BEHAVIOUR OF RANDOMLY INFILLED RC FRAMES WITH SOFT GROUND FLOOR SUBJECTED TO SEISMIC LOADING

Submitted by

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It is hereby declared that except for the contents where specific reference have been made to the work of others, the studies contained in this thesis is the result of investigation carried out by the author. No part of this thesis has been submitted to any other University or educational establishment for a Degree, Diploma or other qualification (except for publication).

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(Md. Taskin Alam)

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ABSTRACT

Multistoried masonry infilled reinforced concrete (RC) frame with open ground floor is a common building construction practice in Bangladesh. Masonry infills in upper floors make the corresponding floors stiffer; resulting stiffness irregularity in RC frames. Consequently stress concentration occurs at open ground floor level in the event of any seismic load. But, this interaction of masonry infill panels with frame elements is often neglected in the conventional design analysis of such structures. Therefore, an extensive analysis has been performed in the present study to determine the seismic performance of masonry infilled RC soft story buildings and to propose appropriate mitigating measures against their earthquake vulnerability.

In the numerical analysis, several soft story 2D frames with variation in floor and span numbers, infill percentages, slenderness of frames as well as randomness of infill positions have been considered to investigate their corresponding seismic performances. The infills are modeled as equivalent diagonal struts. Beams and columns are modeled using two-dimensional elastic frame element. Considered loads during the analyses are dead load, live load, earthquake load and their combinations. Earthquake loads have been applied following both the equivalent static force method (ESFM) and the dynamic response spectrum method (RSM). The base shear, sway pattern and drift demand etc. are evaluated and compared following ESFM as well as RSM. In addition, slenderness of frame and base shear ratio is compared with percentage of infill.

Numerical analysis has revealed sudden increases of sway at soft ground floor level whereas it decreases gradually in the upper floors due to presence of infills. The presences of infills stiffen the upper floors resulting major deflection at ground floor level. Also the base shear is significantly increased in presence of structurally active infill as compared to static analysis. This soft story behavior has been clearly identified in dynamic analysis while conventional static analysis cannot predict such behavior. It has been observed from analysis that randomness in the distribution of infill shows no effect on base shear value. Also the observations clearly indicate that the ground floor columns in soft story buildings are, in general, significantly under-designed for seismic loads found from ESFM and vulnerable during earthquake.

Finally based on the findings of the present study, a magnification factor has been proposed as a function of number of floors to magnify the base shear, moment and shear force found from ESFM. It is expected that design of ground floor columns based on magnified moments and forces will safeguard the soft story buildings from catastrophic failure at the event of earthquakes.

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Brick masonry or unreinforced brick partition wall is commonly used in reinforced concrete structure. Masonry can be considered as a composite material built of relatively strong brick units and weak mortar joints. Since they are usually considered as non-structural elements, their interaction with the bounding frame is often ignored in the design. Neglecting the presence of infill in the calculation of structures subject to horizontal loads leads to an evaluation of stresses in the frames which often far from the real situation and may compromise safety.

In Bangladesh, like other developing countries, construction of multistoried building with soft stories for car parking or other utility services is common. These building are designed as RC frame structures without considering the structural action of infill located on the upper floors. Since the mass is concentrated at upper floors, it makes upper floors heavier than the ground floor and creates the action of inverted pendulum. Infill on the upper floors makes the floors much stiffer in resisting the lateral seismic loads compared to open ground floor. As a result, collapse of ground floor occurs during earthquake or lateral load.

1.2 OBJECTIVES WITH SPECIFIC AIM AND POSSIBLE OUTCOMES

The objective of the study is to investigate the seismic vulnerability of soft ground floor columns of typical RC frames having randomly distributed infill of various amounts on the upper floors. Based on comparative study of the results obtained using equivalent static force method and more rational response spectrum method, an attempt shall be made to provide some simple guidelines for a safer design of soft story columns. The objectives of this study, more specifically, are as follows:

i. To develop finite element models of a series of 2D building frames including infills on the upper floors keeping the ground floor free from infill.

ii. To analyze these building frames where infill is applied randomly in frame panel and 15 numbers of software run are applied to find average base shear using conventional Equivalent Static Force Method (ESFM) as well as Response Spectrum Method (RSM).

iii. To investigate the effect of various amount of infill on base shear, sway pattern, storey drift and moment of building frames considering various parameters i.e. % of infill, slenderness of building frame, number of span, bay and floors.

iv. To make a comparison between the two methods; Equivalent Static Force Method and Response Spectrum Method available in BNBC, 1993 to calculate the earthquake base shear, sway pattern and drift characteristics.

v. To investigate possible structural behavior of ground floor columns under seismic loading (as a soft story) in presence of infill on upper floor.

vi. To determine possible remedies if the vulnerability of such buildings due to earthquake loads is significant.

1.3 OUTLINE METHODOLOGY

To carry out the study, 2D models of reinforced concrete frame will be considered. The ground floor will be kept free of infill to consider the case of soft story. Infill of varying numbers will be applied on the upper floors randomly to account for the diverse pattern of partition walls found in real buildings. For a systematic numerical analysis, ANSYS 10 software will be used to model the RC frame and the infill walls. Considering the geometrical and material properties of the infill panel equivalent strut properties (geometrical and mechanical) would be calculated. The infill would be modeled as diagonal bracings.

The analysis will be carried out for different span lengths and span numbers, number of floors and floors heights etc. For each frame, the effect of different percent of infill on upper floor panels will be studied. For each percentage of infill panels, several random distributions will be studied. Each model frame will be subjected to earthquake loading based on equivalent static force method as well as response spectrum method. Comparison of the results from these two methods will provide us with information on the magnification of base shear on the soft story columns. Based on these information found from the study, an attempt shall be made to provide a guideline for safer design of the columns of soft ground story.

1.4 ASSUMPTIONS FOR MODELING

The investigation is based on the following assumptions :

- Material is linearly elastic and isotropic
- Infill is considered to be present in the frame in a random manner. The number of infill panels ranges from no-infill condition to 80% of the frame panels.
- All dead and live loads are taken in vertical direction while only earthquake load is taken as lateral
- Infill is modeled as diagonal strut with material properties of brick
- Dead load including infill (partition wall) mass contribution is modeled as mass element with vertical acceleration only.
- Weight of infill is taken as per actual percentage of infill.

1.5 ORGANIZATION OF THE THESIS

The thesis is organized into five chapters. Chapter 1 introduces the present study while Chapter 2 focuses on the review of relevant theories, methods of analysis and behavior of RC frame with infill. Chapter 3 illustrates the methodology of developing finite element modeling by software named ANSYS. Chapter 4 is organized with analysis and discussion on the results based on various parameters i.e. bases shear, sway, drift demand, percentage of randomly distributed infill, number of span, number of bay etc. Chapter 5 summarizes the findings of the present study and recommendations for future investigation and extension of this work.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

Bangladesh is a densely populated country. Accommodation is one of the prime needs for this huge population. Due to this reason, a huge number of multistoried buildings are being constructed for providing accommodation. Usually the ground floor is kept open for car parking and guest lounge provision. Thus the ground floor experiences a soft story behavior which is vulnerable for the building. A typical soft ground floor is shown below in figure 2.1a.

Fig.2.1a A Building with soft ground floor

The Bangladesh National Building Code (BNBC, 1993) classifies a soft story as one whose lateral stiffness is less than 70% of the story immediately above or below as or less than 80% as stiff as the average stiffness of the three floors above it. Hence irregularities of lateral stiffness in vertical direction are occurred. This is due to presence of infills on upper floors while keeping the ground floor open. Thus soft ground floor column becomes weak and less capable to withstand lateral force like earthquake force due to stress concentration at soft ground floor columns. This may leads to structural damage or failure, which in turn results in the collapse of the structure.

Fig.2.1b shows soft story mechanism where there is infill in the upper floors by keeping the ground floor open. Thus stress is concentrated to the ground floor and consequently makes the ground floor weaker to withstand the upper floor load.

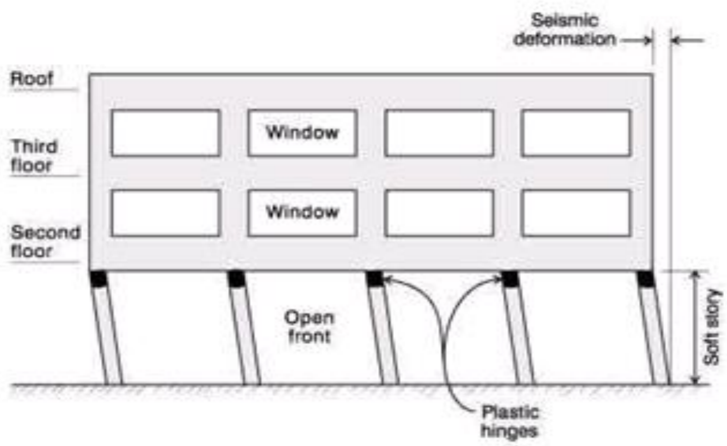


Fig.2.1b Soft story mechanism

2.2 RESPONSE OF MASONRY INFILLED RC FRAME UNDER LATERAL LOAD

The major requirement of using masonry infill (MI) wall includes partitioning, providing building envelop, avoiding fire hazard, temperature and sound barrier etc. Stiffness is developed on the upper floors due to interaction between masonry infilled walls and the surrounding frame when lateral force acts on the structure. This stiffening action is usually not considered design of structure.

The presence of MI in RC frames changes the lateral-load transfer mechanism of the structure from predominant frame action to predominant truss action (Murty and Jain 2000), as shown in Fig. 2.2, which is responsible for reduction in bending moments and increase in axial forces in the frame members.

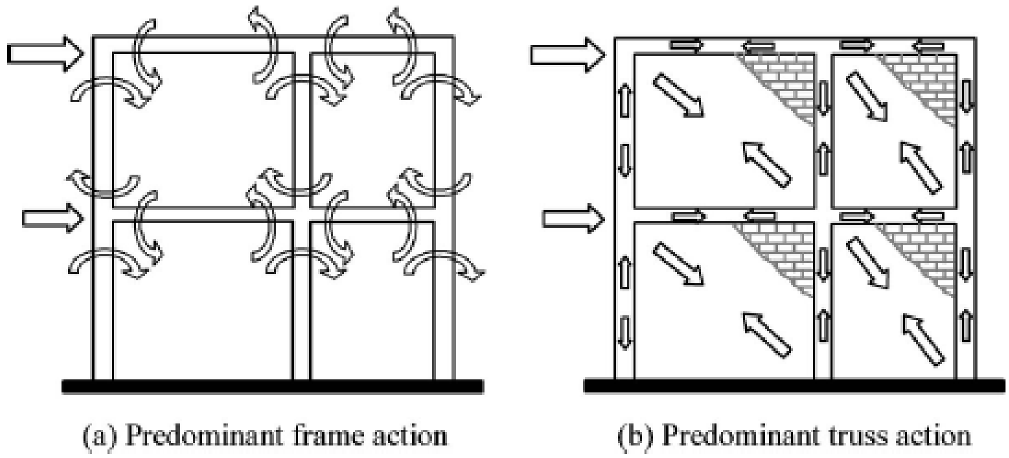


Fig. 2.2 Change in lateral load transfer mechanism due to masonry infills (Murty and Jain 2000).

Masonry infill walls confined by reinforced concrete (RC) frames on all four sides play a vital role in resisting the lateral seismic loads on buildings. The behavior of masonry infilled frames has been extensively studied (Murty and Jain 2000; Smith and Coul, 1991; Moghaddam and Dowling 1987) in attempts to develop a rational approach for design of such frames. Experimentally MI walls was found to have a very high initial lateral stiffness and low deformability (Moghaddam and Dowling 1987).

The beneficial effects of the interaction between masonry infills and structural elements for seismic performance of existing frame buildings were noted in previous studies. Researchers have concluded that proper use of infills in frames could result in significant increases in the strength and stiffness of structures subjected to seismic excitations (Mehrabi et al. 1996, Klingner and Bertero 1978, Bertero and Brokken, 1983). However, the locations of infill in a building must be carefully selected to avoid or minimize torsional effects as well as soft story effect. Architectural restrictions have to be considered when assigning these locations.

The high in-plane rigidity of the masonry wall significantly stiffens the relatively flexible frame. Therefore, a relatively stiff and tough bracing system is resulted. The wall braces the frame partly by its inplane shear resistance and partly by its behavior as a diagonal bracing strut as shown in Fig. 2.3.

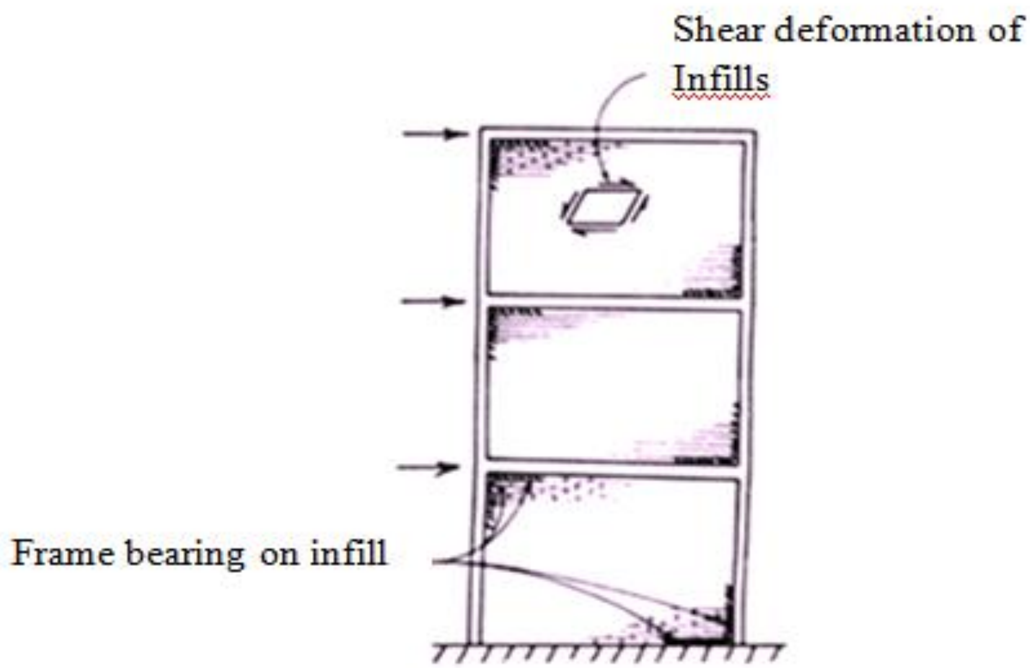


Fig.2.3 Interactive behavior of frame and infill

The frame of Fig. 2.4 shows such mode of behavior. When the frame is subjected to horizontal loading, it deforms with double-curvature bending of the columns and beams. The translation of the upper part of the column in each story and the shortening of the leading diagonal of the frame cause the column to lean against the wall as well as to compress the wall along its diagonal. It is roughly analogous to a diagonally braced frame, shown in Fig. 2.4

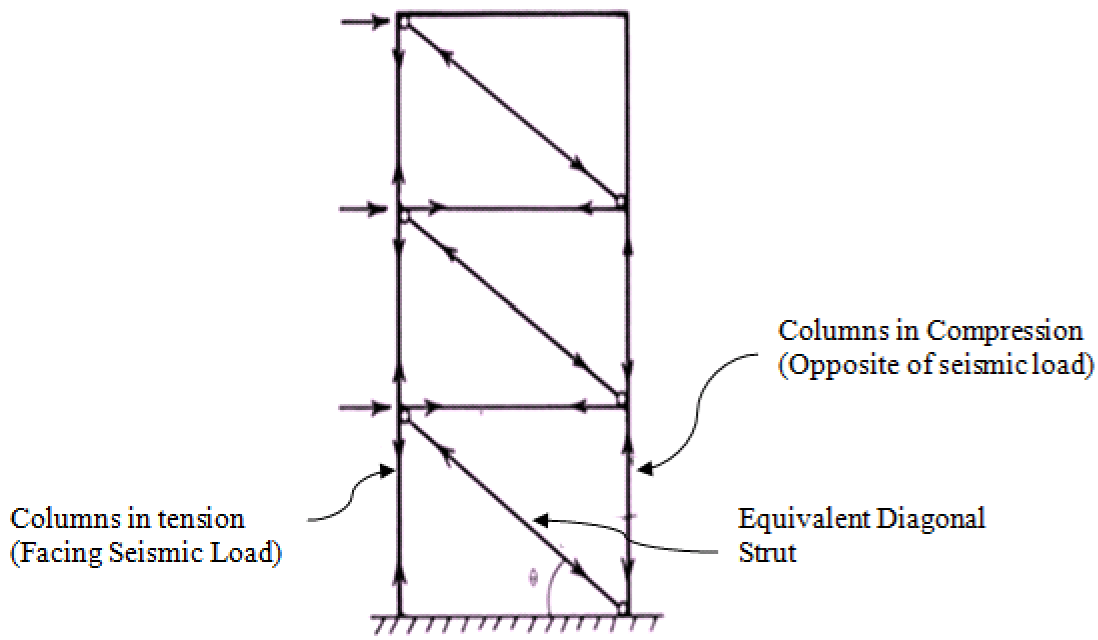


Fig. 2.4 Analogous braced frame

The nature of the forces in the frame can be understood by referring to the analogous braced frame shown in fig. 2.4. The windward column or the column facing the seismic load first is in tension and the leeward column or the other side of the building facing seismic load last is in compression. Since the infill bears on the frame not as exactly a concentrated force at the corners, but over the short lengths of the beam and column adjacent to each compression corner, the frame members are subjected also to transverse shear and a small amount of bending. Consequently the frame members or their connections are liable to fail by axial force or shear and especially by tension at the base of the windward column.

The potential modes of failure of masonry infilled frame structure are occurred due to the interaction of infill walls with frame.

The failure criteria are mentioned below:

- Tension failure of tensioning column due to overturning moments.
- Flexure or shear failure of the columns.
- Compression failure of the diagonal strut.

- Diagonal tension cracking of the panel and
- Sliding shear failure of the masonry along horizontal mortar beds.

Failure modes are described by the figures 2.5 and 2.6. The perpendicular tensile stresses are caused by the divergence of the compressive stress trajectories on the opposite sides of the leading diagonal as they approach the mid region of the infill. The shear failure of wall steps down through the joints of masonry and participated by the horizontal shear stresses in the bed joints. The diagonal cracking is initiated at and spreads from the middle of the infill, where the tensile stresses are a maximum, tending to stop near the compression corners, where the tension is suppressed the diagonal cracking of the wall is through the masonry along a line or line parallel to the loading diagonal and caused by tensile stresses perpendicular to the loading diagonal. The perpendicular tensile stresses caused by the divergence of the compressive stress trajectories on opposite sides of the loading diagonal as they approach the middle region of the infill. The diagonal cracking is initiated and spreads from the middle of the infill while the tensile stresses are at maximum tending to stop near the compression corners, where the tension is suppressed.

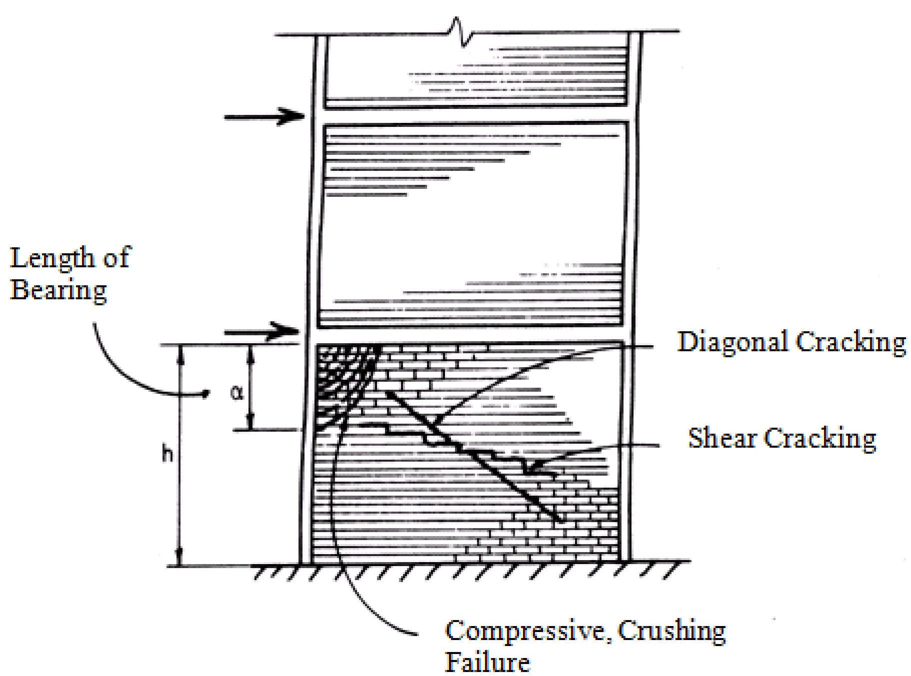


Fig. 2.5 Modes of infill failure

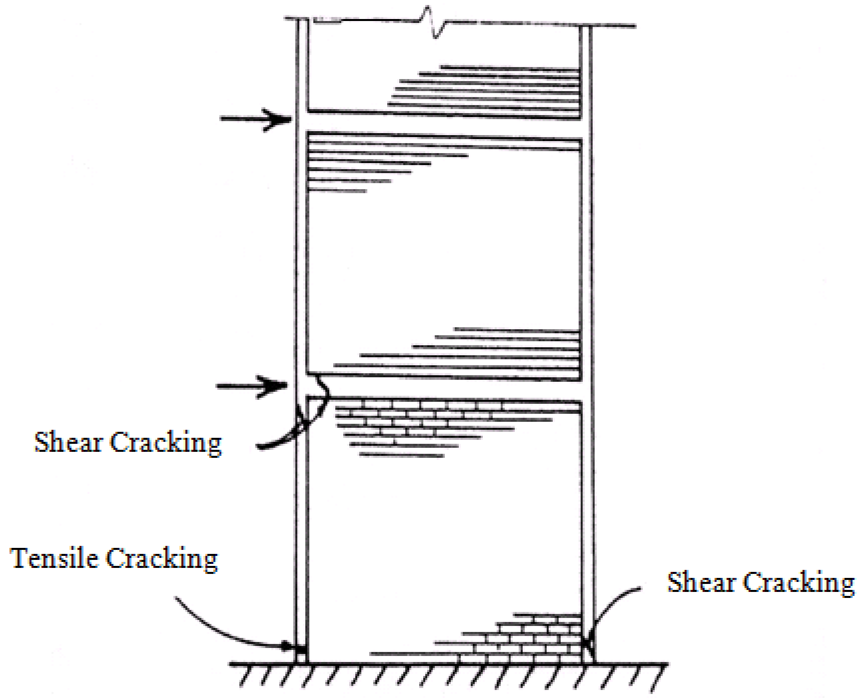


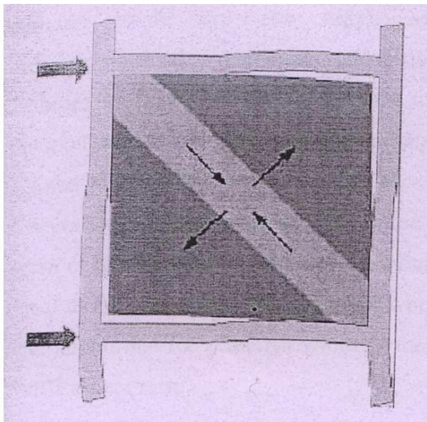
Fig. 2.6 Modes of frame failure

2.3 COMPUTATIONAL ANALYSIS AND EXPERIMENTAL STUDIES OF INFILL PANEL

Modeling of RC structures along with infill panels are based mainly on finite element methods and sophisticated material models. The modeling of infill panel with reinforced concrete frame can be broadly categorized into two approaches: a) equivalent diagonal strut approach and b) continuum approach.

2.3.1 Review of Past Analytical Studies

The first published research on modeling of infill panel as an equivalent diagonal strut was by Holmes (1961). He proposed a method for predicting the deformations and strength of infilled frames based on the equivalent diagonal strut concept. According to his assumption the infill wall acts as a diagonal compression strut, as shown in Fig. 2.7(a), of the same thickness and elastic modulus as the infill with a width equal to one-third the diagonal length. He also concluded that, at the infill failure, the lateral deflection of the infilled frame is small compared to the deflection of the corresponding bare frame.



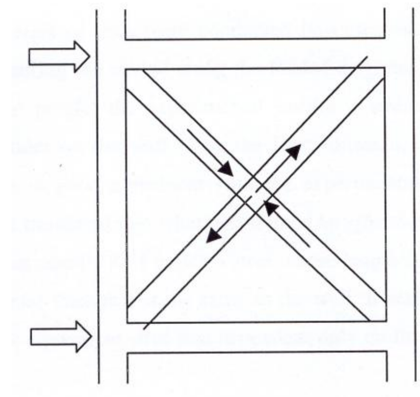


Fig. 2.7(a) The diagonal compression strut of masonry infill

Fig. 2.7 (b) Material modeling of masonry infill as diagonal strut

Riddington and Smith (1977) conducted an extensive series of plane stress finite element analyses of laterally loaded infilled frames.Barua and Mallick (1977) used FE to analyze infilled frames and their technique was similar to the method proposed by Sachanski (1960) except that a finite element technique was used to determine stiffness coefficients of the boundary nodes of the infill. Dawe and Charalambous (1983) presented a finite element technique where standard beam and membrane elements were used to model the frame and the infill wall, respectively. Liauw and Kwan (1983, 1985) developed a plastic theory of non-integral (without shear connectors) infilled frames in which the stress redistribution towards collapse was taken into account and the friction is neglected for strength reserve. The theory was based on the findings from non-linear finite element analysis and experimental investigation. Seah (1998) suggested an analytical technique, in which the steel frame was modeled using elastic beam-column elements connected with nonlinear rotational, shear, and nominal springs.

Saneinejad and Hobbs (1995) proposed a method of analyzing masonry infilled steel frames subjected to in-plane loading. The method utilized the data generated from previous experiments as well as the results of a series of non-linear FE analyses. The proposed method accounts for both the elastic and plastic behavior of infilled frames and predicts the strength and stiffness of the infilled frames. The method also accounted for various parameters like different wall aspect ratios and different beam to-column stiffness and strength.

Madan et al. (1997) further extended the work of Saneinejad and Hobbs (1995) by including a smooth hysteretic model for the equivalent diagonal strut. The proposed analytical development assumes that the contribution of masonry infill panel as shown in Fig. 2.8(a) to the response of the infilled frame can be modeled by replacing the panel by a system of two diagonal masonry compression struts as shown in Fig. 2.8(b).

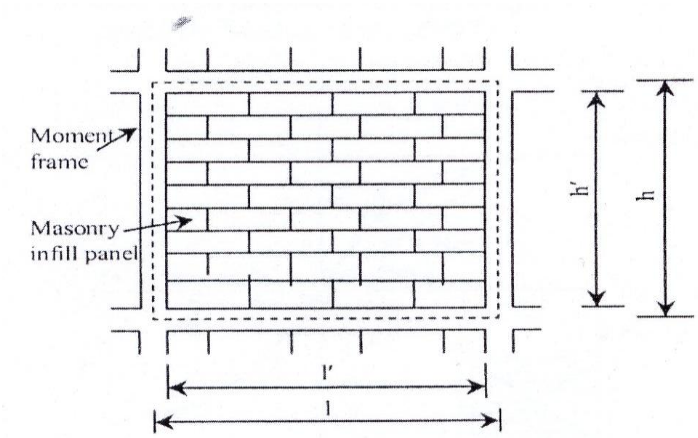


Fig. 2.8 (a) Masonry infilled frame sub-assemblage in masonry infill panel frame

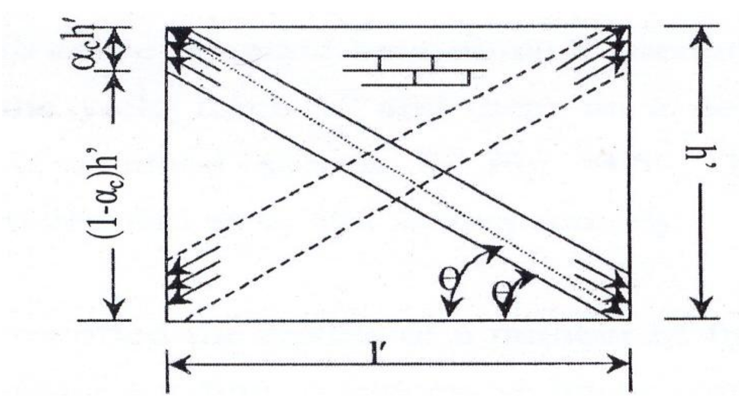


Fig. 2.8 (b) Masonry infill panel in frame structure

Arlekar, Jain and Murty (1997) highlighted the importance of explicitly recognizing the presence of the open ground floor in the analysis of the building. The error involved in modeling such buildings as complete bare frames, neglecting the presence of infills in the upper floor, is brought out through the study of an example building with different analytical models.

The stress strain relationship for masonry in compression as shown in Fig. 2.8(c) is used to determine the strength envelope of the equivalent strut. The individual masonry struts are considered ineffective in tension.

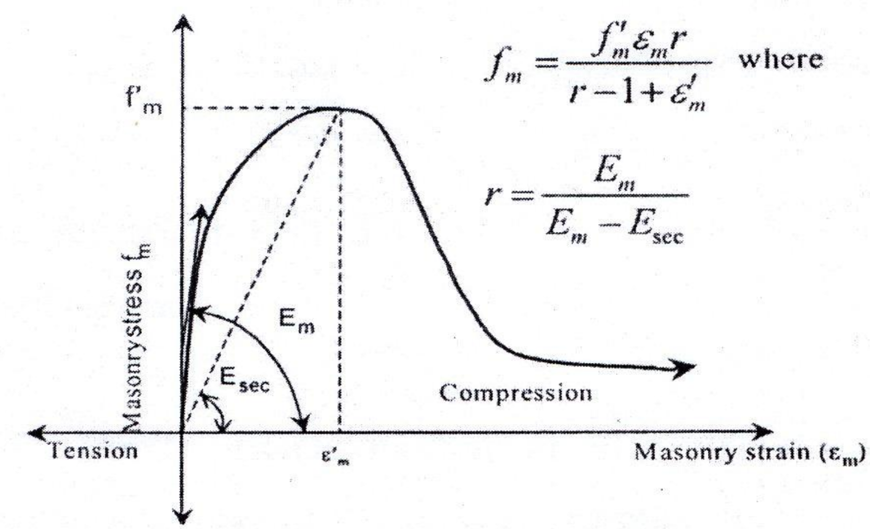


Fig. 2.8 (c) Constitutive model for masonry infill panel by Madan et. al. (1997)

But the combination of both diagonal struts provides a lateral load resisting mechanism for the opposite lateral directions of loading. The lateral force-deformation relationship for the structural masonry infill panel is assumed to be a smooth curve bounded by a bilinear strength envelope with an initial elastic stiffness until the yield force V_y, and there on a degraded stiffness until the maximum force V_m, is reached shown in Fig. 2.8(d). The corresponding lateral displacement values are denoted as u_y and u_m respectively.

