particular. Real and accidental torsional effects must be considered for all structures. Therefore, all structure must be treated as 3-dimensional system. Structures with irregular plans, vertical setbacks or soft stories will cause no additional problems if a realistic 3-dimensional computer model is created (E. L. Wilson, 2002). Different structural system is being employed to resist effectively the gravity as well as lateral loading. Strength, rigidity and stability are the main factors to be considered. The choice of structural system depends on many factors such as architectural planning, material to be used, construction methods, type of lateral load to be resisted, and the height of building etc. So far as the lateral load such as earthquake loading is concerned, a building can be considered as a vertical cantilever. Thus, the effect of lateral force is more pronounced as the height of the building increases, (Smith and Coull, 1991).

For a low rise building the rigid frame system has been used extensively and is quite popular throughout the world. This system essentially is a beam and column configuration joined by moment-resisting connections. The beams and columns in a rigid frame system are modeled using 3-dimensional beam elements with 6 DOFs at each node.

The in-plane stiffness of the floor systems of most building structures are extremely high compared to the stiffness of framing members. As a result, the in-plane deformations of beams can often be neglected, and columns, braces and walls connected to a given diaphragm will be constrained to move as one single unit in the lateral directions. This property is widely used in structural analysis to reduce the size of the system equations of buildings with such rigid floor types. When the “Rigid Diaphragm” option is selected for a given floor in any finite element(FE) based program, a transformation of coordinates and degrees of freedom is carried out to arrive at a system equation that allocates only three in-plane degrees of freedom for that particular diaphragm (STAAD.Pro Manual, CSI Analysis Reference Manual). But, it is important to realize that there are many instances where the rigid-diaphragm assumption cannot be used. Buildings with light metal deck floors, short shear-wall buildings are some of the examples where the floors cannot be assumed rigid diaphragms. Research has shown, for example, in the case of short shear-wall buildings such an assumption could give erroneous results because the walls have a comparable lateral stiffness to that of the floor diaphragms (Rutenberg, 1980, Boppana and Naiem, 1985).

Though with the advent of modern structural analysis tools such as finite element method (FEM) and faster computer, the computer analysis of structure has advanced significantly; linear elastic analysis is still the preferred method of analysis in the design offices as it is simple and allows the superposition of actions and deflections of various load cases. Although nonlinear methods of analysis have been developed, their use at present for high-rise building is more for research than for the design office (Smith and Coull, 1991; FEMA450, 2003).

2. Loadings on structure

Normally in an earthquake prone area the structure is designed for gravity as well as seismic load. Gravity loads are due to the self-weight of the structure, superimposed dead load and occupancy of the building. The dead loading is calculated from the designed member sizes and estimated material densities. The magnitudes of live loading specified in the codes are estimates based on a combination of experience and results of typical field surveys.

Earthquake loading consists of the inertial forces of the building mass that result from the shaking of its foundation by a seismic activity. Earthquake resistant design concentrates particularly on the translational inertia forces, whose effects on a building are more significant than the vertical or rotational shaking component. The intensity of earthquake is related inversely to their frequency of occurrence; severe earthquakes are rare, moderate ones occur more often, and minor ones are relatively frequent. Although it might be possible to design a building to resist the most severe earthquake without significant damage, the unlikely need for such strength in the lifetime of the building would not justify the high additional cost. Consequently, the general philosophy of earthquake resistant design for buildings is based on the principles that they should:

1. Resist minor earthquakes without damage;

2. Resist moderate earthquakes without structural damage but accepting the probability of nonstructural damage;

3. Resist average earthquakes with the probability of structural as well as non-structural damage but without collapse.

Some adjustments are made to the above principles to recognize that certain buildings with a vital function to perform in the event of an earthquake should be stronger.

3. Deflection and story drift

As far as the ultimate limit state is concerned, lateral deflections must be limited to prevent second order P-Delta effects due to gravity loading being of such a magnitude as to precipitate collapse. In terms of serviceability limit state, deflections must be maintained at a sufficiently low level firstly to allow the proper functioning of nonstructural components such as elevators, doors, etc. and secondly to avoid distress in the structure, to prevent excessive cracking and consequent loss of stiffness, and to avoid any redistribution of load to non load bearing partitions, infills, cladding or glazing (Smith and Coull, 1991). The Indian code IS 1893 restricts the maximum inter-story drift of 0.004 times the story height and the maximum displacement of 0.002 times the height of structure.

4. Fixity of base

If there is no structure, motion of the ground surface is termed as free field ground motion. In normal practice, the free field motion is applied to the structure base assuming that the base is fixed. But this is valid only for structures on rock sites. It may not be an appropriate assumption for soft soil sites. Presence of a structure modifies the free field motion since the soil and the structure interact, and the foundation of the structure experiences a motion different from the free field ground motion. Soil structure interaction accounts for this difference between the two motions. The soil structure interaction generally decreases lateral seismic forces on the structure, and increases lateral displacements and secondary forces associated with P-delta effect. For ordinary buildings, the soil structure interaction is usually ignored (IS 1893, 2002). IS 1893: Part 1 (2002) refers the soil-structure interaction as effects of the supporting foundation medium on the motion of structure. For the purpose of determining seismic loads, it is permitted to consider the structure to be fixed at the base (ASCE 7, 2005). Thus, the choice of the support conditions for the structure is essentially governed by the condition of soil on which the structure is founded. The assumption of fixed support may be justified if the structure is built on stiff soil or rock.

5. Infilled frame structure

The infilled frame consists of a steel or reinforced concrete column and girder frame with infill of brickwork or concrete block work. They are usually provided as exterior walls, partitions, and walls around stair, elevator and service shafts and hence treated as non structural elements. But it has been recognized by many studies that it also serve structurally to brace the frame against horizontal loading. The frame is designed for gravity loading only and, in the absence of an accepted design method, the infills are presumed to contribute sufficiently to the lateral strength of the structure for it to withstand the horizontal loading. The simplicity of construction, and the highly developed expertise in building that type of structure have made the infilled frame one of the most rapid and economical structural forms for buildings. Absence of a well recognized method of design for infilled frames have restricted their use for bracing. Thus, it has been more usual to arrange for the frame to carry the total vertical and horizontal loading and to include the infills on the assumption that the infills do not act as part of the primary structure. However, from the frequently observed diagonal cracking of such infill walls it is evident that the approach is not always valid. The walls do sometimes attract significant bracing loads and in doing so, modify the structure's mode of behavior and the forces in the frame (Smith and Coull, 1991)

The use of masonry infill to brace a frame combines some of the desirable structural characteristics of each, while overcoming some of their deficiencies. The high in-plane rigidity of the masonry wall significantly stiffens the frame, while the ductile frame contains the brittle masonry, after cracking, up to loads and displacements much larger than it could achieve without the frame; thus, a relatively stiff and tough bracing system results. The wall braces the frame partly by its in-plane shear resistance and partly by its behavior as a diagonal bracing strut.

When the frame is subjected to lateral loading, the translation of the upper part of the column in each storey 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 as shown in Figure 2. Three potential modes of failure of the wall arise as a result of its interaction with the frame. The first is shear failure stepping down through the joints of the masonry and precipitated by the horizontal shear stresses in the bed joints. The second is a diagonal cracking of the wall through the masonry along a line or lines parallel to the leading diagonal, and caused by tensile stresses perpendicular to the leading diagonal. The diagonal cracking is initiated at and spreads from the middle of the infill, where the tensile stresses are the maximum. In the third mode of failure, a corner of the infill at one of the ends of the diagonal strut may be crushed against the frame due to the high compressive stresses in the corner (Smith and Coull, 1991). These modes of failure are shown in Figure 3

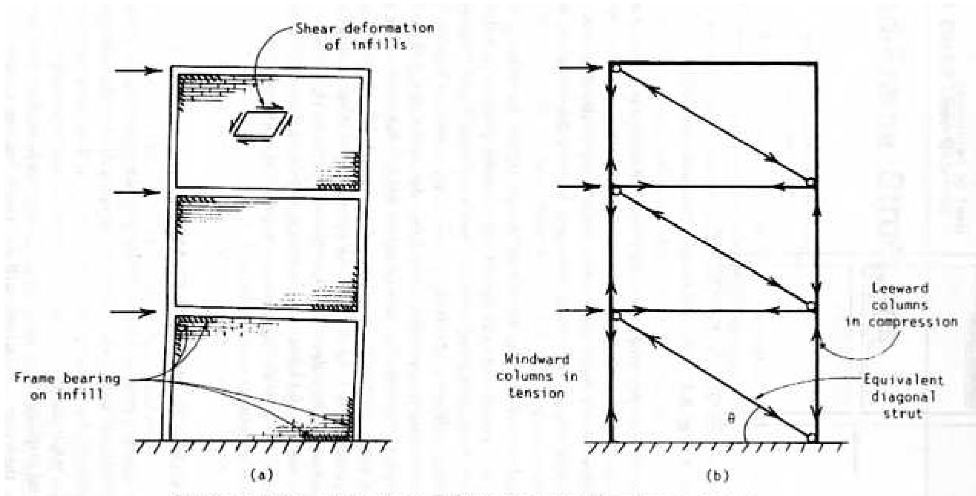


Figure 2 Interactive behavior of frame and infill; analogous braced frame

Source: Smith and Coull (1991)

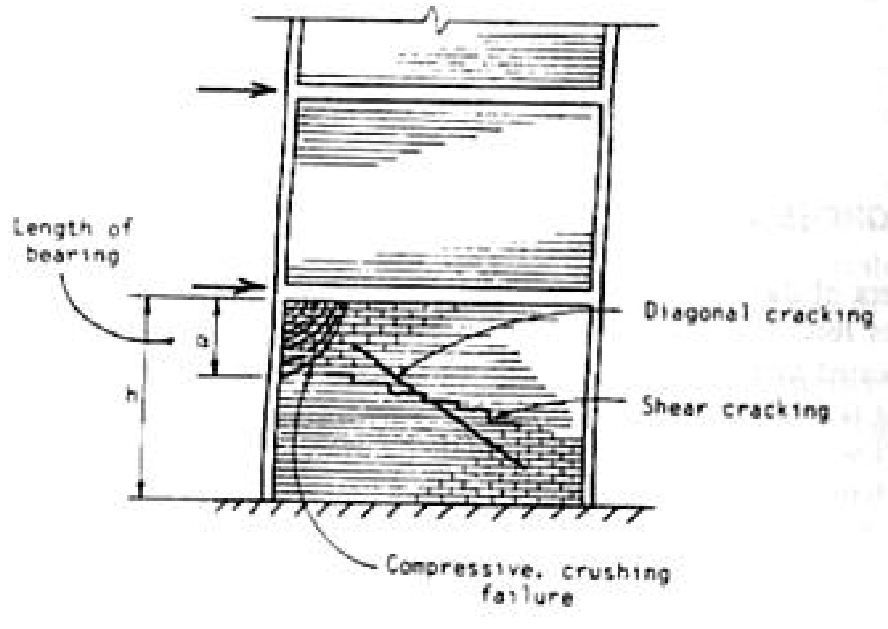


Figure 3 Failure modes of infill

Source: Smith and Coull (1991)

The masonry infill might as well impart some deficiency to the RC frame structure. Irregularities, often unavoidable, contribute to complexity of structural behavior. The masonry infill can drastically alter the intended structural response, attracting forces to parts of the structure that have not been designed to resist them (Paulay and Priestley, 1992).

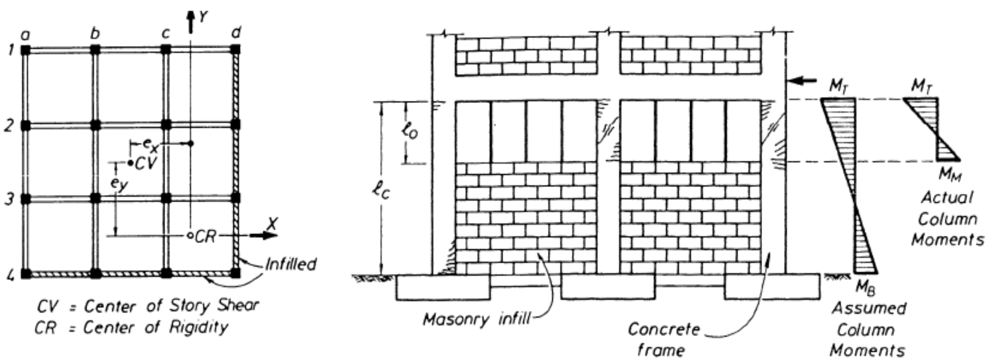


Figure 4 Ill effect of infill
Source: Pauley and Priestley, (1992)

For example, as shown in the left of Figure 4, irregularities in placing of infill walls will cause change in the center of rigidity of the building there by subjecting the building to seismic torsional response. The stiffness of frames with infill increase and consequently, the natural period of these frames will decrease and seismic force will correspondingly increase relative to other frames. Similarly, if the partial infill is provided as shown in the right, the infill will stiffen the frame, reducing the natural period and increasing the seismic force. The design level of shear force in the column will be given by equation (1);

$$V = \frac{M_T + M_B}{l_c} \tag{1}$$

However, in reality, a structure will be subjected to shear force given by;

$$V^* = \frac{M_T + M_M}{l_o} \tag{2}$$

If the structure is not designed for the higher shear force given by equation(2), shear failure can be expected. Thus, if not taken in to account the effect of infill during analysis stage, infill might have some ill effect on the structure.

6. Behavior of brick masonry infilled RC frame

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. A rigorous analysis of infilled structure requires an analytical model of the force deformation response of masonry infills, and number of finite element models has been developed to predict the response of infilled frames (Asteris 2003; Shing *et al*. 1992; Dymiotis *et al*. 2001), such micro-modeling is too time consuming for analysis of large structures. Alternatively, a macro-model replacing the entire infill panel as a single equivalent-strut, by far has become the most popular approach.

An early contribution on the study of complex behavior or masonry infill was by Polyakov (1956). He found that the frame and the infill separate except at two compression corners. He introduced the concept of equivalent diagonal strut and suggested that stresses from the frame to the infill are only transmitted in the compression zone of the infill, with a distribution more typical of a diagonally braced system than a shear wall.

Holmes (1961) proposed replacing the infill by an equivalent pin jointed diagonal strut of the same material and thickness with a width equal to one-third of its diagonal length.

One of the major contributions towards the study of infill wall was by Bryan Stafford Smith (1962). He found that the frame is separated from the infill over three quarters of the length of each side. There remained only one quarter of the length of each side in contact with the infill at the windward top and leeward lower corner and suggested that the infill was behaving approximately as the equivalent structure as shown in Figure 5. Using experimental results and finite difference approximation, he found that the stiffer the frame, the longer is the length of contact and the consequent greater effective stiffness of the infill. In order to express the results in a useable form, he translated the stiffness into an effective width, which is the width of an equally stiff uniform strut whose length is equal to the diagonal of the panel and whose thickness is the same as the panel. He plotted effective width as a proportion of diagonal length for varying side as shown in Figure 6.

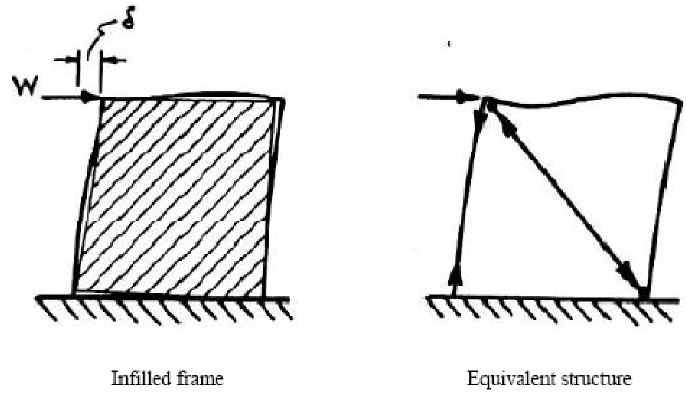


Figure 5 Infilled frame and Equivalent structure
Source: Bryan Stafford Smith (1962)

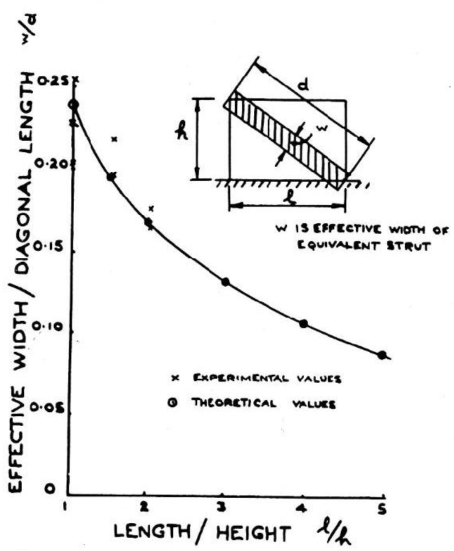


Figure 6 Width of Infill Wall
Source: Bryan Stafford Smith (1962)

Further study by Smith (1967) and Smith and Carter (1969) showed that the ratio, (w/d) also depends on the contact length between column and infill (α) and a dimensionless parameter (λh), which is termed as a relative stiffness of the infill panel to the column. Furthermore, the length of contact and distribution of interaction between the beam and infill is approximately constant, whatever the section of the beam. The relative stiffness of the infill panel to the frame is defined by equation(3). The Contact length is governed by the relative stiffness of the infill to the frame and is given by equation (4) and as shown in Figure 7.

$$\lambda h = h \times \sqrt{\frac{E_m t \sin 2\theta}{4 E_c I_c h_m}} \tag{3}$$

$$\frac{\alpha}{h} = \frac{\pi}{2\lambda h} \tag{4}$$

Where, h is height of column, E_c and E_m are young's modulus of frame and infill panel respectively, t is thickness of infill panel, θ is angle of inclination of diagonal strut with the horizontal, I_c is the moment of inertia of column and h_m is the height of infill. Later Carter (1969) included the effect of nonlinearity of material and studied various modes of failure and concluded that shear cracking is the predominant mode of failure.

Pauley and Priestley (1992) suggested that the effective width shall be one-fourth the diagonal length. FEMA 273 uses the relation proposed by Mainstone (1971) which relates the width w of infill to parameter λh, given by equation (3) and diagonal length d as shown in the equation (5).

$$\frac{w}{d} = 0.175(\lambda h)^{-0.4} \tag{5}$$

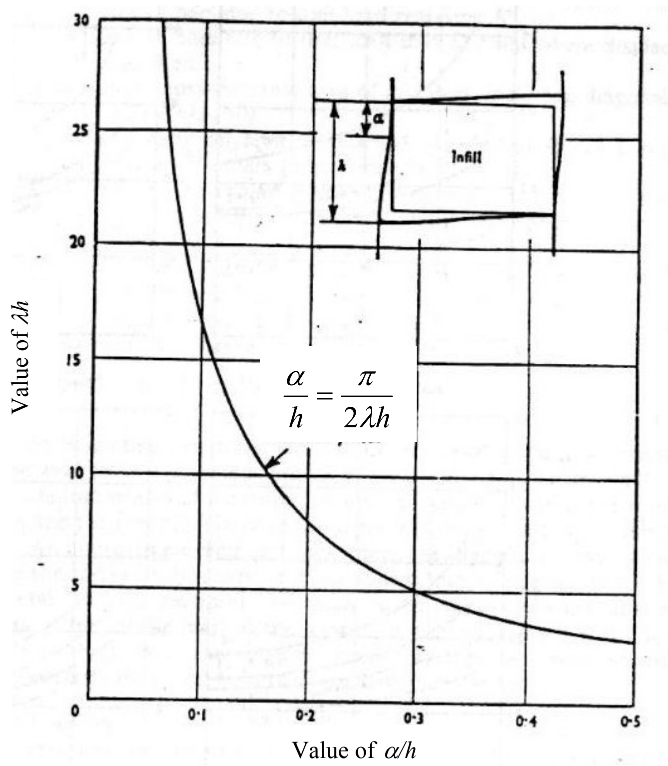


Figure 7 Length of contact as a function of λh
Source: Bryan Stafford Smith (1967)

Saneinejad and Hobbs (1995) developed an equivalent diagonal strut approach for the analysis of steel frames with concrete or masonry infill walls subjected to in-plane lateral load, based on data generated from previous experiments as well as results from a series of nonlinear finite element analyses. The method takes into account the elasto-plastic behavior of infill frames considering the limited ductility of infill materials. Various governing factors such as the infill aspect ratio, shear stress at the infill-frame interface, and relative beam and column strengths are accounted. However, it gives only extreme or boundary values for design purpose.

Armin *et al*. (1996) carried out series of experimental investigation on the influence of masonry infill panels on the seismic performance of RC frames. The experimental results indicated that infill panels can significantly improve the performance of RC frames in terms of load resistance and energy dissipation capability. The study indicated that for a frame that is properly designed for strong seismic loads, infill panels will most likely have a beneficial influence on its performance. It also indicated that infill panels can be potentially used to improve the performance of existing non-ductile frames.

Ghassan Al-Chaar *et al.* (2002) studied experimentally, single-story structures with non-ductile RC frames and infill masonry panels subjected to in-plane loads and found that RC frames with brick infill exhibit significantly higher peak and residual strength and initial stiffness than bare RC frames without compromising any ductility in the load-deflection response. He compared the test result with finite element analysis (FEA) and found that FEA predicted peak load within 8% but residual strength could not be predicted with a high level of confidence.

Hossein Mostafaei and Toshimi Kabeyasawa (2004) did the case study on the Bam telephone center building, with a nonsymmetrical reinforced concrete moment-resisting frame structure. Based on post-earthquake damage assessment results, almost no residual deformations or cracks were observed in the structural elements of the building. However, based on designed base shear coefficient required by Iranian seismic code, nonlinear responses were expected due to such a strong earthquake. Hence, to obtain an analytical answer for the almost linear performance of the building, 3-dimensional nonlinear time history analyses were carried out for north-south and east-west recorded strong motions. The response simulations were performed for different categories of bare frame and infilled frame. The results of the analyses were compared to damage and residual cracks observed on the masonry infill walls. Reasonable correlations were obtained between analytical and observed results. It may be concluded that the presence of masonry infill walls is the main reason for the nearly linear responses of the Bam telephone center building during the earthquake.

Mehmet Emin Kara and Sinan Altin (2006) conducted experimental and analytical investigation on the behavior and strength of non-ductile reinforced concrete (RC) frames strengthened by introducing partial infills under cyclic lateral loading. The RC partially infilled walls introduced to non-ductile RC frames significantly increased the lateral strength, stiffness, and energy dissipation capacity of the frame and resulted in a considerable reduction in the lateral drift. Although having the same aspect ratios, the initial stiffness in the specimen that had an infilled wall connected to both columns and beams of the frame was 45% greater than that of the specimen that had an infilled wall connected only to beams

Mallick and Garg (1971) studied the effect of openings and shear connectors on the behavior of infill panels by using experimental as well as finite element model and found that there is a satisfactory agreement between the two. They concluded that the opening should be located within the middle third of the panel and the opening at either end of a loaded diagonal is undesirable.

T. C. Liaw and S. W. Lee (1977) experimentally investigated and analytically examined the effect of concrete infill with and without openings, and also with and without connectors. They suggested equivalent diagonal strut method for the analysis of frame without connectors and equivalent frame method for the frame with connectors.