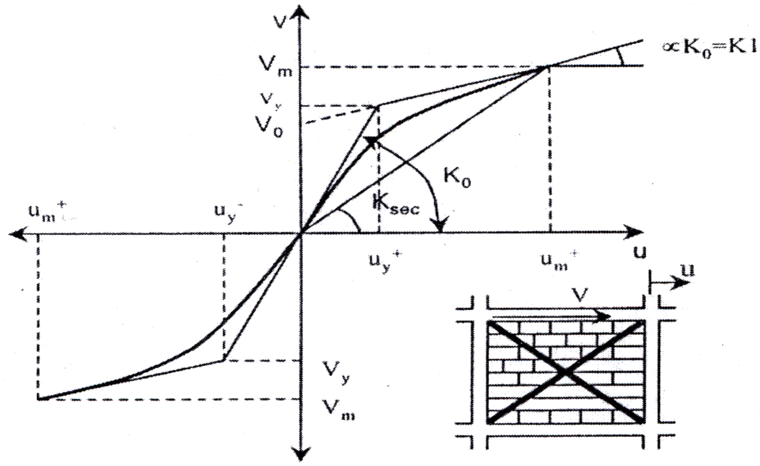


Fig. 2.8 (d) Strength envelope for masonry infill panel by Madan et. al. (1997)

Considering the masonry frame of Fig. 2.8 (d), the maximum lateral force V_m, and the corresponding displacement u_m, in the infill masonry panel (Madan et al.,1997) are

$$V_m^+(V_m^-) \le A_d f_m^{'} \cos\theta \le \frac{vtl'}{(1-0.45\tan\theta)\cos\theta} \le \frac{0.83tl'}{\cos\theta} \qquad (2.1)$$

$$u_m^+(u_m^-) = \frac{\varepsilon_m^{'} L_d}{\cos\theta} \qquad (2.2)$$

in which t = thickness of the infill panel; l' = lateral dimension of the infill panel; $f_m^{'}$ = masonry prism strength; ε' = corresponding strain; θ = inclination of the diagonal strut; v = basic shear strength of masonry; and A_d and L_d = area and length of the equivalent diagonal struts respectively. These quantities can be estimated using the formulations of the "equivalent strut model" proposed by Saneinejad and Hobbs (1995). The initial stiffness K_0 of the infill masonry panel may be estimated using the following formula Madan et al. (1997),

$$K_0 = 2(V_m / u_m) \qquad (2.3)$$

The degradation of strut stiffness from K_0 to K_1 was assumed to be a bilinear curve by Madan et al. (1997). A more rational degradation path would be a smooth curve shown by the heavy solid line in Fig. 2.8 (d). The form of the curve is suggested as given below,

$$V = \frac{(K_0 - K_1)u}{[1 + \{\frac{K_0 - K_1}{V_0} u\}^2]^{1/2}} + K_1 u \tag{2.4}$$

Where, $$V_0 = V_y \frac{K_0 - K_1}{K_0} \tag{2.5}$$

Huang (2005) studied the structural behaviors of low-to-midrise concrete buildings of various configurations with emphases on dynamic properties, internal energy, and the magnitude and distribution of seismic load. Several idealized models were made to represent different structural configurations including pure frame, frames with fully or partially infilled panels, and frames with a soft story at the bottom level, and comparisons were made on the fundamental periods, base shear, and strain energy absorbed by the bottom level between these structures.

M. Helen Santhi, G. M. Samuel Knight (2005) studied two single-bay, three-storied space frames, one with brick masonry infill in the second and third floors representing a soft-story frame and the other without infill were designed and their 1:3 scale models were constructed according to non-seismic detailing and the similitude law.

Rodsin (2005) evaluated the potential seismic performance of building with soft story in an area of low to moderate seismicity regions (such as Australia) by a displacement-based method involving a push-over analysis.

Nagae (2006) studied six storied reinforced concrete building and focused on seismic response of the soft ground floor based on the results on dynamic response analysis.

Amanat and Hoque (2006) studied the fundamental periods of vibration of a series of regular RC framed buildings using 3D FE modeling and modal eigenvalue analysis including the effects of infill. It has been found that when the models do not include infill, as is done in conventional analysis, the period given by the analysis is significantly longer than the period predicted by the code equations justifying the imposition of upper limit on the period by the codes. However, when the effect of infill is included in the models, the time periods determined from eigenvalue analysis were remarkably close to those predicted by the code formulas. It is also observed that the randomness in the distribution of infill does not cause much variation of the period if the total amount of infilled panels is the same for all models. It is also observed that varying total amount of infilled panels causes some changes in the determined period. Based on the findings of the study, some guidelines are suggested for determining the period. The findings of the study have showed 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 as mentioned earlier.

Haque and Amanat (2008) studied behavior of buildings with soft ground floor. They studied the behavior of the columns at ground level of multistoried buildings with soft ground floor subjected to dynamic earthquake loading. Study was done both in ESFM and RSM. For ESFM, lateral sway is almost same for first soft story irrespective of presence of structurally active infill in the upper stories. According to the study, in presence of infill, there is a significant increase in total base shear. For six storied building base shear increases by about 65 percent. For nine and twelve storied building this figure is approximately 113% in both cases. Displacement profiles for both ESFM and RSM have a sudden change of slope at first floor level. The inter-story drift demand is largest in the ground story for all the models for both ESFM and RSM. The mode shape changes significantly when infill is present in the building. Vibration frequency gets almost double when infill is present in the model. Since frequency is significantly increased, it is quite natural that earthquake force on the building would also significantly increase. Thus the study has shown a significant changes in the dynamic characteristics of a building when infill is present.

Haque and Amanat (2009) further made an extensive computational study to find out the behavior of soft storied building as well as their seismic vulnerability. According to their study, the difference of inter story moment and forces are very large. This is due to higher value of shear force and moments at ground floor level which lowered at first floor due to presence of infill. As an example, response spectrum gives shear force and bending moment almost three times higher in soft ground floor than in first floor for an interior column of 6 storied with 50% infill condition. For ESFM, lateral sway is almost same for first soft story irrespective of presence of structurally active infill in the upper stories. In the case of RSM, lateral sway of soft ground story increases with the increase in the number of infilled panels in the upper stories. Sway in upper stories decreases with increase of infilled panels due to increased stiffness of those floors. For the six story model, the drift demand increases up to 45% (11mm for no infill condition to 16mm for 70% infilled condition) for ground floor columns. For nine storied model, the increase in drift demand is about 77% (11mm for no infill condition to 19.5mm for 70% infill condition). Similarly for twelve storied building the drift demand is increased by about 75%.The sway characteristics as revealed by response spectrum method clearly shows that the drift demand of columns of open ground floor are much higher than that predicted by conventional equivalent static force method. It is observed that as percent of infilled panels is inceased from 10% to 70%, base shear increases by about 27% (2.49×10^6 to 2.94×10^6 kN) to 66% (2.49×10^6 to 4.15×10^6 kN) for a six storied building. For nine storied building this magnification of base shear is in the range of 23% to 122%. Similarly, for the twelve storied building the magnification of total base shear is between 20% to 126%.Thus the study shown that base shear is approximately doubled for RC framed buildings with open ground floor.

Hasnain (2009) studied this phenomenon of soft story building. He determined the effect of randomly distributed infills on seismic base shear for RC buildings with soft ground floor. In spite of providing an extensive analysis, his study is also limited due to the following issues:

- Constant beam and column size.
- Application of partition wall load as a constant.
- Equal distribution of total number of infill along the span and bay.

Quayum, Iasmin and Amanat (2009) analyzed RC frames with open ground floor. It was found that the structural responses i.e base shear, column axial force, moment, natural period do not change appreciably by the ESFM analysis for random infill distributions, while they increase noticeably in the RSM analysis.

R. Tasmim and K.M.Amanat (2013) investigated RC frame structure having various percentages of masonry infill on upper floors with no infill and 20% infill on ground floor. The application of infill has been done randomly and the effect of seismic load has been investigated. From the investigation, it has been found that random (irregular) distribution of infill does not cause any significant variation in base shear for commonly occurring range of infill percentage on upper floors (40% and above). Rather than it is the total amount of infill that affects the base shear obtained from dynamic analysis. They showed that a small amount of infill (maximum 20%) application on ground floor normally causes an insignificant reduction on base shear value when compared with the same obtained for fully open ground floor. It has been also found that base shear ratio does not vary significantly with the span number or length, rather than it mainly depends on the number of floors. They suggested a simple expression for base shear ratio as a function of the number of floors which may be used a base shear magnifier. The base shear obtained from ESFM method may be magnified with the suggested base shear multiplier (base shear ratio) to obtain a rational estimation of the base shear for RC framed building with soft ground floor subjected to seismic loading.

2.3.2 Review of Past Experimental Studies

Experimental works of studying infill panels were conducted in the period of very early ages and researches for these are still continuing. Experiment was conducted by Thomas (1953) and Wood (1958) in the United Kingdom. According to the test the result provided ample testimony that a relatively weak infill can contribute significantly to the stiffness and strength of an otherwise flexible frame.

Sachanski (1960) performed tests on model and prototype infilled frames. Based on his test results he proposed an analytical model in which he analyzed contact forces between the frame and the infill by assuming their mutual bond to be replaced by thirty redundant reactions.

Smith (1962) conducted a series of tests on laterally loaded square mild steel frame models infilled with micro-concrete. Monitoring the model deformations during the tests showed that the frame separated from the infill over three quarters of the length of each frame member. These observations led to the conclusion that, the wall could be replaced by an equivalent diagonal strut connecting the loaded corners.

Mallick and Severn (1967) introduced an iterative technique whereby the points of separation between the frame and the infill, as well as the stress distribution along the length of contact between the frame and the infill, were obtained as an integral part of the solution.

Mainstone (1971) presented results of series of tests on model frames with infills of micro-concrete and model brickwork along with a less number of full-scale tests. He found that factors such as the initial lack of fit between the infill and the frame and variation in the elastic properties and strength of the infill can result in a wide variation in behavior even between nominally identical specimens.

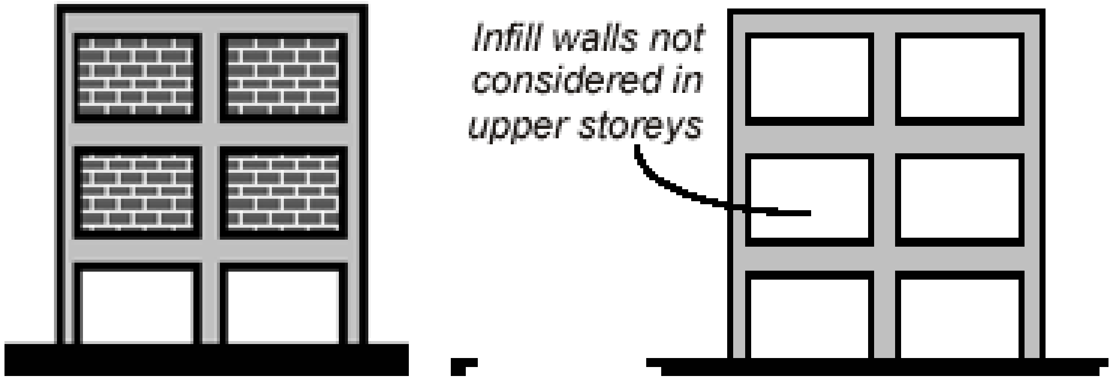
Liauw and Lo (1988) conducted a series of tests on a number of small-scale models of micro concrete infilled steel frames.

Mehrabi and Shing (1997) developed a cohesive dilatants interface model to simulate the behavior of mortar joints between masonry units as well as the behavior of frame to panel interface, and a smeared crack finite element formulation has been used to model concrete in the RC frame and masonry units in the infill panels.

Ghosh and Amde (2002) verified the design of infilled frames to resist lateral loads on buildings in terms of their failure modes, failure loads and initial stiffness using procedures proposed by Riddington (1984) and Pook and Dawe (1986). This verification is made by comparing the results of the analytical procedures of the previous authors with those of a new finite element model for installed frames, which are verified using experimental results.

2.4 EARTHQUAKE EFFECT ON SOFT STORIED BUILDINGS

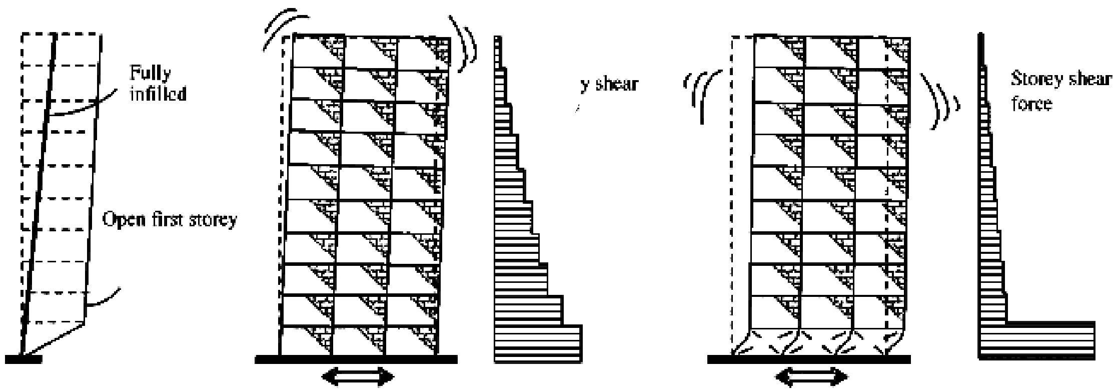
Although soft story provide spaces for car parking and other utilities services but are inherently poor systems due to sudden drop in stiffness and strength in the ground story. In the current practice, stiff masonry walls Fig. 2.9(a) are neglected and only bare frames are considered in design calculations Fig. 2.9(b).



(a) **(b)**

Fig. 2.9 Open ground story building a) actual building b) building being assumed in current design practice

The mode shapes and the corresponding contribution of different modes depend upon the amount and location of infills in the frame because of their high initial stiffness, as shown in Fig. 2.10, where a single frame of the ten-story building is shown.



(a) **(b)** **(c)**

Fig. 2.10 Effects of masonry infills on the first mode shape of a typical frame of a ten story RC building a) Displacement profile b) Fully infilled frame c) Open ground floor frame (EERI, 2001)

In case of a fully infilled frame, lateral displacements are uniformly distributed throughout the height as shown in Fig. 2.10(a and b). On the other hand, in the case of open ground floor buildings, most of the lateral displacement is accumulated at the ground level itself because this floor is the most flexible due to absence of infills (Fig. 2.10 c). Similarly, the seismic story shear forces and subsequently the bending moments concentrate in the open ground story, instead of gradually varying as in fully infilled frame (Fig. 2.10 c and b).

Due to the presence of walls in upper stories makes them much stiffer than the open ground story. Thus, upper story move almost together as a single block and most of the horizontal displacement of the building occurs in the soft ground story itself. In common language, this type of buildings can be explained as a building on chopsticks. Thus, such buildings swing back-and-forth like inverted pendulums during earthquake shaking (Fig. 2.11), and the columns in the open ground story are severely stressed. If the columns are weak (do not have the required strength to resist these high stresses) so that they do not have adequate ductility, they may be severely damaged which may even lead to collapse of the building.

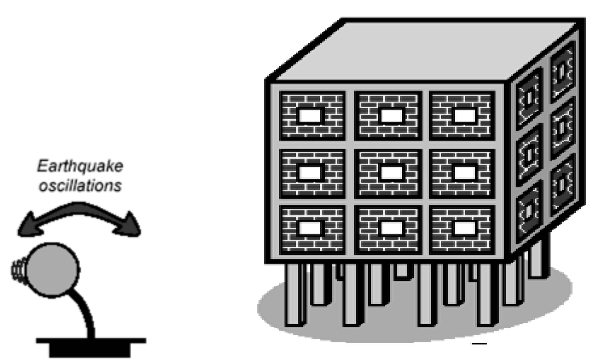


Fig. 2.11 Soft story building act as an inverted pendulum

From the previous history across the world it is being observing that open ground-story buildings have consistently performed poorly during earthquakes. A significant number of buildings with soft story have collapsed (i.e. during 1999 Turkey, 1999 Taiwan, 2001 Bhuj (India) and 2003 Algeria earthquakes, San Fernando 1971 etc.) Alarming amount of damage to the buildings with open basements for parking has been reported during the Northridge Earthquake on

January 17, 1994, as well as Great Hanstin Earthquake of Kobe 1995. Typical examples of such collapses are shown in Figs. 2.12 through 2.15.

Fig. 2.12 Soft story collapse of the ground floor of a multistoried building; Kobe, 1995

Fig. 2.13 Large deflection in soft story due to earthquake; Bhuj 2001

Fig.2.14 Sway mechanisms are often inevitable with soft ground floors; Izmit, Turkey 1999

Fig. 2.15 Failure because of the effect of soft story mechanism; Los Angles, 1994

2.5 BUILDING CODES

Guideline for the design and construction requirements is the building codes which ensure public safety from structural failure and loss of life and wealth. Because of the differences in magnitude of earthquake, geological formations, construction types, Structure types (Height vs Width), Percentage of infill, economical development and other features the seismic design aspects are different in different building codes. The national building codes of different countries can be classified in two broad categories for our discussion. First are those Codes do not consider the features of Masonry Infill walls while designing RC frames and the others are those consider the features of Masonry Infill walls while designing RC frames.

2.5.1 Building Code without Considering Soft Story Phenomenon

A very few codes specifically recommend isolating the MI from the RC frames such that the stiffness of MI does not play any role in the overall stiffness of the frame (NZS-3101 1995, SNIP-II-7-81 1996). As a result, MI walls are not considered in the analysis and design procedure. The isolation helps to prevent the problems associated with the brittle behavior and asymmetric placement of MI. Another group of national codes prefers to take advantage of certain characteristics of MI walls such as high initial lateral stiffness, cost-effectiveness, and ease in construction. These codes require that the beneficial effects of MI are appropriately included in the analysis and design procedure and that the detrimental effects are mitigated. In other words, these codes tend to maximize the role of MI as a first line of defense against seismic actions, and to minimize their potential detrimental effects through proper selection of their layout and quality control.

Most national codes recognize that structures with simple and regular geometry perform well during earthquakes, and unsymmetrical placement of MI walls may introduce irregularities into them. These codes permit static analysis methods for regular short buildings located in regions of low seismicity. However, for other buildings, dynamic analyses are recommended, in which it is generally expected but not specifically required that all components imparting mass and stiffness to the structure are adequately modeled. Most codes restrict the use of seismic design force obtained from dynamic analysis such that it does not differ greatly from a minimum value that is based on the code-prescribed empirical estimate of natural period. This restriction prevents the design of buildings for unreasonably low forces that may result from various uncertainties involved in a dynamic analysis.

Natural period of vibration is an important parameter in the building code equations for determining the design earthquake force by any kind of equivalent static force method. Natural periods of vibration of buildings depend upon their mass and lateral stiffness. Presence of non-isolated MI walls in buildings increases both the mass and stiffness of buildings; however, the contribution of latter is more significant. Consequently, the natural periods of an MI-RC frame are normally lower than that of the corresponding bare frame. Therefore, the seismic design forces for MI frames are generally higher than those for the bare frames. Although, all national codes explicitly specify empirical formulae for the fundamental natural period calculations of bare RC frames, only a few specify the formulae for MI-RC frames.

Several codes—IS-1893 (2002); NBC-105 (1995); NSR-98 (1998); Egyptian code (1988); Venezuelan code (1988); Algerian code (1988); ESCP-1 (1983)—suggest using an empirical formula given by Eqn 2.6 to calculate the natural period of MI-RC frames, T_a in sec.

$$T_a = \frac{0.09h}{\sqrt{d}} \tag{2.6}$$

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

For T_a estimation, French code (AFPS-90 1990) recommends using the most unfavorable of Eqn. 2.6 and the following equation that is specified for masonry buildings:

$$T = 0.06\frac{h}{\sqrt{d}}\sqrt{\frac{h}{2d+h}} \tag{2.7}$$

In Eqn. 2.6 and 2.7, total base width of buildings is used to calculate T_a, which may not be appropriate. For example, d will be equal to the total base dimension for all the frames in Fig. 2.16 irrespective of the distribution of MI in the frame. However, for frame in Fig. 2.16c, it is more appropriate to consider d' as the effective base width, rather than total width d of the building. Therefore, Eqn. 2.6 and 2.7 may not estimate correct T_a values for different frames shown in Fig. 2.16.

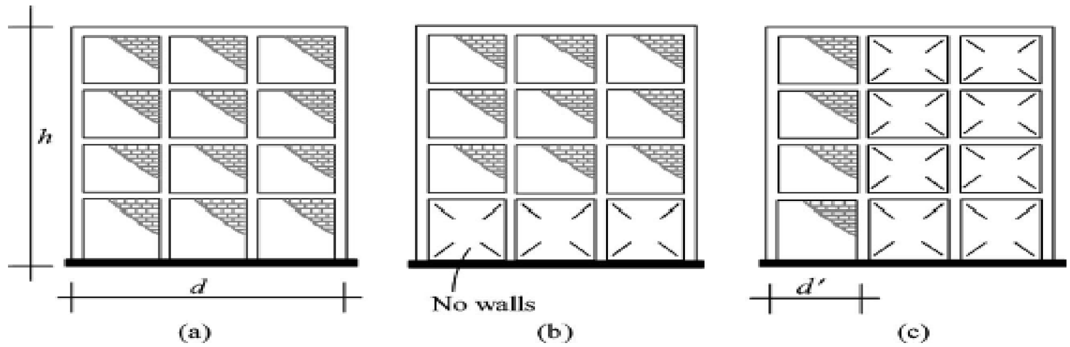


Fig. 2.16 Different arrangements of masonry infill walls in RC frame

2.5.2 Building Code Considering Soft Story Phenomenon

Vertical irregularities are introduced into MI-RC frames due to reduction or absence of MI in a particular story compared to adjacent stories. In general, this gives rise to mass, stiffness, and strength irregularities along height of buildings. A few national codes penalize beams and/or columns of the irregular stories, as they are required to be designed for higher seismic forces to compensate for the reduction in the strength due to absence of MI in the irregular stories.

The Indian seismic code (IS-1893 2002) requires members of the soft story (story stiffness less than 70% of that in the story above or less than 80% of the average lateral stiffness of the three stories above) to be designed for 2.5 times the seismic story shears and moments, obtained without considering the effects of MI in any story. The factor of 2.5 is specified for all the buildings with soft stories irrespective of the extent of irregularities; and the method is quite empirical. The other option is to provide symmetric RC shear walls, designed for 1.5 times the design story shear force in both directions of the building as far away from the center of the building as feasible.

Costa Rican code (1986) requires that all structural-resisting systems must be continuous from the foundation to the top of buildings, and stiffness of a story must not be less than 50% of that of the story below.

2.6 JUSTIFICATION OF PRESENT STUDY

In fact, with most of the buildings constructed without considering seismic resistance, a moderate earthquake could be fatal in populated, unplanned cities like Dhaka. At the backdrop of earthquake vulnerability of structures, the construction of buildings with soft ground story is considered to be even more hazardous. This particular hazardous construction cannot be avoided because of compelling planning and utility requirements. As such developing an improved seismic design for such building structure would ensure a higher degree safety against seismic hazard.

Total seismic base shear as experienced by a building during an earthquake is dependent on its natural period; the seismic force distribution is dependent on the distribution of stiffness and mass along the height. In buildings with soft ground story, the upper story being stiff undergoes smaller inter-story drift. But the inter-story drift in the soft ground story is large, which in turn, leads to concentration of forces at the connections of the story above accompanied by large plastic deformations. In addition, the columns of the soft stories dissipate most of the energy developed during the earthquake. In this process the plastic hinges are formed at the ends of columns, transforming into the soft story mechanism. When this occurs in a lower story, the collapse is unavoidable.

Conventional design is based on static analysis. The basic assumption in static analysis is that only the first mode of vibration of buildings governs the dynamics and the effects of higher modes are not significant; therefore, higher modes are not considered in the analysis. Thus, irrespective of whether the building is regular or irregular, static analysis is incapable of capturing the true dynamic behavior of soft storied building. Therefore a detailed parametric study involving parameters i.e. comparison of base shear, sway, and drift on ratio of building height versus length (H/L), No of bays, and percentage of randomly distributed infill are justified. Such study shall lead to better understanding of the behavior of soft storied building.

CHAPTER 3

FINITE ELEMENT MODELING

3.1 INTRODUCTION

This chapter describes the finite element modeling of reinforced concrete 2D frame with soft story in presence of masonry infill. Selection of element type for modeling this 2D frame including beam, column, and infill is described with proper support condition. Effect of infill is also compared with bare frame. To model the masonry infill, link element is taken as diagonal strut and for load application, mass element is chosen accordingly. For the analysis both equivalent static force method and response spectrum method (RSM) is considered. The comparison of effect of infill between ESFM and RSM is performed to assess the structural characteristics of soft story.

3.2 SOFTWARE USED FOR ANALYSIS

Finite element analysis tools or packages are readily available in the civil engineering field. They vary with the extent of degree of complexity, usability and versatility. Among them ABAQUAS, DIANA, ANSYS, ETABS, STRAND, ADINA, FEMSKI, and STAAD etc are commonly used. ANSYS 10.0 has been used for this research work.

3.3 ASSUMPTIONS FOR MODELING

In reality RC is a composite material with embedded steel reinforcement in concrete. Below assumptions are considered for the present study:

- We assumed linearly elastic homogeneous material for the RC frame that is always steel reinforced in reality.
- According to ACI recommendation, the analysis results for RC frame are accurate enough for this simplification only if appropriate properties of concrete are considered.
- The structural property of masonry infill is modeled as diagonal strut.

3.4 PROPERTIES OF STRUCTURAL COMPONENT

The properties of structural component used in this study are described below:

3.4.1 ELEMENT Properties

The beams and columns are modeled with common two dimensional frame element termed BEAM3 from the ANSYS library. It is basically a two noded frame element having two displacements and one rotational degree of freedom at each node. All beams and columns are rectangular in shape and their dimensions can be easily varied in the code. In this model, the dimension of the internal and external columns can be varied independently.

3.4.2 MASS21 (Structural Mass) for load application

In this present study, dead loads were considered as the self weight of the structure (including beam, column, floor finish, partition wall and slab), uniformly distributed dead load on floor from partition wall and occupancy. These dead loads were converted to equivalent mass and distributed over the beams. In actual condition, the mass is distributed continuously over the beams. In analyzing, the mass needs to be distributed as lump mass at some points. Increased number of lump mass gives more accuracy. In this model, MASS21 from ANSYS library was used to model the lump mass.

3.4.3 LINK8 (3-D Spar or Truss) for diagonal strut

In this model, infill was modeled as diagonal strut. LINK1 from ANSYS library was used to model the diagonal strut. As LINK1 is a uniaxial tension-compression member, it is not needed to mesh.

In the present study, we are using ESFM and RSM for earthquake analysis. Both methods are linear elastic method. Therefore orientation of the diagonal strut shall not have any significance effect on the overall lateral deflection of the frame or base shear. Due to difference in orientation the struts may be either in tension or in compression. This shall effect the local force distribution in beams and columns where the diagonal strut is connected. However the overall structural behavior in terms of lateral sway or base shear shall remain unaffected. Therefore under the present scope of the study, orientation of diagonal strut is not major concern.

3.5 MESHING

In reality, loads are distributed uniformly over floors and so on the beams. In finite element method, uniformly distributed loads are converted into point loads and applied at the nodes. For this reason mesh density is an important criterion to get good results from finite element analysis. As the number of elements is increased, the time for calculation and computation is also increased. The beams were divided into 4 elements and the column was modeled using single element.

In this study, 2D plane frame has been used to model beams and columns. The stiffness formulation of this element is based on the exact solution of the governing fourth order differential equation in flexure. For this reason this element can produced frame results (axial force, shear force and moment) accurately even if a single element is used for a beam or column. Therefore mesh density analysis is not essential for the present study. For the purpose of convenience in distributing the mass element, the beams are divided into 04 (four) elements only.

3.6 CHOICE OF MODEL TO DEVELOP MASONRY INFILL

There are several analytical models of infill available in the literature, which can be broadly categorized as (a) continuum models and (b) diagonal strut models described in the 2nd chapter. For the type of work presented in this study, the diagonal strut model of Saneinejad and Hobbs (1995) has been found to be more suitable. This model has been successfully used by Madan et al. (1997) for static monotonic loading as well as quasi-static cyclic loading. They have also successfully verified the model by simulating the experimental behavior of tested masonry infill frame sub-assemblage. The initial stiffness K_0 of the infill masonry panel is calculated using the formula given by Madan et al. (1997) which is discussed in article 2.3.1 of this thesis.

3.7 SUPPORT CONDITIONS

At foundation level all column ends are considered to act under fixed support condition with all degrees of freedom of the support being restrained.

3.8 LOADS ACTING ON THE STRUCTURE

Various load cases were considered to find the behavior of multistoried RC 2D frame with open soft ground floor. Basic Load cases considered as Dead Load (DL), Live Load (LL), and Earthquake Load (EL). These load cases are combined according to BNBC, 1993.

The Basic load cases are:

- Dead Load (D)
- Live Load (L)
- Earthquake Load, EQ (Static)
- Earthquake Load, EQ (Response Spectrum)

Dead Load (D): Dead load is the vertical load due to the weight of permanent structural and non structural components of a building. For the present study only self weight of beams, columns and slabs are considered as dead load case of the structure.

All vertical loads except self weight of beam, columns and slab are applied as mass on the structure. Total vertical load for floor finish applied on the structure is 1.437 kN/m^2.Total vertical loads for partition wall is variable and is dependent upon the number of floor, number of span, infill thickness (175 mm) and floor height. Floor finish (FF) and Partition wall (PW) loads are applied as mass of the structure and applied at the nodes.

Live Load (L): The temporary load acting on structure as occupancy load is called live load and considered as uniformly distributed surface load (valuing 1.9155 kN/m^2) in vertical direction.

Earthquake Load (E): Earthquake load is applied and analyzed in two methods. Equivalent static force method is used for static analysis and response spectrum method for dynamic analysis. Combinations of loads are considered such as:

- 1.4D
- 1.4D+1.7L
- 1.05D+1.275L+1.4E(static)
- 1.05D+1.275L-1.4E(static)

- 1.05D+1.275L+1.4E(RSM)
- 1.05DL+1.275LL-1.4EL(RSM)

3.9 ANALYSIS OF EARHQUAKE LOAD

Two methods are used to compare the results of seismic load.

- Static analysis (Equivalent static Force Method, ESFM)
- Dynamic Response Analysis (Response Spectrum Method, RSM)

For performing dynamic analysis, it is a prerequisite to determine natural frequencies and mode shape of a structure. Modal analysis is one which determines these two properties.

3.9.1 EQUIVALENT STATIC FORCE METHOD (ESFM)

To calculate seismic lateral forces ESFM is used according to BNBC, 1993. This edition formulated identical approximate formula for calculating period of structure. The empirical relationship for base shear calculation is

$$V = \frac{ZIC}{R} W \tag{3.1}$$

Where,

Z = Seismic zone coefficient

I = Structure importance coefficient

R = Response modification coefficient for structural system

W = Total seismic load

C = Numerical coefficient given by the relation

$$C = \frac{1.25S}{T^{\frac{2}{3}}} \tag{3.2}$$

T = Fundamental period of vibration in sec

S = Site coefficient for soil characteristics

For regular concrete frames, period T may be approximated as

$$T = 0.073(h_n)^{3/4} \tag{3.3}$$

Where, h_n = Height of structure above base in meter

3.9.2 MODAL ANALYSIS

Each structure has its different mode shapes at different frequencies. Modal analysis show how a structure vibrates through its different frequencies and produce different mode shape. The goal of modal analysis in structural mechanics is to determine the natural mode shapes and frequencies of an object or structure during free vibration. Modal Analysis is related with structural frequency. It is pseudo dynamic analysis depending on elastic property. So, modal analysis is not suitable for Non-linear analysis.