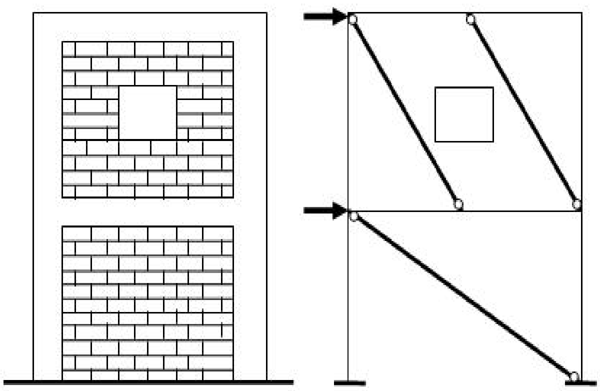


Figure 8 Diagonal strut mechanism for infill with opening
Source: Buonopane *et al.* (1999)

Based on previous experimental and analytical research, Roko Zarnic (1995) prepared mathematical models for full infill as well as infill with opening, using diagonal strut and found it very successful on global response. Later Buonopane *et al.*

(1999) did a series of pseudo-dynamic test on infilled frame with full and partial opening and found that the strut mechanism serves as acceptable idealization for initial stiffness and global behavior. They proposed 2 strut model for infill with opening as shown in Figure 8.

7. Failure Modes

Bryan Stafford Smith (1967) reported two distinct mode of infill failure as shown in Figure 9.

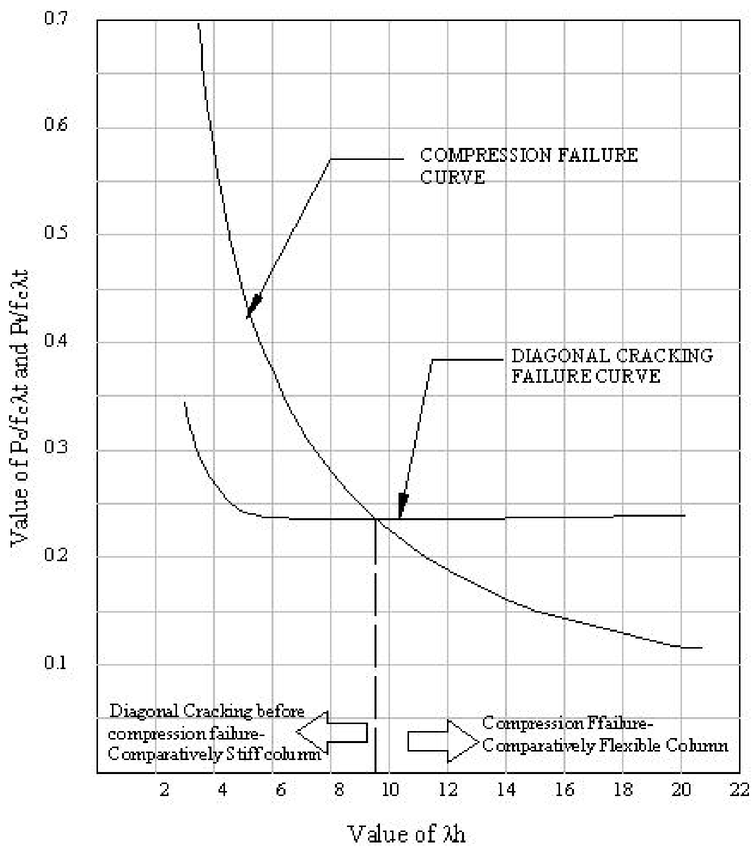


Figure 9 Failure curve of an infill
Source: Bryan Stafford Smith (1967)

1. Tension failure: A crack along the loaded diagonal is started at the center of the infill and is extended towards the corners. This occurs suddenly often with an audible click. This occurs, invariably in the second or final stages of the

load/deflection curve, and is accompanied by a jump in the deflection; however, it is usually possible to restore and increase the load further, without any marked loss in stiffness, to produce eventually a second type of failure.

2. Compressive failure: This consists of a region of crushed mortar in one of the loaded corners, the region extended along the column to the end of the length of contact, but against the beam the infill often remained intact. This invariably defines the collapse of the structure. Whenever λh is less (comparatively stiffer column) than a critical value, a diagonal crack precedes the compressive failure. The strength of the cracking and compressive mode of failure are both increases as λh is reduced. This is shown in the Figure 9.

The modes of failure of multistory infilled frames subject to dynamic load can be distinguished by the fact that whether or not connectors between the infill and the frames were provided. Models with solid infill failed by diagonal compression when there were no connectors, and failed by shear between the frame and the infill when there were connectors. Similarly, models with openings in the infill failed by bending in the lintel beams when there were no connectors, and they failed by shear in the lintel beams when there were connectors (T. C. Liaw, 1979).

Several potential failure modes for infill masonry walls are; firstly a horizontal sliding shear failure of masonry walls, second is the compression failure of diagonal strut, third is the diagonal tensile cracking which does not generally constitute a failure condition, as higher lateral forces can be supported, and lastly the tension failure mode (flexural) which is not usually a critical failure mode for infill wall (Paulay, and Priestley, 1992).

Shear strengths for the first and second critical types of failure mode are obtained for each infill panel, and the minimum value is considered to be the shear strength of the infill wall.

1. Sliding shear failure: If sliding shear failure of the masonry infill occurs, the equivalent structural mechanism changes from the diagonally braced pin-jointed frame to the knee-braced frame as shown in Figure 10. The equivalent diagonal strut compression force R_s to initiate horizontal shear sliding depends on the shear friction τ_f and aspect ratio of the panel. The Mohr-Coulomb failure criteria can be applied to assess the maximum shear strength for this kind of failure mechanism as given by equation(6):

$$\tau_f = \tau_o + \mu N \tag{6}$$

where, τ_o is cohesive capacity of the mortar beds, μ is sliding friction coefficient along the bed joint, and σ_N is vertical compression stress in the infill walls.

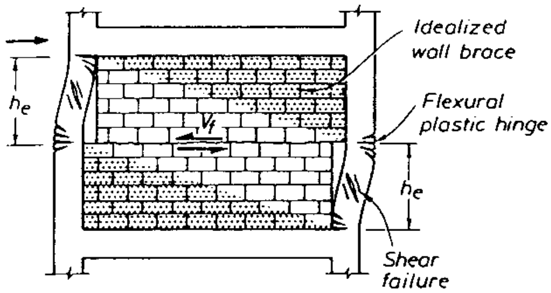


Figure 10 Knee braced frame model for sliding shear failure of masonry infill
Source: Pauley and Priestley (1992)

Applying the panel dimension as shown in Figure 11, maximum horizontal shear force V_f is assessed as follows:

$$V_f = \tau_o t l_m + \mu N \tag{7}$$

Where N is the vertical load in the infill; and is estimated directly as a summation of applied external vertical load on the panel and the vertical component of the diagonal compression force R_s, as shown in Figure 11. It should be assumed that the panel carries no vertical load due to gravity effects, because of difficulties in constructing infill with a tight connection with the overlying beam of the frame, and also because vertical extension of the tension column will tend to separate the frame and panel along the top edge. Consequently, the external vertical load is zero for the infill walls of the building, and only the vertical component of the strut compression force is considered. The maximum shear force V_f that can be resisted by the panel is thus,

$$V_f = \tau_o t l_m + \mu R_s \sin\theta \tag{8}$$

But, from Figure 11,

$$\begin{aligned} &V_f = R_s c\,\text{in}\,\theta = (l_m / d_m); and \\ &h_m / l_m \approx h / l, \end{aligned} \tag{9}$$

$$\therefore R_s = \frac{\tau_o}{1-\mu(h/l)} d_m t \tag{10}$$

Substituting the recommended value of $\tau_o = 0.03 f_m$ and $\mu = 0.3$, we get,

$$\therefore R_s = \frac{0.03 f_m}{1-0.3(h/l)} d_m t \tag{11}$$

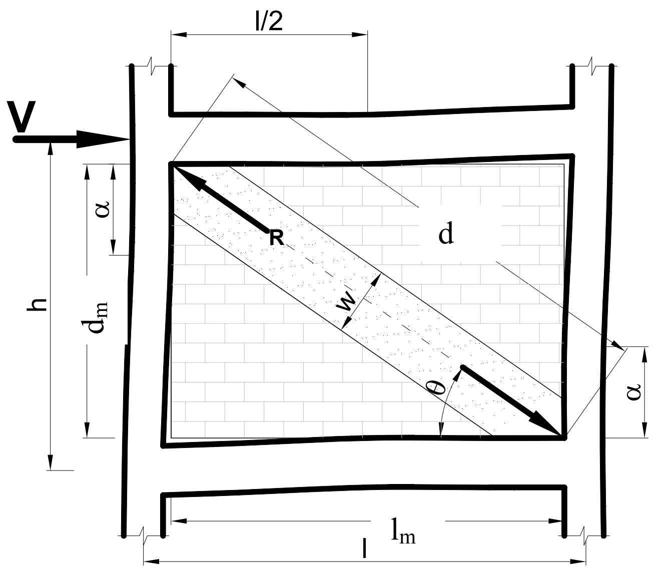


Figure 11 Typical Deformed Infill Subjected Lateral Load with equivalent diagonal bracing

Source: Pauley and Priestley (1992)

2. Compression failure of diagonal strut: For typical masonry infill panels, diagonal tensile splitting will precede diagonal crushing. However, the final panel failure force will be dictated by the compression strength, which may thus be used as the ultimate capacity. The equivalent diagonal strut compression force R_c to initiate compression failure of a diagonal strut is given by.

$$R_c = \frac{2}{3}\alpha t f_m \sec\theta \tag{12}$$

Where,

$$\alpha = \frac{\pi}{2}\left[\frac{4E_c I_c h_m}{E_m t \sin 2\theta}\right]^{\frac{1}{4}} \tag{13}$$

RESEARCH METHODOLOGY

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, relevant design codes, and guidelines of different countries 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 and brick masonry.

2. Data Collection

The study was done with the prevalent construction materials being used in Nepal. Thus, the required experimental and material data necessary to make the analytical model of the brick masonry infill were collected from The Institute of Engineering, Pulchowk Campus, Nepal. The National Building Code of Nepal was collected from the Department of Urban Development and Building Construction, Nepal.

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 Nepal, structure is analyzed for seismic loading as per NBC 105: 1994 Seismic Design of Buildings as well as IS 1893(Part 1): 2002 Criteria for Earthquake Resistant Design of Structures (Part 1: General Provisions and Buildings). The buildings are usually modeled as the 3-dimensional finite element model. 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 effect of RC slab for rigid floor diaphragm action to resist lateral force is taken into account.

For the present study, a hypothetical 10 storey apartment type building with typical floor plan as shown in Figure 12 was considered. This building is not meant to represent any physical buildings. The building is symmetrical in plan with respect to two orthogonal axes and the plan dimension of the building is 25m x 15m and the height of the building is 33.5 m. The grid spacing along both axes is 5m. Thus there are 5 grids along X-axis and 3 grids along Z axis. The floor height is 3.35 m.

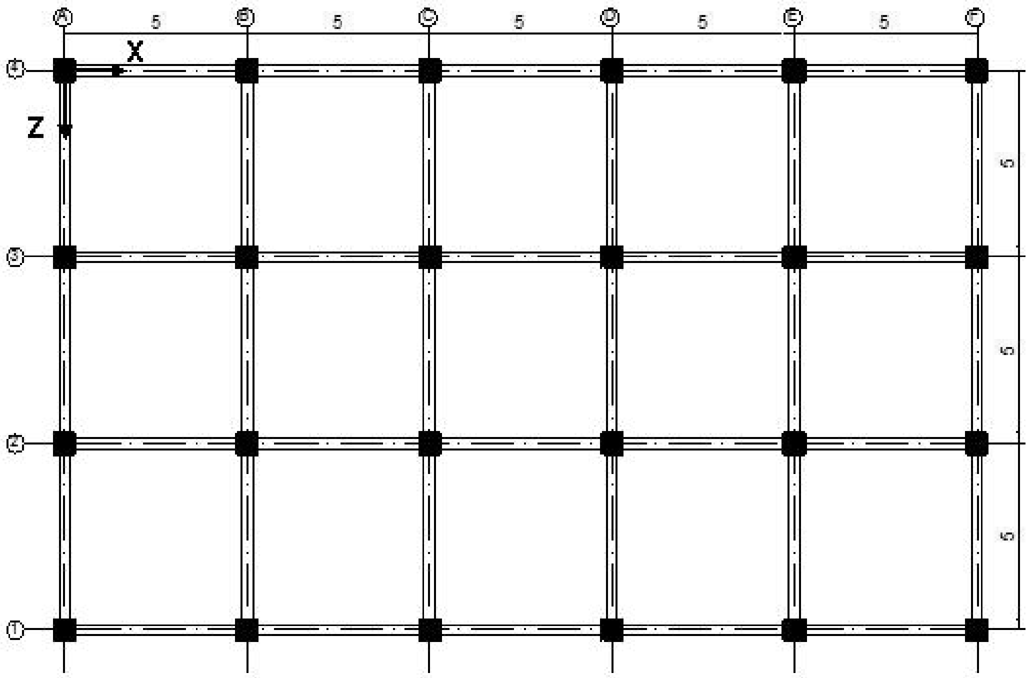


Figure 12 Typical plan of the model being studied

Only the masonry surrounded by beams and columns are considered as infill. For walls in other location, only the weight contribution is considered. 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 not the behavior of infill panel or on the behavioral effect due to minute details. The member sizes are shown in Table 1

Table 1 Member sizes

Structural Members	**Size (mm)**
Column	500 x 500
Beam	300 x 600
Infill panel	230 thick

Initial dimensioning of the beams and columns were made on the basis of bare frame design for full wall case with earthquake load as per IS1893-2002 code such that the structure met the strength and ductility requirements of Indian code, with a limitation that the lateral displacement limit exceeded the allowable value. The same sections were used for the cases of infill with openings. Further, it was assumed that the infill panels were neither integral nor bonding with the frame and the openings are centrally located. Different models with and without infill for full infill panel and infill panel with centrally located openings of 10, 20, 30, 40, and 50 percentage were developed to analyze and to investigate the effect of infill wall on seismic response of the typical structures. For each infill case, bare frame and infill frame models were developed. Both these model were studied with code prescribed time period and calculated time period. The common approaches used for modeling the infill frames are as follows:

i) Bare Frame Method

This is the commonly accepted method of structural analysis and design for buildings with infill panel all around the glove. The only contribution of masonry infill is their masses in the form of non-structural element. Consequently, analysis of the structure is based on the bare frame. In this, the beam and columns are modeled as frame element. Since infills are not considered, their contributions to the lateral stiffness and strength may invalidate the analysis and the proportioning of structural members for seismic resistance on the basis of its results. However, this method is still being widely used in the world even in the earthquake prone areas; and is considered for the comparison in the present study.

ii) Plate Modeling Method

The more rigorous analysis of structures with masonry infilled frame requires an analytical model of force deformation response of masonry infill. This method is probably best suited for this purpose. In this method, beams and columns may be modeled using a frame element whereas; the infill panel could be modeled using a shell element. Interaction between frame and infill, including the effect of initial lack of fit, formation of gaps and slipping between frame and infill after lateral loading could be modeled using an interface element. A number of finite element models have been developed to predict the response of infilled frames (Shing *et al*. 1992, Asteris 2003). Such a micro modeling is too time consuming for analysis of a large structure.

iii) Equivalent diagonal strut Method

Alternatively, a macro-model replacing the entire infill panel as a single equivalent-strut by far has become the most popular approach for analyzing infilled frame systems. In this method, the brick infill is idealized as a pin jointed diagonal strut and the RC beams and columns are modeled as three-dimensional beam elements having 6 degree of freedoms at each node. The idealization is based on the assumption that there is no bond between frame and infill. The brick masonry 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 various factors like; contact length, aspect ratio of the infill and the relative stiffness of frame and the infill.

True, that the macro modeling approach takes into account only the equivalent global behavior of the infill in the analysis and does not permit the study of local effects such as frame-infill interaction within the individual infilled frame subassemblies, which needs detailed micro modeling. However, the macro-modeling approach allows for adequate evaluation of the force-deformation response of the structure and individual components under seismic loading (Madan *et al*. 1997) and may be used to assess the overall response to a sufficient degree of accuracy. Thus, the proposed macro model is better suited for representing the behavior of infills of complex structures with multiple components particularly in cases where the focus is on evaluating the response.

Load cases used:

Dead load: The unit loads used in this study is based on NBC 102: 1994, Unit Weights of Materials. This Nepal Standard for Unit Weight of Materials adopts the Indian Code IS:875 (Part 1) – 1987 Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures, Part 1, Dead Loads-Unit Weights of Building Materials and Stored Materials, (Second Revision).

Imposed Load: the imposed load used in this study is based on NBC 103: 1994, Occupancy Loads. This Nepal Standard for Occupancy Load adopts the Indian Code IS:875 (Part 2) - 1987 Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures, Part 2 Imposed Load, (Second Revision).

Earthquake Load: The Nepal National Building code for the earthquake design is NBC 105: 1994. However, for this study the Indian Standard IS1893 (Part 1): 2002, Criteria for Earthquake Resistant design of Structure, Part 1: General Provisions and Buildings (fifth revision) was used. Static analysis using equivalent lateral force procedure is restricted to regular buildings having height less than 40 m and irregular buildings having height less than 12 m in seismic Zone V which is the most severe zone. Seismic weight of a structure is computed from total dead load and reduced live load and is multiplied by a coefficient from the response spectrum plot shown in Figure 13. The equivalent base shear method is formulated with the assumption that the first mode of vibration governs, which is true for short period structures. Hence, the equations for equivalent base shear method are derived on the assumption that the horizontal displacement of the first mode of vibration increases either linearly or quadratically with height (FEMA450, 2003), the IS 1893 employs the quadratic variation of displacement. Since, the building under study is regular in both horizontal and vertical axis and the height is less than 40m, the seismic coefficient method was used which is defined as follow:

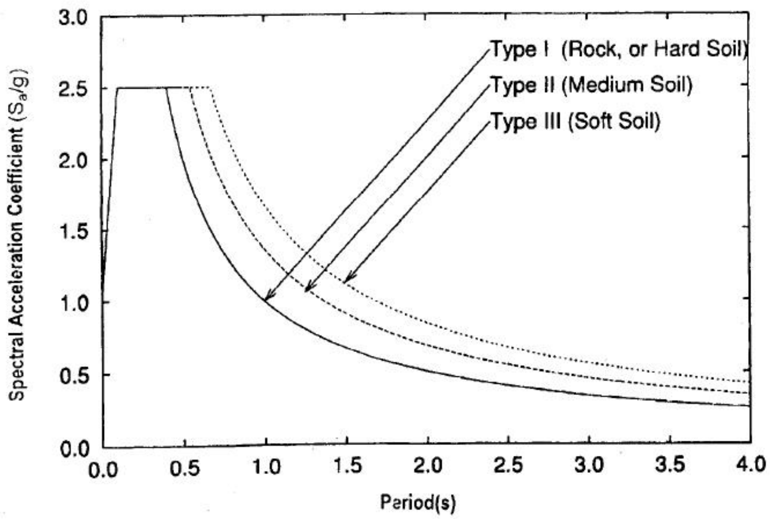


Figure 13 Response spectrum for 5% damping
Source: IS 1893 (Part 1) : 2002

The design base shear V_B which is the total lateral force at the base of a structure is computed in accordance with the clause 7.5.3 of the code which states,

$$V_B = A_h W \tag{14}$$

Where,

$$A_h = \frac{Z}{2}\frac{I}{R}\frac{S_a}{g} \tag{15}$$

Provided that for any structure with T < 0.1 sec, A_h is not less than (*Z/2*) whatever be the value of (*I/R*). Where,

Z = Zone factor = 0.36; I = Importance factor = 1.5; R = Response reduction factor = 5; S_a/g = Average response acceleration coefficient from Figure 13 which depends on the fundamental time period of the building and W = Seismic weight of building, which is the total dead load plus appropriate amount of imposed load.

The approximate fundamental natural period of vibration (T_a), in seconds, of a moment resisting frame building may be estimated by the empirical expression:

$T_a = 0.075h^{0.75}$; For RC frame building,

$T_a = 0.085h^{0.75}$; For steel frame building and

$T_a = 0.09\,h/\sqrt{d}$; For moment resisting frame building with brick infill panels.

Where, (h) is the height of building in meter and (d) is the base dimension of the building at the plinth level, in meter, along the considered direction of the lateral force.

The design base shear (V_B) computed above will be distributed along the height of the building as per the following expression:

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2} \tag{16}$$

Where, Q_i = design lateral force at floor i, W_i = seismic weight of floor i, h_i = height of floor i measured from base, and n = number of story in the building, is the number of levels at which the masses are located

Material Properties to be used

For this study, the material property for concrete, reinforcing bar and brick masonry panels are as follows:

Yield strength of reinforcing bar $f_y = 500\ N/mm^2$ (Fe 500)

<u>For Concrete:</u>

Unit weight = $23.5616\ kN/m^2$

Characteristic compressive strength, f_{ck}, = $M30 = 30N/mm^2$

Tensile strength (flexural strength), $f_{cr} = 0.7\sqrt{f_{ck}} = 3.83\ N/mm^2$

Shear strength, $\tau_c = 3.5\ N/mm^2$

Young's modulus of elasticity, $E_c = 5000\sqrt{f_{ck}} = 27386\ 30\ N/mm^2$

Poisson's ratio, $\nu_c = 0.17$

Shear modulus, $G_c = \dfrac{E_c}{2(1+\nu_c)} = 11703.55\ N/mm^2$

<u>For Brick Masonry Panel</u>

Size of brick = *230mm x 115 mm x 57 mm (9" x 4.5" x 2.25")*, $h_b = 57\ mm$

Horizontal mortar thickness, $j = 18\ mm$

1 course of brick + mortar = *75 mm (3")*

Mortar ratio = *1:5*

Compressive strength of hand molded burnt clay brick, $f_{cb} = 7.5 N/mm^2$

Compressive strength of 1:5 mortar, $f_j = 5 N/mm^2$

Tensile strength of brick, $f_{tb} = 0.1 f_{cb} = 0.75\ N/mm^2$

The compressive strength of masonry prism, f_m can be calculated by the relation given by Paulay and Priestley (1992);

$$f_m = \frac{f_{cb}(f_{tb} + \alpha f_j)}{U_u(f_{tb} + \alpha f_{cb})} = \frac{7.5(0.75 + 0.077 \times 5)}{1.5(0.75 + 0.077 \times 7.5)} = 4.27 N / mm^2$$

Where, $\alpha = \frac{j}{4.1 h_b} = \frac{18}{4.1 \times 57} = 0.077$ and,

U_u = stress non-uniformity coefficient = 1.5

Young's modulus of elasticity, $E_m = 550 f_m = 4125\ N/mm^2$

Poisson's ratio, $\nu_c = 0.12$

4. 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 story shear, story moment, deflection, drift, and member forces. Based on these behaviors, the results of the analysis such as the period of vibration, story shear, story moment, displacement, and story drift and member forces due to earthquake are presented and discussed in the Results and Discussion section.

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 brick infill, only a few published studies provide detailed data about the specimens and the experimental results.