Modal analysis helps the determination of the vibration characteristics of structure. It is an essential part of any elastic dynamic analysis process. The natural frequencies and mode shapes of a structure are important parameters in the design of a structure for dynamic loading conditions. They are also required for spectrum analysis or mode superposition harmonic or transient analysis. Modal analysis is done as a linear type analysis. Any nonlinearity such as plasticity and contact (gap) elements are ignored even if they are defined. There are several mode extraction methods:

- Subspace
- Block Lanczos
- Power Dynamics
- Reduced Method
- Unsymmetric
- Damped

In this analysis Block Lanczos method (Wilson, 2002) is used to extract the mode. The modes that are considered, at least 90 percent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction. To review mode shapes in the postprocessor the modes must be expanded. In the single point response spectrum the modal expansion can be performed after the spectrum analysis based on the significance factor.

3.9.3 DYNAMIC RESPONSE ANALYSIS (RSM)

The dynamic response method conforms to the criteria established is BNBC. The mass and mass moment of inertia of various components of a structure required for dynamic analysis should be calculated based on the seismic dead load *W*. The ground motion representation should be one having 20% probability of being exceeded in 50 years.

Response spectrum method (RSM) is used to analysis dynamic responses of a structure subjected to lateral loadings. In this method, multiple modes of response of a structure are taken into account. A response spectrum is simply a plot of the peak or steady-state response (displacement, velocity or acceleration) of a series of oscillators of varying natural frequency that are forced into motion by the same base vibration. From the resulting plot we can assess the pick of the natural frequency for a particular mass of the linear structure. To get the dynamic impact all significant modes should be considered. The number of mode considered should be at least the number of floors. One such use is in assessing the peak response of buildings to earthquakes. The science of strong ground motion may use some values from the ground response spectrum (calculated from recordings of surface ground motion from seismographs) for correlation with seismic damage.

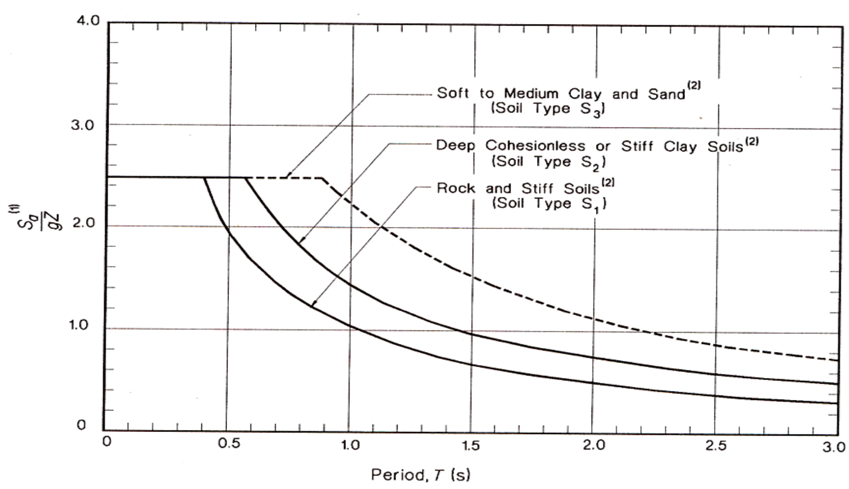
If the input used in calculating a response spectrum is steady-state periodic, then the steady-state result is recorded. Damping must be present, or else the response will be infinite. For transient input (such as seismic ground motion), the peak response is reported. Some level of damping is generally assumed, but a value will be obtained even with no damping.

For single degree freedom system the peak response can be determined directly from the response spectrum for the ground motion without carrying out a response history analysis. But for multiple degrees freedom system the peak response determined directly from the response spectrum for the ground motion is not identical to the RHA result. For this reason response spectrum analysis procedure is for structures excited by a single component of ground motion; thus simultaneous action of the other two components is excluded and multiple support excitations is not considered.

According to BNBC, the response spectrum to be used in the dynamic analysis shall be any one of the following:

- <u>Site specific design spectra:</u> A site specific response spectra shall be developed based on the geologic, tectonic, seismologic, and characteristics associated with the specific site. The spectra shall be developed for a damping ratio of 0.05 unless a different value is found to be consistent with the expected structural behavior at the intensity of vibration established for the site.
- <u>Normalized response spectra:</u> In absence of a site-specific response spectrum, the normalized response spectra shall be used in the dynamic analysis procedure as shown in Figure 3.1.

In absence of a site specific response spectrum the normalized response spectra given in Fig. 3.1 should be used in the dynamic analysis procedure. The analysis should include peak dynamic response of all modes having a significant contribution to total structural response. Peak modal response should be calculated using the ordinates of the appropriate response spectrum curve which corresponds to the modal periods. Maximum modal contributions should be combined in a statistical manner to obtain an approximate total structural response. This is used in the present analysis. Response Spectrum Method is universally accepted method (Wilson, 2002) for design of structure based on dynamic analysis. A few important aspects of Response Spectrum Method are described below.



Note: (1) S_a: Spectral acceleration
g: Acceleration due to gravity
Z: Seismic zone coefficient.
(2) For structures on Soil Type S_4, refer to Sec 2.5.7.1(c) .

Fig. 3.1 Normalized Response Spectra for 5% Damping Ratio (BNBC, 1993)

- **Number of modes**: In case of modal analysis different mode shapes for probable vibration pattern are encountered. Different mode shapes have different frequencies of vibration. Some of the modes are closely spaced showing similar pattern of vibration.

  All significant modes must be included in the analysis of response spectrum. The modes that are considered, at least 90 percent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction. To review mode shapes in the postprocessor the modes must be expanded. In the single point response spectrum the modal expansion can be performed after the spectrum analysis based on the significance factor.

- **Combination of the modes**: The peak member forces, displacements, story forces, shears and base reactions for each mode shall be combined using established procedures in order to estimate resultant maximum values of these response parameters. When two dimensional models are used for analysis, modal interaction effects shall be considered when combining modal maximum. Different mode combination methods for single point response spectrum analysis; such as

  - Square Root of Sum of Squares (SRSS),

  - Complete Quadratic combination (CQC),

  - Double Sum (DSUM), Grouping (GRP),

  - Naval Research Laboratory Sum (NRLSUM).

  Among all these methods CQC method is found suitable for the analysis. The reason is described in article 3.10.

- **Scaling of results**: Base shear for a given direction determined by response spectrum is different from the base shear obtained by equivalent static force method. It should be adjusted which is termed as scaling of results. Scaling of base shear is done according to Bangladesh National Building Code (BNBC, 1993). Base shear of response spectrum is scaled so that this is equal with base shear found from static analysis. As all corresponding parameters including deflections, member forces and moments changes in proportion to the adjusted base shear, scaling is done for 0% infill only. And for other percentage of structurally active infill the same scale factor is used to study effect of infill.

3.10 METHOD OF MODAL COMBINATION

The most conservative method that is used to estimate a peak value of displacement or force within a structure is to use the sum of the absolute of the modal response values. This approach assumes that the maximum modal values, for all modes, occur at the same point in time. Another very common approach is to use the Square Root of the Sum of the Squares, SRSS, on the maximum modal values in order to estimate the values of displacement or forces. The SRSS rule for modal combination developed in E. Rosenblueth's Ph.D. thesis (1951) is

$$r_0 \approx \left(\sum_{n=1}^{N} r_{n0}^2 \right)^{1/2} \tag{3.7}$$

the peak response in each mode is squared, the squared modal peaks are summed, and the square root of the sum provides an estimate of the peak total response. This modal combination rule provides excellent response estimates for structures with well separated natural frequencies. This limitation has not always been recognized in applying this rule to practical problems, and at times it has been misapplied to systems with closely spaced natural frequencies such as piping systems in nuclear power plants and multistory buildings with unsymmetrical plan. For three dimensional structures, in which a large number of frequencies are almost identical, this assumption is not justified.

The relatively new method of modal combination is the Complete Quadratic Combination (CQC) method (Wilson, Kiureghian and Bayo, 1981) that was first published in 1981 is applicable to a wider class of structures as it overcomes the limitations of the SRSS rule. It is based on random vibration theories and has found wide acceptance by most engineers and has been incorporated as an option in most modern computer programs for seismic analysis. The peak value of a typical force can now be estimated, from the maximum modal values, by the CQC method with the application of the following double summation equation:

$$r_0 \approx \left(\sum_{i=1}^{N} \sum_{n=1}^{N} \rho_{in} r_{io} r_{no} \right)^{1/2} \tag{3.8}$$

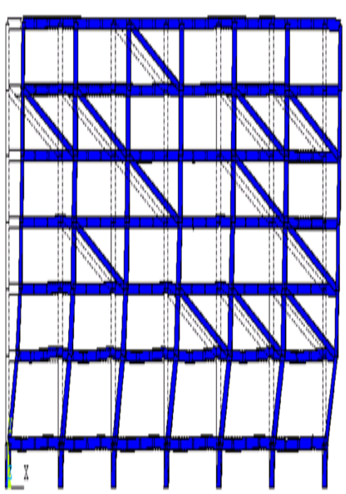
each of the N^2 terms on the right side of this equation is the product of the peak responses in the ith and the $n$nth modes and the correlation coefficient ρ_{in} for these two modes; ρ_{in} varies between 0 and 1and ρ_{in}=1 for $i = n$. Thus Eqn. 3.8 can be rewritten as

$$r_0 \approx \left(\sum_{n=1}^{N} r_{n0}^2 + \underbrace{\sum_{i=1}^{N} \sum_{n=1}^{N}}_{i \neq n} \rho_{in} r_{io} r_{no} \right)^{1/2} \tag{3.9}$$

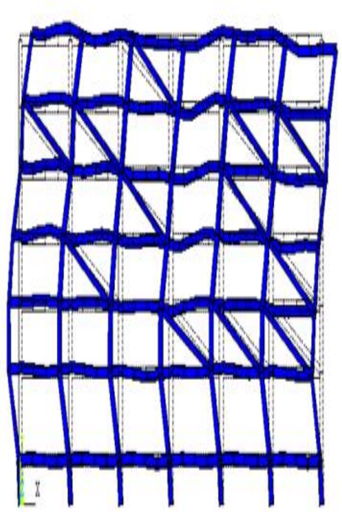
to show that the first summation on the right side is identical to the SRSS combination rule of Eqn. 3.7.

3.11 DIFFERENT MODE SHAPES:

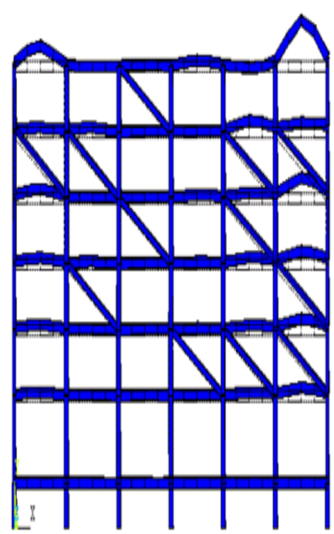
Since modal analysis has been performed hence different mode shapes for probable vibration pattern are encountered. Different mode shapes have different frequencies of vibration. Some of the modes are closely spaced showing similar pattern of vibration. Here some well distinguished mode shapes are featured to give some ideas about the different modes of vibration in dynamic analysis.



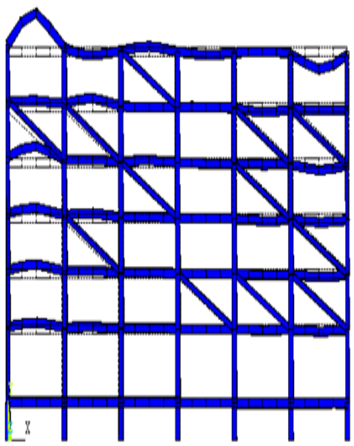
a) 1st Mode Shape



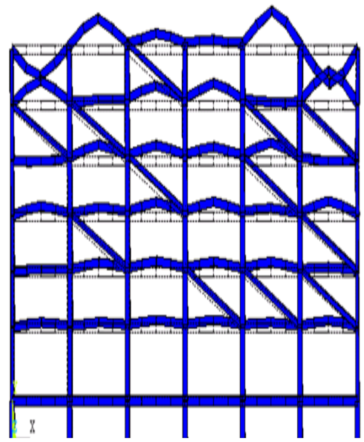
b) 2nd Mode Shape



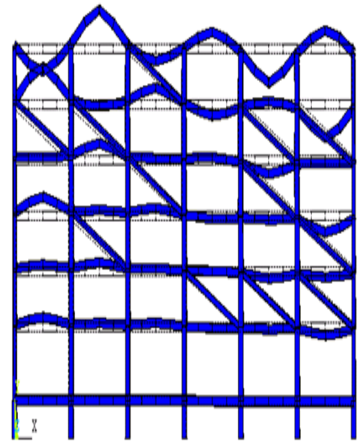
c) 3rd Mode Shape



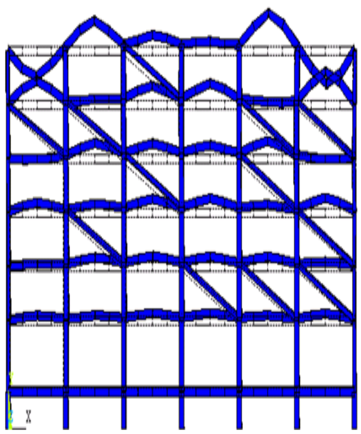
d) 4^{th} Mode Shape



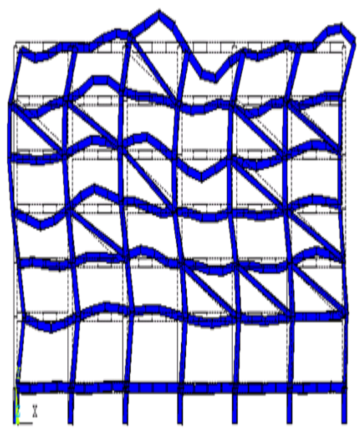
e) 5^{th} Mode Shape



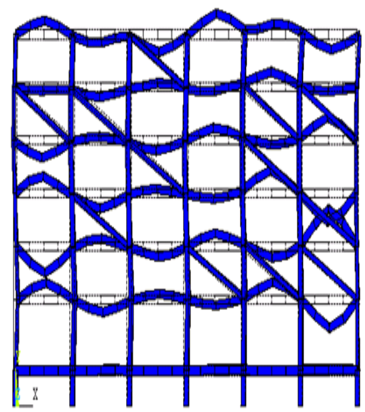
f) 6^{th} Mode Shape



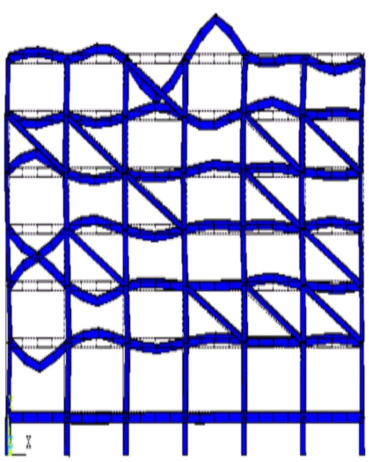
g) 7^{th} Mode Shape



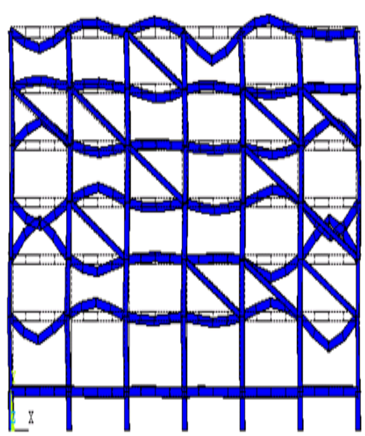
h) 8^{th} Mode Shape



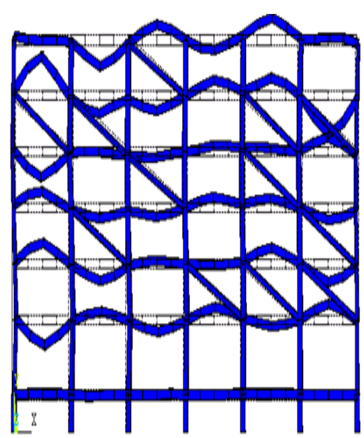
i) 9^{th} Mode Shape



j) 10^{th} Mode Shape



k) 11^{th} Mode Shape



l) 12^{th} Mode Shape

Fig 3.2 Mode shapes of a 6 storied building

3.12 STUDY PARAMETERS

The main objective of this thesis is to study seismic effect of a 2D frame with random infill keeping the ground floor as a soft story. For a wide range of analysis several multiple storied frames with variable span length has been analyzed. Storied considered for this study are 6 storied, 9 storied,12 storied,15 storied and 18 storied with variable span length i.e. 2 spanned,4 spanned,6 spanned,8 spanned and 10 spanned frame has been analyzed separately by ANSYS10. Analysis has been done by considering varying percentage of infill i.e. 0%, 20%, 40%, 60% and 80% which applied randomly. For a particular height of frame with fixed span length, a particular amount of infill is applied randomly. This process is repeated for 15 times and results are taken each time. Then the average value is considered for base shear and sway.

A reinforced concrete moment resisting 2D frame with open ground story and un-reinforced brick infill walls in the upper story is chosen for this study is shown in fig.-3.3. The building is considered to be located in seismic zone II and intended for residential use. The dimensions of structural components were assumed relatively and the material parameters were taken accordingly for normal concrete.

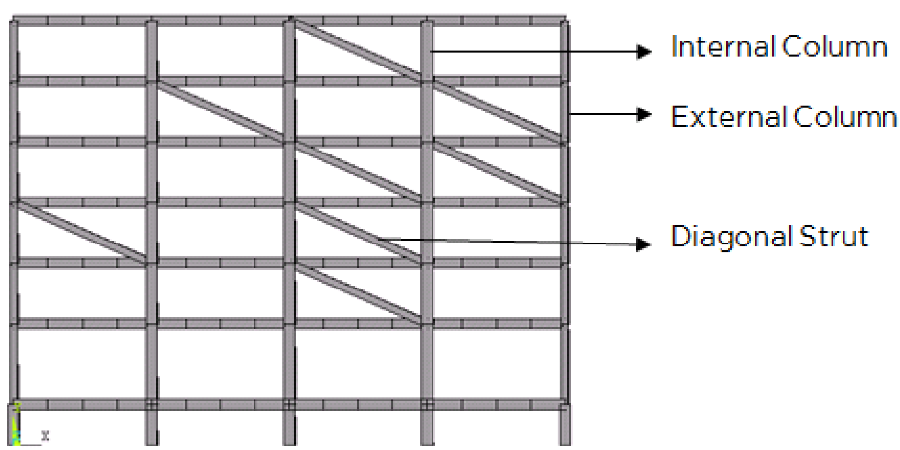


Fig. 3.3 Finite Element modeling of total structure

Table 3.1: Values of parameters of the generalized reference model

Sl No.	Parameter	Value/Dimension
01	Span length	7000 mm
02	Bay width	5000 mm
03	Floor height	3000 mm
04	Slab thickness	177.80 mm
05	Floor finish	1.437 k N/m^2
06	Live load	1.9155 k N/m^2
07	Beam Width	300mm
08	Beam Height	Span length /14
09	Column	as per design requirement
10	Gravitational acceleration	9810 mm/sec^2
11	Number of story	6 , 9, 12, 15 and 18
12	Height of each story	3m
13	Height of ground floor	4m
14	Number of span	2 to 10
15	Number of Bay	1
	Concrete properties	
16	Modulus of elasticity of concrete	21530 N/mm^2
17	Poisson's ratio	0.2
18	Density of concrete	2.44 ton/ m^3
19	Unit weight	24 kN/m^3
	Infill properties	
20	Thickness	175mm
21	Amount of Infill	0% , 20%, 40%, 60% and 80%
22	Infill Pattern	Randomly Distributed

3.12.1 EXAMPLE OF LOAD CALCULATION:

Frame Parameters

Height of column below Grade Beam, H1	= 2 m
Height of column of Ground Floor, H2	= 4 m
Height of column of other floor, FH	= 3 m
Slab thickness, t	= 0.1778 m
Infill Thickness, INFLTH	= 0.175 m
Bay Width, BAYW	= 5 m
Span Length, SPANL	= 7 m
No. of span, NSPAN	= 2
No. of bay, NBAY	= 1
No. of floors, NFLOOR	= 6
Infill %	= 20 %
Unit weight of Concrete, u	= 24 kN/m^3

Beam Parameters

Internal Beam Parameter

Beam Height, BHL	= SPANL/14 or BAYW/14 or 0.3 m (whichever is greater)
	= 0.5 m or 0.357 m or 0.3 m
Here, appropriate value is	= 0.5 m
Beam Width, BWL	= 0.3 m
Beam Height, BHT	= SPANL/14 or BAYW/14 or 0.3 m (whichever is greater)
	= 0.5 m or 0.357 m or 0.3 m
Here, appropriate value is	= 0.5 m
Beam Width, BWT	= 0.3 m

Load Calculation

a) Weight of Floor Finish, FF = 30 psf = 1.437 kN/m^2

Total weight of floor finish, WFF = FF×SPANL×BAYW×NSPAN×NFLOOR

= 603.54 kN

c) Weight of Live Load, LL = 40 psf = 1.9155 kN/m^2

Total weight of live load, WLL = LL×SPANL×BAYW×NSPAN×NFLOOR

= 804.525 kN

d) Weight of Partition wall

Total no of infill panel for drawing infill,

NPANEL=NSPAN (NFLOOR-1) = 10 (except ground floor)

No of infill, NINFILL = NINT (NPANEL×percent/100) = 2

Wt of a single infill, WTINF1=SPANL×FH×INFLTH×U = 88.2 kN

Total weight of infill, WTINFLT=NINFILL×WTINF1 = 176.4 kN

Weight of Infill per floor, WTINFPF=WTINFLT/NFLOOR = 29.4 kN / floor

Weight of partition wall, PW=WTINFPF/ (NSPAN×SPANL×BAYW) = 0.42 kN/m^2

e) Weight of Slab

Total weight of Slab, WS = h×SPANL×BAYW×u×NSPAN×NFLOOR = 1792.224 kN

f) Weight of Beam

Wt of Longitudinal Beam = (BHL×BWL)×SPANL×u×NSPAN×NFLOOR = 302.4 kN

Wt of Transverse Beam = (BHL×BWL)×BAYW× u ×(NSPAN+1)×NFLOOR = 324 kN

Total Weight of Beam = 626.4 kN

Column Parameters:

Wt of (floor finish + partition wall + slab) acting on ground floor column ,

W_{fps} = (FF+PW+ (h×1×1×u))×SPANL×BAYW×NFLOOR = 1286 kN

Wt of beam (longitudinal + transverse) acting on ground floor column,

W_b = ((BAL×SPANL×U)+(BAT×BAYW×U))×NFLOOR = 259.2 kN

Dead load acting on ground floor column, DLC= (W_{fps} + W_b) = 1545.21 kN

Live load acting on ground floor column

LLC=LL×SPANL×BAYW×NFLOOR = 402.26 kN

Total load acting on ground floor column

P_u = (1.4×DLC+1.7×LLC)×1.5 = 4270.71 kN (1.5 is Arbitrary to account for the effect of lateral load))

Fcp = 2.068e4 = 20687.79 kN/m^2 (Concrete strength assumed as 3000 psi which turned into kn/m2)

Roh=0.02 (Steel Ratio, assumed)

Fi=0.7

Fy=4.138e5 = 413755.93 kn/m^2 (Steel yield strength assumed as 60,000 psi which turned into kn/m^2)

Stirrup number = 3

Ccover = 0.038 (Clear cover assumed as 1.5 in)

Bar no = 6

Internal Column Parameters :

A_{gi} = P_u / (0.8×0.7×(0.85×Fcp+Roh×(Fy-0.85×Fcp))) (Internal Column area in mt)

= 0.29897 m^2

ICH= $\sqrt{(A_{gi})}$ = 0.54678 m $\geq$ 0.3 m (ICH=Internal column thickness)

ICH = 0.54678 m

ICW=ICH = 0.54678 m (ICW= Internal column Width)

External Column Parameter:

A_{ge} = (P_u×0.5) / (0.8×0.7× (0.85×Fcp + Roh×(Fy-0.85×Fcp))) (Ext. Column area, m)

= 0.14948 m^2

$ECH = \sqrt{(A_{ge})} = 0.38663\ m \geq 0.3\ m$ (ECH=Ext. Column Thickness)

ECH = 0.38663 m

ECW=ECH = 0.38663 m

Grade Beam Parameter :

GCH=ICH = 0.54678 m (Column Height below grade beam)

GCW=ICW =0.54678 m (Column width below grade beam)

GBH=ICH = 0.54678 m (GBH=Grade beam height)

GBW=ICW = 0.54678 m (GBW=Grade beam width)

g) Weight of Grade Beam

Wt. of Grade Beam in Longitudinal Direction,

WGBL = (GBH×GBW)×SPANL×u×NSPAN = 100.455 kN

Wt. of Grade Beam in Transverse Direction,

WGBT = (GBH×GBW) ×BAYW×u× (NSPAN+1) = 107.631 kN

Total Weight of Grade Beam, WGB = 208.086 kN

h) Weight of column under Grade Beam

Internal Column, WGCI = GCW×GCH×H1× u × (NSPAN-1) = 14.35079 kN

External Column, WGCE = GCW×GCH×H1×u×2 = 28.70158 kN

Total Wt. of column under grade beam, WGC = WGCI+WGCE = 43.052 kN

i) Weight of column above grade beam/ground floor

Internal Column, WIC_g = ICA×H2×u×(NSPAN-1) = 28.701 kN

External Column, WEC_g = ECA×H2×u×2 = 28.701 kN

Total wt. of column above grade beam, $WC_g = WIC_g + WEC_g$ = 57.403 kN

j) Weight of column above ground floor

Internal Column , WIC = ICA×FH×u×(NSPAN-1)×(NFLOOR-1) = 107.631 kN

External Column, $WEC = ECA \times FH \times u \times 2 \times (NFLOOR-1) = 107.631$ kN

Total wt. of column above ground floor, WC = WIC + WEC = 215.262 kN

Now total weight of the structure,

$W = WFF+WTINFLT+WS+WB+WGB+WGC+WC_g+WC$
$= \mathbf{3722.368\ kN}$

Earthquake Parameter:

Z =0.15 (Siesmic zone coefficient for zone-2, BNBC Table 6.2.22)

I = 1 (Structure Importance Coefficient)

R = 8 (Response modification coeffcient for structural system)

S = 1.5 (Site coefficient for soil characterstics)

Ct = 0.073

$h_n = H2+(NFLOOR-1) \times FH = 19$ (Height in meters above the ground to level n)

Structure Period, $T = Ct \times (h_n)^{3/4} = 0.664$

$C = 1.25 \times S / (T)^{2/3} = 2.463 < 2.75$ (OK)

Now Design Base Shear , V = ZICW / R = **171.882 kN**

The additional lateral force assume to approximate the effects of higher nodes of structural vibration, Ft

If, T > 0.7 second then $Ft = 0.07 \times T \times V \leq 0.25V$

or else if $T \leq 0.7$ then Ft = 0

Since, T = 0.664 < 0.7So, Ft = 0

Wt of the portion of the building assumed to be lumped at level x, Wx

$W1 = W_{belowgradebeam} = WGB+WGC = 251.1388$ kN

Wt. of ground floor,

$W2 = W_{ground} = WFF/NFLOOR + WTINFLT/NFLOOR + WB/NFLOOR + WS/NFLOOR + WCg$

$= 590.497$ kN

Wt. of Other Floor,

W3 = Wi = WFF/NFLOOR + WPW/NFLOOR + WB/NFLOOR + WS/NFLOOR + WC / (NFLOOR-1) = 576.146 kN

Determination of Each Story Forces:

The magnitude of each story force ,

$$\mathrm{Fx} = \frac{(V-Ft)Wxhx}{\sum_{i=1}^{n} Wihi} \quad \text{.................. (1)}$$

Now from the equation (1), various values can be summarized in a table as below :

Table 3.2 Storey Forces acting on study frame for ESFM

Story No.	$\mathbf{W}_i$ / h_i	$\mathbf{W}_x$ / h_x	**Base Shear, V**	$\mathbf{V \times W}_x \mathbf{\times} h_x$	$\mathbf{F}_x$
1	3542.98	3542.94	171.882	608967.61	13.02
2	5185.31	5185.32	171.882	891262.76	19.06
3	6913.75	6913.76	171.882	1188350.35	25.42
4	8642.19	8642.20	171.882	1485437.93	31.77
5	10370.63	10370.64	171.882	1782525.52	38.13
6	12099.07	12099.07	171.882	2079613.11	44.48
$\sum$ $\mathbf{W}_i$ / h_i =	**46753.93**			$\sum F_x$ =	**171.88**

Again the base shear and storey forces found from software analysis are given below,

Base Shear :

```
Reaction_different lcases:
Deadload                    :        3722.195320KN
Liveload                    :         804.525418KN
Earthquake(static)          :        -171.880347KN
Earthquake(rsm)             :         203.251786KN
```

And Storey Forces :

```
Floor Ht=  6.0 m:ESF=      13.025 kn
Floor Ht=  9.0 m:ESF=      19.063 kn
Floor Ht= 12.0 m:ESF=      25.417 kn
Floor Ht= 15.0 m:ESF=      31.771 kn
Floor Ht= 18.0 m:ESF=      38.125 kn
Floor Ht= 21.0 m:ESF=      44.479 kn
```

This is equal to the value found from above manual calculation. (Checked).

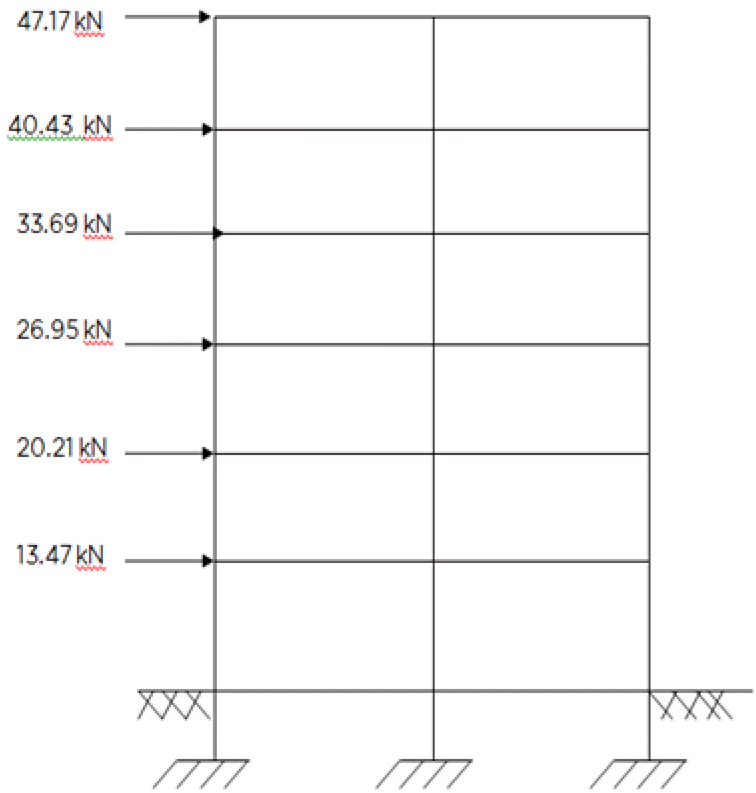


Fig.3.4 Story Forces determined by ESFM method.