Details of the Experimental Specimen

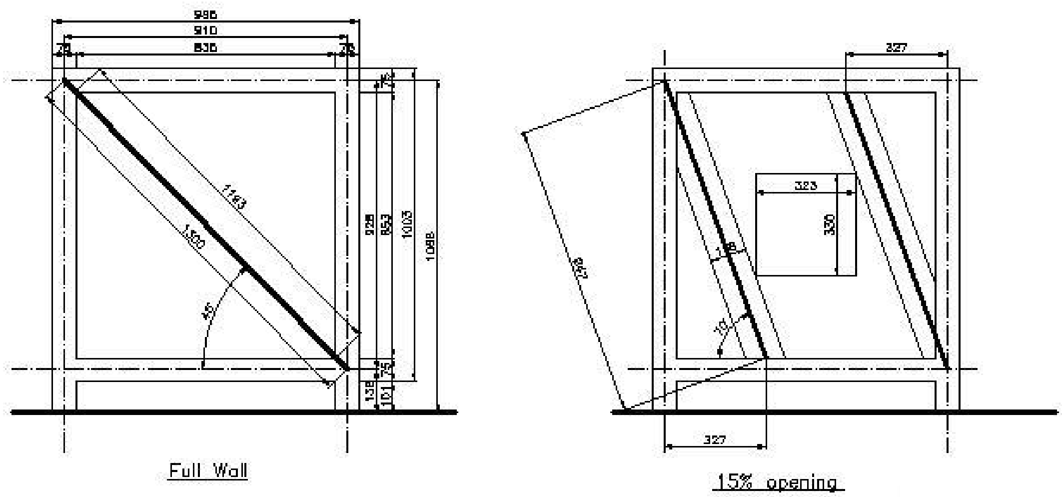
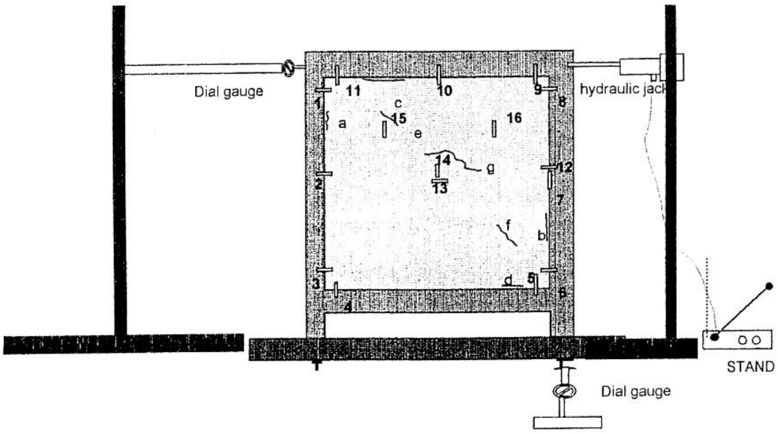


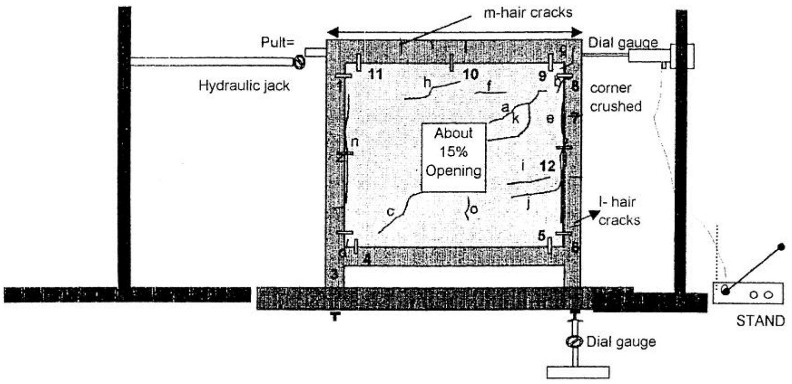
Figure 14 Geometry of test specimen

Sumat Shrestha (2005) prepared 4 models in 1:3 reduced scale single bay single story model of RC frame with unreinforced full infill panel as well as infill panel with central opening of 15%, 50% and 70%. 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 brick 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 opening and 15% central opening are shown in Figure 15 and Figure 16.



Strain Guage No.	Coordinate	
	x(mm)	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 15 Test setup for infill RC frame with no opening
Source: Sumat Shrestha (2005)



Strain Gauge No	Coordinate	
	x(mm)	y(mm)
1	75	850
2	75	500
3	75	150
4	150	75
5	830	75
6	910	150
7	910	500
8	910	850
9	830	930
10	500	930
11	150	930
12	910	440

Figure 16 Test setup for Infill RC frame with 15% opening
Source: Sumat Shrestha (2005)

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 Figure 14, and properties of materials are listed in Table 2. 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 Properties of materials

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

Analytical Study of Specimens without opening

The specimen for infill frame without opening was modeled using equivalent diagonal strut as shown in Figure 17 using three different strut widths as proposed by Holmes, Pauley & Priestley and FEMA273. The experimental as well as analytical results are shown in Figure 18.

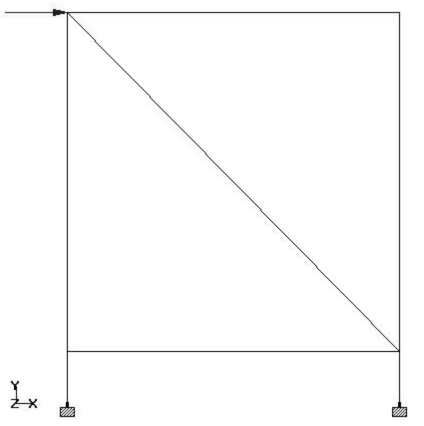


Figure 17 Analytical model for full wall

As seen from the Figure 18, 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.

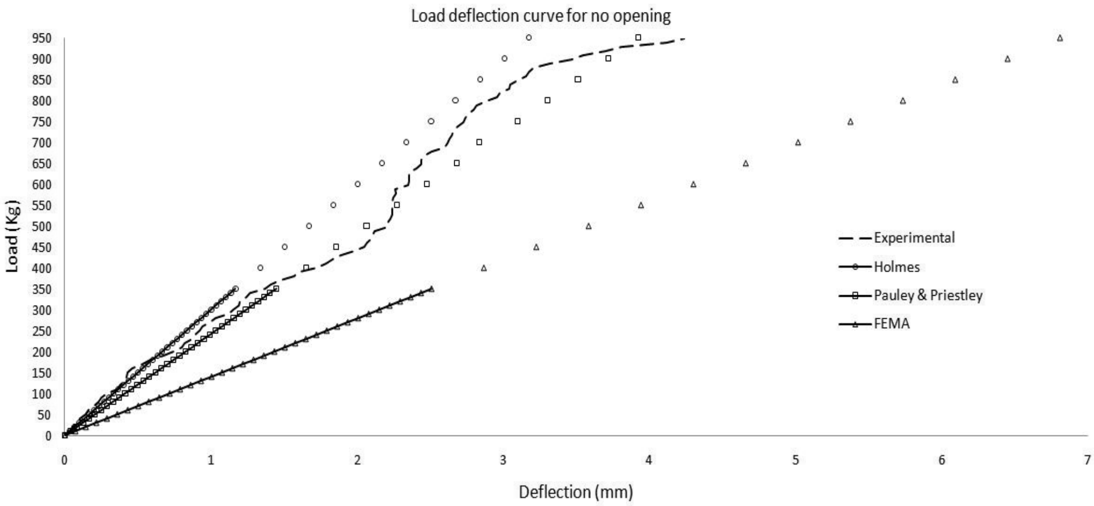


Figure 18 Load deflection curve for full wall case

Analytical Study of Specimens with central opening of 15%

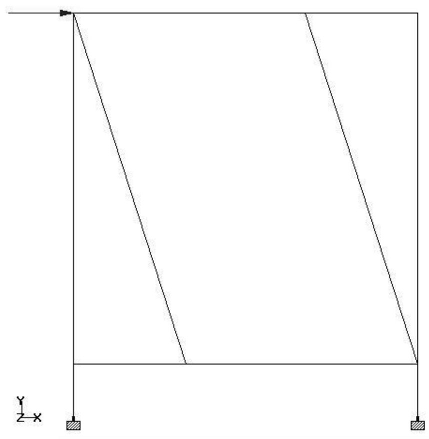


Figure 19 Analytical model for 15 % central opening

The infill frame specimen with opening was also modeled using all the three different strut widths as proposed by Holmes, Pauley & Priestley and FEMA273. The type of equivalent diagonal strut model used in this case was as suggested by

Buonopane *et al*. (1999) as shown in Figure 19. The experimental as well as analytical results are shown in Figure 20.

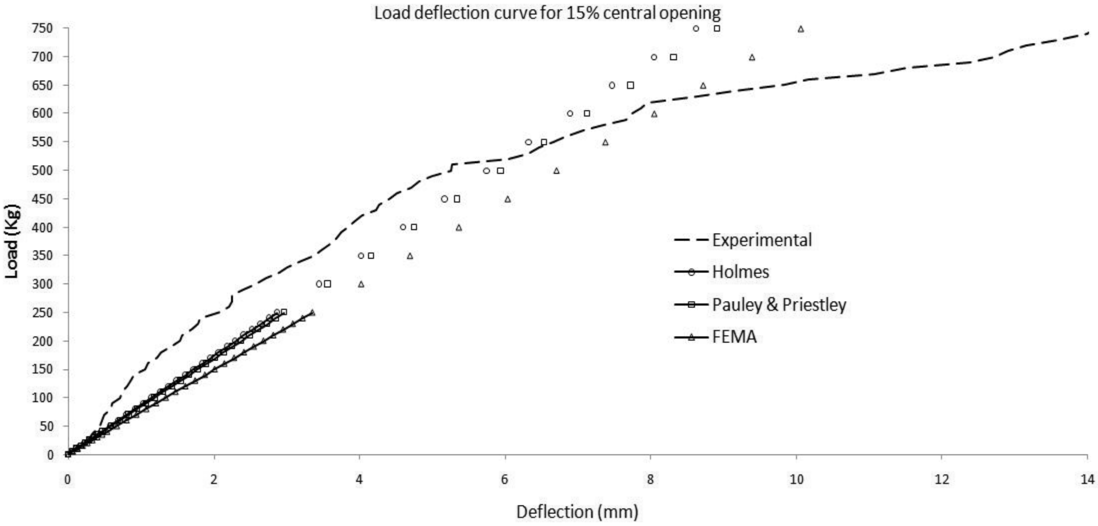


Figure 20 Load deflection curve for specimen with 15% central opening

In this case, all the three models gave less stiffness than the experimental value. This might be because in the 2-strut model for opening; perfect truss mechanism is not formed as in the case of infill with no opening. The stiffness predicted by Holmes and Pauley & Priestley models are very close and even the stiffness given by FEMA model is not very different. However, since the Pauley & Priestley model gave the reasonable stiffness in the case of infill without opening, the same model is chosen for the case of infill with openings.

RESULTS AND DISCUSSIONS

The bare frame and infill frame for full and partial infill with central opening of various sizes were studied analytically. Based on the results obtained from the numerical analysis, the behavior of different structural systems in terms of fundamental time period, design lateral force, story shear, story moment, deflection profile with height, maximum horizontal displacement at roof level, story drift ratio, and the member forces are compared in the following pages.

The results of analytical study for full and partial infill are presented into two sections. In the first section, only the findings of the effects of full infill based on Holmes, Pauley & Priestley, and FEMA 273 are studied and compared with bare frame model. In the second section, effects of full and partial infill of different opening sizes are studied with Pauley and Priestley model and compared with bare frame model.

First the comparison of fundamental time period between bare frame models and respective infill frame models for different opening sizes is presented. This is followed by the presentation of comparative study of seismic excitation in terms of design lateral force, story shear and story moment of different bare frame and infill frame models. Next, the structural responses of different bare and infill models in terms of displacement, maximum roof level displacement and inter story drift are compared. In this, the responses of all the models from the seismic coefficient method are discussed. Lastly, the member forces of structural member due to combined effect of gravity and seismic loading for both the bare and infill frame for all the opening sizes are studied and discussed.

Although from the verification chapter the Pauley & Priestley model with effective width of one-fourth the diagonal seems most appropriate strut model, Holmes and FEMA model were also considered for the case of full infill panel. Thus, four different models, a bare frame and three types of strut models were considered. A rigid floor diaphragm which still retains the bending flexibility was used to model floor slab.

1. Effect of Full Infill Wall Panel on RC Frame Structure

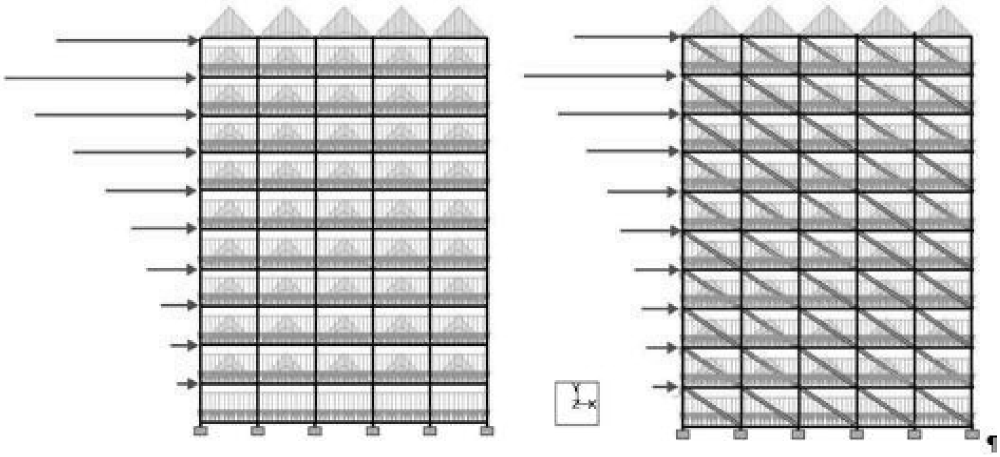


Figure 21 Bare and infilled frame with full wall

1.1 Fundamental Time Period and Base Shear

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 Indian code IS 1893 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 3, and hence smaller lateral forces. However, when the effect of infill is included, the time periods determined from analysis were found to be close to the one computed form the code formulas. 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.

The Pauley and Priestley model gave the closest match. The time period predicted by this model in X-direction was almost the same from code value, whereas: it predicted stiffer structure than the code in the Z-direction. This suggests that the time period is not only the function of height and width of the structure, as given in the code formula. FEMA model gave the least conservative value, whereas; Holmes model gave the most conservative one. This agrees with the verification model.

Table 3 Comparison of Time Period for different model types

Model Type	**Time Period in second X**			**Time Period in second Z**		
	Calculated	**IS Code**	**Ratio**	**Calculated**	**IS Code**	**Ratio**
Bare frame	1.70998	0.603	283.58%	1.78997	0.77847	229.93%
Holmes	0.5484	0.603	90.95%	0.63961	0.77847	82.16%
Pauley & Priestley	0.60581	0.603	100.47%	0.69396	0.77847	89.14%
FEMA 273	0.80529	0.603	133.55%	0.88797	0.77847	114.07%

Table 4 Comparison of Base Shear for different model types

Model Type	**Base Shear in X(kN)**			**Base Shear in Z (kN)**		
	Calculated	**IS Code**	**Ratio**	**Calculated**	**IS Code**	**Ratio**
Bare frame	1201	3404	35.28%	1148	2638	43.50%
Holmes	3743	3404	109.95%	3209	2638	121.63%
Pauley & Priestley	3390	3404	99.58%	2959	2638	112.14%
FEMA 273	2550	3404	74.91%	2312	2638	87.63%

Similarly, as shown in Table 4, the base shear calculated on the basis of bare frame model gave a much lesser value than the code; whereas, base shear from the infill model were comparable with the code value. Here also, the Pauley & Priestley model gave the closet match with the code value whereas; the FEMA model was the least conservative and the Holmes model gave the most conservative value.

1.2 Design lateral force, story shear and story moment

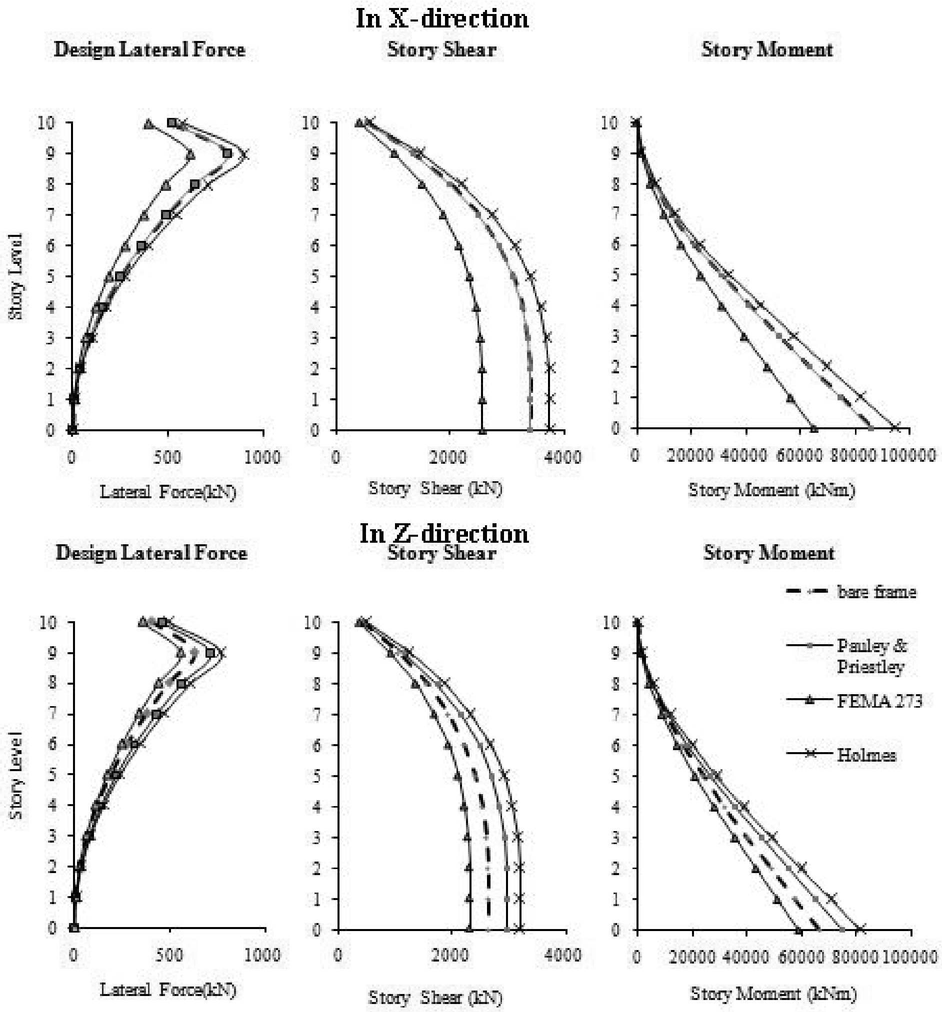


Figure 22 Design lateral force, story shear and story moment in X & Z-direction

Since, from the comparison of fundamental time period and base shear it is clear that the bare frame model with analytically computed time period predicts too flexible structure than that by the code, further comparison were done for bare frame with code prescribed time period and infill frame with Holmes, Pauley & Priestley, and FEMA 273 model with analytically computed time period.

Figure 22 shows the comparison between the bare frame model with infill model by Holmes, Pauley & Priestley, and FEMA 273. Even in this case the Pauley and Priestley model gave the closest match whereas, the Holmes model gave around 10% higher values and FEMA model gave around 25% lesser value than by the code formula in X-direction. Whereas in the Z-direction FEMA model predicted about 12% less value from that of code, both Holmes and Pauley & Priestley model predicted a stiffer structure than that from the code by about 22% and 12% respectively.

1.3 Lateral displacement and inter-story drift

Next, the effect of infill on the lateral displacement and inter-story drift were studied for bare frame model and all the 3 infill models as suggested by Holmes, Pauley & Priestley, and FEMA 273. The floor displacements are presented in Table 5 and the inter-story drifts are presented in Table 6. These are also presented in Figure 23 and Figure 24.

Table 5 and Table 6 show the comparative study of seismic demand in terms of lateral displacement and inter-story drift amongst all the three types of infilled model and the bare frame model. As discussed earlier, the bare frame model was analyzed with the code prescribed time period whereas; the infilled ones were analyzed with the calculated time period. The lateral displacement predicted by FEMA model is the maximum which are about 17% in X and 23% in Z-direction of that predicted by the bare frame model. In this case, Holmes model gave the least value which is about 12% in X and 17% in Z-direction of the bare frame model. Pauley & Priestley model predicted about 13% and 19% of the bare frame model.

Table 5 Floor displacement in X & Z direction

Floor	Average displacement in X direction				Average displacement in Z direction			
	Holmes	Pauley & Priestley	FEMA 273	Bare Frames	Holmes	Pauley & Priestley	FEMA 273	Bare Frames
0	0	0	0	0	0	0	0	0
1	0.0957	0.1085	0.1482	0.7466	0.0950	0.1084	0.1497	0.6095
2	0.2069	0.2343	0.3239	1.862	0.2144	0.2426	0.3356	1.5453
3	0.3251	0.366	0.5028	3.0149	0.348	0.3897	0.5302	2.5231
4	0.4478	0.5013	0.6826	4.1494	0.4925	0.5463	0.7303	3.4915
5	0.5714	0.6363	0.8587	5.2351	0.6432	0.7075	0.9305	4.4234
6	0.692	0.7669	1.0256	6.239	0.795	0.8681	1.1242	5.2900
7	0.8051	0.8881	1.1769	7.1218	0.9427	1.0224	1.3045	6.0576
8	0.906	0.9947	1.3057	7.8384	1.0809	1.1643	1.4635	6.6878
9	0.9896	1.0809	1.4039	8.3405	1.2037	1.2876	1.5926	7.1403
10	1.0513	1.1415	1.4649	8.6055	1.3063	1.3867	1.6849	7.3976

Similarly, as seen in Table 6 the inter-story drift as predicted by FEMA are the maximum of 24% in X and 36% in Z-direction of that predicted by the bare frame model. Here also, the drift predicted by the Holmes model is the least, closely followed by Pauley & Priestley models.

Thus, the infill panel reduces the seismic demand of a RC moment resisting frame structure. Figure 23 and Figure 24 shows the comparison between all the three models with the bare frame model with analytically calculated time period as well as the code prescribed one. The lateral displacement and inter-story drift are 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 as seen in 2003 Bam earthquake (Hossein Mostafaei and Toshimi Kabeyasawa, 2004).