3.12.2 DEFORMED SHAPES FOR DIFFERENT LOADING PATTERN:

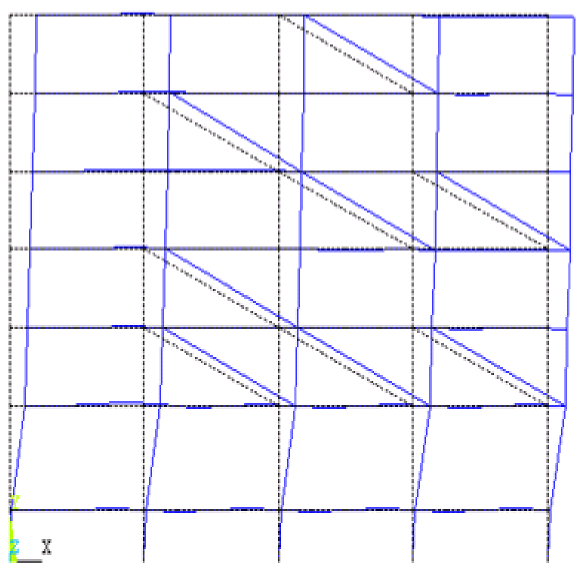


Fig. 3.5 (a) Deflected shape due to EQ for 40%infill of 6 storied frame.

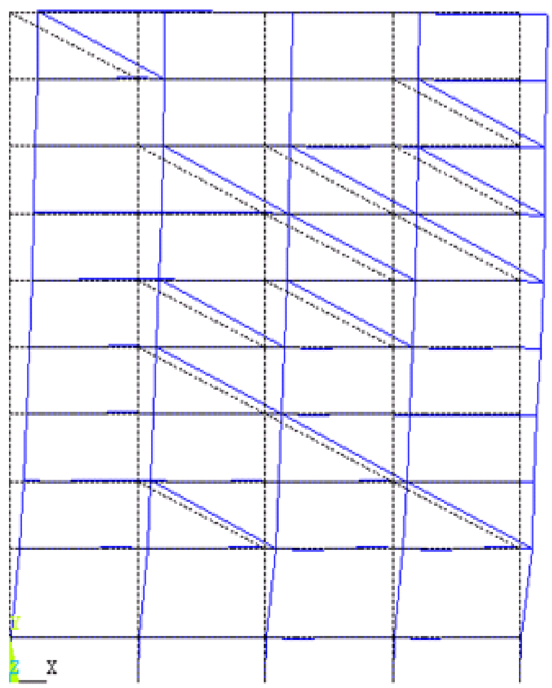


Fig. 3.5 (b) Deflected shape due to EQ for 40%infill of 9 storied frame.

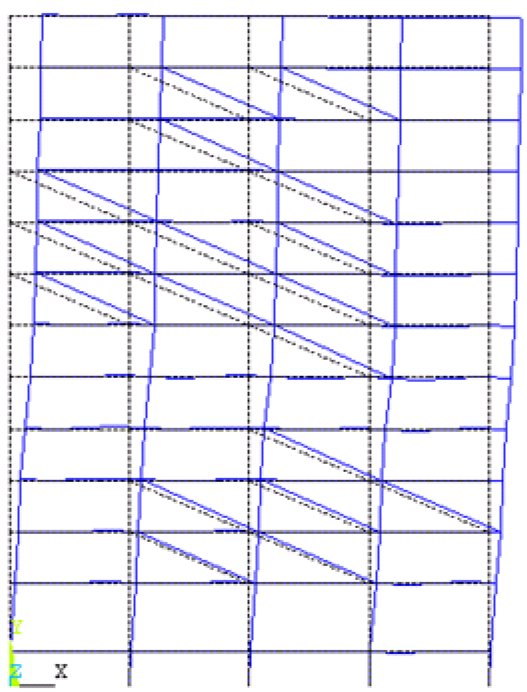


Fig. 3.5 (c) Deflected shape due to EQ for 40%infill of 12 storied frame.

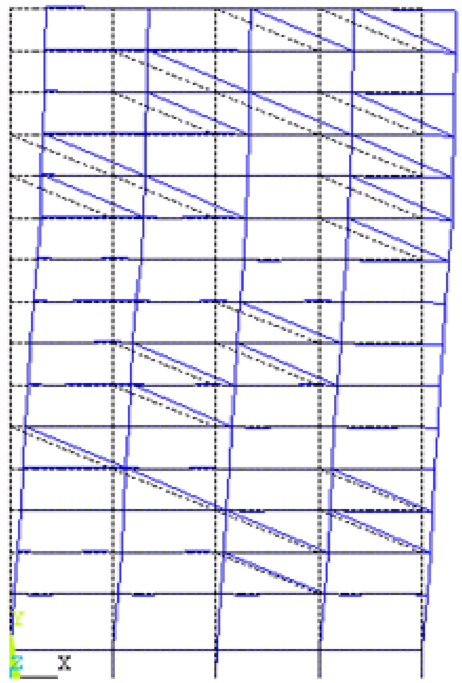


Fig. 3.5 (d) Deflected shape due to EQ for 40%infill of 15 storied frame.

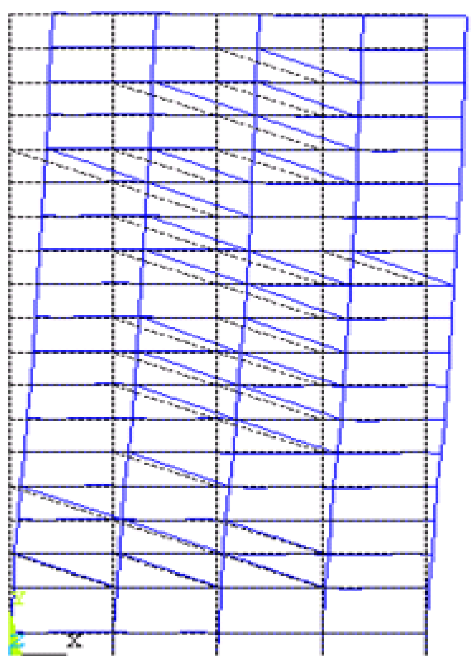
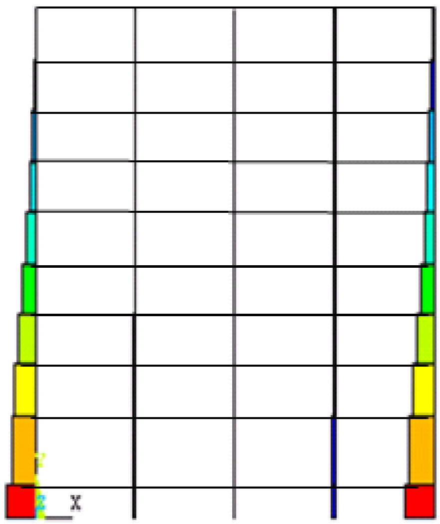


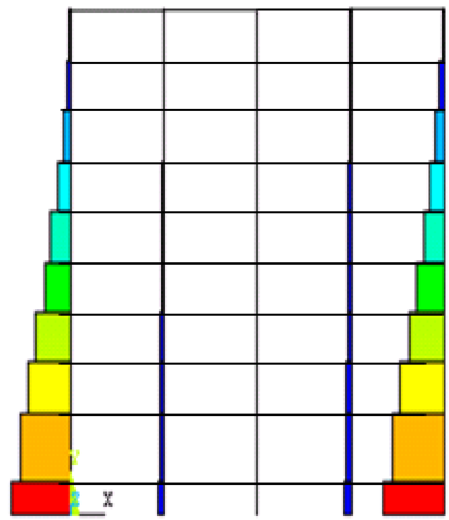
Fig. 3. 5 (e) Deflected shape due to EQ for 40%infill of 18 storied frame.

3.12.3 STUDY OF MEMBER FORCES

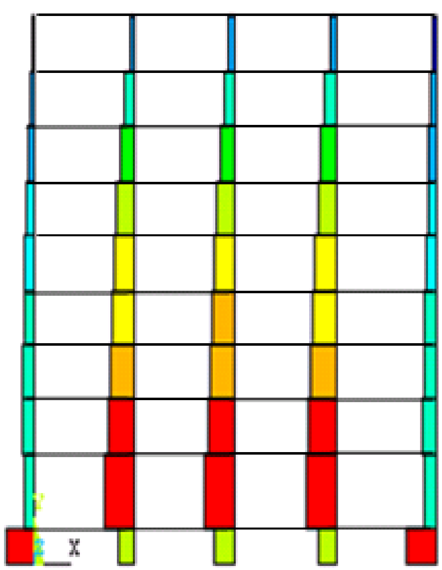
Shear force, axial force and bending moment diagram for a frame of 9 storied building are shown in Fig. 3.6. Comparison of frames without infill and with 50% infill is presented here for both equivalent static force method and response spectrum method. These diagrams reveal the nature of forces and moments developed in the columns.



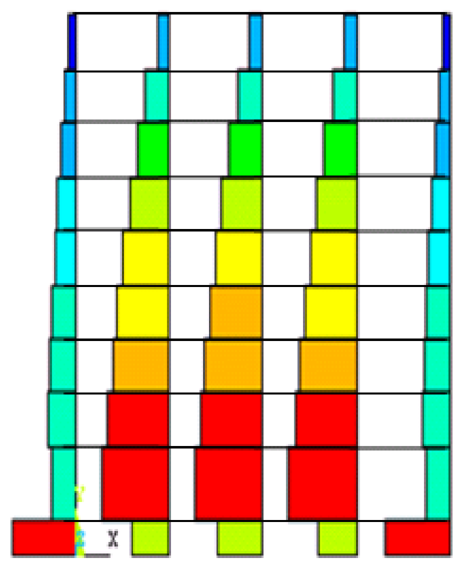
a-1 Axial force (Bare Frame) ESFM



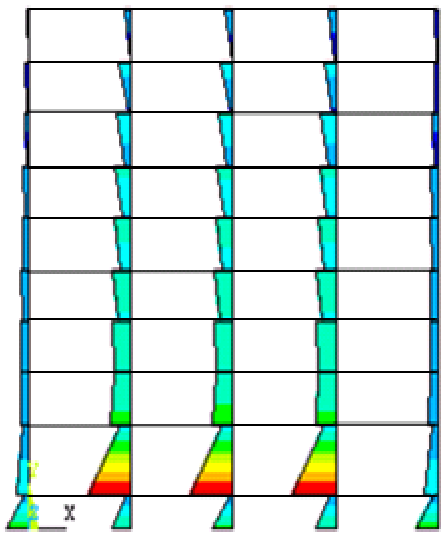
b-1 Axial force (Bare Frame) RSM



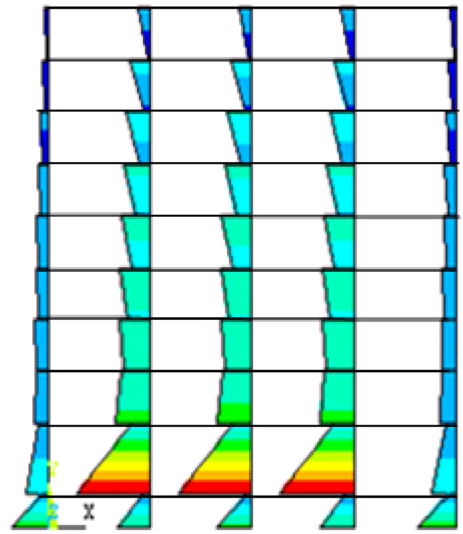
a-2 Shear force (Bare Frame) ESFM



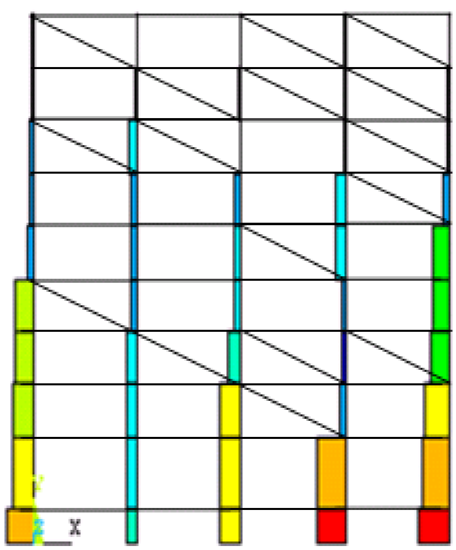
b-2 Shear force (Bare Frame) RSM



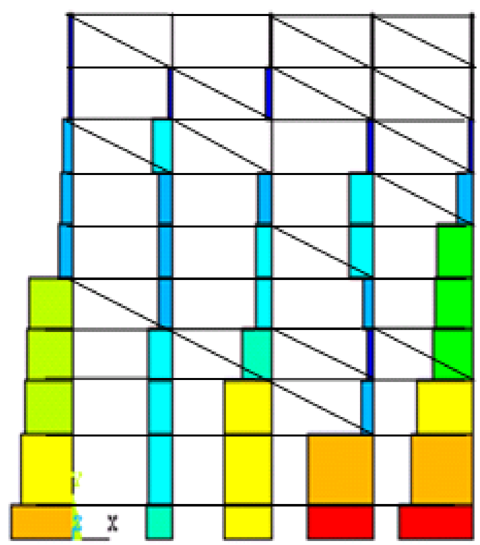
a-3 Bending Moment (Bare Frame) ESFM



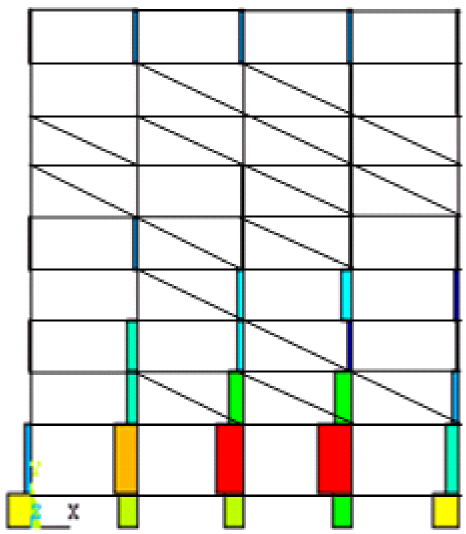
b-3 Bending Moment (Bare Frame) RSM



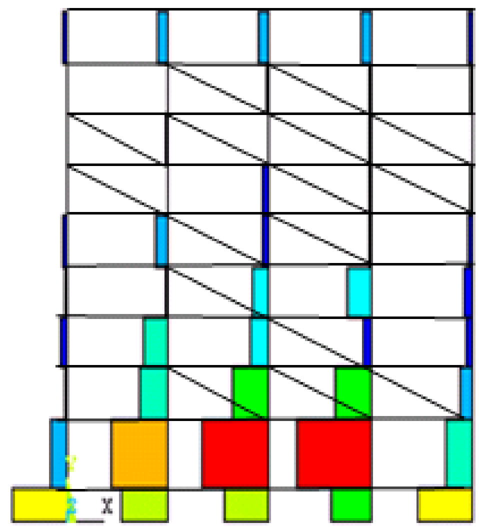
a-4 Axial force (50% Infill) ESFM



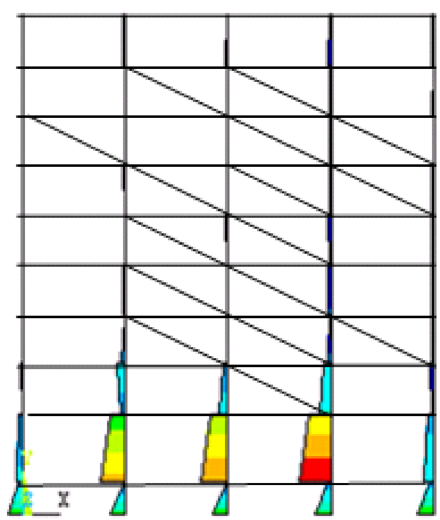
b-4 Axial force (50% Infill) RSM



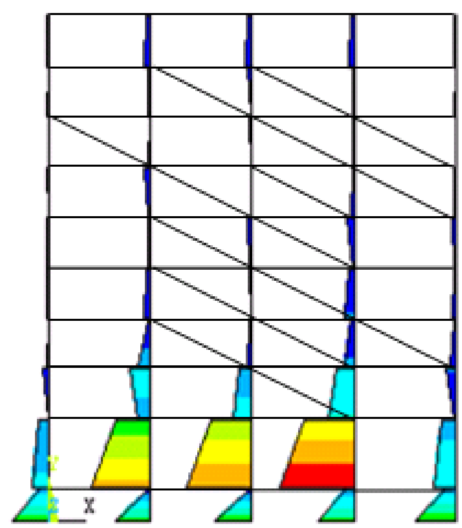
a-5 Shear force (50% Infill) ESFM



b-5 Shear force (50% Infill) RSM



a-6 Bending Moment (50% Infill) ESFM



b-6 Bending Moment (50% Infill) RSM

Fig.3.6 Moment and Force diagram of 9 storied frame for ESFM & RSM loading.

The findings from these figures can be summarized as follows.

Axial Force Diagram: For the case of frame without infill, the axial force is very nominal. But for 50% infill, it shows that there is a prominent effect of infill for earthquake loading. Value is significant for response spectrum method. In RSM for 50% structurally active infill, it is found that axial force is significant above ground floor while for bare frame it is insignificant. The reason is that floor finish and partition load is applied on the structure as mass and infill is placed as diagonal strut. So this diagonal struts increases the axial force. As earthquake loading is applied laterally so values of axial force is not much.

Values of shear force and bending moment are much higher for response spectrum loading. For this reason mixed frame action does not show any effect for shear force and bending moment.

Shear Force Diagram: For shear force, the diagram shows major change in force in upper stories due to presence of infill. In upper stories the shear force decreases and almost becomes zero while in ground floor the shear force increases. Fig. 3.6a (2) gives shear force 103KN for model without infill in a ground floor column for ESFM whereas for the same column with 50% infill this value is 126 KN, has shown in fig 3.6b (2) in ESFM. The value in RSM is 304 KN has shown in fig.3.6b (5).

The infill makes the structure stiffer to deflect. As shown in the mode shape 1 deflection is concentrated in the open ground floor only. Presence of infill stiffens the upper stories which makes deflection concentration and so shear concentration in the open ground floor.

Bending Moment Diagram: Bending moment increases in the first ground floor whereas reduces in upper floors after placement of infill. Infill is placed as diagonal strut which stiffens the structure. As a result bending moment decreases in those stories where infill is placed.

As shear force increases in the open ground floor due to presence of infill in upper floors, consequently bending moment shows same pattern of change. Due to pendulum effect deflection is concentrated in ground floors so the moment is also concentrated.

3.13 Random distribution of infill:

Infill position plays a significant role to contribute on structural modification. In current 2D analysis the random infill position is featured for each case so that base shear can be found for different position of infill after each time run.

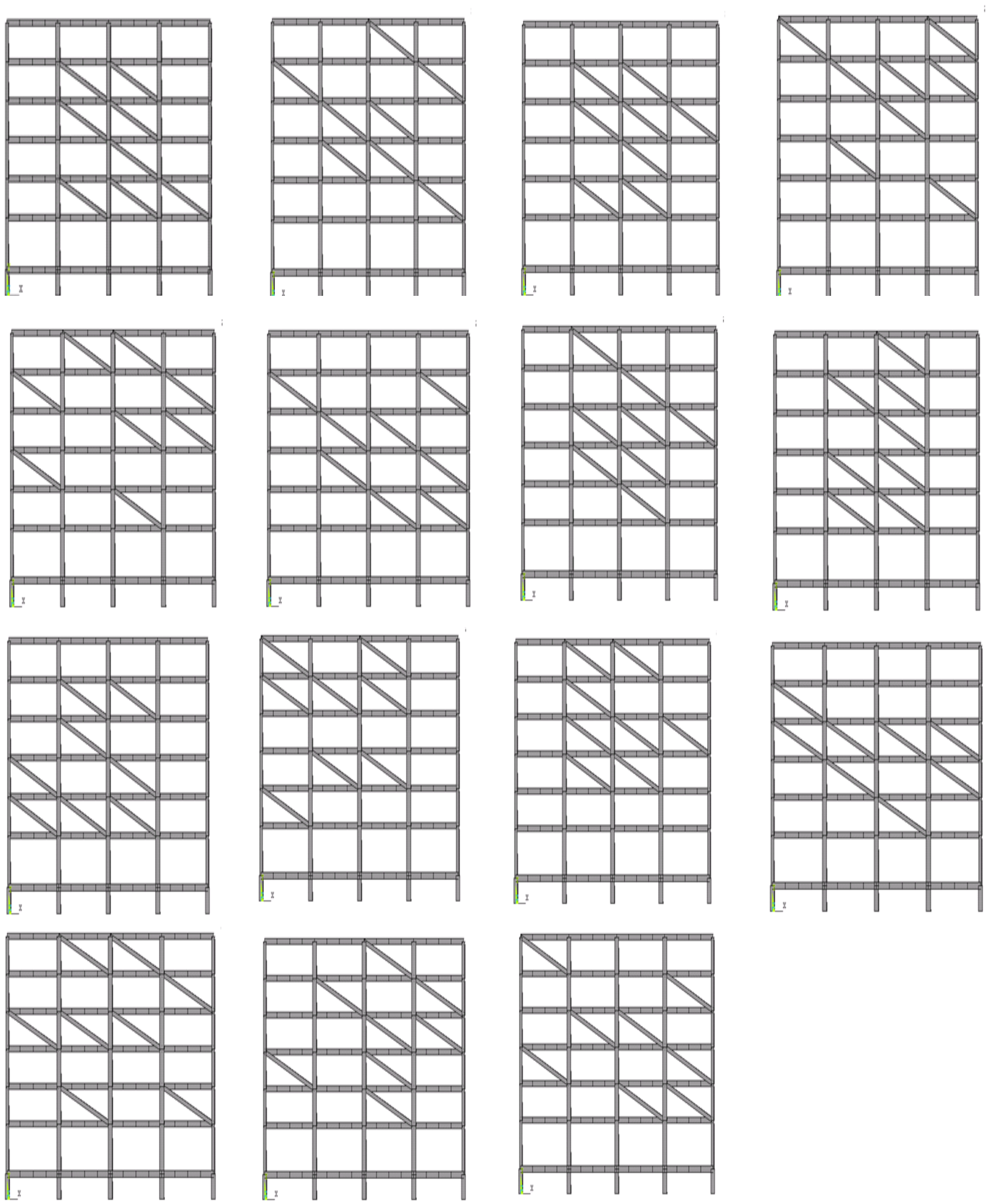


Fig.3.7 Different patterns of 40% random infill application (6 storied building)

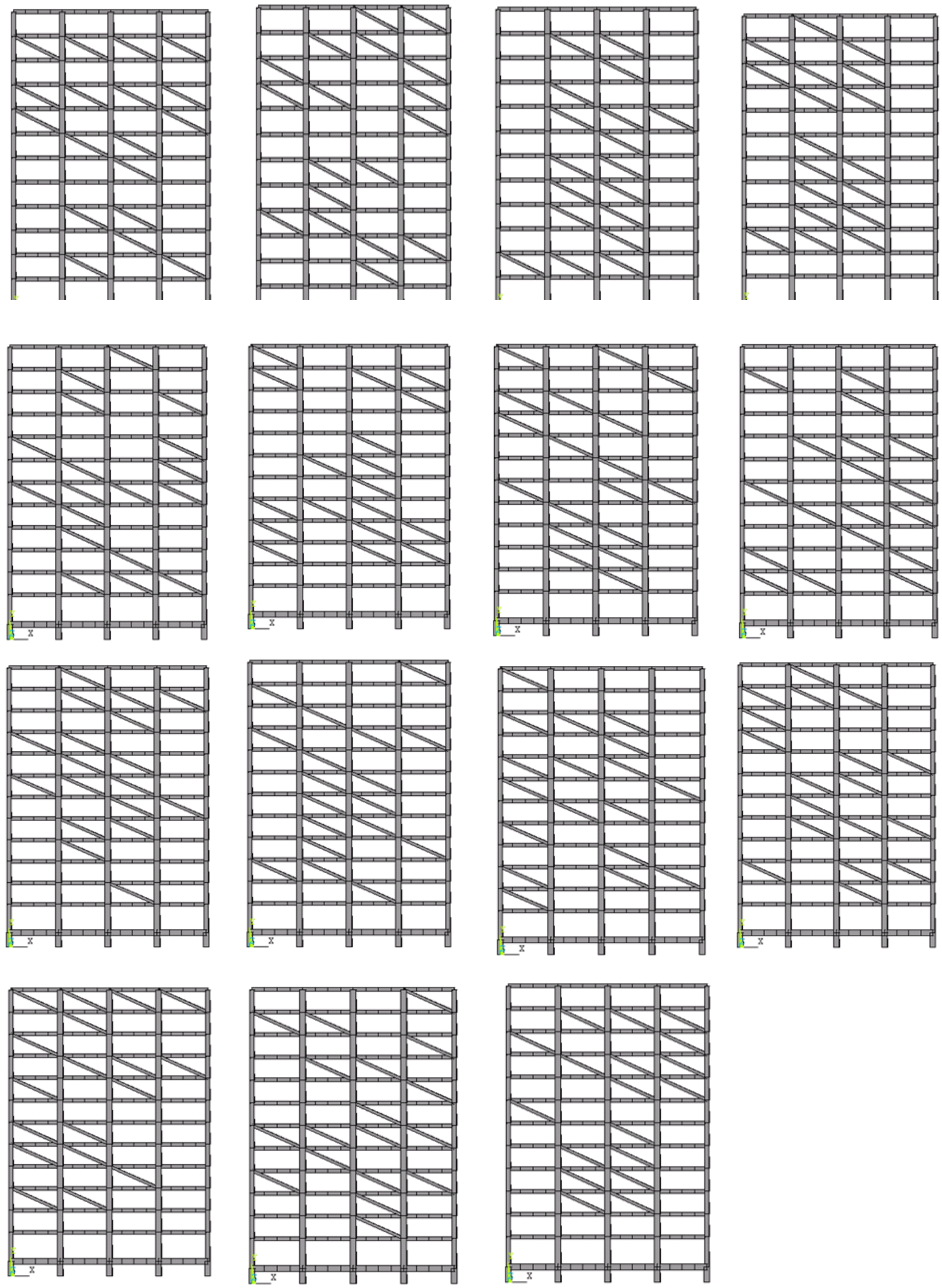


Fig.3.8 Different patterns of 40% random infill application (12storied building)

3.14 REMARKS

Total analysis procedure of the present study starting from software used, elements used for Finite Element Modeling to the applied loads and load cases for observing the effect of soft ground floors are described in this chapter. Finally parameters of reference models are described and analyzed with some parameter i.e. $V_{avg(rsm)}/V_{esfm,}$ Height/Length, Base Shear and Sway pattern. The analysis is based on the comparison between without infill and variable percentage of randomly distributed infill**.**

CHAPTER 4

PARAMETRIC STUDY AND DISCUSSION

4.1 INTRODUCTION

Seismic performance of a soft story 2D frame has been analyzed in the current study considering varying percentage of randomly distributed infill on the upper floors. Other parameters i.e. building height, numbers of span, load combination etc are also considered in the study. The parameters are selected in such a way that practical behavior of RC frame is reflected in the model. Results of analyzing of varying parameters are described in this chapter. The effect of randomly applied varying percentage of infill i.e 0%, 20%, 40%, 60% and 80% has been studied for 6, 9, 12, 15 and 18 storied frame with open ground floor. The effect on base shear due to various number of span with respect to height of the frame i.e. slenderness of frame (frame height versus length, H/L) is also considered both in static (ESFM) and dynamic (RSM) method. Results were compared for base shear, drift, sway, base shear versus slenderness of frame.

4.2 STRUCTURAL PARAMETERS FOR MODELS

Presence of infill in the structures changes the mass and stiffness. Base shear of building during earthquake is dependent on its natural period; whereas the time period of a building is basically a function of its mass and stiffness. Thus any structural or building parameter that changes the stiffness of mass shall have influence on the period as well as base shear and sway. In the present study the varying parameters are percentage of infill, frame height and number of span.

The values of parameters for reference model have been described in the article 3.12 in Table 3.1. As per that reference 6, 9, 12, 15 and 18 storied 2D frame models are studied with different infill percentages to observe soft ground floor effect due to lateral force.

4.3 PARAMETERS OF MODELS FOR FRAMES

For the current 2D frame, 15 times software run were executed (except bare frame) for a particular height of frame with certain amount of randomly distributed infill. Then an average value of base shear was taken from all of those 15 results. For each frame the columns are designed considering all design loads. Considered load cases are dead load, live load, earthquake load. The infill is placed as diagonal strut model. The infill percentages are 0%, 20%, 40%, 60%, and 80%. Number of span considered as 2, 4, 6, 8 and 10 spanned for a particular height of the frame i.e; 6, 9, 12, 15 and 18 storied. Various parameters of these buildings are given a tabular form in Table 4.1.

Infill and mass locations are so placed to avoid accidental torsional effects following Bangladesh National Building Code (BNBC, 1993). According to BNBC to avoid accidental torsion the vertical lateral load resisting elements should be parallel to or symmetric about the major orthogonal axes of the lateral force-resisting system. Each infill is applied as mass on the structure and the infill effect is placed as diagonal strut.

Table 4.1: Parameters of Study Frames

Parameters	**Values**
Number of bay	1
Number of span	2, 4, 6, 8 and 10
Number of story	6, 9, 12, 15 and 18
Span×Bay (mm)	7000×5000
Height of story (mm) (1^{st} floor to top floor)	3000
Height of ground floor (mm)	4000
Ht of col under Grade Beam(mm)	2000
Beam height (mm)	Max. of Span length/14 or Bay width /14 or 300
Beam Width (mm)	300
Column (External and Internal) size (mm×mm)	As per design
Height of grade beam (mm)	As per design
Infill Thickness (mm)	175
Slab thickness (mm)	177.80
Infill Percentage (%)	0% , 20%, 40%, 60% and 80%
No. of run for each span with respect to fixed % of infill	15

4.3.1 Effect of randomness of infill on base shear

Infill has been applied randomly and base shear value is taken for the same application. In this way software has run for 15 times. Each run is a combination of fixed height, fixed span and fixed percentage of infill, just changing the infill location each times. Then bar chart has plotted using those base shear value.

Table 4.2: Base shear variation of 9 storied with 10 spanned frame for random application of Infill (in different percentage) with soft ground floor

Upper floor infill percentage	**20%**	**40%**	**60%**	**80%**
Base shear values by RSM for different patterns of infill application (KN)	1659	2507	2738	2954
	1961	2227	2747	2959
	1757	2403	2719	2957
	1721	2179	2735	2945
	1745	2451	2722	2951
	1792	2294	2737	2964
	1655	2372	2735	2949
	1772	2427	2749	2951
	1880	2341	2753	2960
	1831	2358	2750	2949
	1704	2362	2732	2955
	1714	2266	2738	2946
	1768	2446	2747	2946
	1846	2420	2751	2944
	1681	2156	2726	2957
Average Base shear by RSM (KN)	1766	2347	2739	2952
Maxima (KN)	1961	2507	2753	2964
Minima (KN)	1655	2156	2719	2944
Standard deviation	82.64	100.19	10.44	5.92
Percentage (%) variation with Avg.	4.68	4.27	0.38	0.20

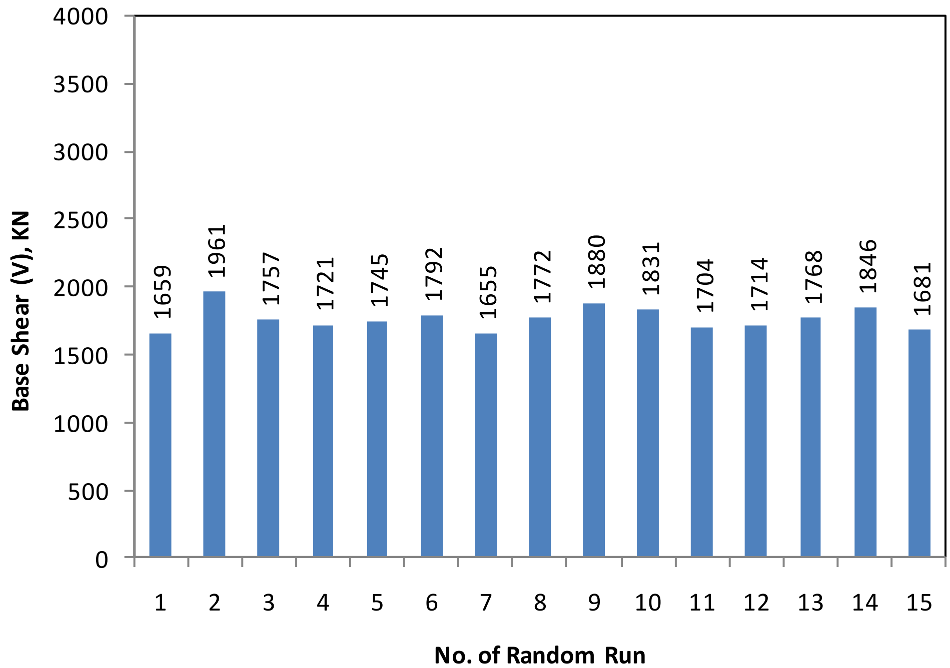


Fig.4.1 Variation of Base Shear for 20% Infill 9 Floor10 Spanned Frame

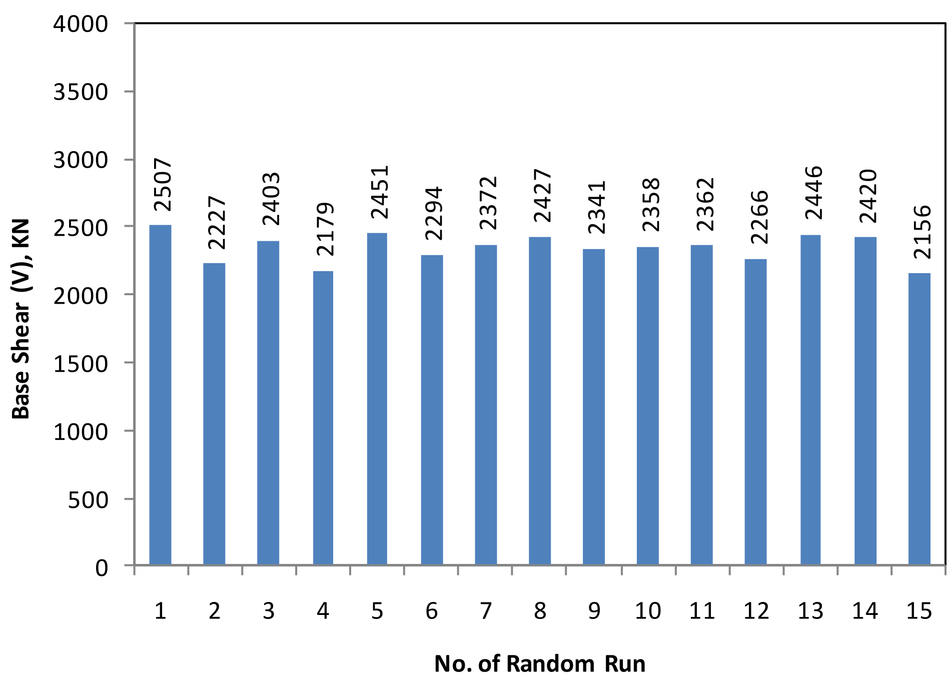


Fig.4.2 Variation of Base Shear for 40% Infill 9 Floor10 Spanned Frame

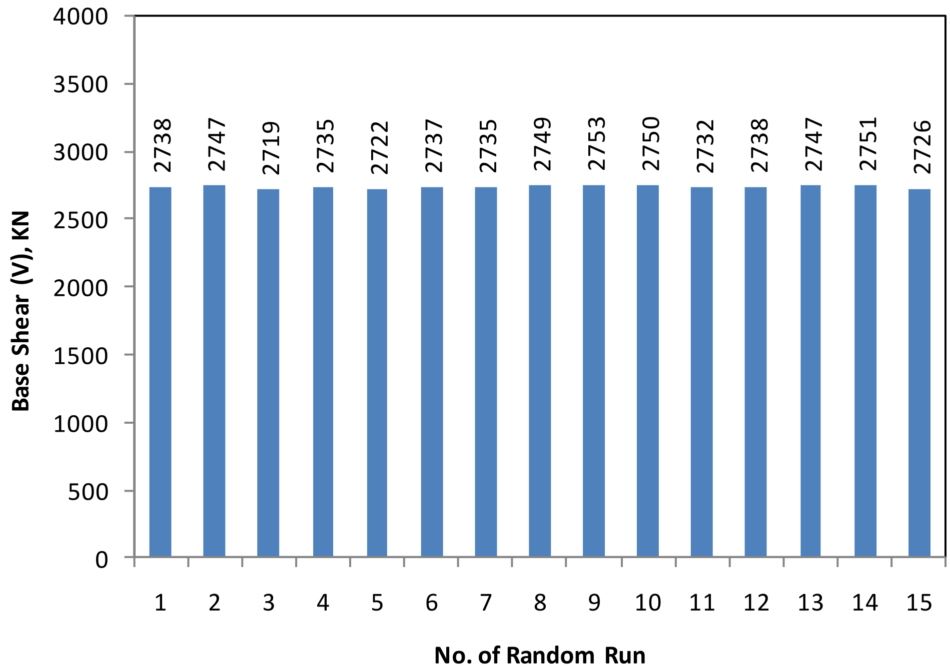


Fig.4.3 Variation of Base Shear for 60% Infill 9 Floor10 Spanned Frame

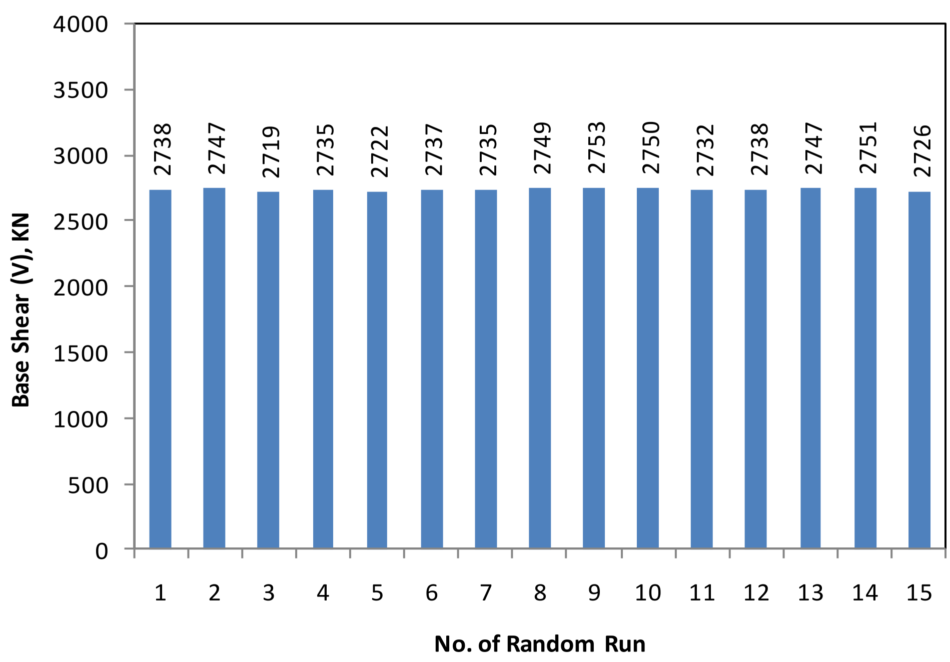


Fig.4.4 Variation of Base Shear for 80% Infill 9 Floor 10 Spanned Frame

Table 4.3: Base shear variation of 18 storied with 6 spanned frame for random application of Infill (in different percentage) with soft ground floor

Upper floor infill percentage	20%	40%	60%	80%
Base shear values by RSM for different patterns of infill application (KN)	1727	2363	2897	3362
	1677	2356	2913	3418
	1673	2471	2862	3366
	1810	2300	2895	3367
	1745	2346	2926	3373
	1620	2210	2927	3415
	1678	2454	2899	3421
	1670	2258	2927	3417
	1668	2252	2855	3453
	1705	2396	2936	3386
	1724	2389	2887	3391
	1677	2334	2873	3385
	1807	2369	2896	3398
	1719	2381	3004	3418
	1851	2437	2931	3375
Average Base shear RSM(KN)	1717	2354	2909	3396
Maxima (KN)	1851	2471	3004	3453
Minima (KN)	1620	2210	2862	3362
Standard deviation	61.35	72.27	35.31	25.51
Percentage (%) variation with Avg.	3.57	3.07	1.21	0.75

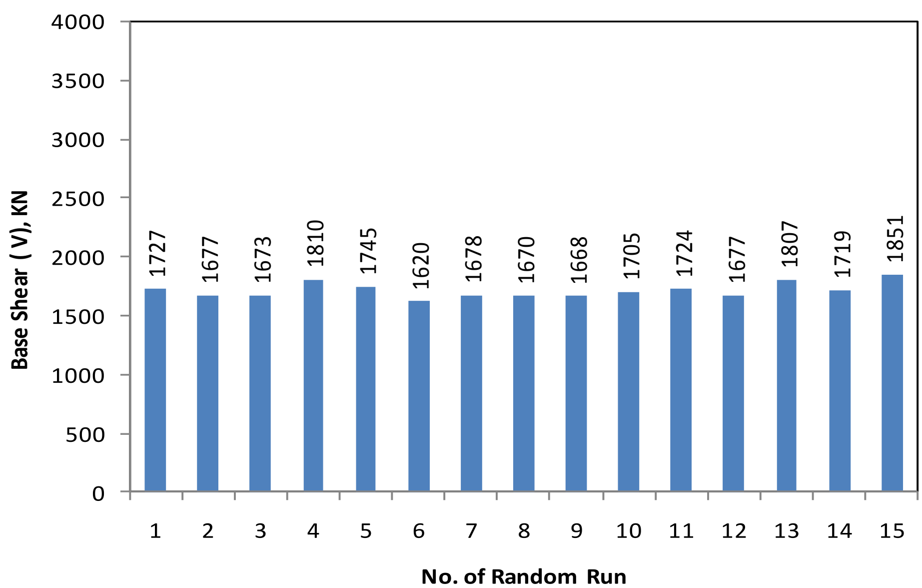


Fig.4.5 Variation of Base Shear for 20% Infill 18 Floor 6 Spanned Frame

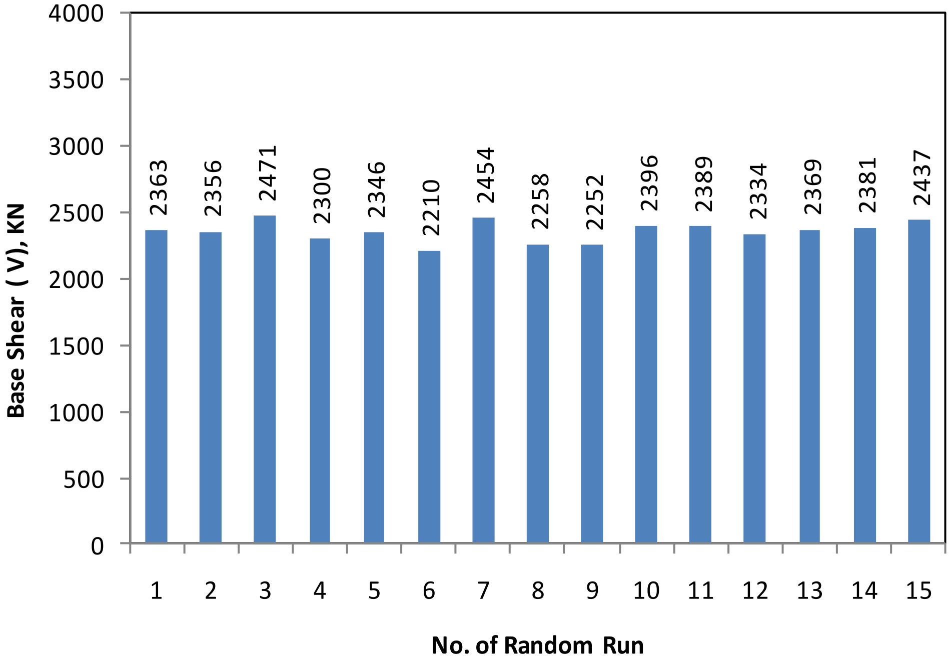


Fig.4.6 Variation of Base Shear for 40% Infill 18 Floor 6 Spanned Frame

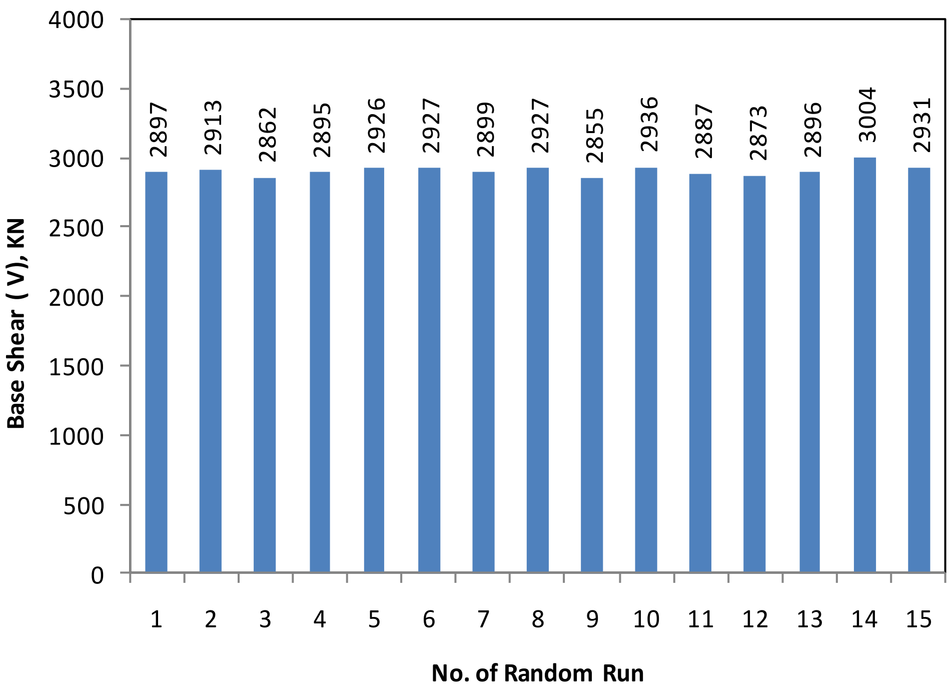


Fig.4.7 Variation of Base Shear for 60% Infill 18 Floor 6 Spanned Frame

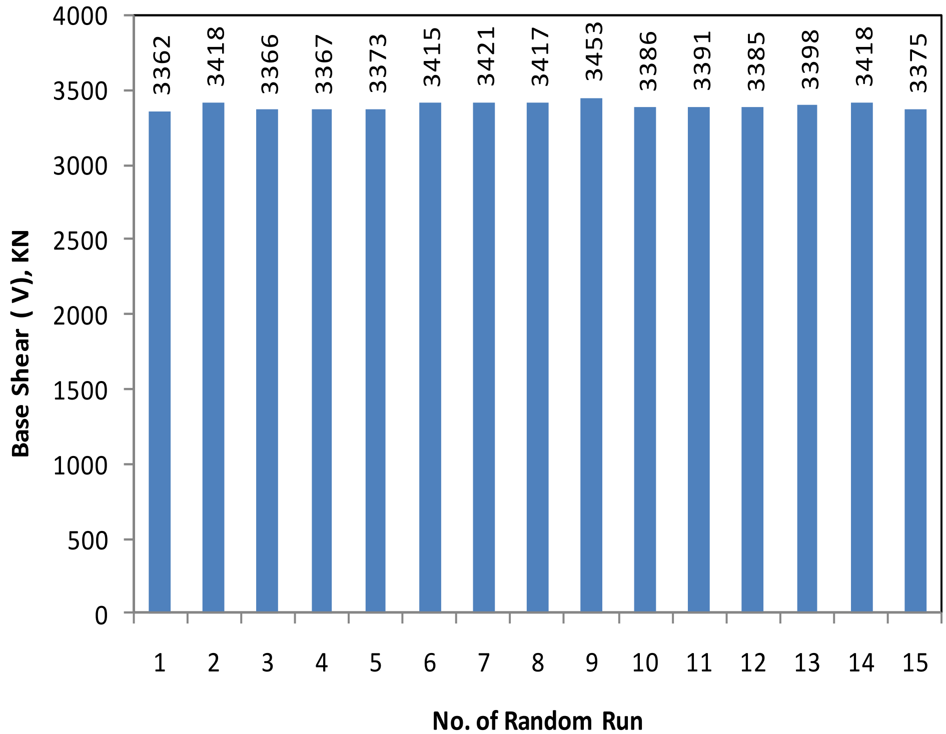


Fig.4.8 Variation of Base Shear for 80% Infill 18 Floor 6 Spanned Frame

From the results obtained (shown in table 4.2 and table 4.3 and fig.4.1 to fig. 4.8) we find that value of base shear does not change significantly due to the change of infill location for a certain percentage of infill. When the infill percentage is increased i.e 60% or 80%, then the variation of base shear is insignificant. Hence an average value of base shear has taken from the values of 15 numbers of software run.

4.3.2 Effect of Variation of Infill Percentage on Story Sway

In the present study, lateral force (earthquake force) is considered which has dominant effect. Story sway due to earthquake is compared here.

First earthquake load is applied in one direction. Sway is plotted for the frame for the load cases. Two load cases are considered for earthquakes i.e. load case-3 for static load (ESFM) and load case-4 for response spectrum (RSM). Sway for Equivalent Static Force Method (ESFM) and Response Spectrum Method (RSM) is shown in fig.4.9 to 4.18

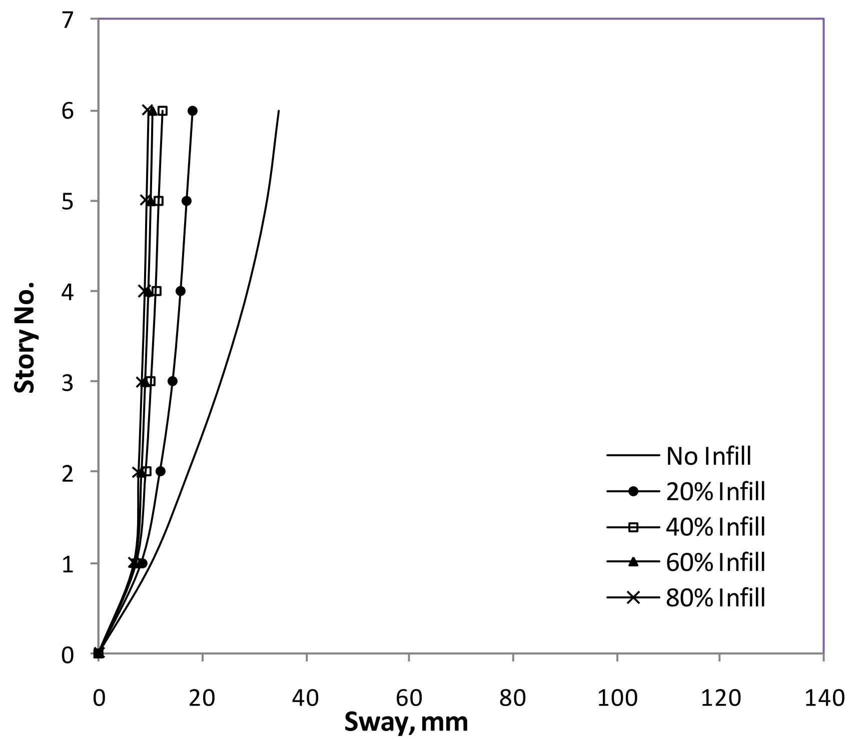


Fig.4.9 Story Sway for Diff. Infill percent of 6 storied and 4 spanned Frame by ESFM

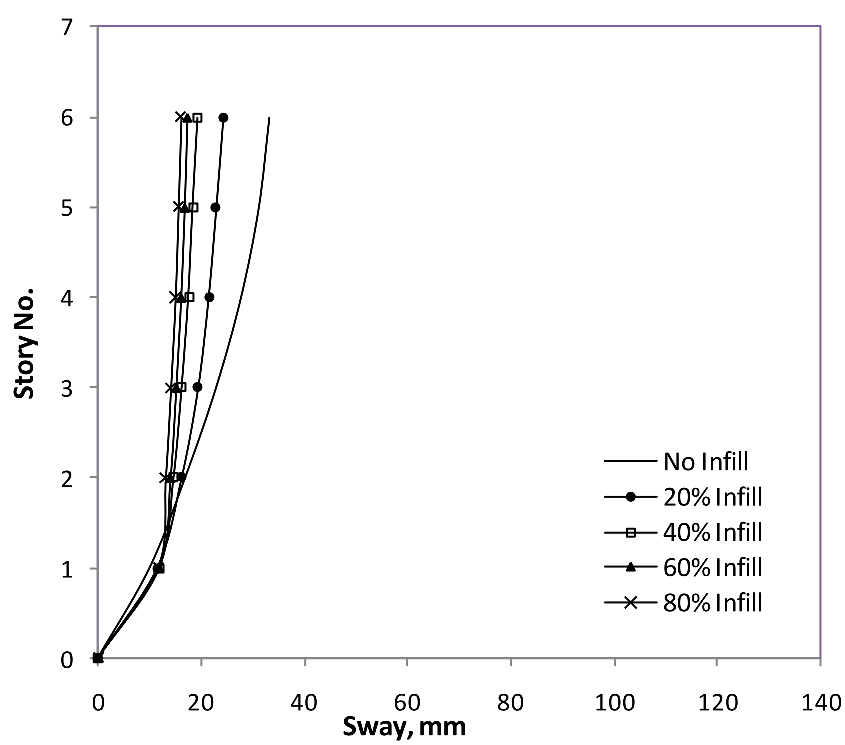


Fig.4.10 Story Sway for Diff. Infill percent of 6 storied and 4 spanned Frame by RSM

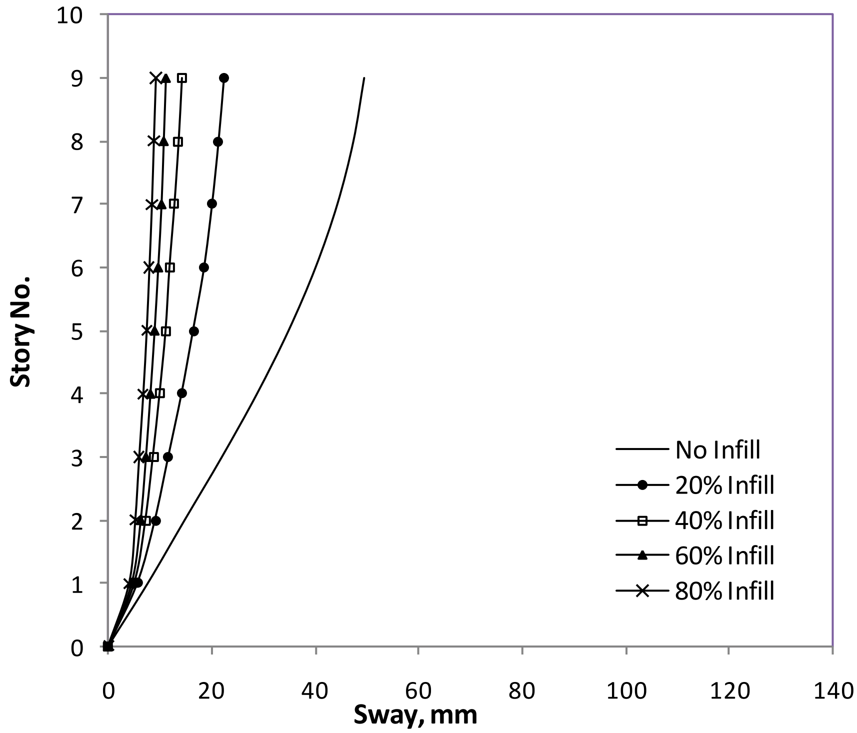


Fig.4.11 Story Sway for Diff. Infill percent of 9 storied and 4 spanned Frame by ESFM

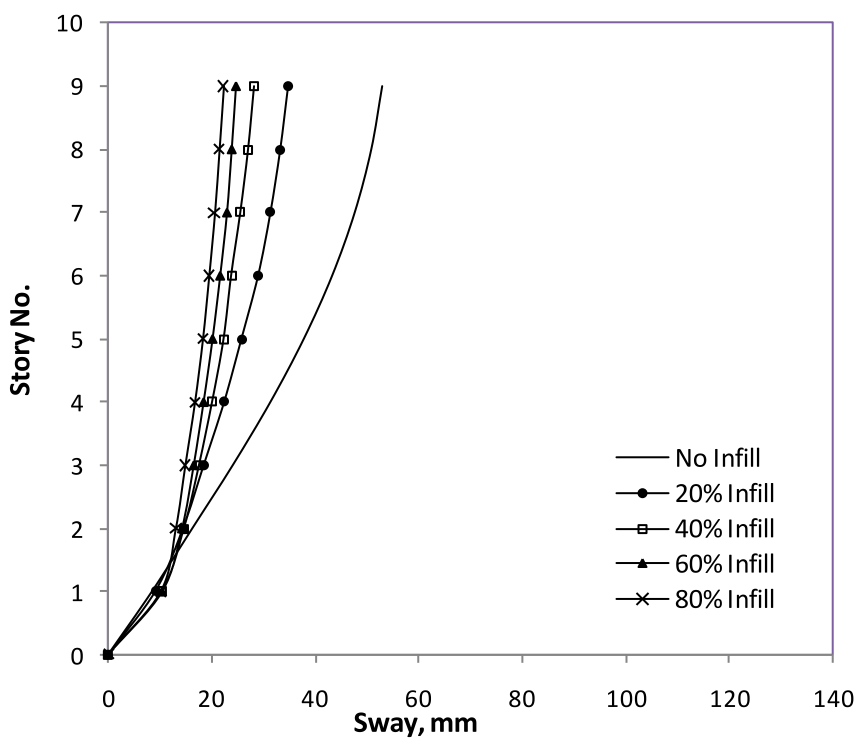


Fig.4.12 Story Sway for Diff. Infill percent of 9 storied and 4 spanned Frame by RSM

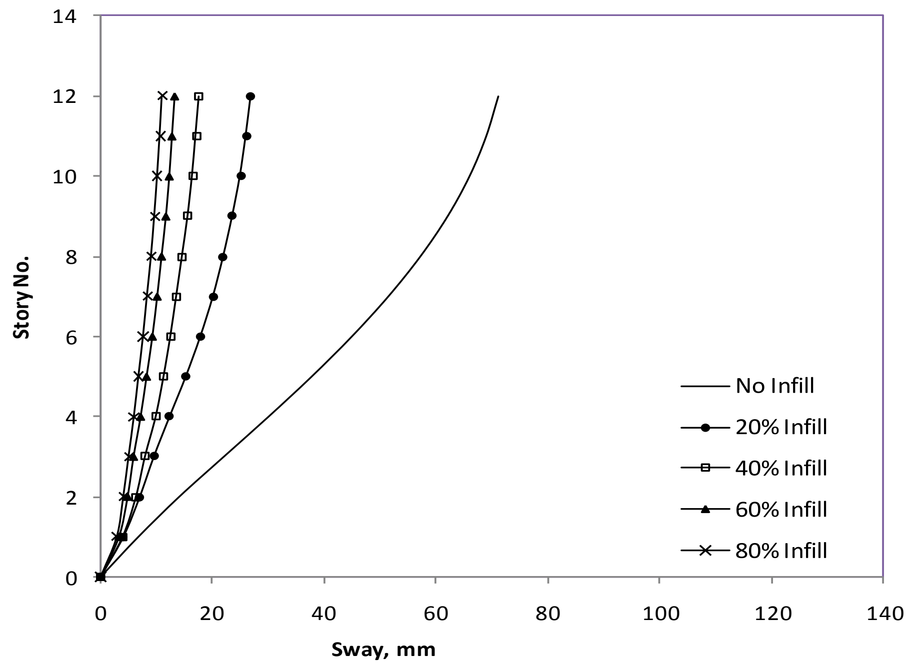


Fig.4.13 Story Sway for Diff. % Infill of 12 storied and 4 spanned Frame by ESFM

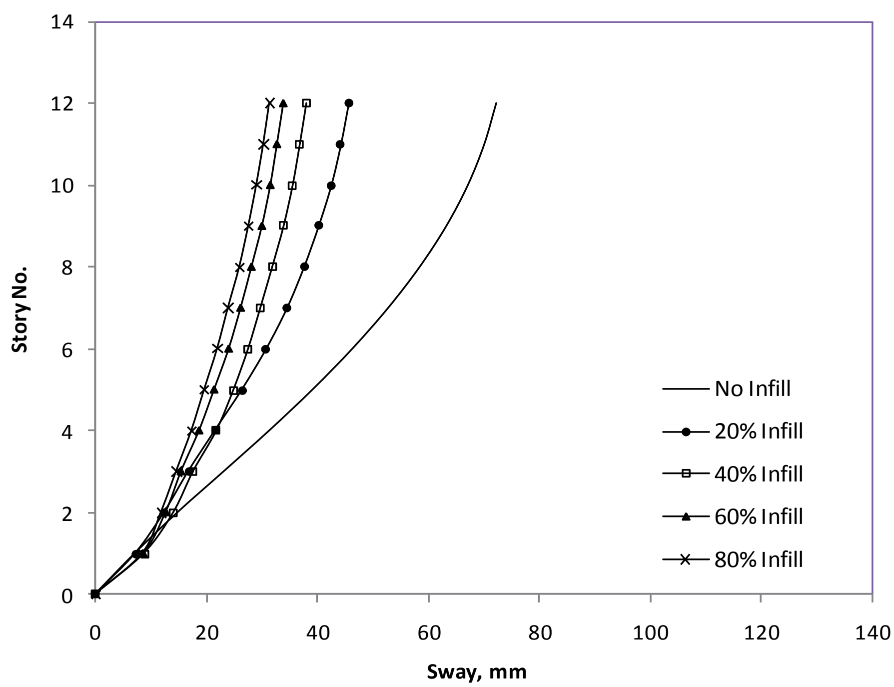


Fig.4.14 Story Sway for Diff. % Infill of 12 storied and 4 spanned Frame by RSM

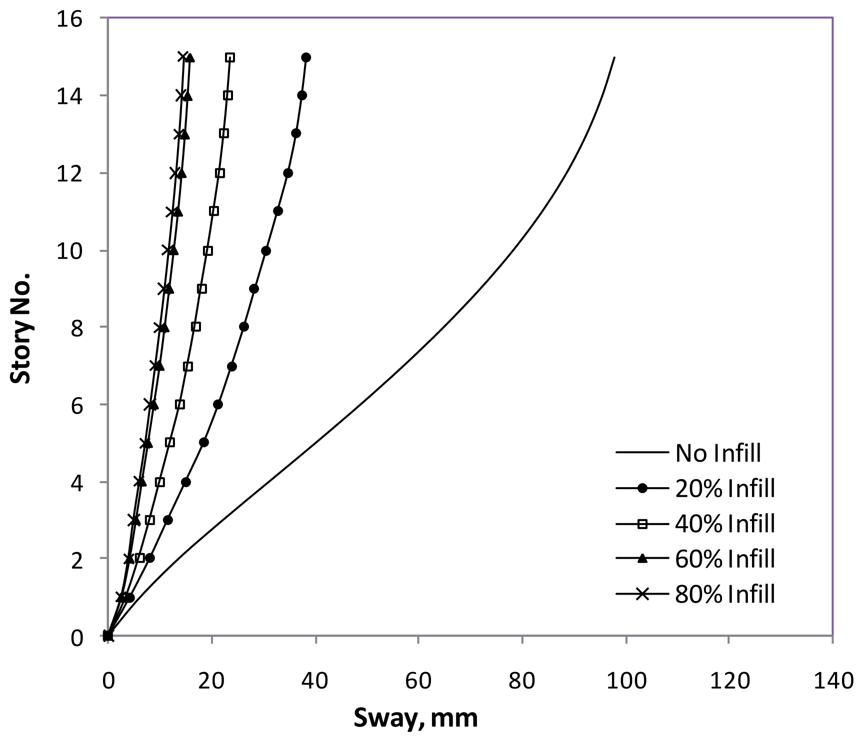


Fig.4.15 Story Sway for Diff. Infill percent of 15 storied and 4 spanned Frame by ESFM