Table 6 Inter story drift in X & Z direction

Floor	Inter story drift in X direction				Inter story drift in Z direction			
	Holmes	Pauley & Priestley	FEMA 273	Bare Frames	Holmes	Pauley & Priestley	FEMA 273	Bare Frames
0	0	0	0	0	0	0	0	0
1	0.0957	0.1085	0.1482	0.6095	0.095	0.1084	0.1497	0.6095
2	0.1112	0.1258	0.1757	0.9357	0.1194	0.1343	0.1859	0.9357
3	0.1182	0.1317	0.1789	0.9778	0.1337	0.1471	0.1946	0.9778
4	0.1227	0.1353	0.1798	0.9685	0.1445	0.1566	0.2002	0.9685
5	0.1236	0.1351	0.1761	0.9319	0.1507	0.1612	0.2001	0.9319
6	0.1206	0.1305	0.1669	0.8666	0.1518	0.1606	0.1938	0.8666
7	0.1131	0.1212	0.1514	0.7676	0.1477	0.1543	0.1803	0.7676
8	0.1009	0.1066	0.1288	0.6302	0.1381	0.1419	0.159	0.6302
9	0.0836	0.0862	0.0982	0.4525	0.1229	0.1233	0.1291	0.4525
10	0.0616	0.0606	0.0609	0.2574	0.1026	0.0991	0.0923	0.2574

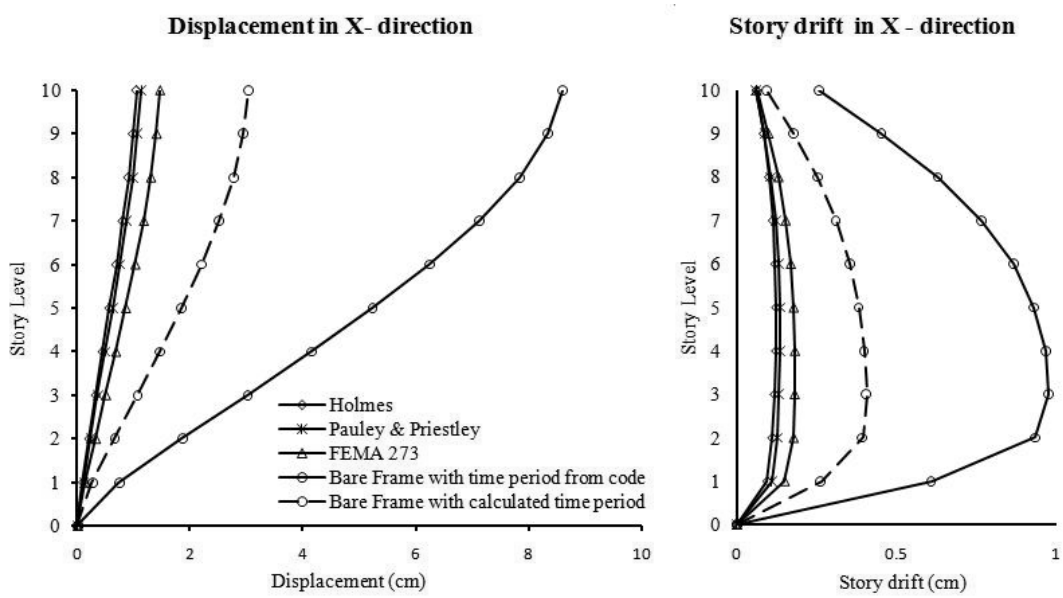


Figure 23 Lateral displacement and story drift in X-direction

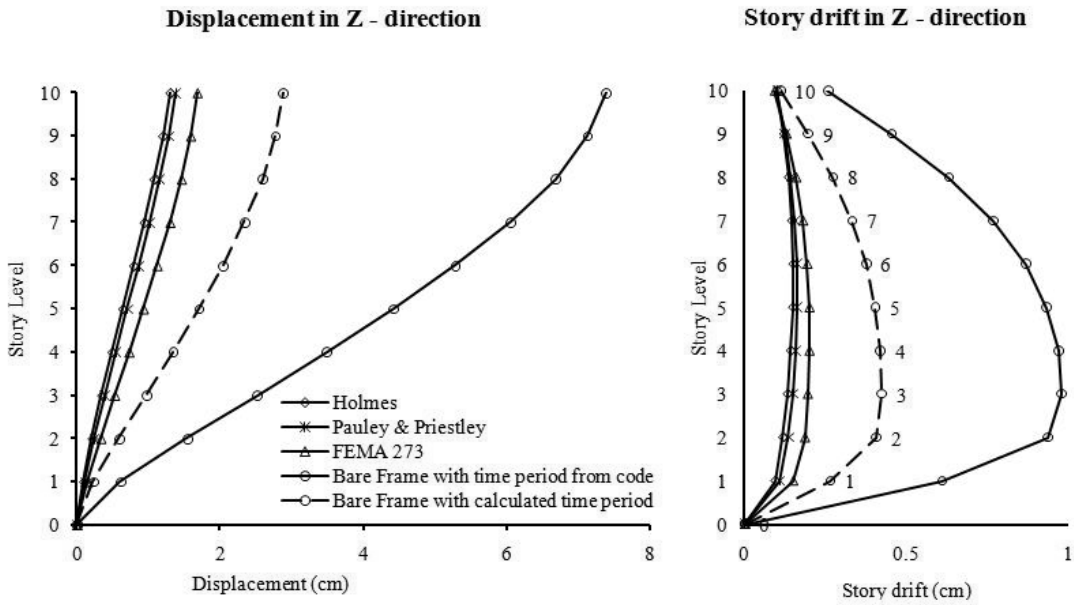


Figure 24 Lateral displacement and story drift in Z-direction

1.4 Member 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.

The floor wise axial forces for the corner column for the seismic load case are presented in Table 7. Generally, for the bottom floors where the axial force is large, FEMA model 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 (Table 8). Even in this category, FEMA model showed the most conservative values. The reduction here is about 40% as compared to bare frame model whereas; the Holmes model was the least conservative which gave less than 20% value.

Table 7 Axial forces for corner column for seismic load case in X-direction

Floor	Bare frame	FEMA 273	Pauley & Priestley	Holmes
1	537.651	699.519	646.27	621.922
2	466.388	611.519	571.569	554.726
3	389.085	515.008	489.447	479.627
4	313.357	420.813	407.235	402.999
5	241.236	329.348	325.242	325.283
6	174.373	242.494	245.278	248.32
7	114.81	162.92	169.917	174.686
8	65.056	94.18	102.666	107.886
9	28.005	40.726	48.052	52.474
	6.944	8.391	12.068	14.502

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 as shown in Table 9 and Table 10. The FEMA model predicted about 35 % of the bare frame model whereas; Holmes model predicted only about 13%.

The Holmes model gives the largest effective width and FEMA gives the least. 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.

It is seen that FEMA model is the most conservative ones so far as predicting the lateral displacement and member forces are concerned. However, as seen from the verification model and the comparisons of time period, base shear, design lateral forces, story shear and moment Pauley & Priestley model is the most realistic one and hence, to study the effect of partial infill panel on a frame structure, the Pauley & Priestley prescribed strut width is chosen.

Table 8 Shear force and bending moment in corner column for seismic load case

Floor	**Shear**				**Moment**			
	Bare frame	**FEMA 273**	**Pauley & Priestley**	**Holmes**	**Bare frame**	**FEMA 273**	**Pauley & Priestley**	**Holmes**
1	78.8	33.2	17.5	13.5	186.0	74.5	39.4	30.6
2	62.7	15.0	6.9	5.2	110.4	24.5	11.5	8.8
3	61.4	16.4	8.1	6.3	103.5	28.4	14.5	11.4
4	58.8	15.5	7.6	5.9	97.3	26.0	13.2	10.3
5	55.5	14.6	7.3	5.7	90.3	24.0	12.2	9.6
6	50.8	13.3	6.7	5.2	80.7	21.2	10.8	8.5
7	44.4	11.5	5.9	4.6	67.8	17.4	9.0	7.1
8	35.6	9.0	4.7	3.7	50.9	12.4	6.6	5.3
9	24.9	6.0	3.3	2.7	30.2	6.3	3.6	3.0
10	7.2	-0.4	0.0	0.2	3.1	-3.1	-1.2	-0.7

Table 9 Shear force in edge beam for seismic load case

Beam	**Bare frame**	**FEMA 273**	**Pauley & Priestley**	**Holmes**
25	-71.265	-23.128	-11.62	-8.916
27	-63.612	-21.311	-10.892	-8.391
29	-64.292	-21.694	-11.14	-8.601
31	-63.612	-21.35	-10.957	-8.463
33	-71.265	-22.754	-11.103	-8.336

Table 10 Bending moment in edge beam for seismic load case

Beam	Distance	Bare frame	FEMA	Pauley & Priestley	Holmes
25	0	-188.35	-61.357	-30.867	-23.692
	5	167.972	54.282	27.232	20.89
27	5	-158.49	-53.22	-27.236	-20.992
	10	159.575	53.337	27.223	20.965
29	10	-160.73	-54.24	-27.857	-21.511
	15	160.729	54.232	27.843	21.496
31	15	-159.58	-53.382	-27.312	-21.059
	20	158.486	53.369	27.474	21.254
33	20	-167.97	-53.338	-25.925	-19.421
	25	188.354	60.432	29.591	22.258

2. Effect of Full and Partial Infill Wall Panel on RC Frame Structure

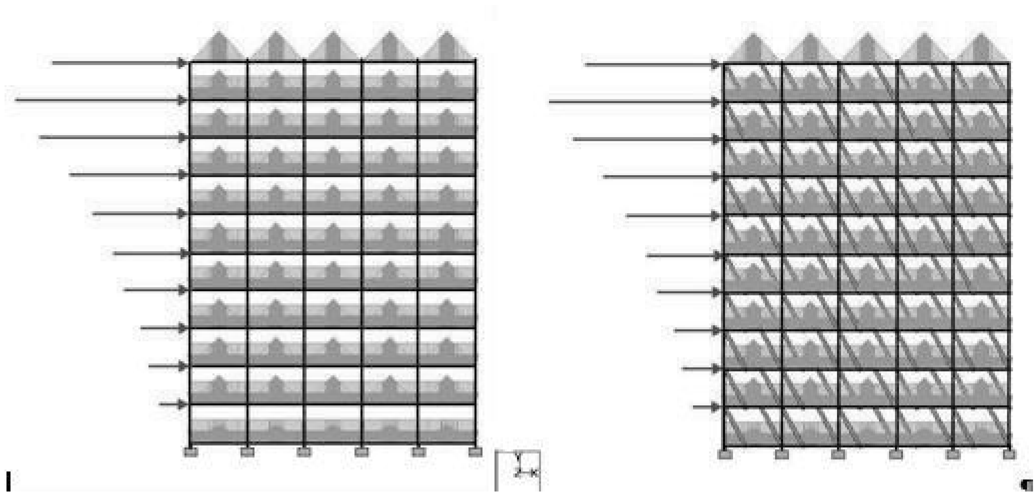


Figure 25 Bare and infilled frame for wall with central opening

For this case, in addition to full infill panel, centrally located square opening of 10%, 20%, 30%, 40% and 50% were considered. A rigid floor diaphragm which still retains the bending flexibility was used to model floor slab. The model considered were a bare frame model wherein, only the mass effect of infill panel was considered; and equivalent strut proposed by Buonopane *et al*. where infill panels were replaced by pin jointed diagonal strut. The Pauley and Priestley proposed strut width was considered.

2.1 Fundamental Time Period an0d Base Shear

Similar to the case of infill panel without opening, even for the case of infill panels with openings, the fundamental time period for all the opening cases are studied as the base shear, design lateral load, story shear, and story moments depends on this property. As the bare frame models gives significantly longer time period than predicted by the code equations as shown in Figure 26, and hence smaller lateral forces; most codes imposes an upper limit to the same. However, when the effect of infill is included, the time periods determined from analysis for smaller openings were found to be close to code formulas whereas, it is close to the bare frame for the large opening. 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.

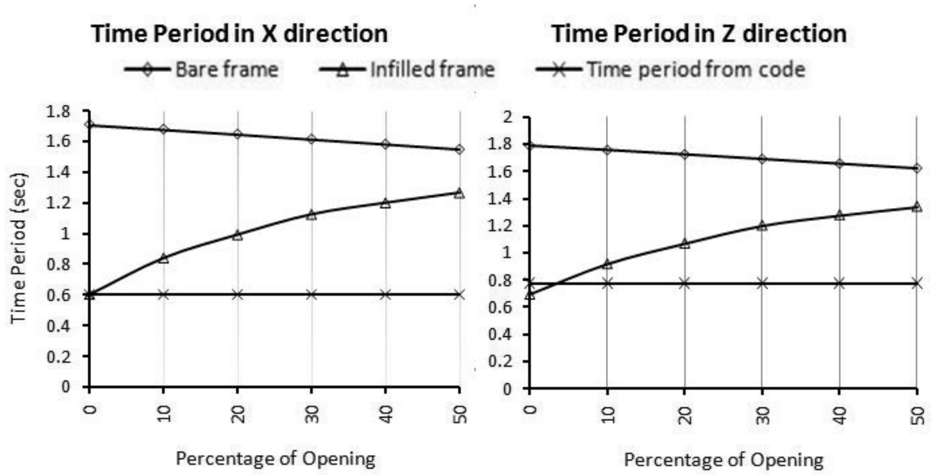


Figure 26 Time period in X & Z-direction for infilled frame

Similarly, as shown in Figure 27, the base shear calculated on the basis of bare frame model gave a much lesser value than the code. When the effect of infill is considered, the base shear varies with the opening size. For no opening, the base shear given by the infilled frame closely matches with the bare frame with code prescribed time period model; whereas, as the opening size increases, the base shear from the infill model were comparable with the bare frame with calculated time period.

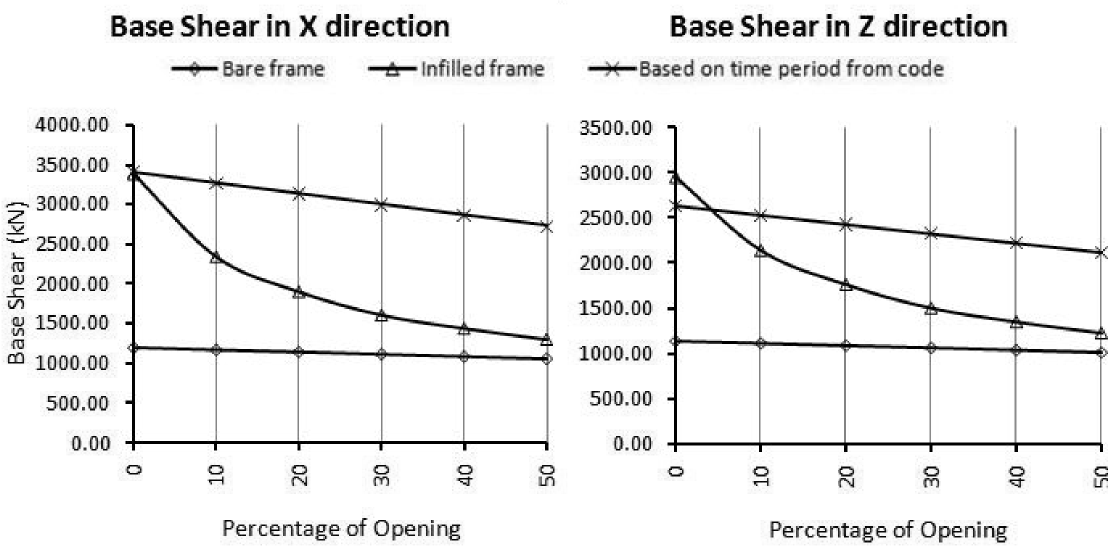


Figure 27 Base shear in X & Z-direction for infilled frame

2.2 Design lateral force, story shear and story moment

Since, from the comparison of fundamental time period and base shear it is clear that the bare frame model with analytically computed time period predicts too flexible structure than that by the code, further comparison were done for bare frame with code prescribed values of time period and infill frame with Pauley & Priestley model with analytically computed time period.

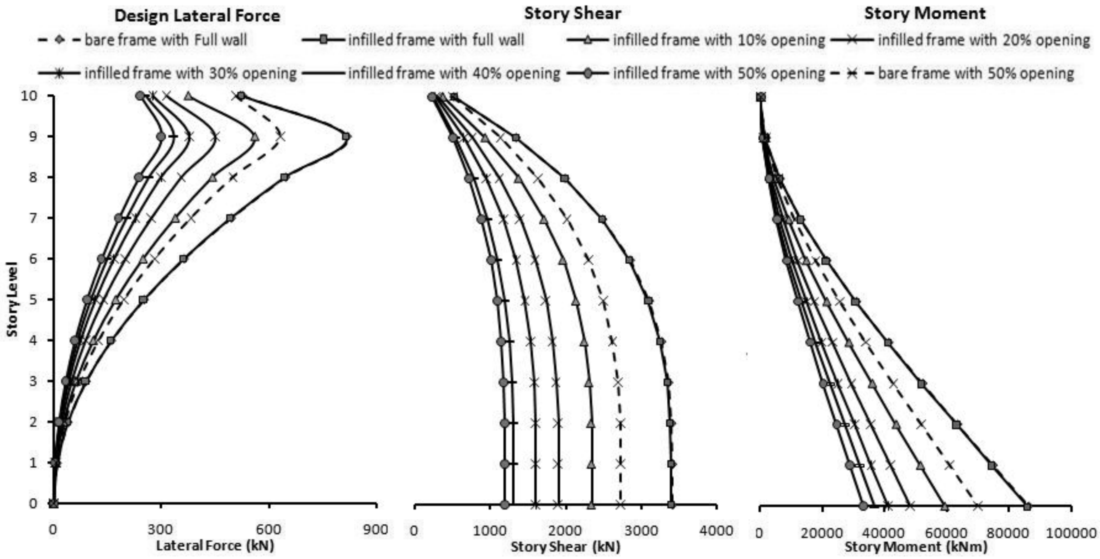


Figure 28 Design lateral force, story shear and story moment in X-direction

Figure 28 shows the comparison for the design lateral forces, story shear and story moment between the bare frame models with infill models. The design lateral forces, story shear and story moment for infilled model were compared with bare frame model with no opening and with 50% opening, which are the two extreme cases for this study. For the case of full infill, both bare frame as well as infill model gave almost the same value whereas for infill with opening, the infill model gave a much lesser value. With the introduction of infill, the total design lateral force, story shear and story moment were reduced by 28%, 39%, 46%, 50%, and 52% respectively for 10%, 20%, 30%, 40%, and 50% opening. Although, study of the experimental and analytical model showed that the analytical model for infill with opening predicts softer structure, reduction in design forces suggests that the code prescribe rather a conservatively high value.

2.3 Lateral displacement and inter-story drift

Next, the effect of infill on the lateral displacement and inter-story drift were studied. In the seismic analysis of a building structure this is one of the important parameter to access the seismic demand of a building structure. Also, many building codes give an upper limit to both lateral displacement as well as story drift. As noticed during past earthquakes, buildings designed using a conventional approach without taking in to account the effect of masonry panel had performed well as shown by the case study of the Bam Telephone Center Building by Hossein Mostafaei and Toshimi Kabeyasawa (2004). Based on post-earthquake damage assessment results, almost no residual deformations or cracks were observed in the structural elements of the building. However, based on designed base shear coefficient required by Iranian seismic code, nonlinear responses were expected due to such a strong earthquake. It may be concluded that the presence of masonry infill walls is the main reason for the nearly linear responses of the Bam telephone center building during the earthquake.

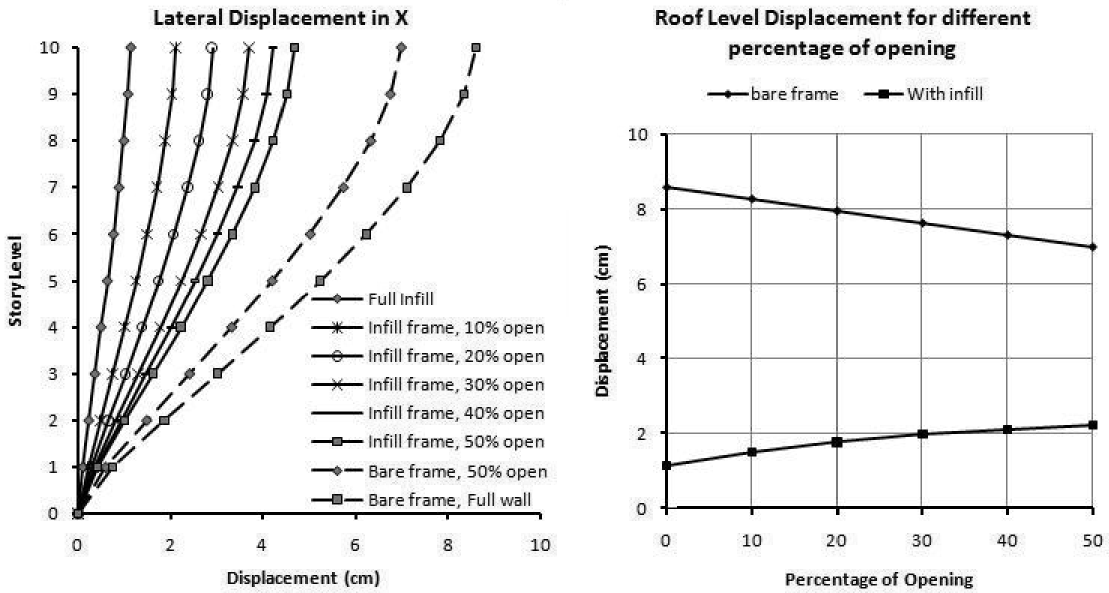


Figure 29 Average lateral displacement and roof level displacement

The average lateral floor displacements and roof level displacements are presented in Figure 29. The comparison was made for the combined effect of gravity and earthquake load combination as required by the Indian code IS 1893. This code limits the inter story drift to 0.004 times the story height and the maximum displacement to 0.002 times the total height of the building. The hypothetical apartment building was analyzed and designed in a conventional approach using the bare frame model. The sizes of the columns were chosen such that the lateral displacement was greater than permitted by the code.

In general, the effect of infill panel is to reduce the seismic demand of a building structure both in terms of lateral displacement as well as inter story drift. As expected, the full infill has a better response during earthquake excitation, whereas; as the size of opening in the infill panel increase, the effect of infill on the structure decreases. As shown in Table 11, the frame with full panel, the infill model predicts the lateral displacement of 1.14 mm which is about 85 % less when compared to the bare frame model (8.6 mm). Similarly, for the case of infill panels with opening size ranging from 10, 20, 30, 40, and 50% respectively, the lateral displacements for infill models are reduced by about 80, 77, 75, 70 and 67% of respective bare frame model.

Table 11 Roof level displacement for different opening size

Opening %	Bare frame	Infill frame	% reduction
0	8.6055	1.1415	87%
10	8.291	1.513	82%
20	7.9661	1.7663	78%
30	7.6377	1.9885	74%
40	7.3123	2.1126	71%
50	6.9868	2.2199	68%

Similarly, as seen in Figure 30 the inter-story drift as predicted by infill models shows a similar improvement in the seismic demand of the respective bare frame model.

The infill panel reduces the seismic demand of the structure. This probably is the cause of building designed in conventional way behaving near elastically even during strong earthquake as seen in 2003 Bam earthquake (Hossein Mostafaei and Toshimi Kabeyasawa, 2004).

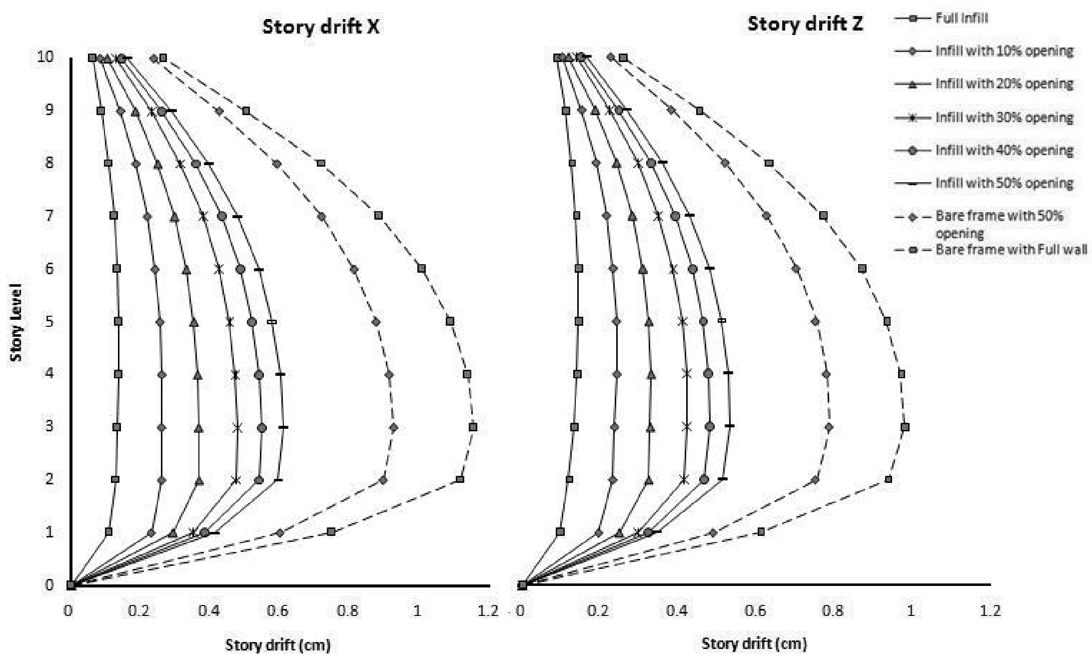


Figure 30 Story drift in X & Z-direction

2.4 Member Forces

Next, the effect of infill on the member forces in beams and columns were studied. This is one of the most important parameter in the design of any building structure. The member forces are important in sizing the section of structural members and to limit the ratio of the reinforcement to be provided and also to limit the drift and displacement. For example, the Indian code IS 13920 : 1993 (Ductile detailing of reinforced concrete structures subjected to seismic forces) limits the steel ratio on any face of flexural member between a minimum value of $0.24\sqrt{f_{ck}}/f_y$ and a maximum value of 2.5%. 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.