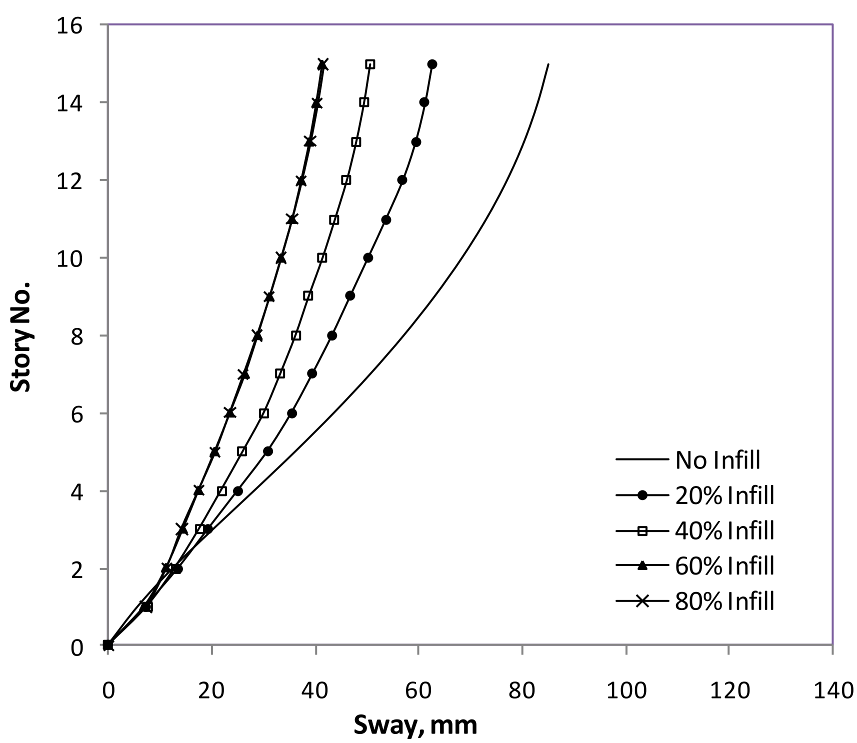


Fig.4.16 Story Sway for Diff. Infill percent of 15 storied and 4 spanned Frame by RSM

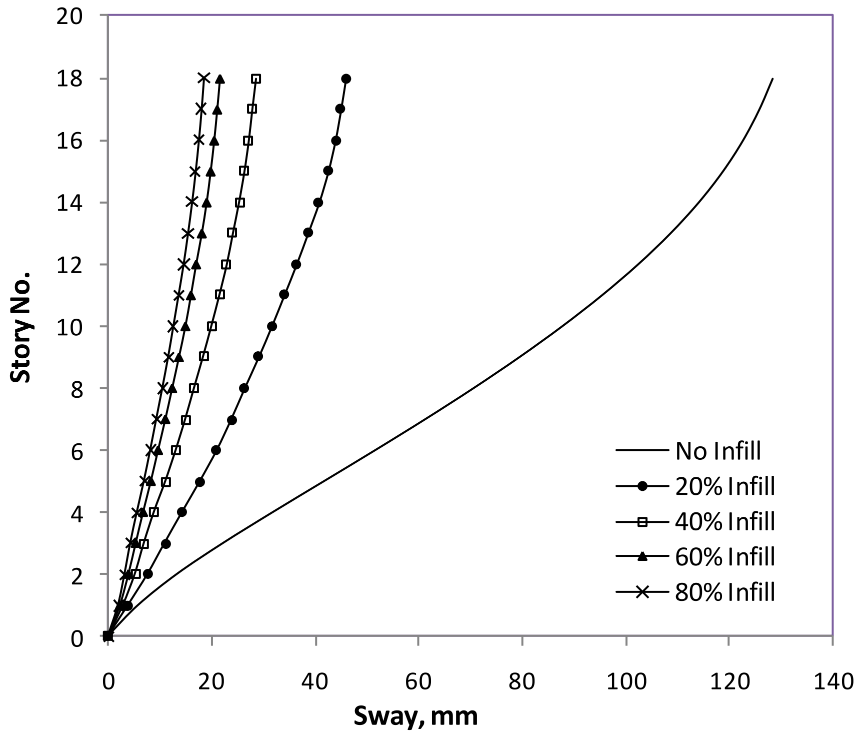


Fig.4.17 Story Sway for Diff. Infill percent of 18 storied and 4 spanned Frame by ESFM

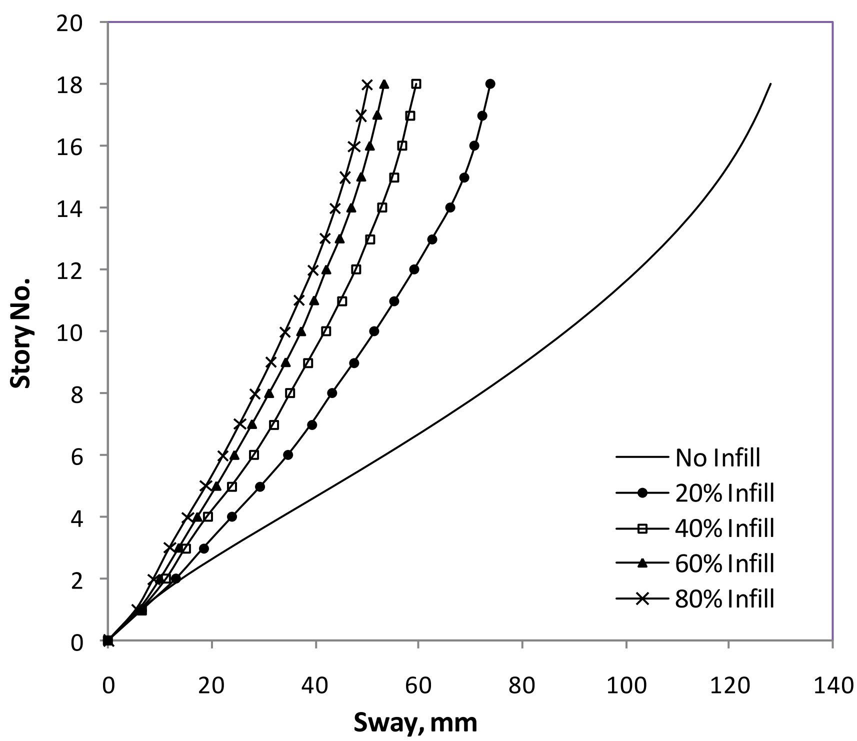


Fig.4.18 Story Sway for Diff. Infill percent of 18 storied and 4 spanned Frame by RSM

Findings from the graphs are stated below:

- Sway increases suddenly at first floor in RSM method while it is not significant in ESFM. The abrupt changes in the slope of the profile are due to the stiffness irregularity between the ground floor and upper floors

- Sway remains almost same both for ESM and RSM method for bare infilled frame for a particular height irrespective of number of span. This means that sway is not influenced by number of span or analytical method (i.e. ESFM or RSM) if the frame does not contain any infill.

- The infill act as equivalent diagonal strut which is responsible to increases the story stiffness. Both for ESFM and RSM lateral sway is the highest for frame with 0% infill and it reduces gradually with the increase of infill due to increased stiffness of the story for the presence of infill.

- Sway profiles for both ESFM and RSM have a sudden change of slope at first floor level. The inter-story drift demand is largest in the ground story for all the models for both ESFM and RSM.

- Lateral sway is almost same for soft story irrespective of percentage of structurally active infill in the upper stories. This scenario is same for ESFM and RSM method.

- Sway value in RSM method is always higher than the value found from ESFM method. For 6 storied 2 spanned frames with 60% infill, ESFM shows 7mm sway for soft ground floor while it is 14mm for RSM. For 9 storied frame these values are 6mm and 12mm respectively, for 12 storied frame it is also 8 mm and 10 mm respectively, for 15 storied frame it is 3mm and 7mm respectively and for 18storied frame it is 2mm and 5mm respectively. So it is found that sway determined by dynamic method is always higher than the value found from static analysis due to the consideration of pendulum effect.

- Variation of sway between top floor and ground floor due to various percentage of infill (40% and more) is not significant for 6 and 9 storied frame. But for 12 storied or more, the variation is significant, top floor deviated significantly than ground floor. The variation is prominent in RSM method.

4.3.3 Effect of Span Length & Infill Percentage on Story Drift

Story drift for ESFM and RSM method are determined for height of 6, 9, 12, 15 and 18 floor with varying number of span starting from 2 to 10 with an incremental value of 2. Then the graph are plotted as story number versus drift (mm) for ESFM and RSM method as shown below from Fig. 4.19 to 4.34.

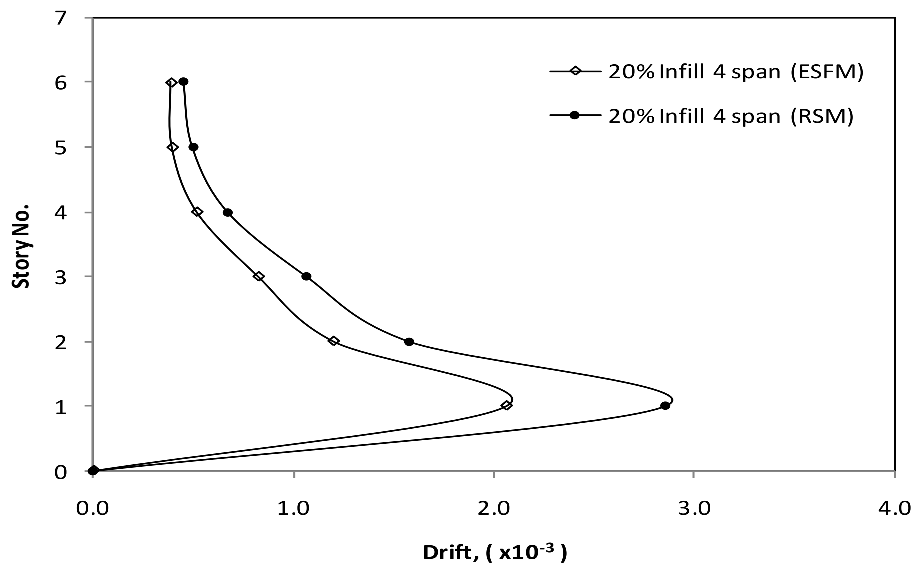


Fig. 4.19 Comparison of Drift ESFM vs RSM for 20% Infil 6 Floor and 4 spanned Frame

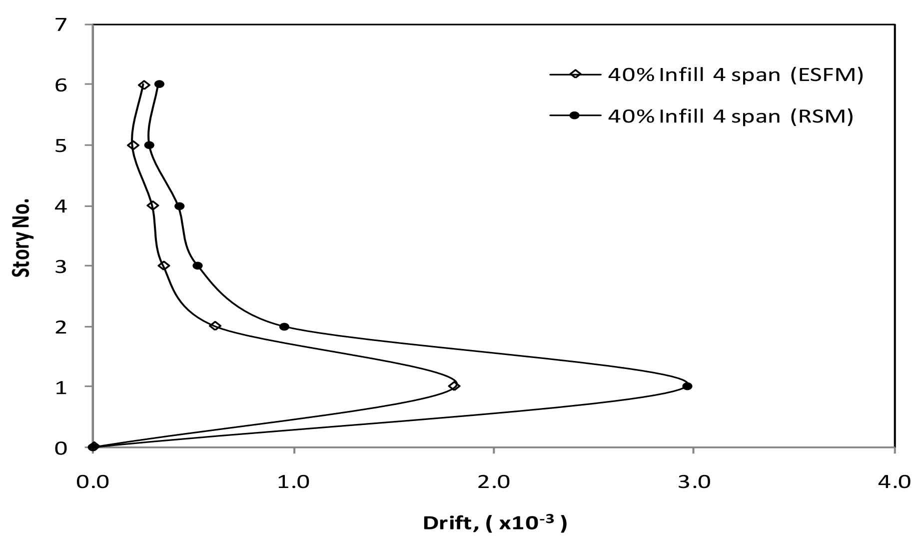


Fig.4.20 Comparison of Drift ESFM vs RSM for 40% Infill 6 Floor and 4 spanned Frame

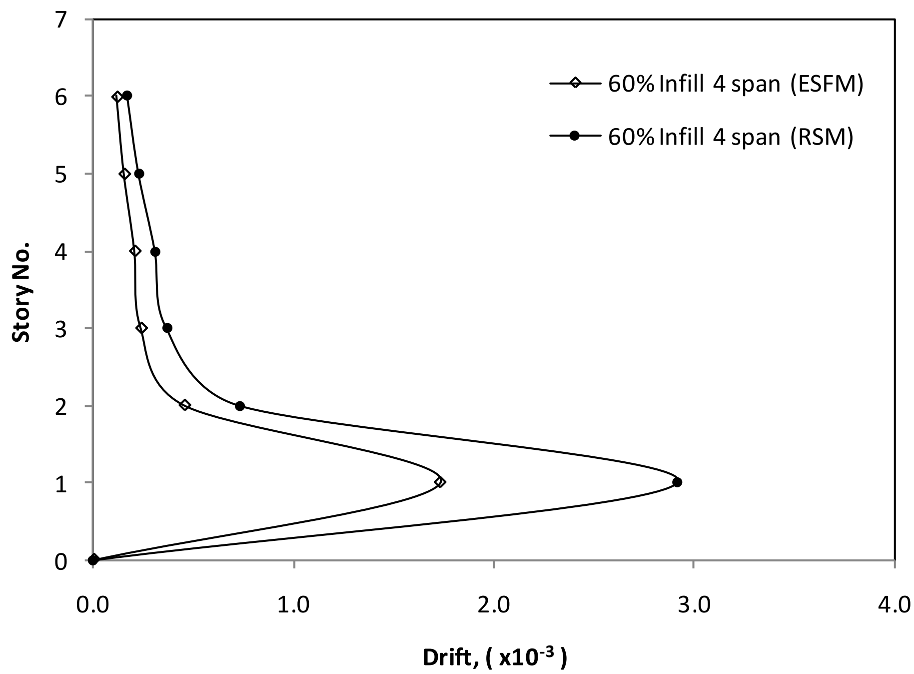


Fig.4.21 Comparison of Drift ESFM vs RSM for 60% Infill 6 Floor and 4 spanned Frame

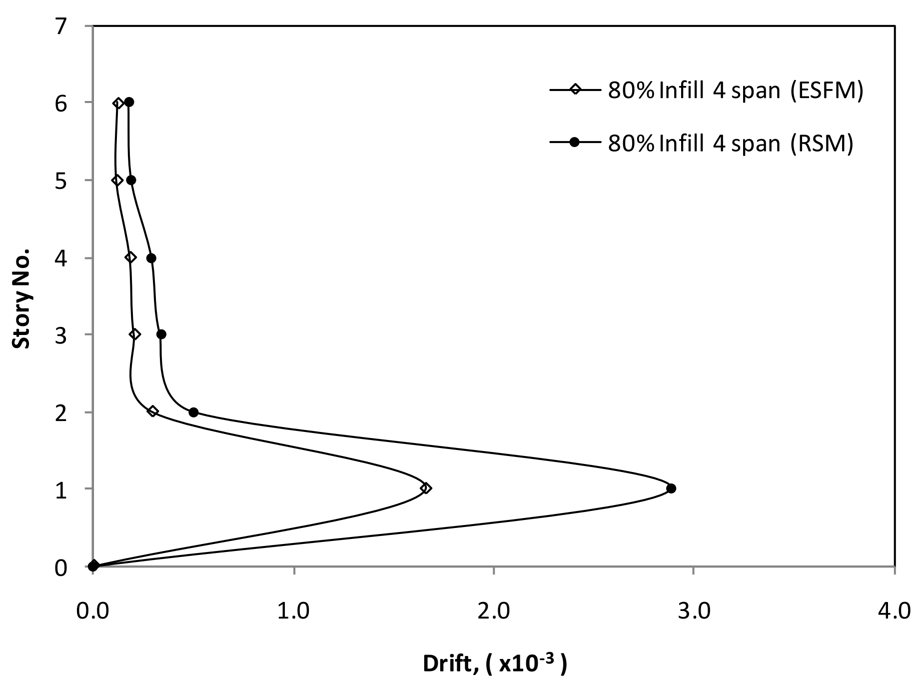


Fig.4.22 Comparison of Drift ESFM vs RSM for 80% Infill 6 Floor and 4 spanned Frame

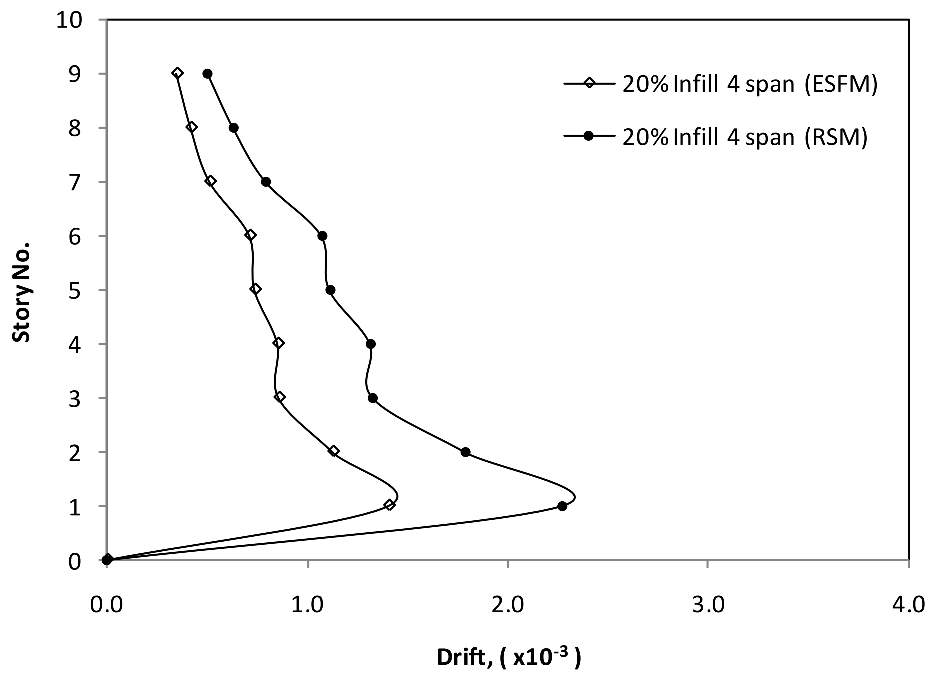


Fig.4.23 Comparison of Drift ESFM vs RSM for 20% Infill 9 Floor and 4 spanned Frame

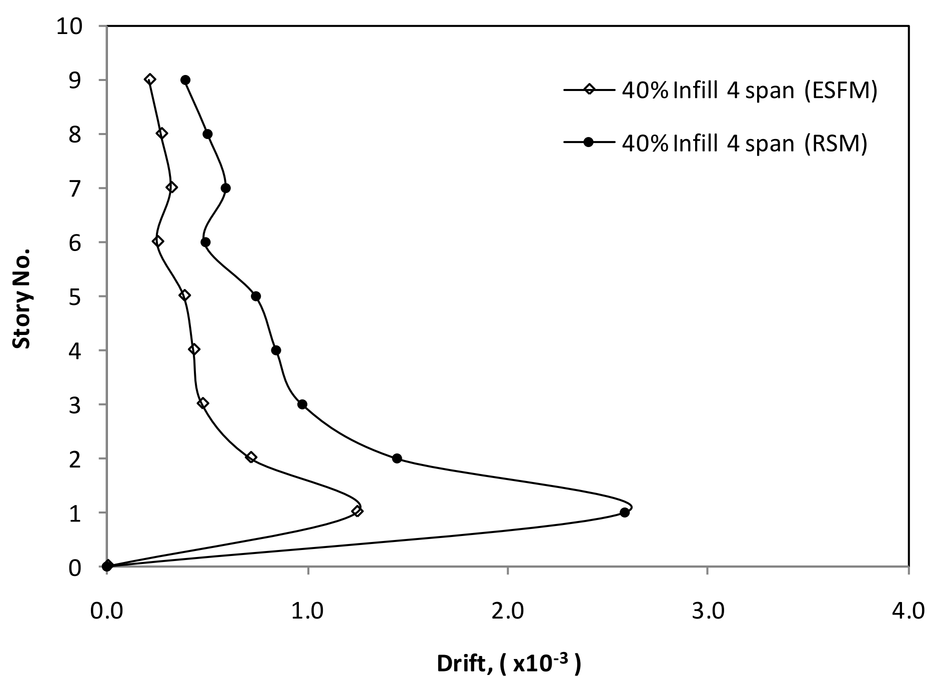


Fig.4.24 Comparison of Drift ESFM vs RSM for 40% Infill 9 Floor and 4 spanned Frame

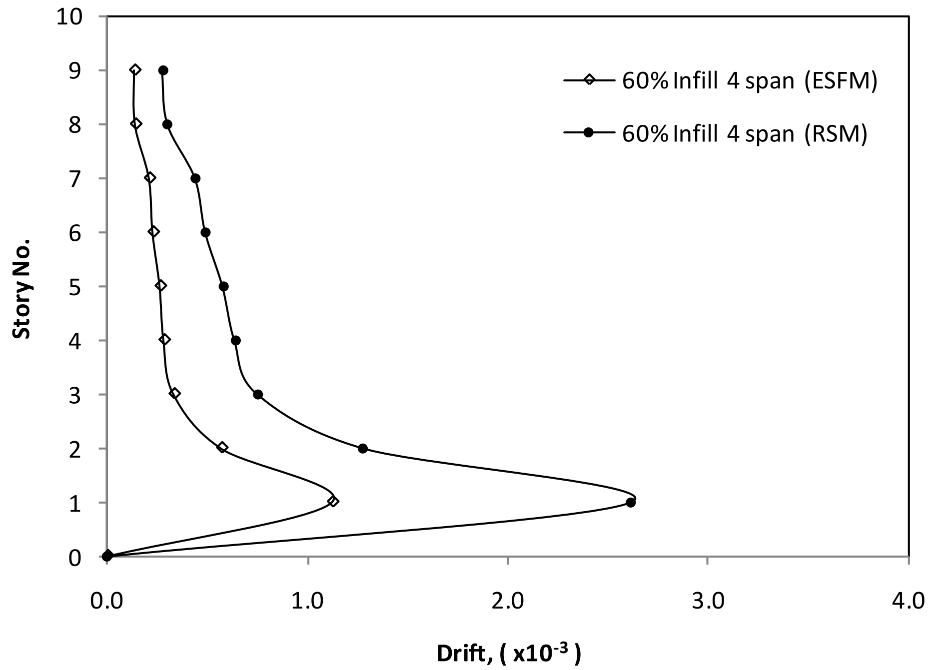


Fig.4.25 Comparison of Drift ESFM vs RSM for 60% Infill 9 Floor and 4 spanned Frame

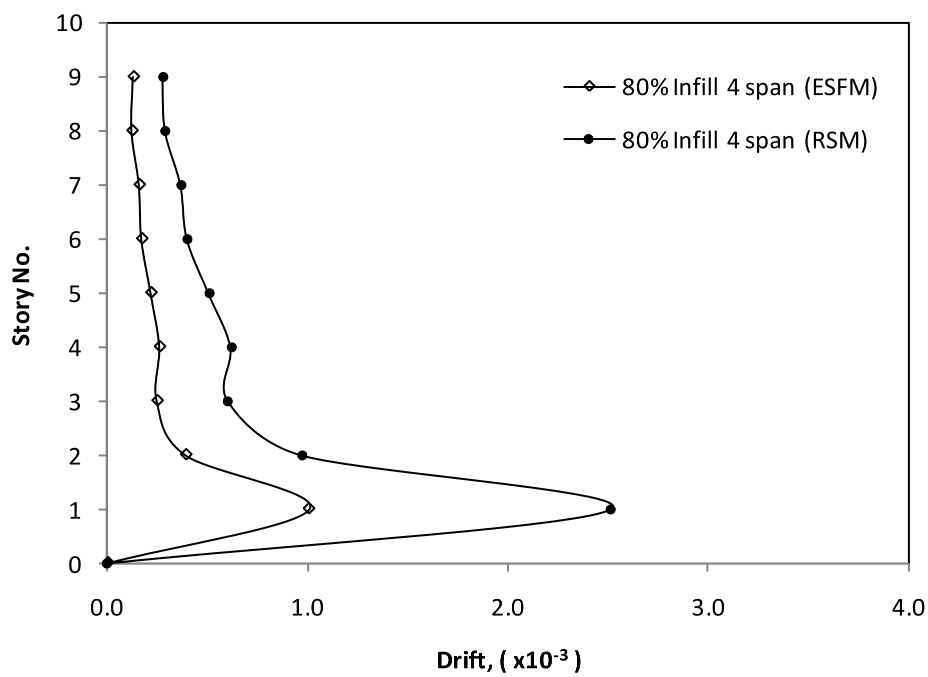


Fig.4.26 Comparison of Drift ESFM vs RSM for 80% Infill 9 Floor and 4 spanned Frame

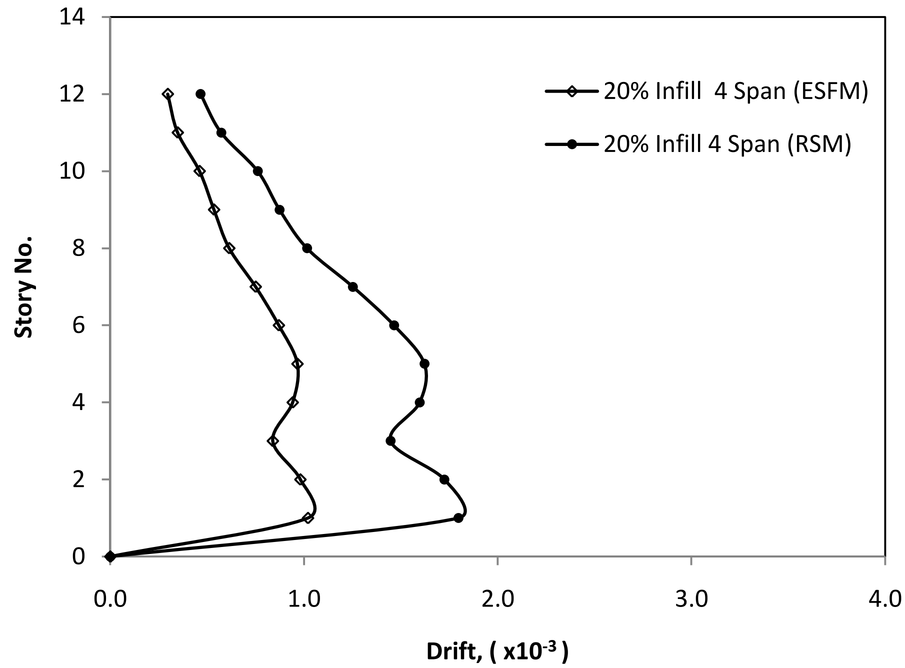


Fig.4.27 Comparison of Drift ESFM vs RSM for 20% Infill 12 Floor and 4 spanned Frame

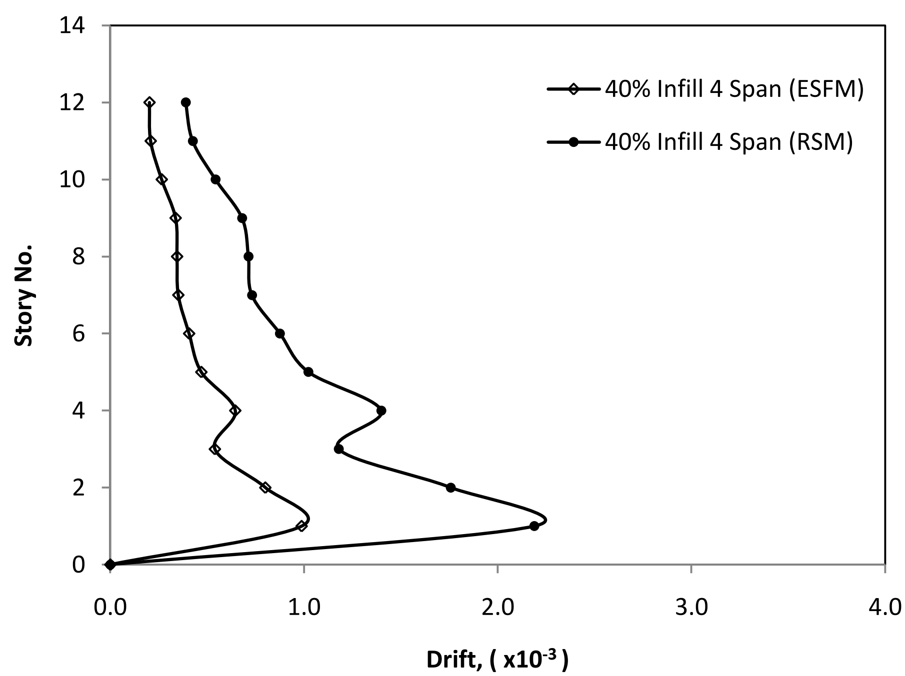


Fig.4.28 Comparison of Drift ESFM vs RSM for 40% Infill 12 Floor and 4 spanned Frame