Axial loads in columns

The axial loads in columns are compared for bare frame model with code prescribed time period and the infill frame model with analytically computed time period for all the opening cases and presented in Table 12.

The floor wise axial forces for the corner column for the load combination 1.2(DL + LL + EQx) for all opening cases are presented in the Table 12. Generally, axial force computed from the strut model is larger than that computed form the bare frame model. The increase in axial force is the largest for the lower floor and goes on decreasing with increase in floor level.

Table 12 Axial force in corner columns for seismic combination in X direction

height	Full wall		10% open		20% open		30% open		40% open		50% open	
	bare	infill	bare	infill	bare	infill	bare	infill	bare	infill	bare	infill
0	992	1158	956	1276	921	1290	886	1281	851	1250	817	1212
3.35	940	1068	907	1159	874	1172	841	1167	808	1142	775	1111
6.7	891	978	860	1048	829	1057	797	1052	766	1029	735	1002
10.05	833	882	804	934	775	940	745	935	716	915	687	891
13.4	762	780	735	814	709	818	682	812	656	795	630	775
16.75	677	672	653	689	630	690	607	685	583	670	560	653
20.1	574	554	554	557	535	556	515	551	496	540	477	527
23.45	448	424	434	417	419	415	405	411	390	403	376	394
26.8	297	277	288	267	280	265	271	262	263	258	255	254
30.15	112	105	111	99	110	100	109	101	108	102	107	103

Shear force and bending moments in columns

To study the effect of infill panels on the member force of a moment resisting RC frame structure, firstly the shear force and bending moments are studied for the case of full infill panel. Three typical columns are selected for the study; they are corner column, edge column and middle column. The comparison is made for the load combination 1.2(DL + LL + EQx). The bending moment and shear force diagram for corner, edge and middle columns are presented in Figure 31, Figure 32, and Figure 33 respectively.

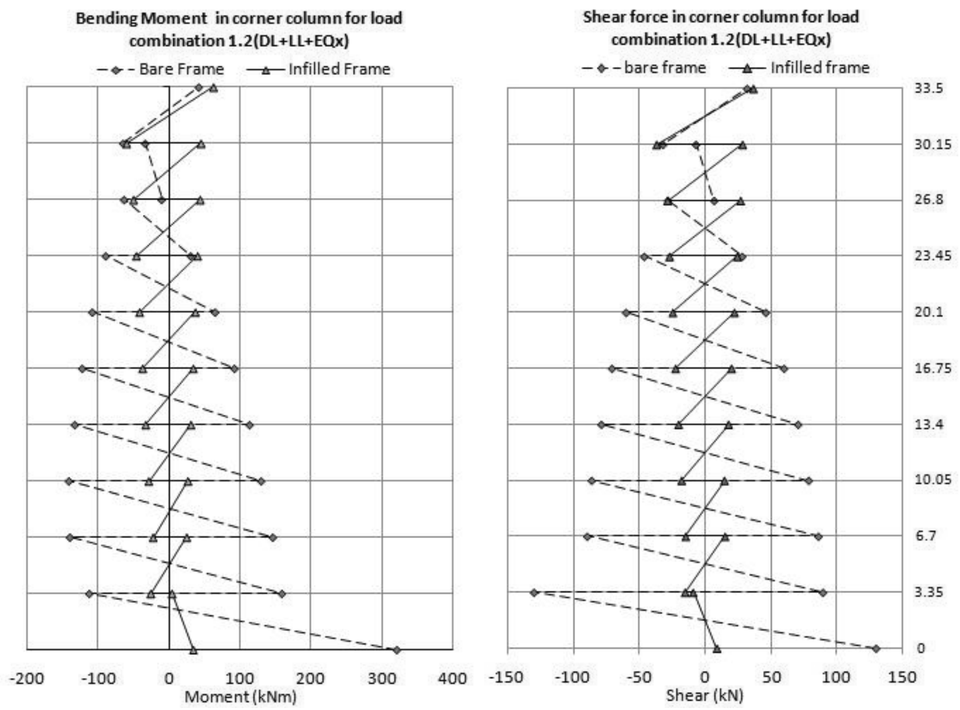


Figure 31 Comparison of member forces in corner column for full infill

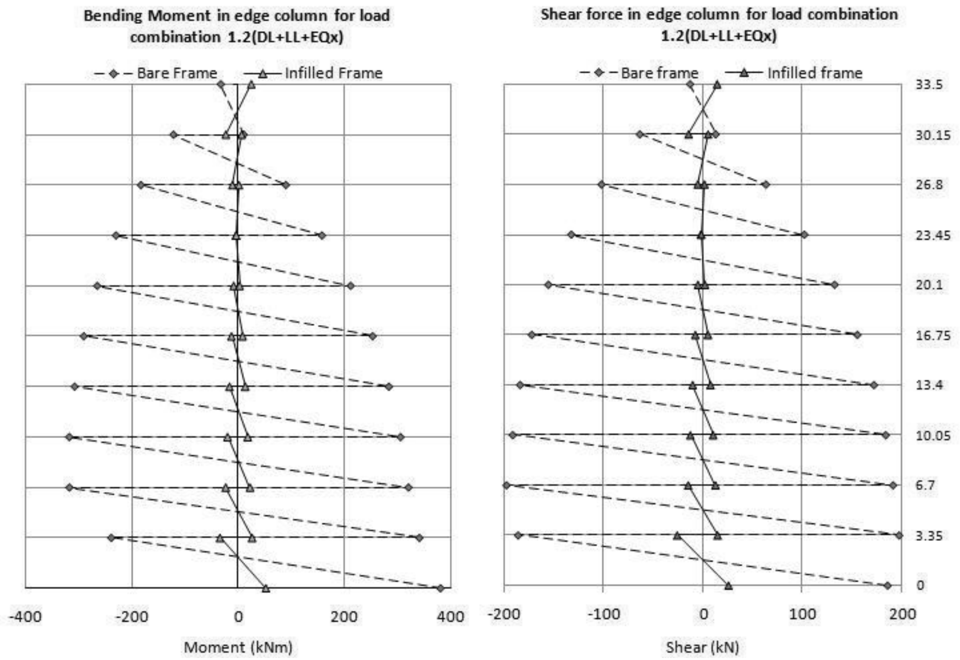


Figure 32 Comparison force in edge column for full infill

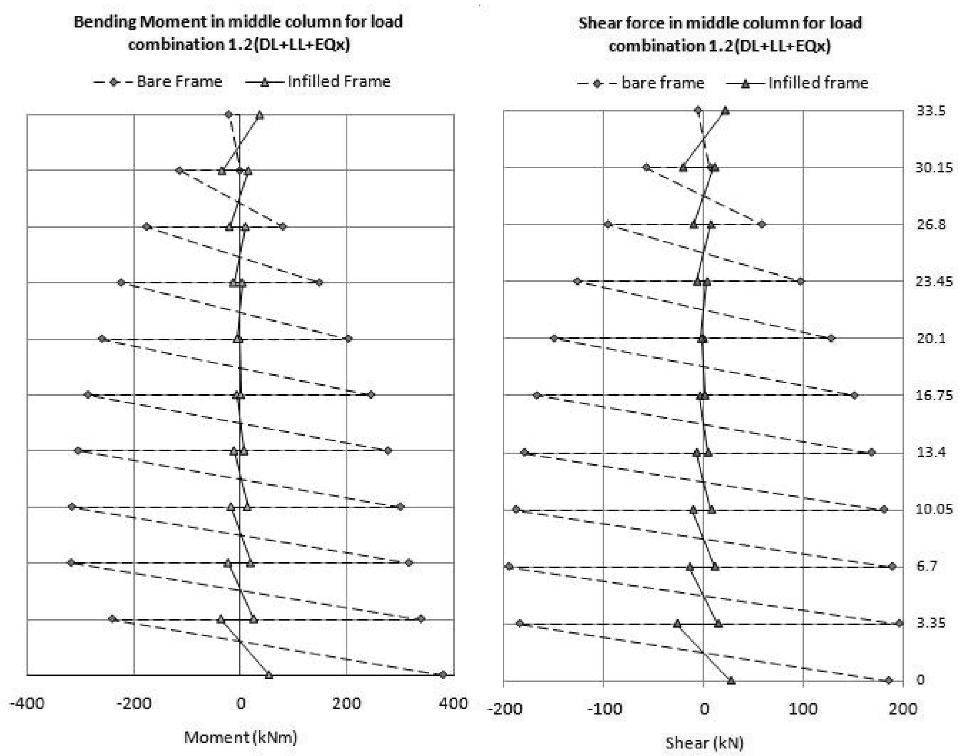


Figure 33 Comparison force in middle column for full infill

In the case of corner column, bare frame model predicts the maximum moment of about 320 kN-m at the bottom floor which is reduced by about 90% to about 34 kN-m in the case of infill model. 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 55% at the upper floors. As can be seen from the Figure 31, for the top two floors there is almost no reduction in the bending moment. Also, the bending moment shows a reversal of sign in some of the floors. This is true for all the cases of openings and position of columns in the building. The most probable cause of this can be due to the fact that there is a clear inflection point in the displacement profile of the building as shown in the Figure 23 and Figure 29.

Now, consider the figures for the case of edge column (Figure 32) and middle column (Figure 33). 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 the three columns as predicted by infill model to that by the bare frame model is same as that of bending moment.

Thus, from above discussion and from all the three figures, it is quite clear that the effect of infill on frame is to reduce the shear force and bending moments. In general for all the three columns, both shear force and bending moments are reduced by a huge margin. At the lower floors the reduction is more than 90%, which decrease to about 55% in the case of corner columns but remains about the same for edge and middle column even at the upper floors. For all the three columns, the trend is the same and hence for the case of infill with opening, only the middle column will be presented. 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.

The bending moments and shear forces for middle columns for the cases of infill panel with openings are presented in Figure 34 through Figure 38. The maximum values of bending moments predicted by bare frame models are 380, 365, 350, 335,320, and 305 kN-m respectively for 0, 10, 20, 30, 40, and 50% openings of infill panels. Now, compare these values with the bending moments predicted by respective infill frames; these are 54, 81, 85, 90, 92, and 94 kN-m, which in terms of percentage reduction are about 85%, 78%, 76%, 73%, 71%, and 69%. The bending moments for all the cases of opening and for bare frame and infill frame are shown in the Table 13. Comparing these values, it is observed that there is reduction in moments as an effect of introduction of infill panels in all the cases. However, as can be observed from the Figure 34 through Figure 38 and from Table 13, as the size of opening increases the reduction of moments decreases. Thus, the effect of infill panels is to reduce the member forces on columns of a RC frame in general, but the effect reduces as the opening size increases.

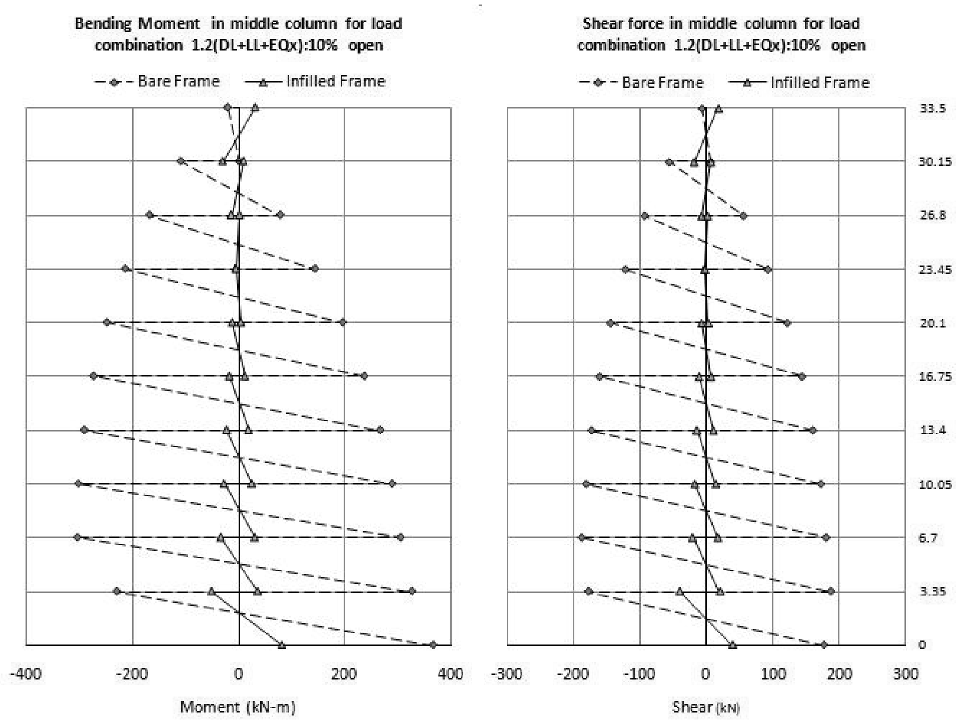


Figure 34 Comparison of Member forces for structure with 10% opening

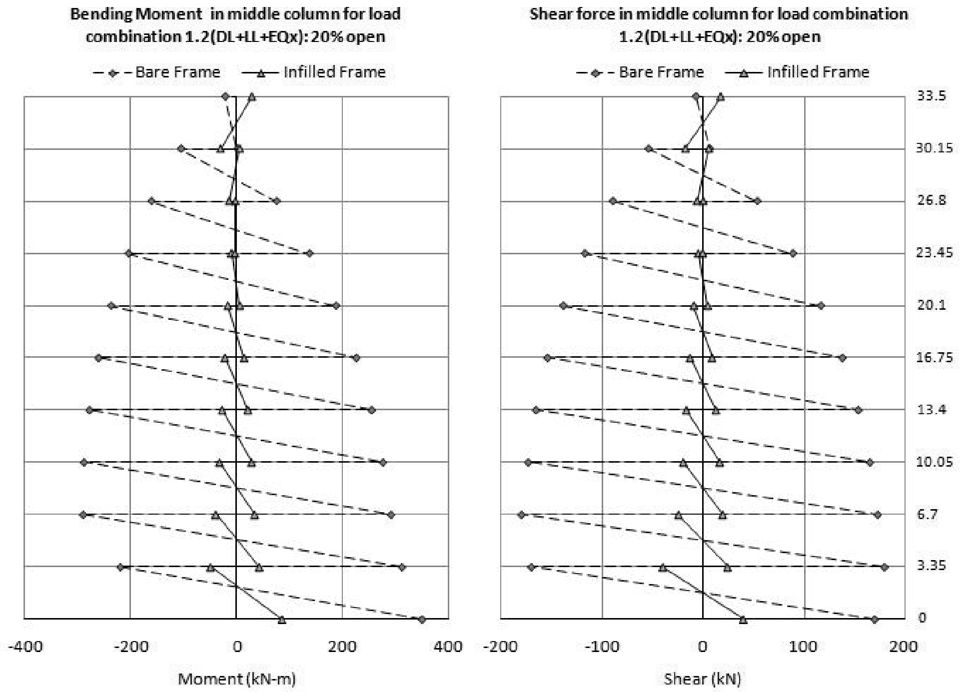


Figure 35 Comparison of member forces for structure with 20% opening

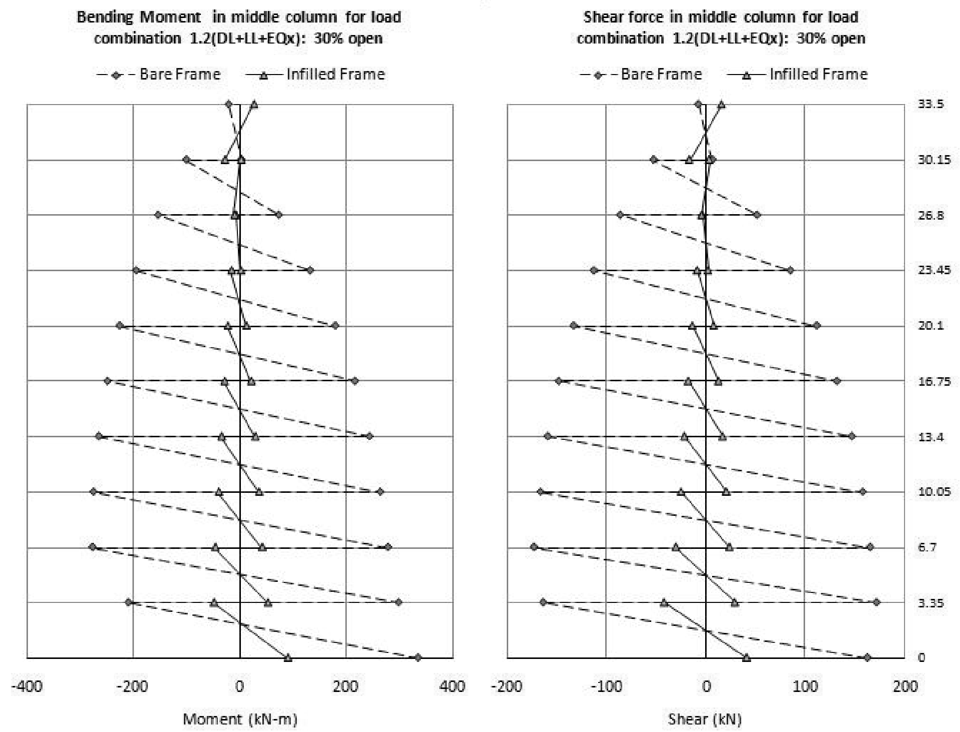


Figure 36 Comparison of member forces for structure with 30% opening

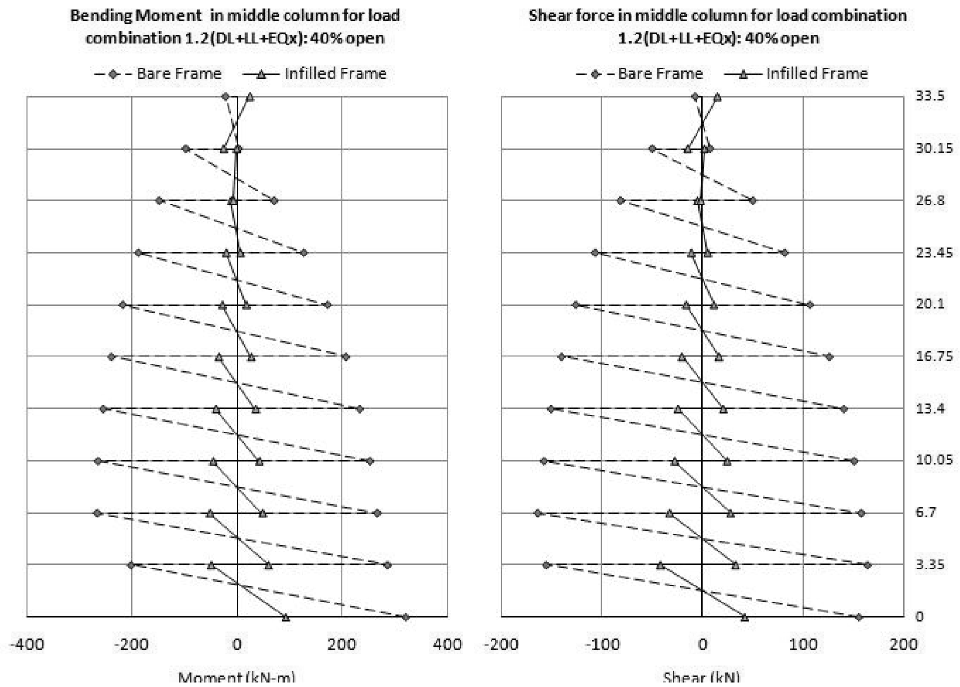


Figure 37 Comparison of member forces for structure with 40% opening

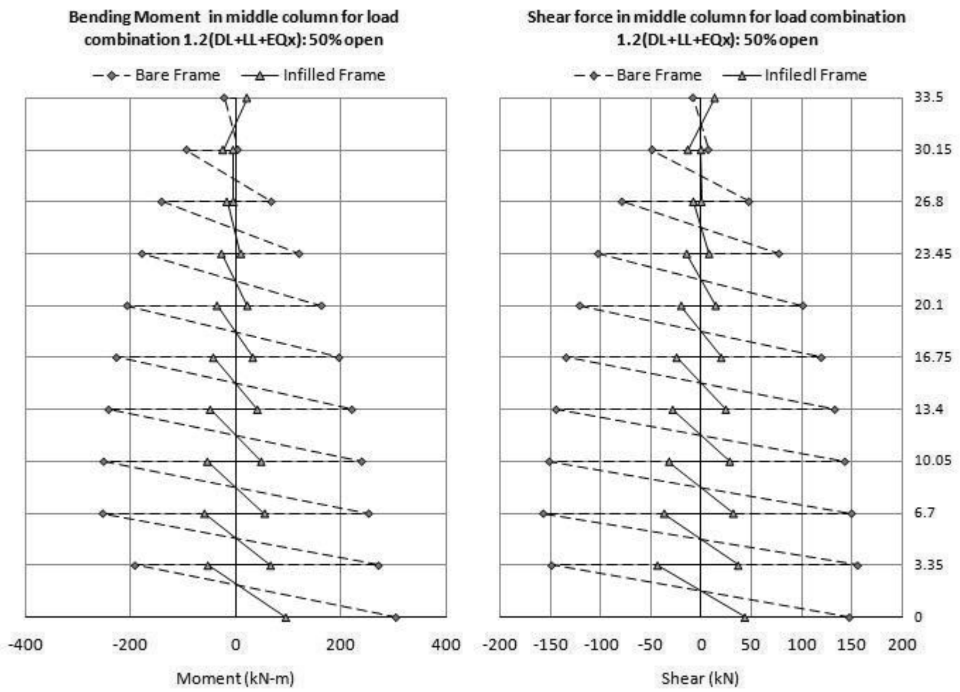


Figure 38 Comparison of member forces for structure with 50% opening

Table 13 Bending moments for bare frame and infill models for middle columns for all opening cases

height	0% opening		10% opening		20% opening		30% opening		40% opening		50% opening	
	bare	**infill**	**bare**	**infill**	**bare**	**infill**	**bare**	**infill**	**bare**	**infill**	**bare**	**Infill**
0	380.911	54.648	365.851	81.686	350.726	85.744	335.473	89.819	320.348	92.102	305.223	94.398
3.35	-238.984	-35.183	-229.485	-51.235	-219.945	-48.733	-210.325	-49.226	-200.786	-50.321	-191.247	-52.055
3.35	339.752	25.841	326.223	35.754	312.634	42.673	298.932	52.29	285.344	59.097	271.756	65.871
6.7	-315.967	-22.014	-303.315	-33.906	-290.606	-39.042	-277.792	-46.684	-265.085	-52.349	-252.378	-58.235
6.7	317.307	19.85	304.574	30.183	291.783	33.797	278.884	41.597	266.092	47.945	253.3	54.698
10.05	-314.425	-16.521	-301.744	-27.766	-289.005	-32.004	-276.159	-39.976	-263.421	-46.298	-250.682	-52.965
10.05	301.011	13.877	288.893	24.716	276.714	28.317	264.431	35.678	252.25	41.732	240.067	48.245
13.4	-302.981	-11.312	-290.707	-22.947	-278.374	-27.058	-265.937	-34.82	-253.604	-41.079	-241.269	-47.725
13.4	277.672	7.681	266.503	18.217	255.272	21.636	243.943	28.689	232.705	34.521	221.464	40.788
16.75	-284.523	-6.166	-272.987	-17.669	-261.39	-21.725	-249.694	-29.254	-238.093	-35.33	-226.49	-41.767
16.75	246.05	1.481	236.231	11.416	226.345	14.456	216.372	20.949	206.476	26.395	196.573	32.255
20.1	-258.343	-0.997	-247.917	-12.091	-237.427	-15.989	-226.846	-23.097	-216.348	-28.837	-205.845	-34.895
20.1	203.786	-5.065	195.82	3.81	187.783	6.34	179.67	12.006	171.616	16.859	163.549	22.104

Table 13 (continued)

height	0% opening		10% opening		20% opening		30% opening		40% opening		50% opening	
	bare	**infill**	**bare**	**infill**	**bare**	**infill**	**bare**	**infill**	**bare**	**infill**	**bare**	**Infill**
23.45	-222.367	4.454	-213.515	-5.836	-204.594	-9.495	-195.591	-15.953	-186.656	-21.166	-177.711	-26.641
23.45	149.026	-12.21	143.502	-4.891	137.902	-3.031	132.243	1.497	126.615	5.528	120.971	9.923
26.8	-174.6	10.567	-167.895	1.454	-161.112	-1.888	-154.261	-7.423	-147.455	-11.889	-140.635	-16.554
26.8	80.798	-19.487	78.296	-14.59	75.741	-13.635	73.145	-10.572	70.55	-7.597	67.931	-4.287
30.15	-113.129	15.798	-109.176	8.867	-105.158	6.197	-101.089	2.056	-97.036	-1.318	-92.961	-4.837
30.15	-0.197	-33.368	0.666	-30.572	1.515	-30.191	2.349	-28.464	3.151	-26.436	3.924	-24.136
33.5	-20.901	37.208	-21.135	30.912	-21.343	28.894	-21.528	26.158	-21.682	23.586	-21.802	20.855

Shear force and bending moments in beams

Similar to the case of columns, to study the effect of infill on beams of a moment resisting RC frame structure, the shear force and bending moments are studied for the case of full infill panel. For the purpose of comparison, one each typical beams at the periphery and at the middle are selected. Also for this case of full opening, beams at three different floor levels are selected; they are floor level 2, 5 and roof. The comparisons were made for the load combination 1.2(DL + LL + EQx). The bending moment and shear force diagram for the above load case for these two beams at different floor levels are shown in Figure 39 and Figure 40.

First, the comparison is done for the case of peripheral beams. The maximum bending moments of peripheral beam at floor level 1 and 5 are about 410 kN-m when the effect of infill is not considered. This is reduced to about 120 kN-m when the effect of infill is taken into account, which is about 70% reduction. Further, in the case of roof level beams the maximum bending moment by considering bare frame is about 85 kN-m whereas, it is about 65 kN-m in the case of infill model which is about 24% reduction. Similarly for the middle beams, the maximum floor moment was about 453 kN-m from the bare frame analysis and 163 kN-m from infill frame analysis, which is about 64 % reduction. The roof level moments by bare frame analysis was 118 kN-m and 83 kN-m from infill frame analysis which is about 29% reduction.

Similarly, shear forces in peripheral beams at both floor levels were reduced to about 98kN form 210 kN and the shear force at roof level was reduced to about 46 kN from 54 kN, which are about 56% and 15% reduction when the effect of infill was considered. Likewise, the shear forces in middle beams were reduced from about 235 kN to 121 kN at both floor levels and to 93 kN from 118 kN at roof level which is about 50% and 20 % reduction respectively. Thus, in general the shear force and bending moments in beams are reduced by the introduction of infill panels which are not taken into account in the conventional design practice.

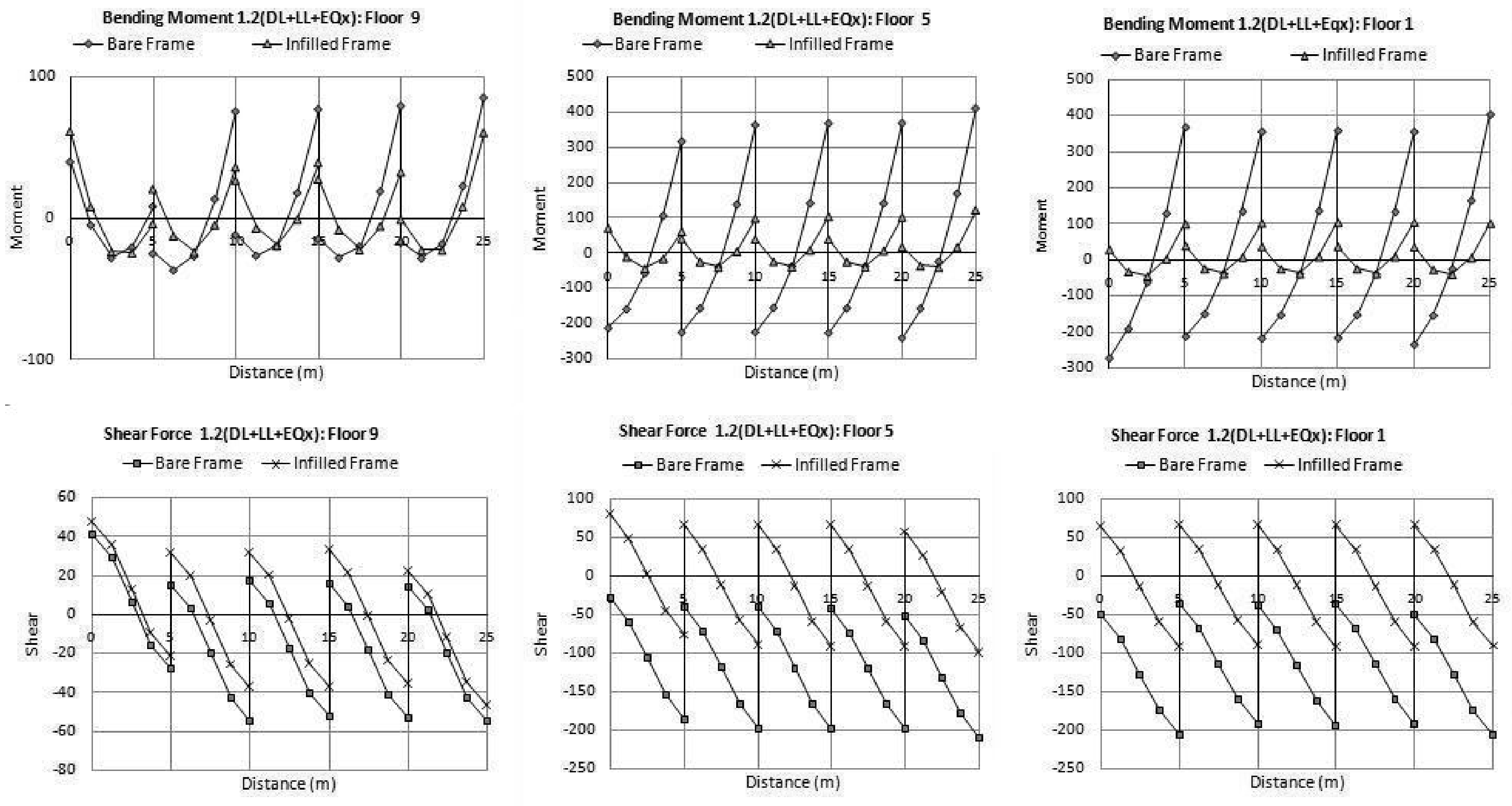


Figure 39 Member forces in edge beam for full infill

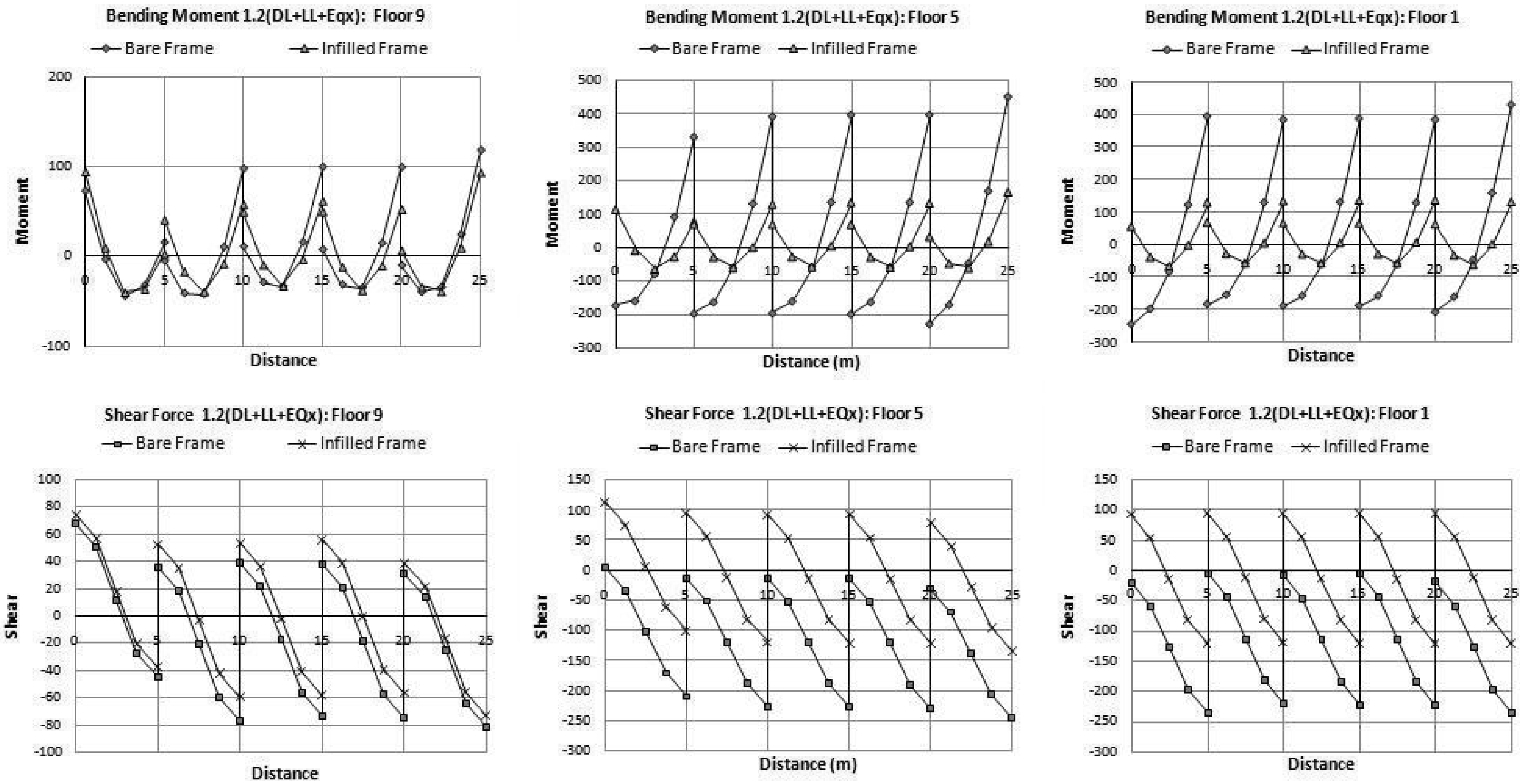


Figure 40 Member forces in middle beam for full infill

Now to study the effect of opening sizes of the infill panel to the member forces of RC frame, the results of bare frame models are compared with the respective infill frame models with various opening sizes. Similar to the case of infill with no opening, even in the case of infill with openings, the effect of infill seems to be very less in the roof beams and hence, these are not compared. The member force diagram for the infill model with 10 to 50% opening size is shown in Figure 41 through Figure 45.

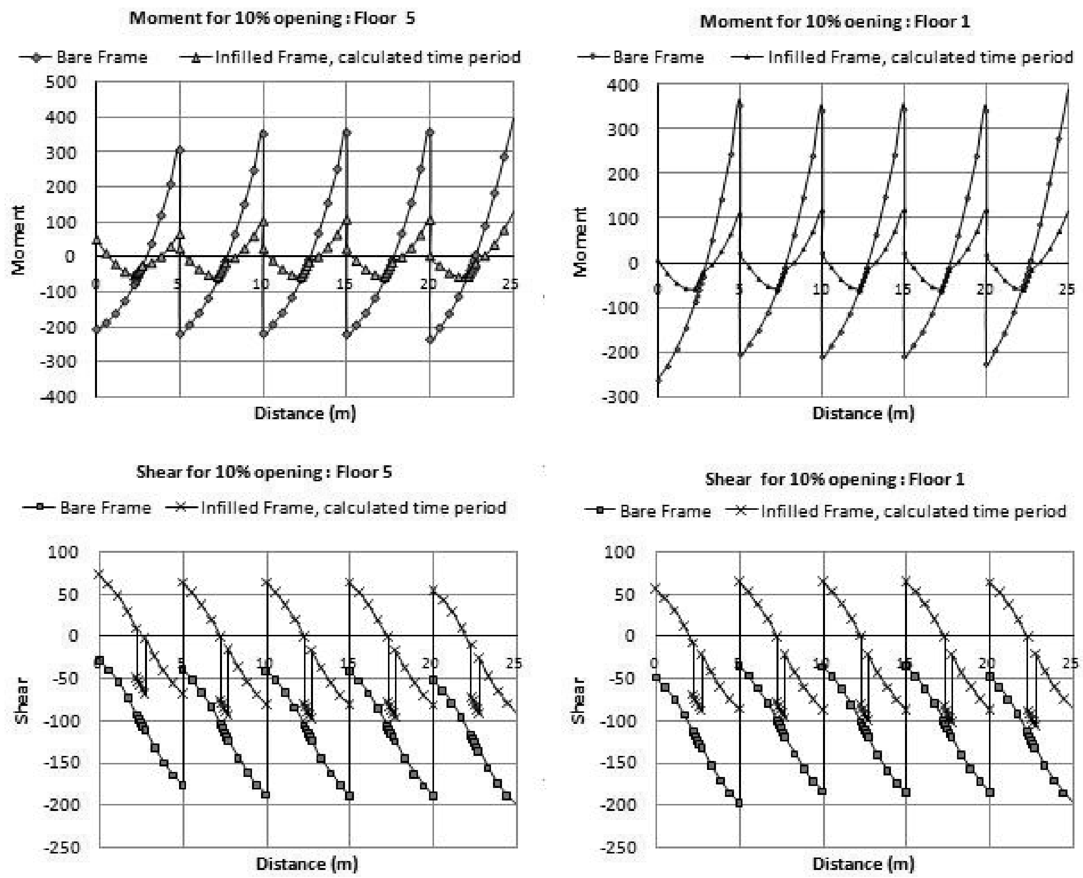


Figure 41 Member forces in edge beam for 10% opening 1.2(DL+LL+EQx)

The member force diagram for the infill with 10% centrally located opening is shown in Figure 41. For this case, the maximum moment for the seismic load combination in peripheral floor beams as predicted by bare frame model is about 395 kN-m. The infill model predicts a lesser value of about 127 kN-m, which is about

68% less than the bare frame model. Similarly, the maximum shear force in edge beams computed using bare frame model is about 200 kN whereas, that from infill model is about 92 kN.

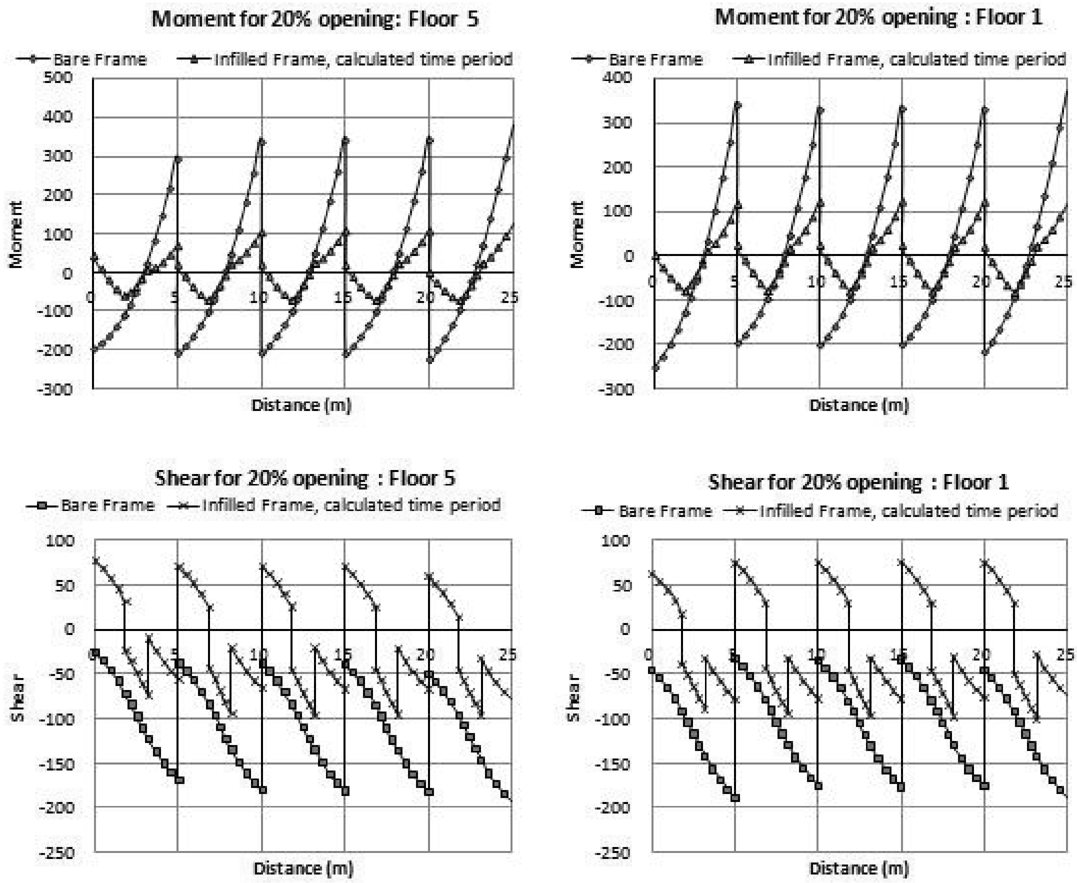


Figure 42 Member forces in edge beam for 20% opening 1.2(DL+LL+EQx)

The member force diagram for the infill with 20% centrally located opening is shown in Figure 42. In the peripheral floor beams, the bare frame model predicted the maximum moment of about 380 kN-m for the seismic load combination. The infill model predicts a lesser value of about 127 kN-m, which is about 67% less than the bare frame model. Similarly, the maximum shear force in edge beams computed using bare frame model is about 192 kN whereas, that from infill model is about 78 kN.

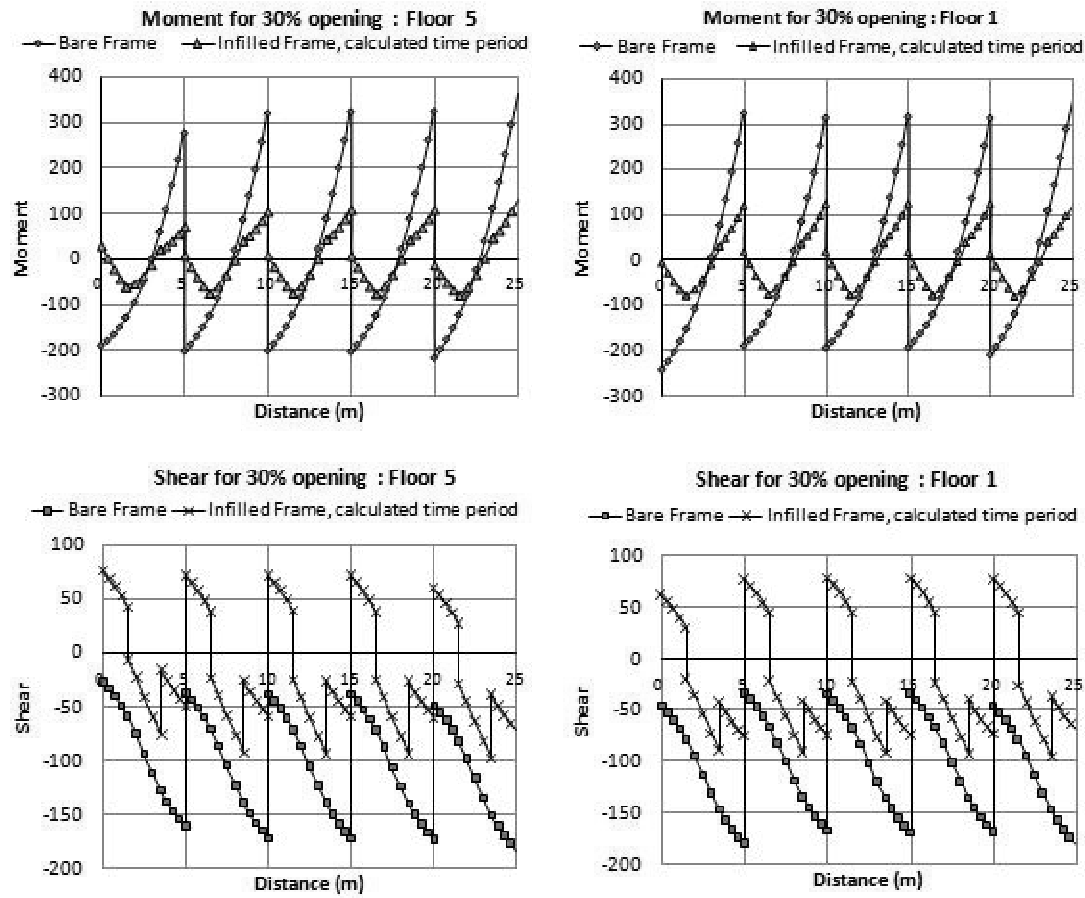


Figure 43 Member forces in edge beam for 30% opening 1.2(DL+LL+EQx)

The member force diagram for the infill with 30% centrally located opening is shown in Figure 43. In the peripheral floor beams, the bare frame model predicted the maximum moment of about 364 kN-m for the seismic load combination. The infill model predicts a lesser value of about 131 kN-m, which is about 64% less than the bare frame model. Similarly, the maximum shear force in edge beams computed using bare frame model is about 183 kN whereas, that from infill model is about 71 kN.

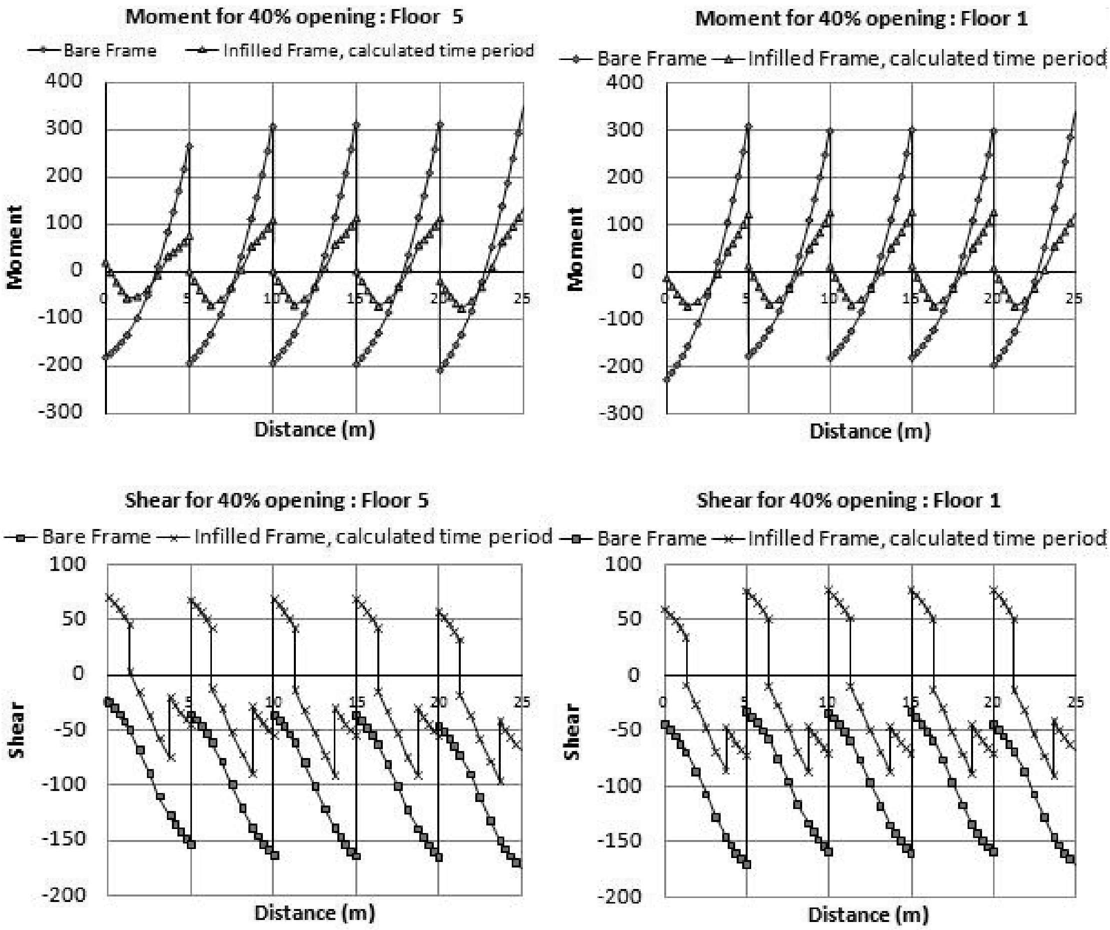


Figure 44 Member forces in edge beam for 40% opening 1.2(DL+LL+EQx)

The member force diagram for the infill with 40% centrally located opening is shown in Figure 44. In the peripheral floor beams, the bare frame model predicted the maximum moment of about 347 kN-m for the seismic load combination. The infill model predicts a lesser value of about 134 kN-m, which is about 64% less than the bare frame model. Similarly, the maximum shear force in edge beams computed using bare frame model is about 175 kN whereas, that from infill model is about 67 kN.