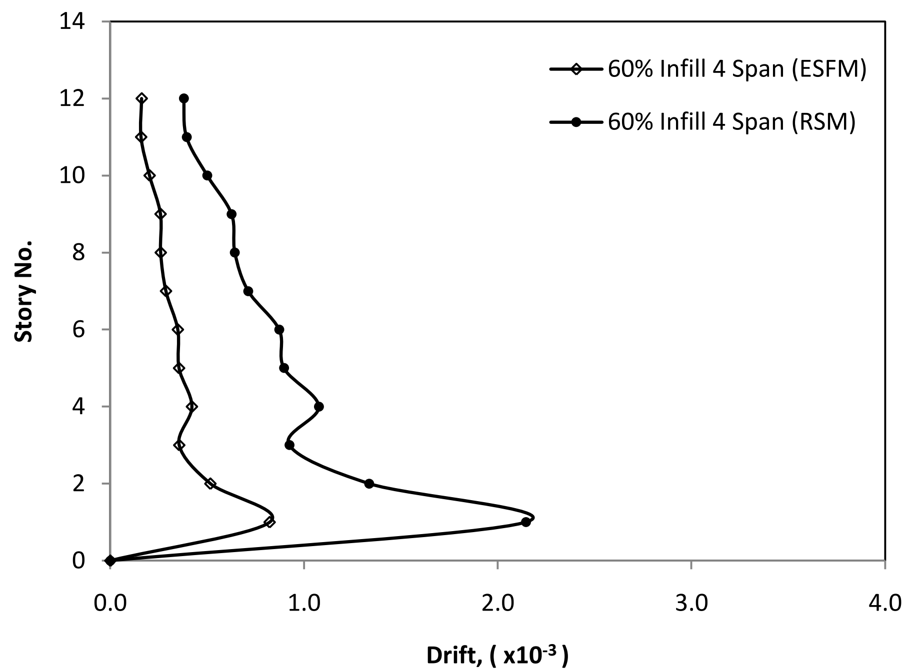


Fig.4.29 Comparison of Drift ESFM vs RSM for 60% Infil 12 Floor and 4 spanned Frame

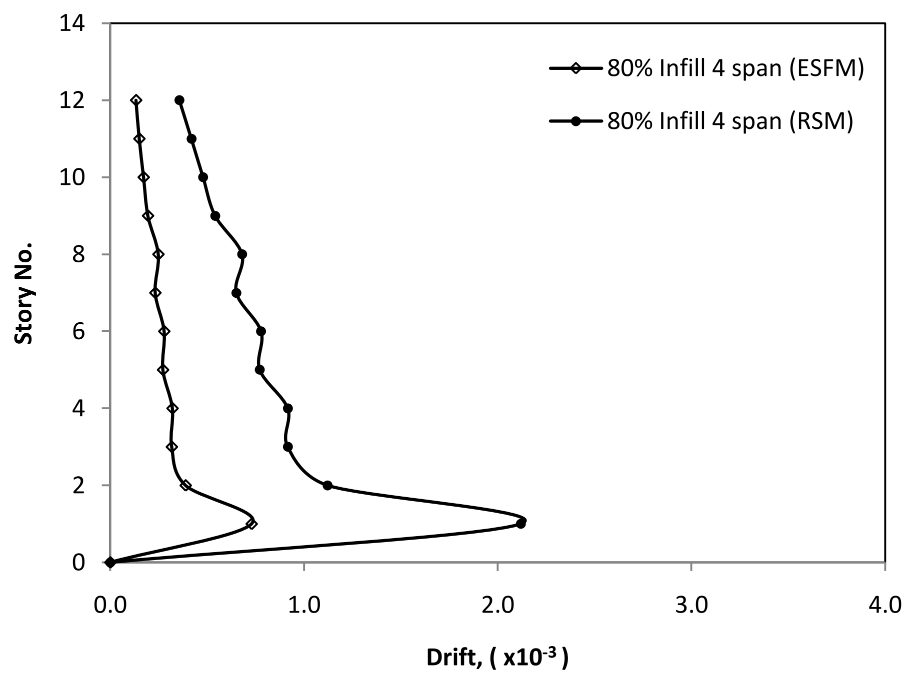


Fig.4.30 Comparison of Drift ESFM vs RSM for 80% Infil 12 Floor and 4 spanned Frame

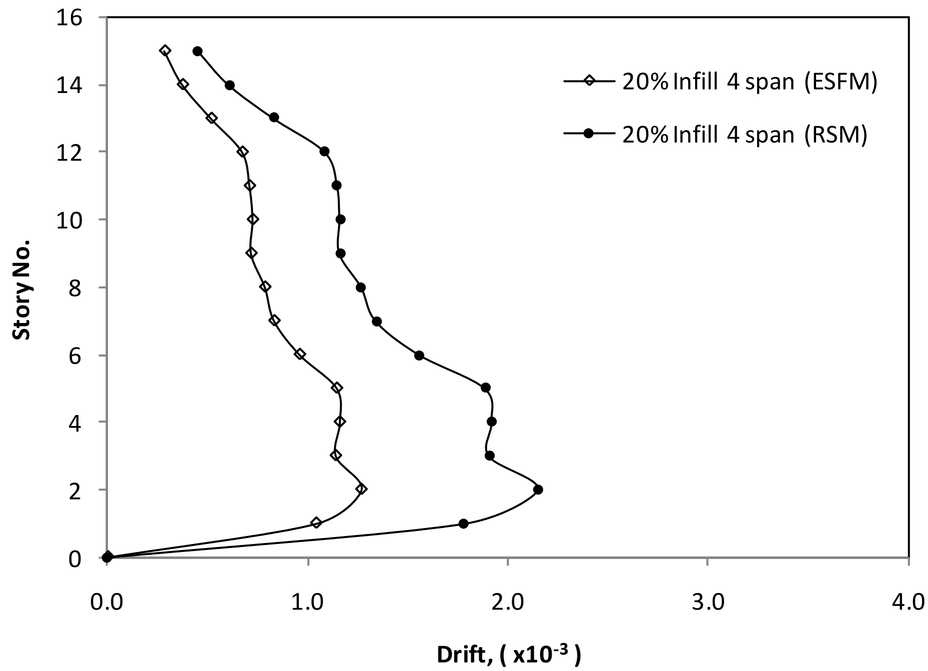


Fig.4.31 Comparison of Drift ESFM vs RSM for 20% Infil 15 Floor and 4 spanned Frame

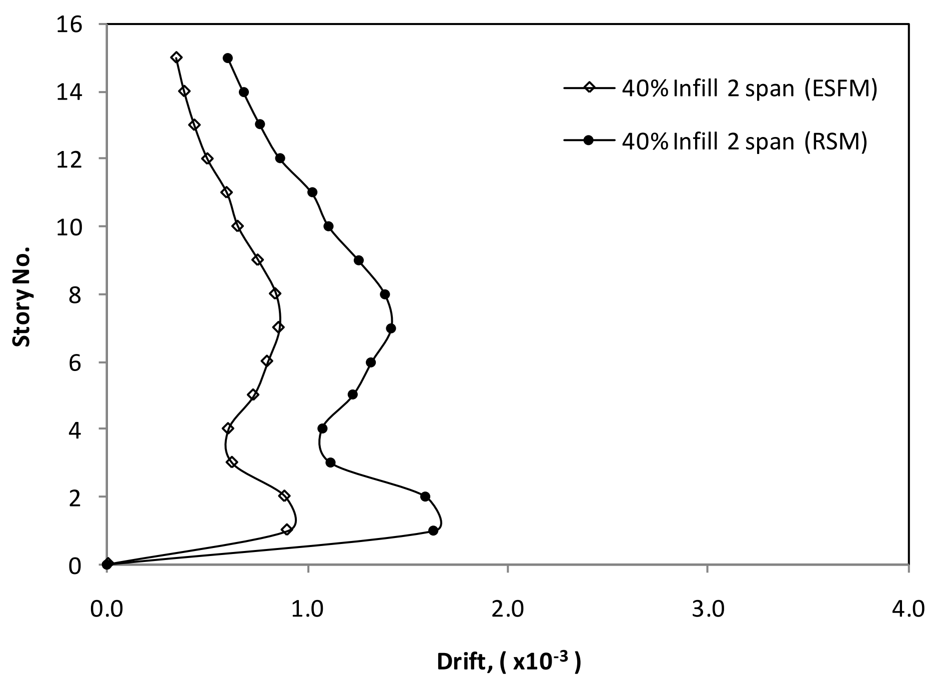


Fig.4.32 Comparison of Drift ESFM vs RSM for 40% Infil 15 Floor and 4 spanned Frame

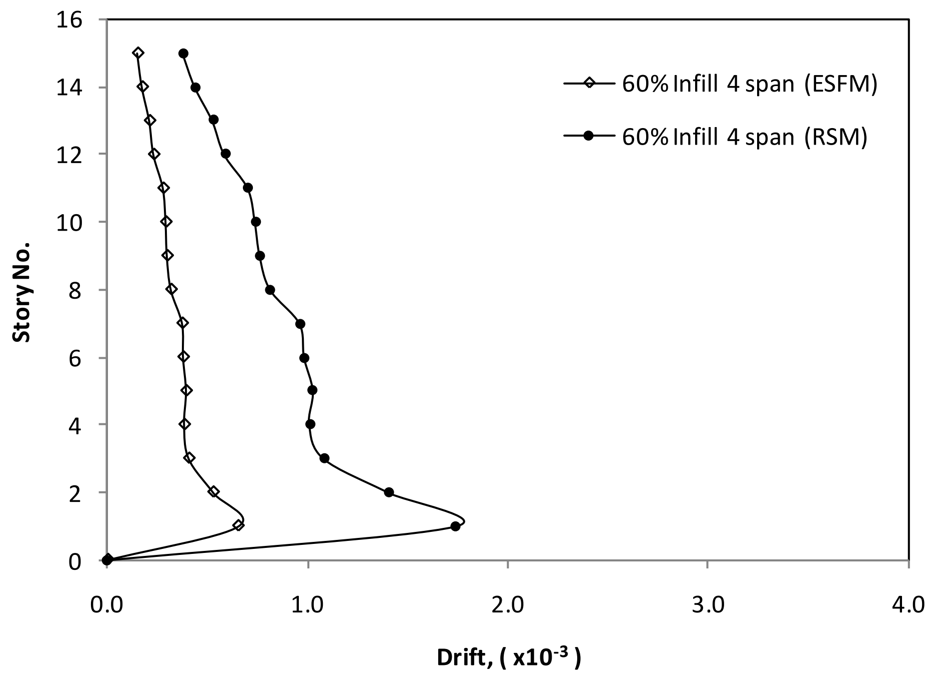


Fig.4.33 Comparison of Drift ESFM vs RSM for 60% Infil 15 Floor and 4 spanned Frame

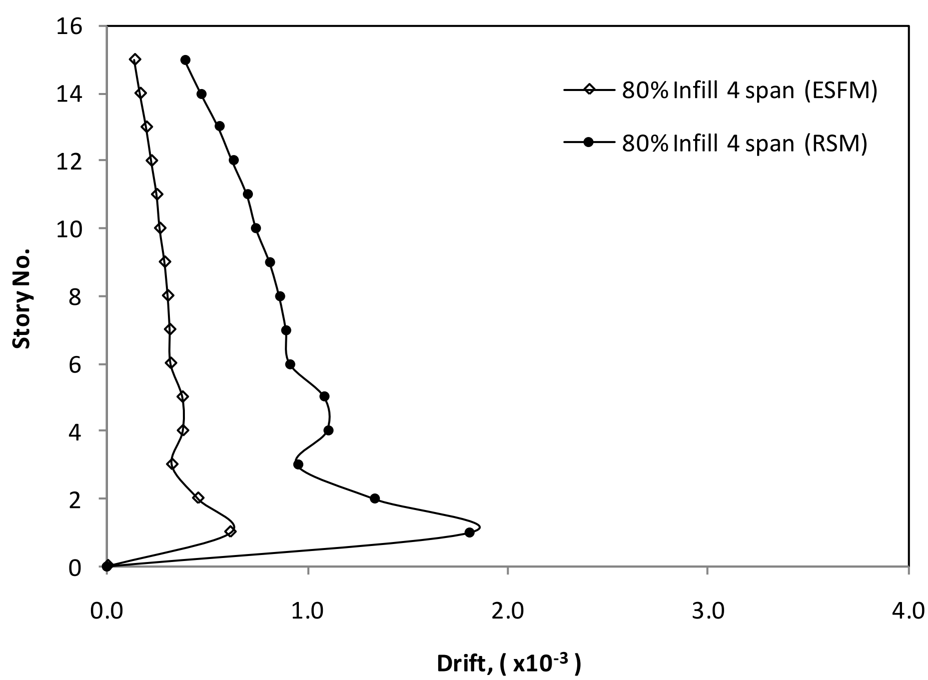


Fig.4.34 Comparison of Drift ESFM vs RSM for 80% Infil 15 Floor and 4 spanned Frame

From the above figures from 4.19 to 4.34, it has been seen that drift value is higher in the soft ground floor. Since infills are located on the upper stories keeping the ground floor open, hence load concentration occur at ground floor. Thus ground floor becomes weaker than other floors and deflection occurs severely.

The findings from the graphs are mentioned below:

For 6 storied Frame

- For a 6 storied frame, drift is higher in the soft ground floor with comparison to other floor for all spanned (i.e 2, 4, 6, 8, 10) and infill percentage.
- Drift value is highest for 2 spanned 40% infilled frame. For ESFM this value is 2.125 and for RSM is 3.35.Since the frame is 2 spanned hence it experiences as slender frame and thus deflection is higher for this 2 spanned slender frame.

For 9 storied Frame

- For all percentage of infill, drift is higher for 2 spanned frames both for ESFM and RSM method. This is due to slenderness of the frame.
- Highest value is obtained from ground floor. The value is 1.55 found for 40% infilled frame for ESFM method. For RSM method it is 2.85.

For 12 storied Frame

- For 20% infill and 2 spanned frames, Drift is higher in 5th floor for both ESFM and RSM method and the values are 1.6 and 2.35 respectively. And these are the highest value.
- For other percentage of infill of different spanned except 2 spanned frames, drift is higher in the ground floor level.

For 15 storied Frame

- For 20% infill and 2 spanned frames, Drift is higher in 5th floor for both ESFM and RSM method and the values are 1.55 and 2.2 respectively. This is similar to 12 storied frames. For other spanned from 4 to 10, drift is higher in 2nd floor.
- For other percentage of infill with different span, drift is higher in ground floor.

For 18 storied Frame

- For 20% and 40% infill and 2 spanned frames, Drift is higher in 6^{th} floor for both ESFM and RSM method and the values are 1.55 and 2.2 respectively. For other spanned from 4 to 10, drift is higher in 2^{nd} floor.
- For other percentage of infill with different spanned frame, Drift is higher in soft ground floor.

Based on the above findings, highest drift values are summarized below in table 4.4

Table 4.4 Maximum drift values for different floor level

Sl. No	Frame Height	Floor level corresponding to highest value	Number of span corresponding to highest value	Percentage of Infill corresponding to highest value	Highest Value (ESFM)	Highest Value (RSM)
1	6 Storied	Ground Floor	2	40%	2.125	3.35
2	9 Storied	Ground Floor	2	40%	1.55	2.85
3	12 Storied	5th Floor	2	20%	1.6	2.35
4	15 Storied	5th Floor	2	20%	1.55	2.2
5	18 Storied	6th Floor	2	20% & 40%	1.55	2.2

4.3.4 Effect of Slenderness (H/L) on Base shear

Slenderness of frame i.e. value of height by span length (H/L) has an effect on the base shear found by ESFM and RSM. Base shear for ESFM and RSM method are determined for certain height of 6, 9,12, 15 and 18 floor with varying number of span from 2 to 10 with a incremental value of 2. Then the graph are plotted as Base shear versus H/L as shown below from fig. 4.35 to 4.39

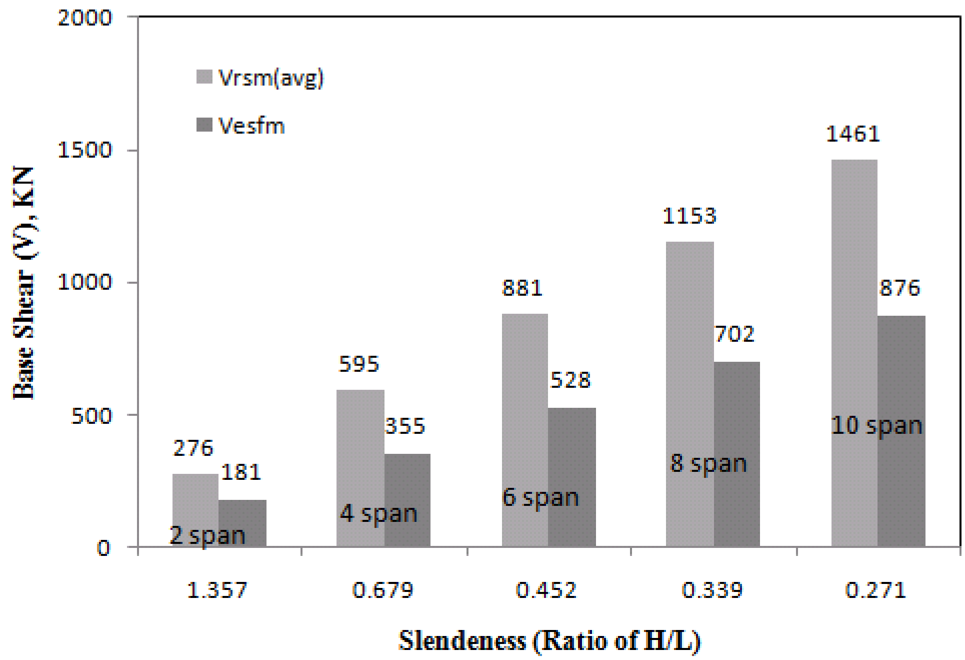


Fig. 4.35 Comparison of Base Shear with Slenderness for 40% Infill 6 Floor

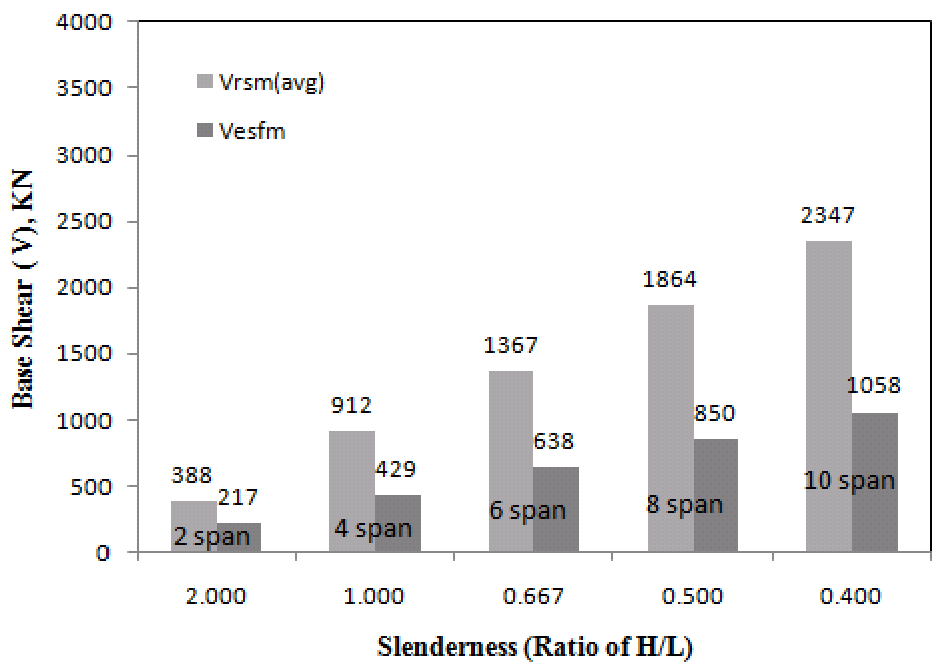


Fig. 4.36 Comparison of Base Shear with Slenderness for 40% Infill 9 Floor

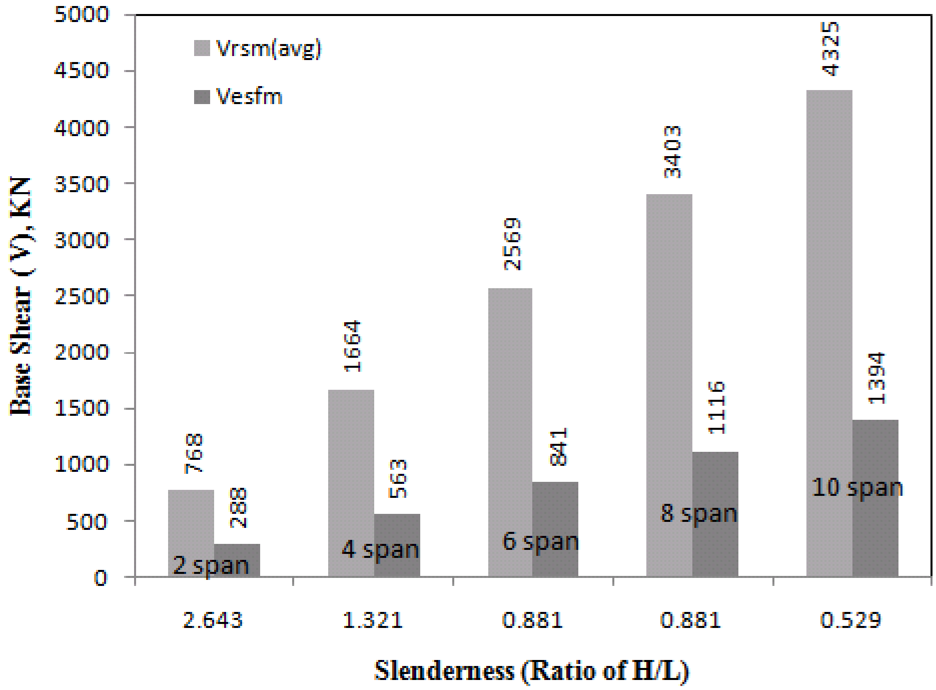


Fig. 4.37 Comparison of Base Shear with Slenderness for 40% Infill 12 Floor

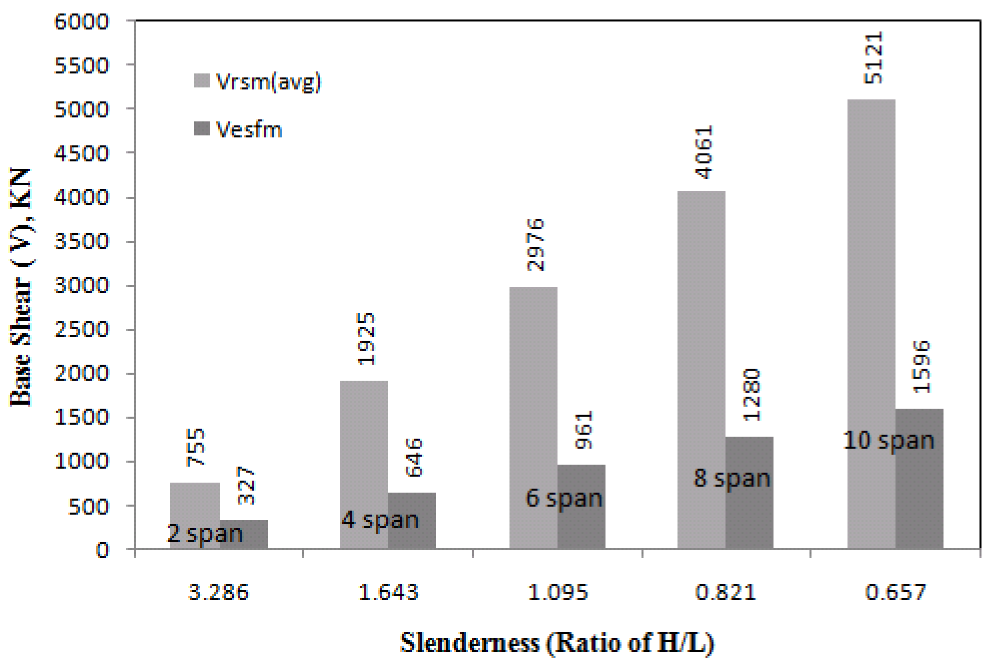


Fig. 4.38 Comparison of Base Shear with Slenderness for 40% Infill 15 Floor

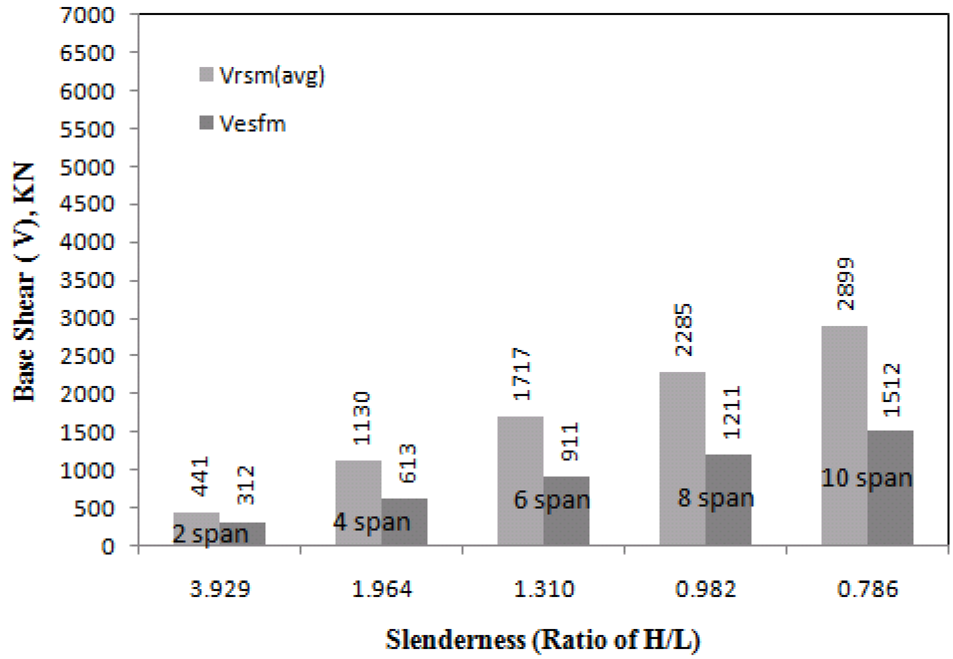


Fig.4.39 Comparison of Base Shear with Slenderness for 40% Infill 18 Floor

Findings from the graph are as below:

- Value of base shear in RSM method is always higher than the value found in ESFM method considering slenderness of frame, percentage of infill and number of spans.
- For a value of H/L 3.929 (i.e. slender frame), the ratio of $V_{avg(rsm)}$ / V_{esfm} is found 1.4 for 18 storied frame with 20% infill. Whereas the value is almost 2.0 for the same frame having 80% of infill. So, base shear in RSM is significant for higher percentage of infill. This is due to more load is applied for infill and thus increases base shear.