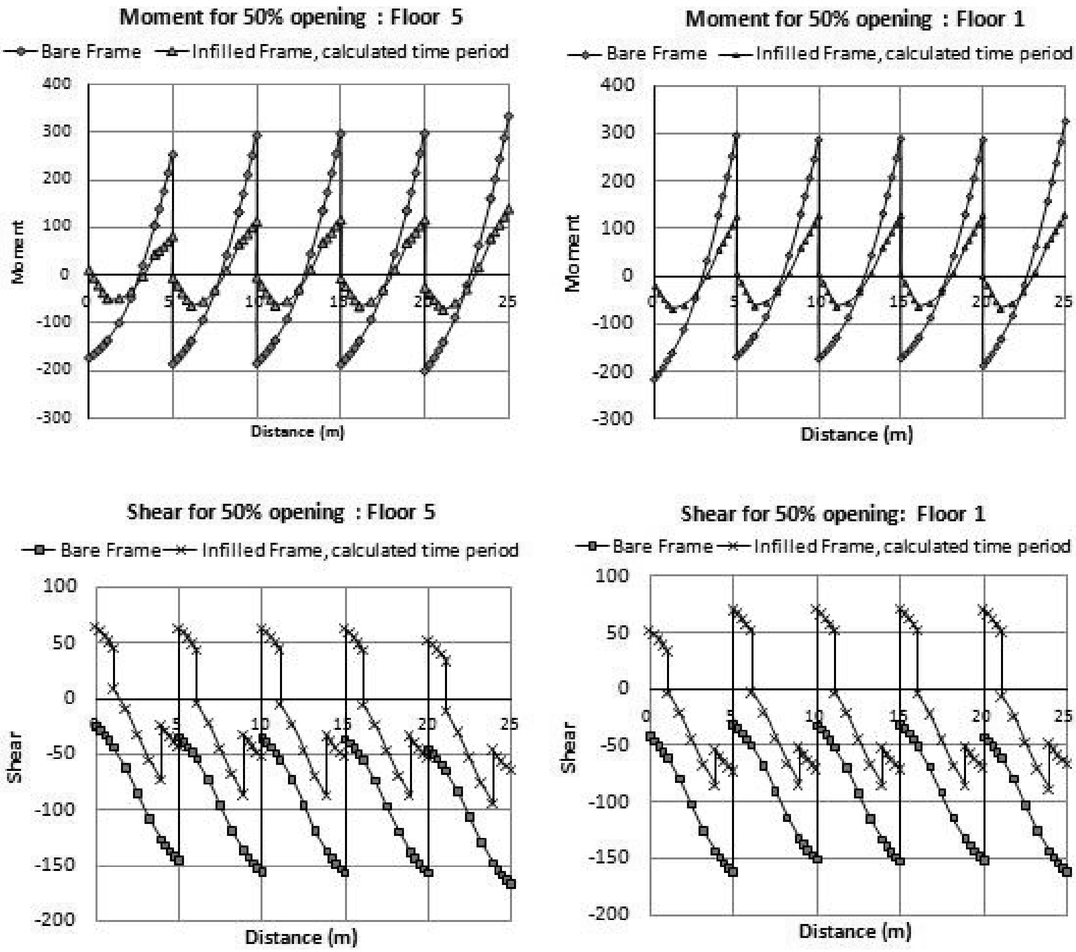


Figure 45 Member forces in edge beam for 50% opening 1.2(DL+LL+EQx)

The member force diagram for the infill with 50% centrally located opening is shown in Figure 45. In the peripheral floor beams, the bare frame model predicted the maximum moment of about 332 kN-m for the seismic load combination. The infill model predicts a lesser value of about 137 kN-m, which is about 61% less than the bare frame model. Similarly, the maximum shear force in edge beams computed using bare frame model is about 166 kN whereas, that from infill model is about 65 kN. One interesting fact about the reduction of member forces when the infill is considered is that the decrease in member forces is more towards the end span of each beam. On the other hand in the mid span of the beams, just above the opening, there is not much difference in member forces computed based on bare frame model and infill model.

CONCLUSIONS AND RECOMMENDATIONS

Conclusion

In most of the developing countries around the world, most of the multi-story buildings consist of moment resisting reinforced concrete frames. The vertical space created by 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 in cement mortar. The infill panels are not usually an integral part of the moment resisting structure. Mostly, these masonry walls have openings in them due to functional demand such as doors and windows.

Past studied and experience gained during past earthquakes showed the beneficial as well as ill effects of infill walls. Infill walls increase the global lateral strength. Infill wall also increases damping of the structure and hence increases the energy dissipation capacity. It decreases the inter-story drift and hence the total lateral deflection of the structure. However, infill posses some ill effects on the building structure such as soft story, short column effect and torsion.

However, design engineers tend to neglect the strength and stiffness effect of infill while designing a building structure and treat these masonry infills as a non-structural component. This is mainly due to the lack of generally accepted seismic design criteria. In fact, many building codes of the world do not provide specifications to design the infill walls. Neglecting the effect of infill walls while designing can lead to erroneous or uneconomical design. Since early fifties, many researchers have developed a number of micro as well as macro models to compute the lateral strength, stiffness and deformation capacity, but there exists considerable variation in results obtained by these models. By far, the most popular analytical model has been the equivalent diagonal strut model. In this method, the brick infill is idealized as a pin jointed single diagonal strut and the RC beams and columns are modeled as a three-dimensional beam elements having 6 degree of freedoms at each node. The idealization is based on the assumption that there is no bond between frame and infill. The brick masonry 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 various factors like; contact length, aspect ratio of the infill and the relative stiffness of frame and the infill. Various researchers had proposed different strut width, however in the present study the effective width as suggested by Holmes, Pauley and Priestley, and FEMA 273 were 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 a small effort towards the understanding of the effect of infill wall, both full and partial to the moment resisting RC framed structure under seismic loading condition. Three analytical models; bare frame, infill with code prescribed time period, and infill with calculated time period were prepared for full infill and their results compared. Later on, for case of partial infill walls only two models viz, bare frame with code prescribed time period and infill frame with analytically calculated time period were used. The main conclusions are summarized below:

1. Though the span lengths in both X & Z axis were the same, the X axis has more number of spans. Fundamental time period and base shear computed for the full infill case closely matched with the code prescribed value, Pauley & Priestley model giving the closest match. Time period and base shear in X-axis were much close to the code value but infill frame predicted a stiffer structure than the code in the Z-axis. As the size of opening was increased the analytical model gave more flexible structure than the code. At 50% opening the time period and base shear was closer to the bare frame model with calculated time period.

2. The seismic excitation in terms of design lateral force, story shear and story moment computed from infill model closely matches with the code value for the full infill case. For infill model with opening, the design seismic excitations were reduced,

the reduction being more for the infill with larger opening and vice versa. This is due to the fact that the frame with larger opening has less stiffness and hence smaller lateral force.

3. From the analytical study it is observed that the seismic demand of a structure in terms of inter-story drift and hence the average displacement as well as maximum displacement at roof level of a structure is greatly improved by the introduction of infill walls. As the full infill provides largest stiffness increase, the maximum roof level displacement predicted by the infill model is reduced by around 85% compared to the bare frame model. The roof level displacement for 10, 20, 30, 40, and 50% opening with infill model are reduced by 82, 78, 74, 71 and 68% respectively as compared to the respective bare frame model. Thus, the frame with full infill has a better response during earthquake excitation than the one with partial infill

4. The effect of infill wall is to change the predominantly a frame action of a moment resisting frame structure towards a truss action. The axial forces in columns are increased in infill frame model compared to a bare frame model.

5. The response of a structure in terms of shear forces and bending moments are greatly improved in an infill model. Both shear force and bending moments are reduced greatly by the introduction of infill panels. The response is better in a full infill panel than a partial infill.

6. The shear force and bending moments are reduced by a greater margin in the lower columns. There was almost no decrease in shear and moment at the top most columns. Even in this case the response is better in a full infill panel than in the infill panel with opening.

7. In the case of beams, the reduction in bending moment is more pronounced in lower floor up to 5^{th} floor. At the roof level there is very marginal difference form bare frame model and infill model

8. In general, the infill panel placed symmetrically seems to have a beneficial response on a building structure under seismic loading.

Recommendations

1. Bare frame models gives significantly longer time period than predicted by the code equations, and hence smaller lateral forces. Thus, building codes impose an upper limit on the natural period determined from a rational numerical analysis by empirical equations. However, when the effect of infill is included, the time periods determined from analysis for smaller openings were found to be close to code formulas whereas, it is close to the bare frame with analytical time period for the large opening. 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 and equivalent static method for the seismic analysis. This could be extended to nonlinear properties of infill and dynamic analysis to cater for the structure with horizontal as well as vertical irregularity.

4. The study was carried out for full and partial infill with centrally located square opening of different sizes. This can be extended to partial infill with opening size of different aspect ratio which will be practically applicable.

5. Further study on partial infill with openings at various locations could lead to valuable information regarding the practical aspect of design work.

6. 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.

7. The present study was based on the symmetrical placement of infill panel. It would be of great practical benefit for the designers if this can be extended to irregular and/or unsymmetrical placement of infill panels on a multi-story building frame. This might lead to an insight on soft story and short column effect due to the presence of infill panel.

8. 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.

9. 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 does not 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.

LITERATURE CITED

Al-Chaar , G., M. Issa and S. Sweeney. 2002. Behavior of Masonry Infilled Non-ductile Reinforced Concrete Frame. **Journal of Structural Engineering** ASCE, 128 (8): 1055-1063.

ASCE 7. 2005. **Minimum Design Loads for Buildings and Other Structures Including Supplement 1**. ASCE/SEI 7-05, American Society of Civil Engineers, Reston, Virginia, USA.

Buonopane, S.G and R.N. White. 1999. Pseudodynamic testing of masonry infilled reinforced concrete frame. **Journal of Structural Engineering** ASCE, 125 (6): 578-589.

FEMA 273. 1997. **NEHRP Guidelines for the Seismic Rehabilitation of Buildings**. Building Seismic Safety Council, Federal Emergency Management Agency, Washington (DC).

FEMA 306. 1998. **Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings, Basic Procedures Manual**. Applied Technology council, Federal Emergency Management Agency, Washington (DC).

FEMA 450. 2003. **NEHRP Recommended Provisions for Seismic Regulations for New Buildings and other Structures**. Building Seismic Safety Council, Federal Emergency Management Agency, Washington (DC).

Holmes, M. 1961. Steel Frames with Brickwork and Concrete Infilling. **Proceedings of Institution. Of Civil Engineers** London, Vol 19: 473-478.

IS 875. 1987. **Code of practice for design loads (other than earthquake) for buildings and structures**. 2nd Revision, Bureau of Indian Standards, New Delhi.

IS 1893 (Part 1): 2002. **Criteria for Earthquake Resistant Design of Structure**, Bureau of Indian Standards. New Delhi.

Kaushik, H.B., D.C. Rai and S.K. Jain. 2006. Code Approaches to Seismic Design of Masonry-Infilled Reinforced Concrete Frames: A State-of-the-Art Review. **Earthquake Spectra** EERI, 22 (4): 961-983.

Liauw, T.C. 1979. Test on Multistory Infilled Frames Subject to Dynamic Lateral Loading. **ACI Structural Journal**. Title no. 76-28: 551-563.

Liauw, T.C. and S.W. Lee. 1977. On The Behavior and The Analysis of Multistory Infilled Frames Subjected to Lateral Loading. **Proc. Institute of Civil, Engineers**. Part 2. Vol 63: 651-656.

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.

Mallick, D.V. and R.P. Garg. 1971. Effect of Openings on the Lateral Stiffness of Infilled Frame. **Proceedings of Institute of Civil Engineers**, 49(6): 193-209.

Mehrabi, A B., P.B Shing, Micheal P. Schuller and James L. Noland. 1996. Experimental Evaluation of Masonry Infilled RC Frames. **Journal of Structural Engineering,** ASCE, 122 (3): 228-237.

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

Mostafaei, H. and T. Kabeyasawa. 2004. Effect of infill masonry walls on the seismic response of RC buildings subjected to the 2003 Bam earthquake strong motion: A case study of Bam telephone center. **Bulletin of earthquake research institute**, University of Tokyo, 79: 133-156.

Murthy, C.V.R. and S.K. Jain. 2000. Beneficial influence of masonry infills on seismic performance of RC frame buildings. **Proceedings of 12th World Conference on Earthquake Engineering**, New Zealand, Paper No. 1790

NBC 201. 1994. **Mandatory Rules of Thumb Reinforced Concrete Buildings With Masonry Infill**. Department of Urban Development and Building Construction, Nepal

NBC 205. 1994. **Seismic Design of Buildings in Nepal**. Department of Urban Development and Building Construction, Nepal

Pauley, T. and M.J.N. Priestley, 1991 . **Seismic Design of Reinforced Concrete and Masonry Buildings**. John Wiley & Sons, Inc.

Shrestha, S. 2005. **Interaction between RC Frames & Brick Masonry Infill Wall**. M.S Thesis, Tribhuvan University, Institute of Engineering, Pulchowk Campus.

Smith, B.S. 1962. Lateral Stiffness of Infilled Frames. Proceeding of the American Society of civil Engineering, **Journal of Structural Engineering,** ASCE, 88 (ST6): 183-199.

Smith, B.S. 1966. Behavior of Square Infilled Frames. Proceeding of the American Society of civil Engineering **Journal of Structural Engineering**, ASCE, 92 (ST1): 381-403.

Smith, B.S. 1967. Method For Predicting The Lateral Stiffness And Strength Of Multi-Storey Infilled Frames. **Building Science** Vol. 2: 247-257.

Smith, B.S. and A. Coull. 1991. **Tall Building Structures: Analysis and Design**. John Wiley & Sons, Inc.

Wilson, E.L. 2002. **Three Dimensional Static and Dynamic Analysis of Structures – A Physical Approach with Emphasis on Earthquake Engineering.** 3rd Edition. Computers and Structures, Inc. Berkeley, California, USA.

Zarnic, R. 1995. Modeling of response of Masonry Infilled Frames,. **10th European Conference on Earthquake Engineering**, Vol 1: 1481-1486.

APPENDICES

Appendix A

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 non structural 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 storey 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 as shown in Figure 2. 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. However, different researcher had proposed different values for the effective width.

Holmes (1961) proposed replacing the infill by an equivalent pin jointed diagonal strut of the same material and thickness with a width equal to one-third of its diagonal length. Pauley and Priestley (1992) suggested that the effective width shall be one-fourth the diagonal length. FEMA 273 use the relation proposed by Mainstone (1971) which relates the width w of infill to parameter λh (B. S. Smith,1967) and given by equation (A1) and diagonal length d as shown in the equation (A2).

$$\lambda h = h \times \sqrt{\frac{E_m t \sin 2\theta}{4 E_c I_c h_m}} \qquad \text{(A1)}$$

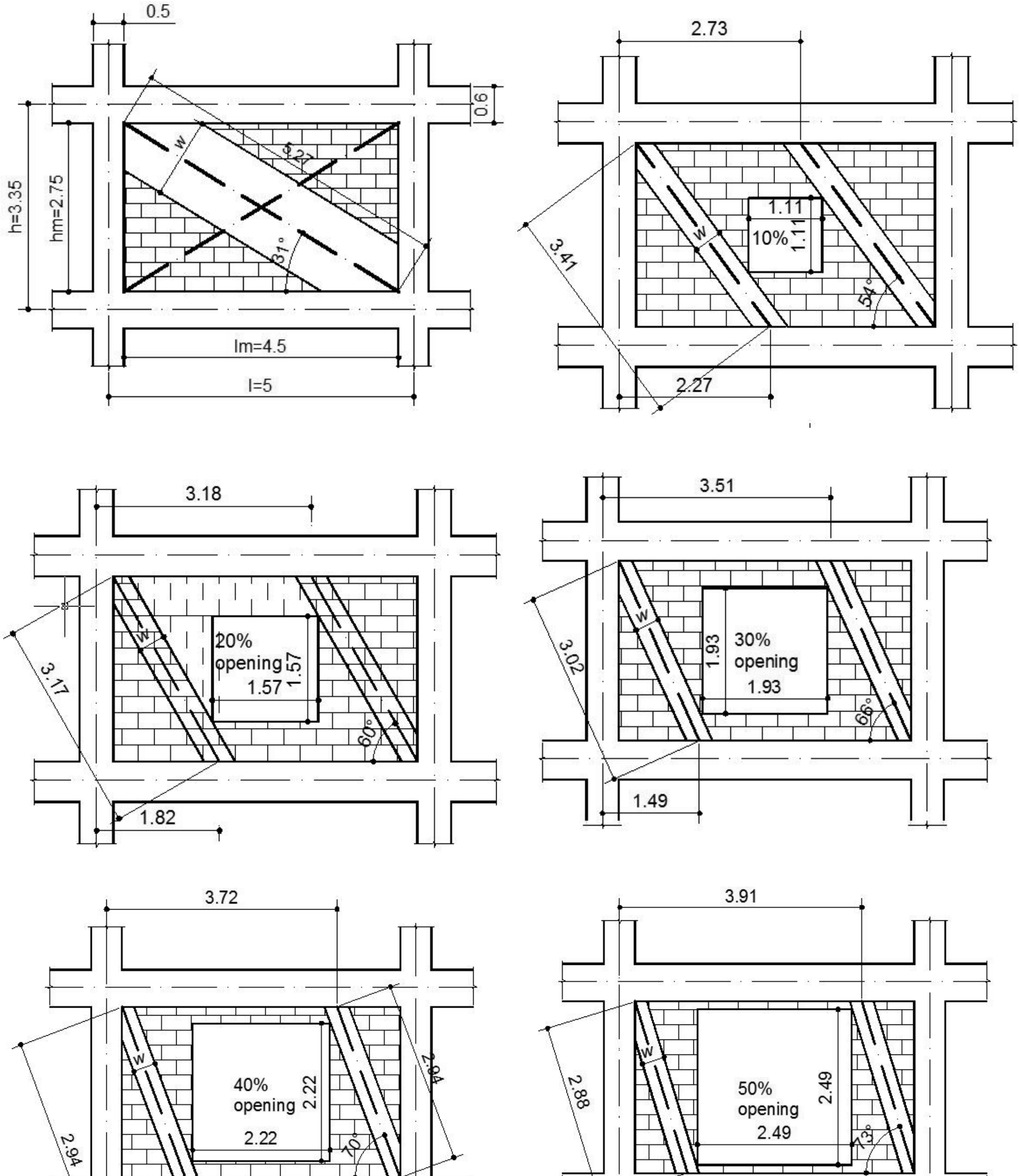
$$\frac{w}{d} = 0.175 (\lambda h)^{-0.4} \qquad \text{(A2)}$$

Where, h is height of column, E_c and E_m are young's modulus of frame and infill panel respectively, t is thickness of infill panel, θ is angle of inclination of diagonal strut with the horizontal, I_c is the moment of inertia of column and h_m is the height of infill.

Thus, the effective width as proposed by Holmes and Pauley & Priestley can be found by just knowing the diagonal length whereas for FEMA the full geometry has to be known. The effective widths calculated are shown in Appendix Table A 1.

Appendix Table A 1 Effective width of diagonal strut

Opening %	**Opening size (mm*mm)**	**diagonal length (mm)**	**diagonal angle (degree)**	**effective width**		**effective width from FEMA**	
				Holmes (mm)	**Pauley & Priestley (mm)**	**λ**	**width (mm)**
0	-	5270	31^{o}	1758	1320	9.076E-04	592
10	1110 x 1110	3410	54^{o}	568	430	9.237E-04	380
20	1570 x 1570	3170	60^{o}	528	400	9.000E-04	356
30	1930 x 1930	3020	66^{o}	503	378	8.694E-04	344
40	2220 x 2220	2940	70^{o}	490	368	8.476E-04	339
50	2490 x 2490	2880	73^{o}	480	360	8.087E-04	338



Appendix Figure A 1 Opening size with diagonal length and diagonal angle

Appendix B

Loadings

Loading for structure

STAAD.Pro has extensive load generation facilities to generate floor loads for dead and live load as well as earthquake load. However, the load intensity for dead and live load has to be provided in order to generate member loads on beams and earthquake load.

Gravity Loading

As the software has the capability to generate loads on beam, we need only to provide the loading intensities for the floor. The loading intensities for dead load are 4.85 kN/m^2 at floor level and 5.05 kN/m^2 at roof level. These include, self-weight of slab, partition loads, finishing, and service loads. For the case of live loads, applied loading intensities are 3 kN/m^2 at floor levels and 0.75 kN/m^2. All these loads are computed as per the IS 875 (Part 1 and 2).

Seismic Loading

As defined in IS 1893, the seismic weight of each floor is its full dead load plus appropriate amount of imposed load. At floor level, for 3 KPa imposed loading, only 25% is taken for computing seismic loading. At the roof level no imposed load is considered for computing seismic load. The procedure for seismic analysis using seismic coefficient method is outline below.

After computing the seismic weight of the structure, the next step would be to find out the time period of the structure. The IS 1893 imposes and upper limit to the fundament time period by the empirical equations as follows;

$T_a = 0.075h^{0.75}$; For RC frame building,

$T_a = 0.085h^{0.75}$; For steel frame building and

$T_a = 0.09\, h/\sqrt{d}$; For moment resisting frame building with brick infill panels. Where, *(h)* is the height of building in meter and (*d)* is the base dimension of the building at the plinth level, in meter, along the considered direction of the lateral force. For the present case this equation should be used to compute the time period.

Then, the design base shear V_B which is the total lateral force at the base of a structure is computed in accordance with the clause 7.5.3 of the code which is given by equation (B1),

$$V_B = A_h W \tag{B1}$$

Where, W = total seismic weight of the building and A_h is given by the equation (B2).

$$A_h = \frac{Z}{2}\frac{I}{R}\frac{S_a}{g} \tag{B2}$$

Where

Z = Zone factor = (0.10 for zone II, 0.16 for zone III, 0.24 for zone IV, and 0.36 zone V). Zone I has been removed from the code in the present revision.

I = Importance factor = 1.0 (for general buildings, and 1.5 for important buildings like hospitals etc.).

R = Response reduction factor = 1.5 ~ 5 for frame with ductile detailing as per IS 13920

S_a/g = Spectral acceleration coefficient, read from Figure 13 corresponding to fundamental natural time period. For the present case, $T_a = 0.09\, h/\sqrt{d}$.

But for any structure with T < 0.1 sec, A_h is not less than (*Z/2*) whatever the value of (I /R) be.

Now, once the design base shear is known, this has to be distributed to all the floors as a design lateral force. The IS 1893 use the parabolic distribution of base shear to floors using the equation (B3).

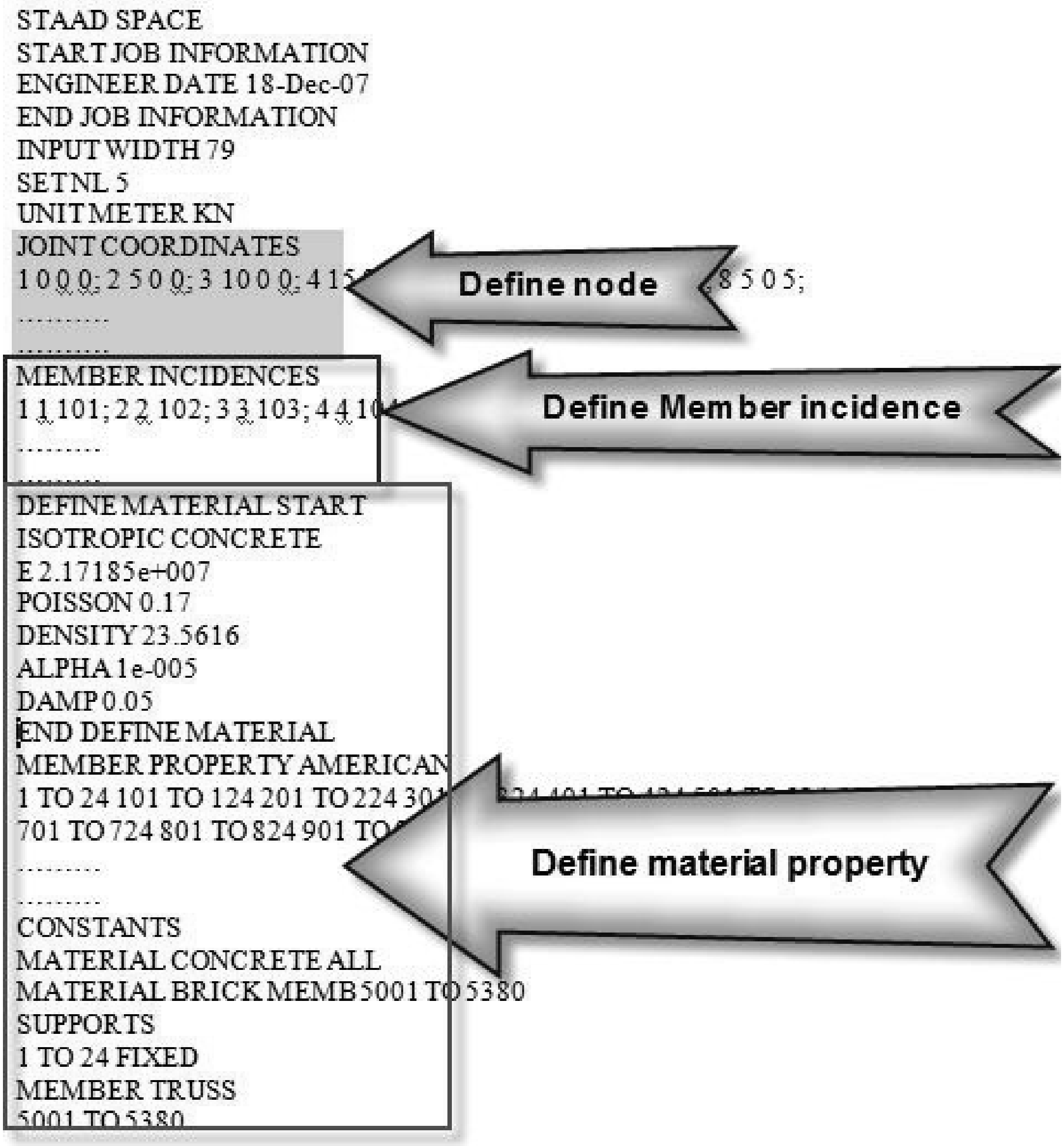
$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2} \tag{B3}$$

Where, Q_i = Design lateral force at floor *i*, W_i = Seismic weight of floor *i*, h_i = Height of floor *i* measured from base, *n* = Number of stories in the building,

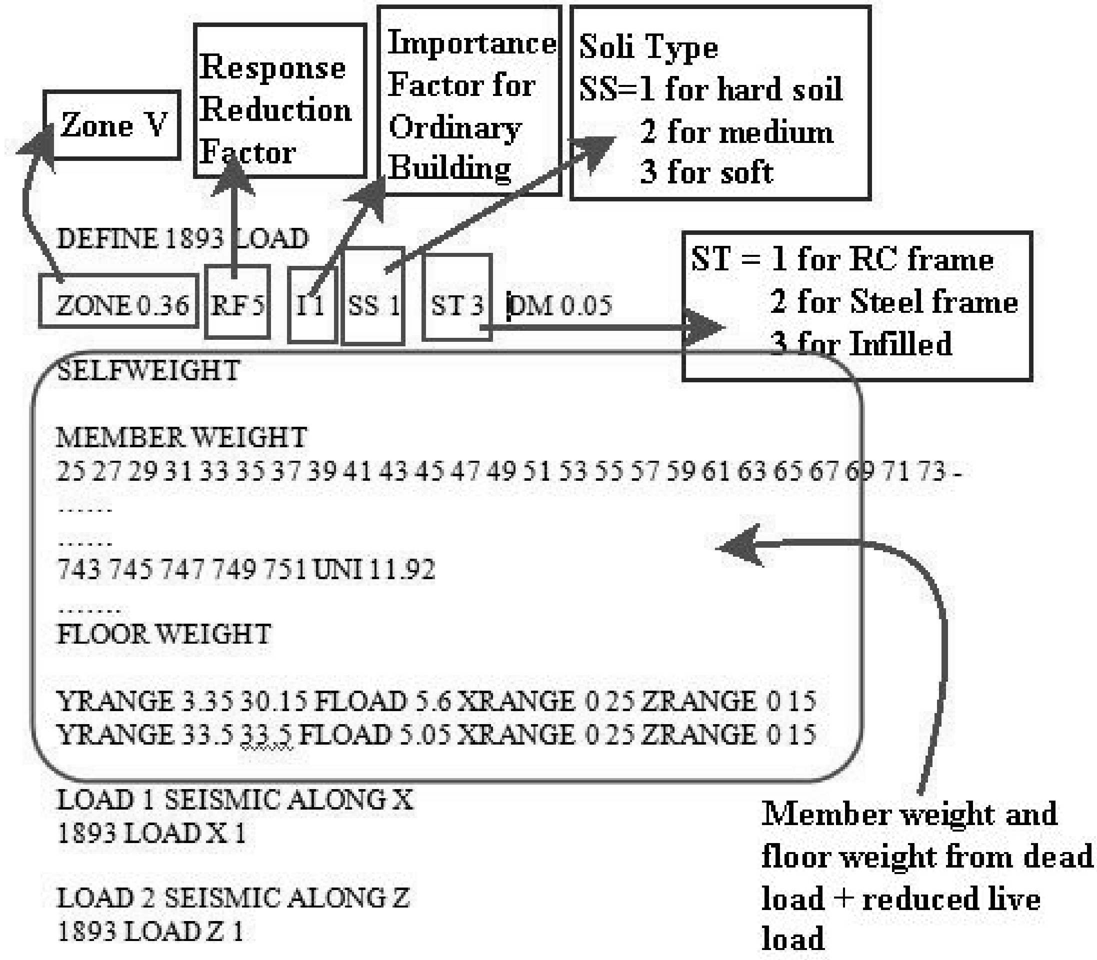
Seismic Load generation using STAAD.Pro

The figure B1 shows the command required to generate the geometry of the structure. The command required to generate earthquake loading is shown in figure B2.

User has to provide seismic zone coefficient, response reduction factor and importance factor. Based on ST value the program will calculate the fundamental time period (T_a). Based on the SS value and the time period the program then calculate S_a/g form the Figure 13. The seismic weight is calculated from the weight data provide by the user through DEFINE 1893 Load command. The weight data must be in the order shown. The program then calculates the base shear using equation (B1). The total base shear is then distributed at different level using the equation (B3) by the program. It is required by the program that the seismic load cases in the two orthogonal directions X and Z should be case 1 and case 2 respectively for the analysis using the coefficient method.



Appendix Figure B 1 STAAD.Pro command to generate geometry of the model.



Appendix Figure B 2 STAAD.Pro command for earthquake loading based on IS 1893.

Appendix C

Sample Output

Appendix Table C 1 Design lateral load for different openings in X-direction

Floor	full wall		10% open		20% open		30% open		40% open		50% open	
	bare frame	**infilled frame**	**bare frame**	**infilled frame**	**bare frame**	**infilled frame**	**bare frame**	**infilled frame**	**bare frame**	**infilled frame**	**bare frame**	**infilled frame**
Roof	525.72	523.28	522.54	374.30	519.18	315.00	515.53	276.22	511.63	257.06	507.41	241.77
9	817.97	814.18	780.99	559.42	743.58	451.15	705.93	378.24	668.68	335.96	631.52	300.91
8	646.46	643.46	617.08	442.01	587.52	356.46	557.78	298.86	528.34	265.45	498.97	237.75
7	495.15	492.86	472.45	338.42	449.82	272.92	427.05	228.81	404.51	203.24	382.03	182.03
6	363.79	362.10	347.11	248.63	330.48	200.51	313.75	168.11	297.19	149.32	280.67	133.74
5	252.63	251.46	241.05	172.66	229.50	139.24	217.88	116.74	206.38	103.69	194.91	92.87
4	161.68	160.93	154.27	110.50	146.88	89.12	139.44	74.71	132.09	66.36	124.74	59.44
3	90.95	90.53	86.78	62.16	82.62	50.13	78.44	42.03	74.30	37.33	70.17	33.43
2	40.42	40.23	38.57	27.63	36.72	22.28	34.86	18.68	33.02	16.59	31.19	14.86
1	10.11	10.06	9.64	6.91	9.18	5.57	8.72	4.67	8.26	4.15	7.80	3.72
Base	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	3404.87	3389.09	3270.48	2342.63	3135.49	1902.38	2999.37	1607.07	2864.39	1439.14	2729.40	1300.51

Appendix Table C 2 Design lateral load for different openings in Z-direction

Floor	full wall		10% open		20% open		30% open		40% open		50% open	
	bare frame	**infilled frame**	**bare frame**	**infilled frame**	**bare frame**	**infilled frame**	**bare frame**	**infilled frame**	**bare frame**	**infilled frame**	**bare frame**	**infilled frame**
Roof	407.22	456.81	404.76	342.91	402.16	292.48	399.33	258.57	393.04	241.28	393.04	228.15
9	633.6	710.75	604.95	512.52	575.98	418.9	546.81	354.07	489.17	315.35	489.17	283.95
8	500.74	561.72	477.99	404.95	455.09	330.99	432.05	279.76	386.5	249.16	386.5	224.36
7	383.55	430.25	365.96	310.04	348.43	253.41	330.79	214.19	295.92	190.77	295.92	171.78
6	281.79	316.1	268.87	227.79	255.99	186.18	243.03	157.36	217.41	140.15	217.41	126.2
5	195.69	219.52	186.71	158.19	177.77	129.29	168.77	109.28	150.98	97.329	150.98	87.64
4	125.24	140.49	119.5	101.24	113.77	82.746	108.01	69.939	96.626	62.291	96.626	56.09
3	70.447	79.026	67.217	56.946	63.998	46.545	60.757	39.341	54.352	35.039	54.352	31.551
2	31.31	35.122	29.874	25.31	28.443	20.687	27.003	17.485	24.156	15.573	24.156	14.022
1	7.827	8.781	7.469	6.327	7.111	5.172	6.751	4.371	6.039	3.893	6.039	3.506
Base	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL =	2637.4	2958.6	2533.3	2146.2	2428.7	1766.4	2323.3	1504.4	2114.2	1350.8	2114.2	1227.3

Appendix Table C 3 Story shear in X-direction for different openings

Floor	full wall		10% open		20% open		30% open		40% open		50% open	
	bare frame	infilled frame	bare frame	infilled frame	bare frame	infilled frame	bare frame	infilled frame	bare frame	infilled frame	bare frame	infilled frame
10	525.72	523.28	522.54	374.30	519.18	315.00	515.53	276.22	511.63	241.77	507.41	222.21
9	1343.68	1337.46	1303.54	933.72	1262.77	766.15	1221.47	654.46	1180.31	542.68	1138.92	498.78
8	1990.14	1980.92	1920.62	1375.73	1850.29	1122.61	1779.24	953.32	1708.65	780.43	1637.90	717.30
7	2485.30	2473.78	2393.07	1714.14	2300.11	1395.53	2206.29	1182.13	2113.16	962.46	2019.92	884.61
6	2849.08	2835.88	2740.18	1962.77	2630.59	1596.04	2520.04	1350.24	2410.35	1096.19	2300.60	1007.52
5	3101.71	3087.34	2981.22	2135.43	2860.09	1735.29	2737.92	1466.98	2616.73	1189.06	2495.51	1092.88
4	3263.40	3248.27	3135.49	2245.94	3006.97	1824.40	2877.36	1541.70	2748.81	1248.50	2620.25	1147.51
3	3354.34	3338.80	3222.27	2308.09	3089.59	1874.53	2955.80	1583.72	2823.11	1281.94	2690.42	1178.24
2	3394.76	3379.03	3260.84	2335.72	3126.31	1896.81	2990.66	1602.40	2856.13	1296.80	2721.61	1191.90
1	3404.87	3389.09	3270.48	2342.63	3135.49	1902.38	2999.37	1607.07	2864.39	1300.51	2729.40	1195.31
0	3404.87	3389.09	3270.48	2342.63	3135.49	1902.38	2999.37	1607.07	2864.39	1300.51	2729.40	1195.31

Appendix Table C 4 Story shear in Z-direction for different openings

Floor	full wall		10% open		20% open		30% open		40% open		50% open	
	bare frame	**infilled frame**	**bare frame**	**infilled frame**	**bare frame**	**infilled frame**	**bare frame**	**infilled frame**	**bare frame**	**infilled frame**	**bare frame**	**infilled frame**
10	407.22	456.81	404.76	342.91	402.16	292.48	399.33	258.57	241.28	228.15	393.04	210.62
9	1040.81	1167.56	1009.72	855.43	978.13	711.39	946.14	612.64	556.63	512.11	882.21	472.76
8	1541.56	1729.28	1487.70	1260.38	1433.23	1042.37	1378.19	892.39	805.79	736.46	1268.71	679.89
7	1925.10	2159.53	1853.66	1570.43	1781.66	1295.78	1708.98	1106.58	996.56	908.24	1564.63	838.46
6	2206.89	2475.63	2122.53	1798.21	2037.65	1481.96	1952.01	1263.94	1136.71	1034.44	1782.04	954.97
5	2402.58	2695.14	2309.25	1956.40	2215.42	1611.25	2120.78	1373.22	1234.04	1122.08	1933.01	1035.88
4	2527.81	2835.63	2428.74	2057.63	2329.19	1694.00	2228.79	1443.16	1296.33	1178.17	2029.64	1087.66
3	2598.26	2914.66	2495.96	2114.58	2393.19	1740.54	2289.55	1482.50	1331.37	1209.72	2083.99	1116.79
2	2629.57	2949.78	2525.83	2139.89	2421.63	1761.23	2316.55	1499.99	1346.94	1223.74	2108.15	1129.73
1	2637.40	2958.56	2533.30	2146.22	2428.74	1766.40	2323.31	1504.36	1350.84	1227.25	2114.19	1132.97
0	2637.40	2958.56	2533.30	2146.22	2428.74	1766.40	2323.31	1504.36	1350.84	1227.25	2114.19	1132.97

Appendix Table C 5 Story moment in X-direction for different openings

Floor	full wall		10% open		20% open		30% open		40% open		50% open	
	bare frame	**infilled frame**	**bare frame**	**infilled frame**	**bare frame**	**infilled frame**	**bare frame**	**infilled frame**	**bare frame**	**infilled frame**	**bare frame**	**infilled frame**
Roof	0	0	0	0	0	0	0	0	0	0	0	0
9	1761.145	1752.985	1750.522	1253.888	1739.256	1055.247	1727.032	925.3471	1713.957	861.1343	1699.817	809.9295
8	6262.487	6233.469	6117.365	4381.834	5969.519	3621.846	5818.943	3117.801	5667.989	2847.738	5515.209	2627.891
7	12929.46	12869.55	12551.42	8990.516	12167.98	7382.599	11779.4	6311.427	11391.96	5723.599	11002.16	5242.321
6	21255.2	21156.71	20568.2	14732.89	19873.35	12057.63	19170.46	10271.57	18471.03	9280.301	17768.91	8466.549
5	30799.62	30656.92	29747.79	21308.17	28685.84	17404.37	27612.58	14794.88	26545.69	13337.21	25475.91	12138.79
4	41190.36	40999.5	39734.88	28461.87	38267.15	23217.58	36784.6	19709.26	35311.73	17741.49	33835.86	16122.16
3	52122.73	51881.22	50238.78	35985.76	48340.51	29329.32	46423.75	24873.94	44520.26	22368.08	42613.71	20304.64
2	63359.78	63066.2	61033.38	43717.88	58690.65	35609	56325.67	30179.41	53977.69	27119.73	51626.62	24599.12
1	74732.23	74385.96	71957.18	51542.54	69163.8	41963.31	66344.38	35547.45	63545.73	31926.96	60744.01	28943.39
Base	86138.54	85739.41	82913.28	59390.33	79667.71	48336.28	76392.28	40931.14	73141.43	36748.08	69887.51	33300.1

Appendix Table C 6 Story moment in Z-direction for different openings

Floor	full wall		10% open		20% open		30% open		40% open		50% open	
	bare frame	infilled frame	bare frame	infilled frame	bare frame	infilled frame	bare frame	infilled frame	bare frame	infilled frame	bare frame	infilled frame
Roof	0	0	0	0	0	0	0	0	0	0	0	0
9	1364.177	1530.297	1355.949	1148.762	1347.223	979.8214	1337.752	866.2062	1316.671	808.2981	1316.671	764.3059
8	4850.901	5441.609	4738.495	4014.459	4623.968	3362.965	4507.331	2918.533	4272.057	2673.009	4272.057	2479.858
7	10015.11	11234.68	9722.3	8236.746	9425.275	6854.907	9124.281	5908.043	8522.233	5372.415	8522.233	4947.012
6	16464.2	18469.1	15932.07	13497.67	15393.83	11195.77	14849.37	9615.083	13763.73	8710.884	13763.73	7989.613
5	23857.28	26762.45	23042.55	19521.68	22219.94	16160.34	21388.61	13849.29	19733.55	12518.87	19733.55	11454.99
4	31905.91	35791.18	30778.53	26075.6	29641.59	21558.03	28493.23	18449.58	26209.14	16652.91	26209.14	15213.96
3	40374.08	45290.55	38914.82	32968.68	37444.38	27232.92	35959.69	23284.17	33008.43	20995.62	33008.43	19160.83
2	49078.26	55054.66	47276.28	40052.52	45461.57	33063.74	43629.68	28250.55	39989.8	25455.71	39989.8	23213.4
1	57887.32	64936.42	55737.83	47221.15	53574.03	38963.85	51390.14	33275.51	47052.09	29967.97	47052.09	27312.95
Base	66722.6	74847.61	64224.39	54410.98	61710.32	44881.3	59173.21	38315.11	54134.62	34493.28	54134.62	31424.23

Appendix Table C 7 Displacement at floor in X-direction for different openings

Floor	Full wall		10% opening		20% opening		30% opening		40% opening		50% opening	
	bare frame	Infill frame	bare frame	Infill frame	bare frame	Infill frame	bare frame	Infill frame	bare frame	Infill frame	bare frame	Infill frame
0	0	0	0	0	0	0	0	0	0	0	0	0
1	0.7466	0.1085	0.7173	0.1632	0.6877	0.1776	0.6578	0.188	0.6282	0.1926	0.5986	0.1965
2	1.862	0.2343	1.789	0.3473	1.7152	0.401	1.6408	0.4424	1.567	0.4632	1.4932	0.4801
3	3.0149	0.366	2.8973	0.5313	2.7778	0.6236	2.6573	0.6989	2.5379	0.7383	2.4185	0.7709
4	4.1494	0.5013	3.9883	0.7161	3.8241	0.8446	3.6585	0.952	3.4944	1.0094	3.3303	1.0575
5	5.2351	0.6363	5.033	0.8967	4.8264	1.0588	4.6179	1.1961	4.4114	1.2704	4.2049	1.333
6	6.239	0.7669	6	1.0677	5.7547	1.2599	5.5071	1.4242	5.2619	1.5137	5.0167	1.5895
7	7.1218	0.8881	6.8518	1.2228	6.5733	1.4404	6.2922	1.6277	6.0137	1.7303	5.7354	1.8175
8	7.8384	0.9947	7.5452	1.3546	7.2412	1.5916	6.934	1.7966	6.6299	1.9095	6.326	2.0059
9	8.3405	1.0809	8.034	1.455	7.714	1.7034	7.3907	1.9197	7.0706	2.0396	6.7506	2.1427
10	8.6055	1.1415	8.291	1.513	7.9661	1.7663	7.6377	1.9885	7.3123	2.1126	6.9868	2.2199

Appendix Table C 8 Displacement at floor in Z-direction for different openings

Floor	Full wall		10% opening		20% opening		30% opening		40% opening		50% opening	
	bare frame	**Infill frame**	**bare frame**	**Infill frame**	**bare frame**	**Infill frame**	**bare frame**	**Infill frame**	**bare frame**	**Infill frame**	**bare frame**	**Infill frame**
0	0	0	0	0	0	0	0	0	0	0	0	0
1	0.6095	0.1084	0.5856	0.1647	0.5614	0.1802	0.5371	0.191	0.5129	0.1959	0.4887	0.1995
2	1.5453	0.2426	1.4848	0.3593	1.4236	0.4153	1.3618	0.458	1.3006	0.4796	1.2394	0.4959
3	2.5231	0.3897	2.4247	0.5581	2.3248	0.6534	2.2241	0.7307	2.1242	0.7717	2.0244	0.8031
4	3.4915	0.5463	3.3561	0.7617	3.2181	0.893	3.0789	1.0026	2.9409	1.0621	2.803	1.1083
5	4.4234	0.7075	4.2528	0.9642	4.0785	1.1283	3.9025	1.2675	3.7282	1.344	3.5539	1.4039
6	5.29	0.8681	5.0875	1.1597	4.8798	1.3523	4.6702	1.5175	4.4625	1.6091	4.2549	1.6811
7	6.0576	1.0224	5.828	1.3408	5.5916	1.5566	5.3528	1.7434	5.1163	1.8476	4.8799	1.9298
8	6.6878	1.1643	6.4377	1.4998	6.1787	1.7319	5.9171	1.9345	5.6581	2.0482	5.3991	2.1383
9	7.1403	1.2876	6.8778	1.6278	6.6044	1.8676	6.328	2.0791	6.0544	2.1986	5.7809	2.2938
10	7.3976	1.3867	7.1271	1.7138	6.8482	1.9541	6.5663	2.1686	6.287	2.2908	6.0074	2.3888

CIRRICULUM VITAE

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