4.3.5 Base Shear Magnification vs. H/L

Slenderness of frame i.e. value of height by span length (H/L) has compared here to find the effect on the base shear by ESFM and RSM method. Base shear for ESFM and RSM method are determined for frame with floor height 6, 9, 12, 15 and 18 floors with varying number of span starting from 2 to 10 with an incremental value of 2. Then the graph are plotted as Base shear versus H/L as shown below from fig. 4.40 to 4.59

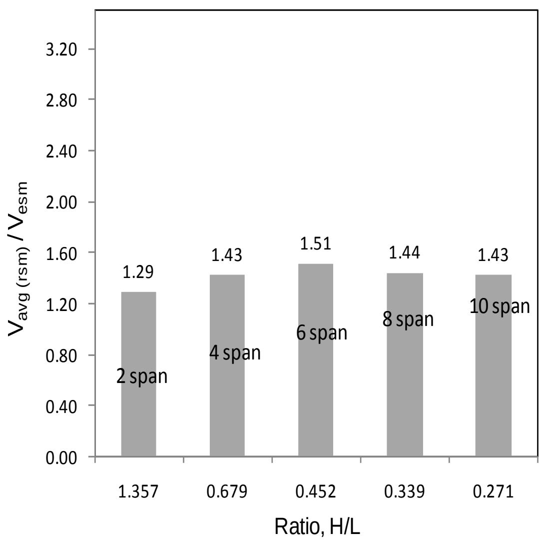


Fig.4.40 Base Shear Magnification vs H/L for 20% Infill 6 Floor

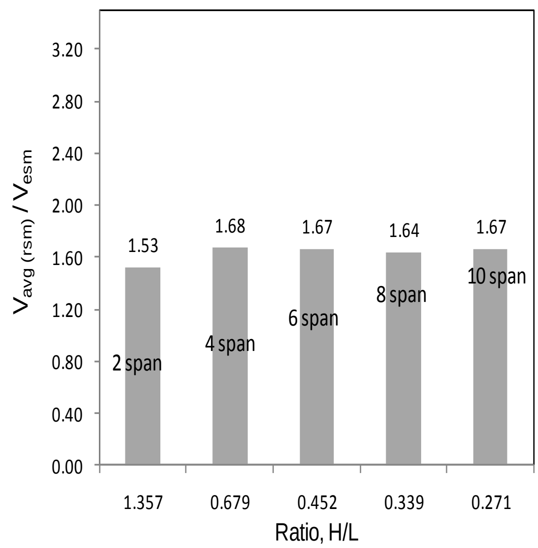


Fig.4.41 Base Shear Magnification vs H/L for 40% Infill 6 Floor

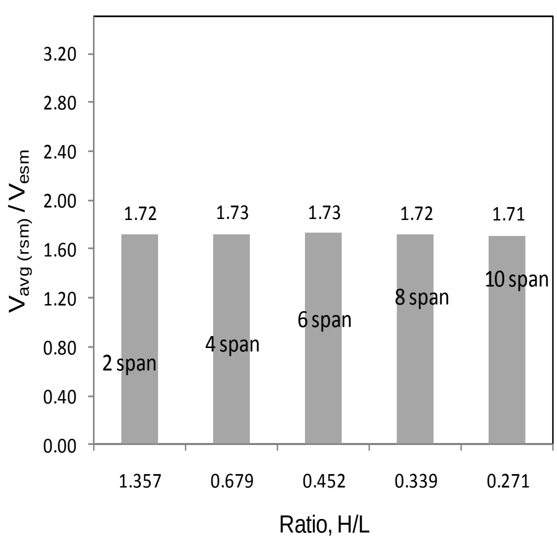


Fig.4.42 Base Shear Magnification vs H/L for 60% Infill 6 Floor

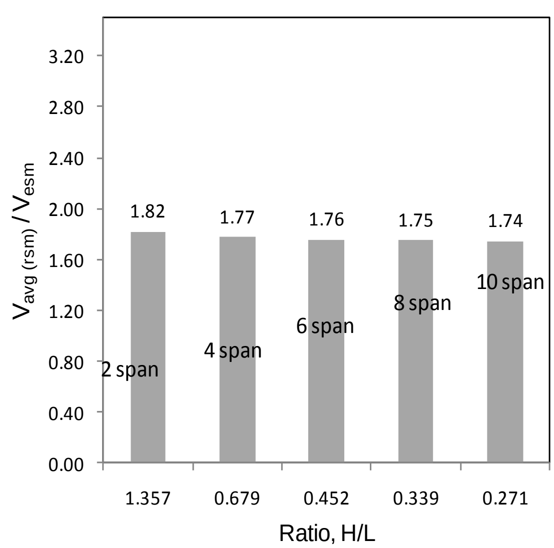


Fig.4.43 Base Shear Magnification vs H/L for 80% Infill 6 Floor

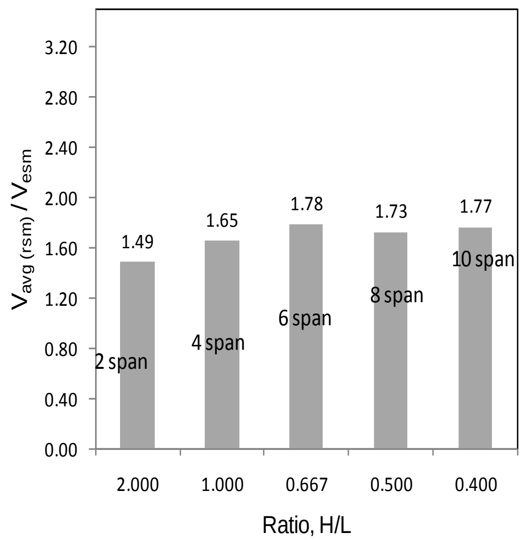


Fig.4.44 Base Shear Magnification vs H/L for 20% Infill 9 Floor

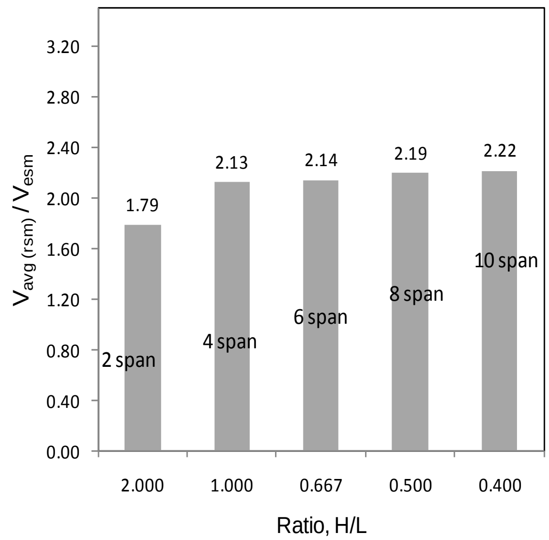


Fig.4.45 Base Shear Magnification vs H/L for 40% Infill 9 Floor

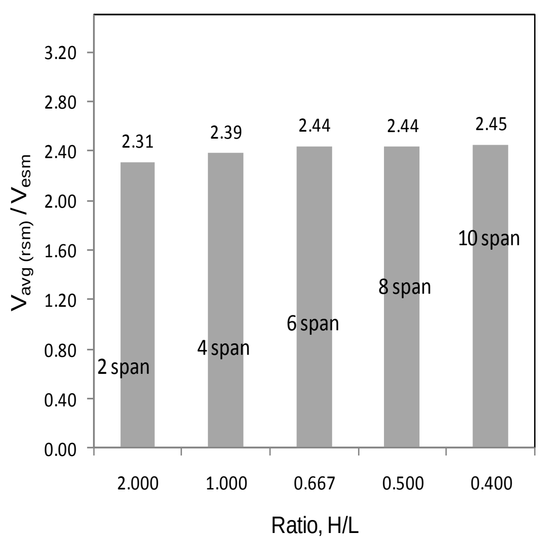


Fig.4.46 Base Shear Magnification vs H/L for 60% Infill 9 Floor

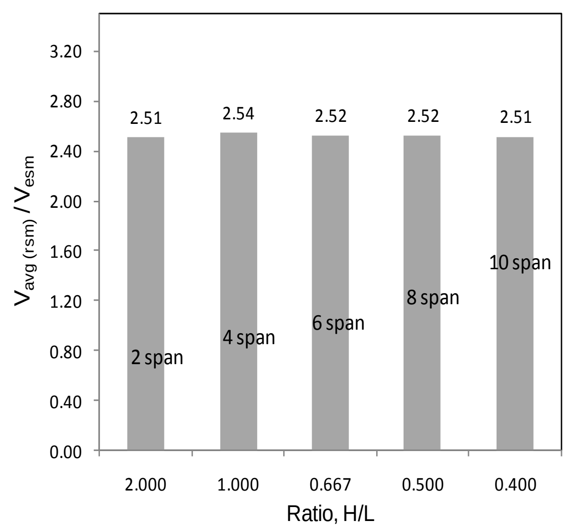


Fig.4.47 Base Shear Magnification vs H/L for 80% Infill 9 Floor

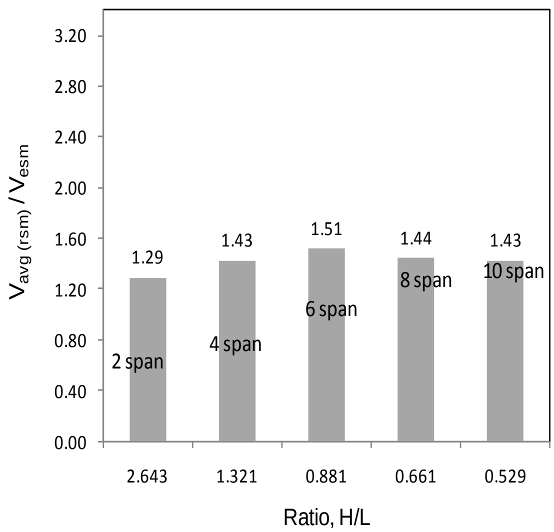


Fig.4.48 Base Shear Magnification vs H/L for 20% Infill 12 Floor

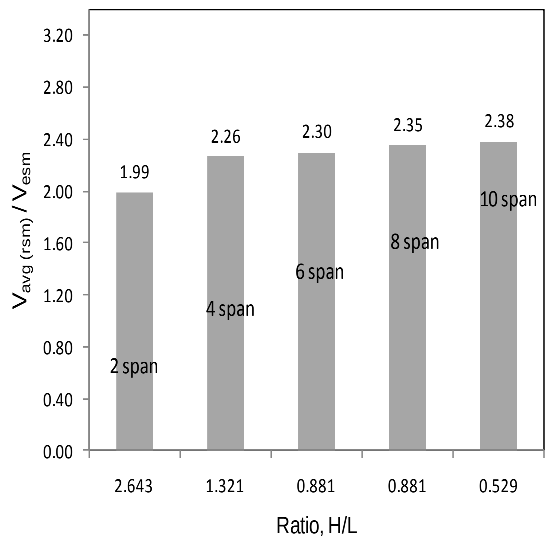


Fig.4.49 Base Shear Magnification vs H/L for 40% Infill 12 Floor

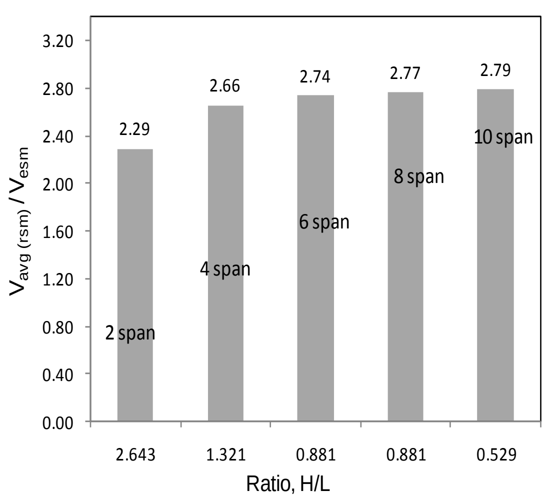


Fig.4.50 Base Shear Magnification vs H/L For 60% Infill 12 Floor

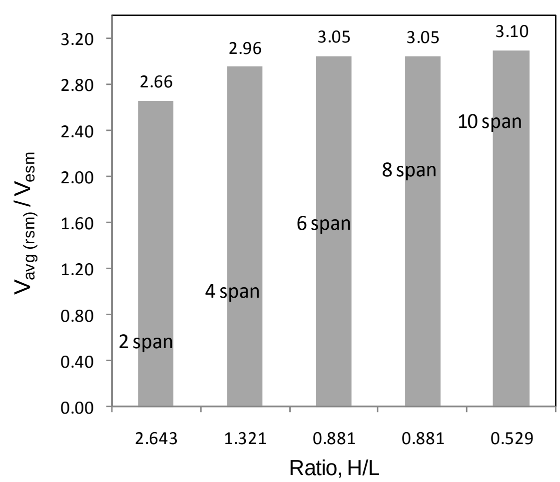


Fig.4.51 Base Shear Magnification vs H/L For 80% Infill 12 Floor

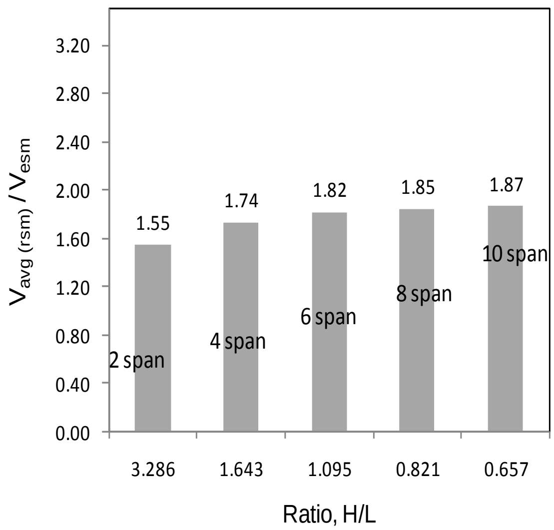


Fig.4.52 Base Shear Magnification vs H/L for 20% Infill 15 Floor

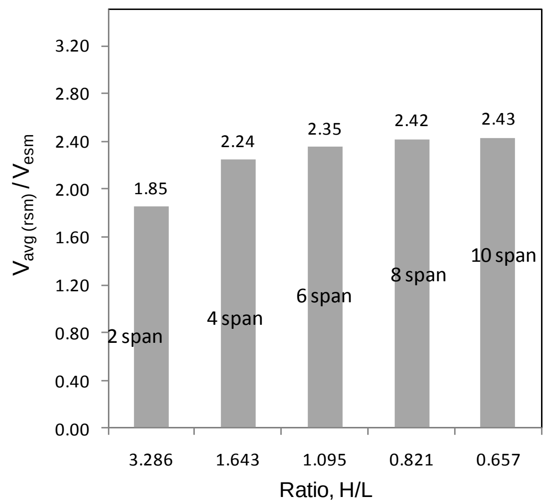


Fig.4.53 Base Shear Magnification vs H/L For 40% Infill 15 Floor

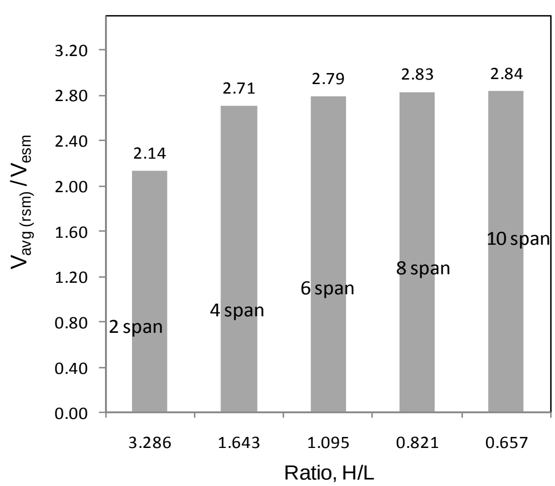


Fig.4.54 Base Shear Magnification vs H/L For 60% Infill 15 Floor

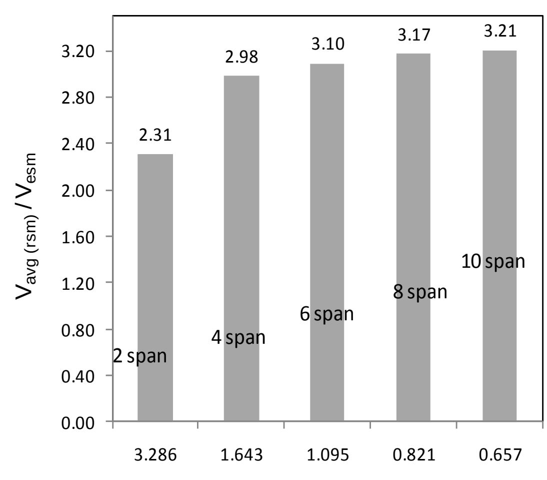


Fig.4.55 Base Shear Magnification vs H/L for 80% Infill 15 Floor

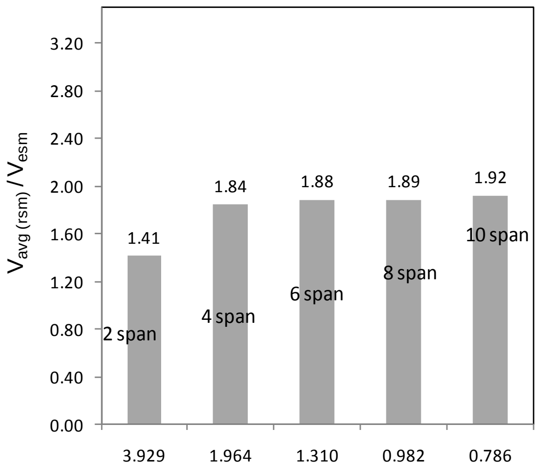


Fig.4.56 Base Shear Magnification vs H/L For 20% Infill 18 Floor

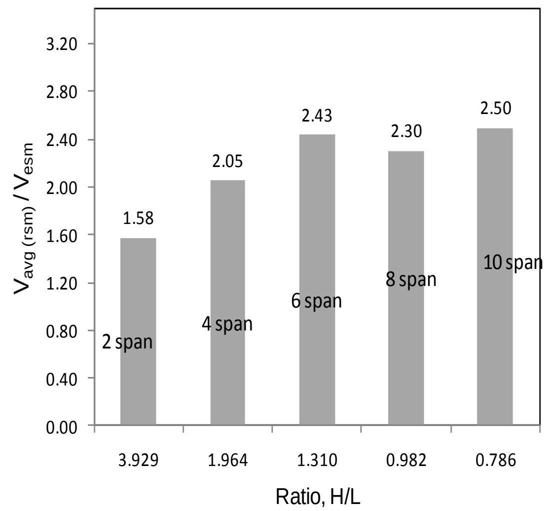


Fig.4.57 Base Shear Magnification vs H/L For 40% Infill 18 Floor

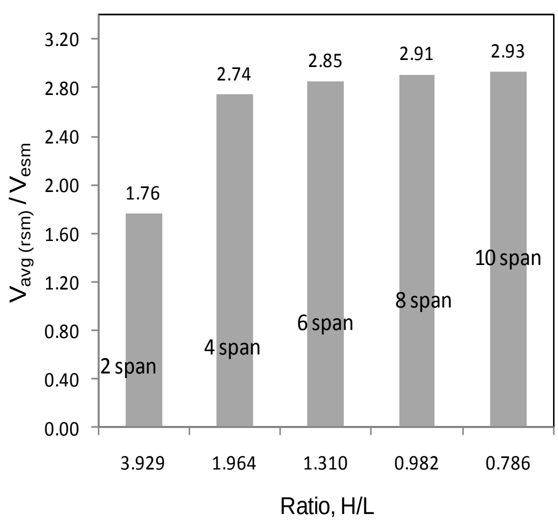


Fig.4.58 Base Shear Magnification vs H/L for 60% Infill 18 Floor

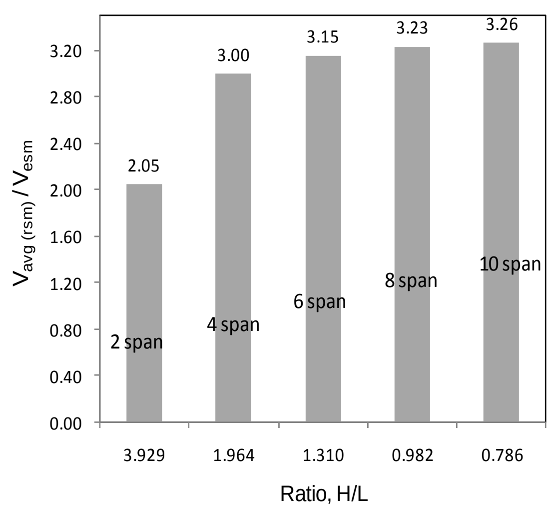


Fig.4.59 Base Shear Magnification vs H/L for 80% Infill 18 Floor

Findings of the above graph are mentioned below:

Frame height up to 10 floors

- For frame with infill percentage of 20% and 40% , the value $V_{avg\ (rsm)}$ / V_{esm} is high for spanned 4 to 10. For 2 spanned frames the value is quite lower.
- For 60% and 80% infill the value $V_{avg\ (rsm)}$ / V_{esm} are almost same for all spanned frame i.e effect of increasing span length after 6 spanned is not so prominent on base shear value.

Frame height above 10 floors

- The value $V_{avg\ (rsm)}$ / V_{esm} increase gradually with the increase of span length.
- The above increment is prominent for 4 spanned to 10 spanned frames with respect to 2 spanned frames. From the graph it has been found that for a frame of 18 floor with 80% infill, the value $V_{avg\ (rsm)}$ / V_{esm} is 2.05 for 2 spanned frame. But for the same frame with 4, 6, 8, 10 spanned, the values are 3.00, 3.15, 3.23, 3.26 respectively.

4.3.6 Base Shear Magnification VS Percentage (%) Infill

Base shear magnification is the ratio of base shear value of response spectrum method (RSM) and equivalent static force method (ESFM). It is denoted as β_0 which is defined below by the eqn. 4.1

$$\beta_0 = \left(V_{RSM(Avg.)} / V_{ESFM} \right) \tag{4.1}$$

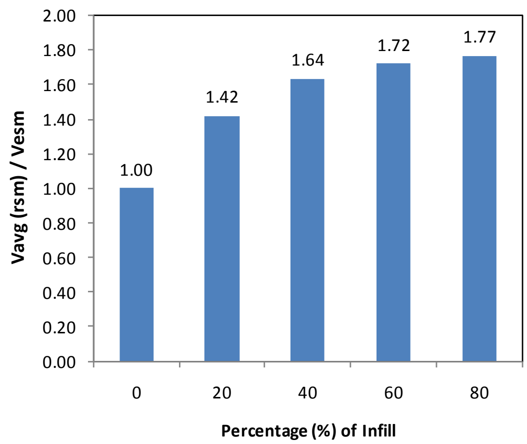


Fig.4.60 Base shear magnification vs % Infill 6 storied frame.

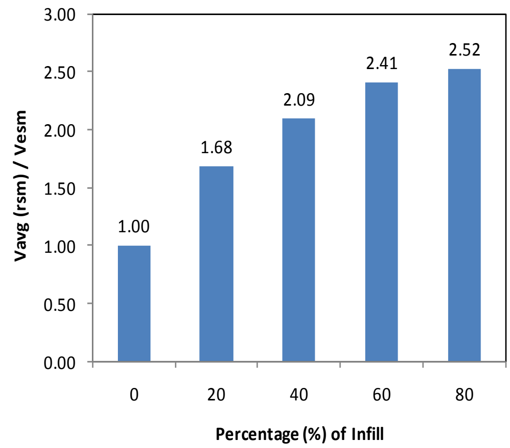


Fig.4.61 Base shear magnification vs % Infill 9 storied frame.

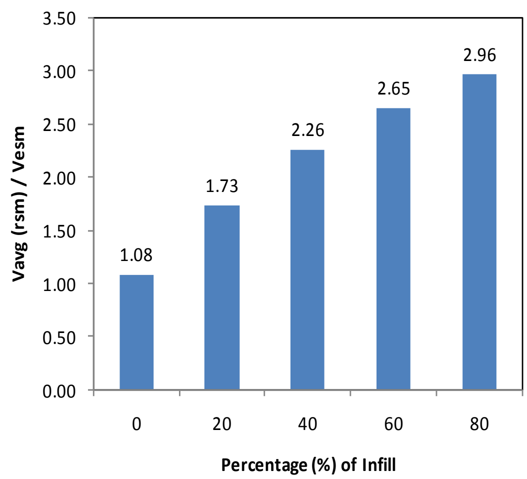


Fig. 4.62 Base shear magnification vs % Infill 12 storied frame.

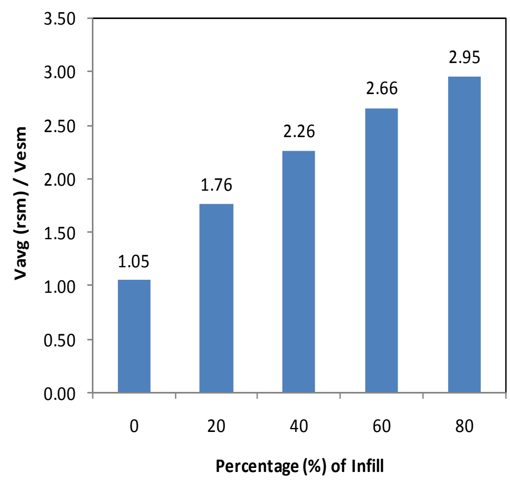


Fig. 4.63 Base shear magnification vs % Infill 15 storied frame.

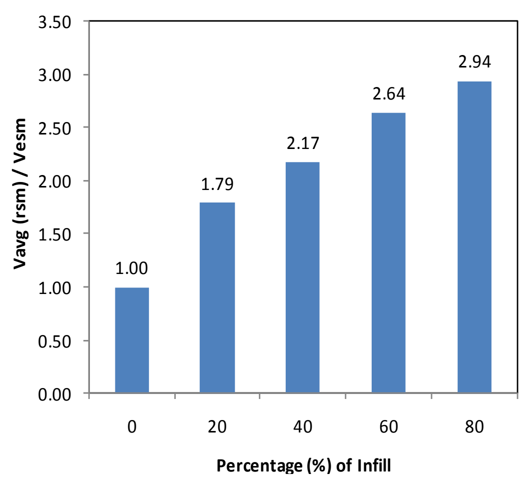


Fig. 4.64 Base shear magnification vs % Infill 18 storied frame.

Base shear magnification has determined for 6, 9, 12, 15 and 18 storied frame. Infill was applied as 20%, 40%, 60% and 80%. Here value of magnification factor, β_0 is adopted as average from the value of magnification factor of 2, 4, 6, 8 and 10 spanned frame. Then the graphs are plotted as magnification factor versus percentage of infill which is shown from Figure 4.60 to 4.64.

It has been observed from Figure 4.60 to 4.64 that the base shear magnification were almost the same for 12, 15 and 18 storied frame with presence of 20%, 40% , 60%, 80% infill. So, the variation in base shear magnification is not significant for higher storied (i.e. 12, 15 and 18 storied) frame. The value differs slightly for 9 storied frame where as the value is the lowest for 6 storied frames.

4.4 PROPOSAL FOR BASE SHEAR MAGNIFICATION

Figure 4.65 shows the variation of base shear ratio, β_0, for different amount of infill on upper floors for 6, 9, 12, 15 and 18 storied frame. It has been found from the figure 4.60 to 4.65 that base shear magnification is almost same for 12, 15 and 18 storied frame. Hence base shear magnification for 12th storied frame is considered here as the highest value and this represents as the value of 15th and 18th storied frame. Except for 20% infill on upper floors, it can be observed that in all other cases the relation between β_0 and number of floors is almost a linear one. It is also observed that for 40%, 60% and 80% amount of infill, the relationship between β_0 and number of floors remains in a narrow band. From practical experience as well as indicated by other researchers (Amanat and Hoque, 2006), the most probable percentage amount of structurally active infill in a real RC framed building shall be between 40% to 60%. Therefore, we can derive a relationship between base shear ratio, β_0, and number of floors by considering average base shear value corresponding to 40% and 60% infill. Then a graph is plotted as base shear ratio for 50% infill versus number of floors which is shown in fig.4.65. A simple linear regression suggests that, Base shear magnification, $\beta_0 = 0.129\, N_f + 0.97 \leq 2.52$ (4.2)

Where N_f = Number of floor ($6 \leq N_f \leq 18$)

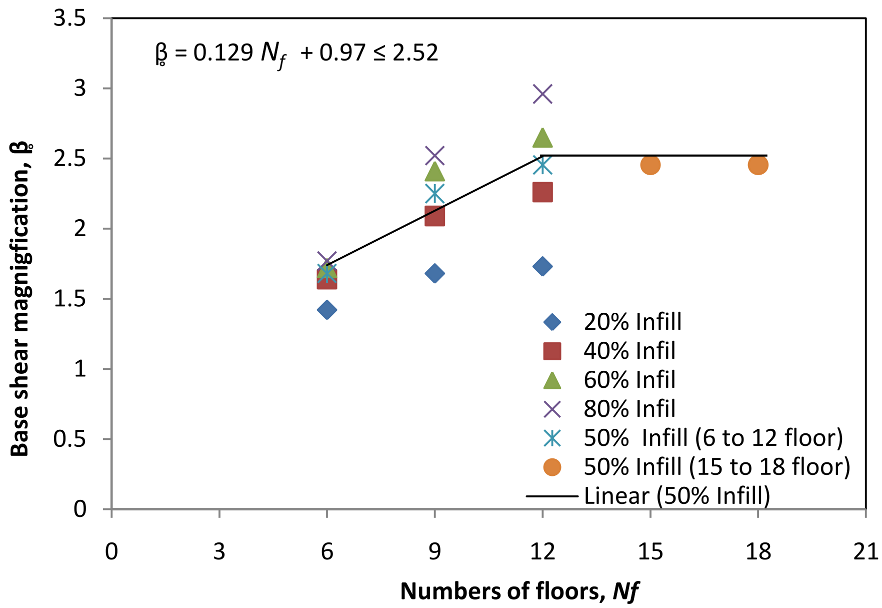


Fig. 4.65 Base shear magnification, β_0, as a function of number of floors and percent amount of Infill.

The above simplified expression for base shear ratio, β_0 as function of only the number of floors of a building, N_f (where $6 \leq N_f \leq 18$), shall enable a designer to rationally estimate the appropriate base shear for commonly occurring RC framed buildings with open (soft) ground floor using the conventional static force method (ESFM) by simply multiplying the obtained base shear ratio, β_0, obtained from the above equation 4.2. Thus β_0 , may be considered as a magnification factor to be applied to the base shear obtained from equivalent static force method to get a reasonable estimate of the correct base shear. This shall lead to a safe design of the columns of building with soft ground floor.

In order to use for design purpose we may increase it by 10% for additional safety (Goel and Chopra 1997, Goel and Chopra 1998). Thus we denote the finally suggested multiplier as

$$\beta = 1.1\beta_0 \tag{4.3}$$

Thus the equation 4.1 can be rewritten as

$$\beta = 0.15 N_f + 1.1 \leq 2.77 \quad (4.4)$$

Thus the design procedure would be to determine base shear on the basis of conventional Equivalent Static Force Method (ESFM) and analyze the structure for shear, axial force and moment in the conventional manner. The above equation 4.4 of magnification factor derived from base shear may consider applying for moment and shearing for soft storied column since moment and shear is increased with the increase of base shear. Then at the time of load combination we can apply the factor β obtained from eqn. 4.4 to get the design shear and moment for ground floor columns as follows.

$$1.05D+1.275L+ \beta\, 1.4E \text{ (Static)} \quad (4.5)$$

$$1.05D+1.275L- \beta\, 1.4E \text{ (Static)} \quad (4.6)$$

The above combination shall replace the conventional combination related to earthquakes for the design of open ground floor columns. It should be kept in mind that combinations related to other types of loading (wind load) should also be considered in the design process of ground floor columns.

4.5 APPLICABILITY OF ESFM FOR SOFT GROUND FLOOR BUILDING

Earthquake is always a dynamic phenomenon capturing the true structural behavior during earthquake, thus measures a full transient dynamic analysis involving natural non-linearity, time dependent ground acceleration etc. Such analysis is, even by present standards is extremely complicated, time consuming and the results are quite difficult to interpret and therefore such analysis is generally not recommended for a common design practices. Instead, an easier method called ESFM (equivalent static force method) has been formulated by researchers and accepted in design codes. ESFM method is purely intended for the purpose of design only. And it has its own scopes and limitations. For example it is only applicable for a building having regular stiffness characteristics. This method is not, as defined in codes, applicable for soft story buildings. Due to presence of open ground floor such building are becoming more common. However due to absence of definite code provisions for such structures and due to the complexity involved in dynamic analysis, design engineers are in most cases applying, though inappropriate, ESFM. In the preceding section the inappropriateness of ESFM has been elaborately identified. In order to overcome this limitation of ESFM for soft story building, a proposal for base shear magnification factors, $\boldsymbol{\beta}$ has been suggested. When the $\boldsymbol{\beta}$ shall be used to modify the moments and shears due to earthquake application of equivalent static force method, a more rational and safer design would be obtained and the limitations of ESFM would be overcome.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

In the present study a wide range of computational analysis has been performed on reinforced concrete frame having various percentage of masonry infill on upper floors with soft ground floor. The application of infill has been applied randomly and the effect of seismic load has been investigated. The analysis was carried for multistoried 2D frames for different percentage of infill on upper floors. For each individual case at least 15 runs have been made to study the variation on base shear for randomly applied infill. Static (ESFM) analysis does not reflect actual dynamic behavior of a structure subjected to lateral loading. Hence the analysis is performed by both ESFM and RSM so that a comparison can be made on the actual masonry infilled reinforced concrete soft story behavior with the current design practice. Based on the investigation, modification factor has been recommended on various cases of infill application. This modification factor might help the designers to adopt in their designing in order to stay in safe side during earthquake.

5.2 CONCLUSIONS

The investigation of this study indicates a different characteristic behavior of infilled RC frame with open ground floor. The summary of the findings can be tabulated as follows:

- Randomness of distribution of structurally active infill has no effect on base shear value.
- The value of base shear increases with the increase of floors, span and percentage of infill. Because these parameters are related to the increase of the dead load of the total structure.
- Sway increases suddenly at soft ground floor level whereas it decreases on the upper floors due to the presence of infill in upper floors. In presence of infill in upper story the sway pattern shows a major change. The presence of infill makes the upper floor rigid and as a result the major deflection is concentrated in the ground floor.

- Drift demand is high for soft story frame in response spectrum method (RSM). This drift demand increases with the increase of frame height, percentage (%) of infill and number of span.
- For tall buildings (i.e. 12 storied or more) with 4 to 10 spanned frame, magnification of base shear is almost same whereas the value is quite lower for 2 spanned frame.
- Equivalent static force method is incapable of predicting the soft story behavior even in presence of infill in the analysis model.
- The finding of the study i.e. base shear magnification factor can help designer to produce safe design of buildings with open ground floor.

5.3 RECOMMENDATIONS

The behavior of RC framed buildings with open ground floor has been presented in this thesis. Based on the findings suggestion is made which is elaborated in chapter 4. The proposed suggestion is to magnify the base shear by a factor, based on the number of floor of the structure. Since moment and shear are increased with the increase of base shear, hence the magnification factor is applicable for moment and shear of soft story column. Then the designing of ground floor columns will be safer to withstand the intensified earthquake force resulting from the shaking of much stiffer upper floors.

5.4 SCOPE FOR FURTHER STUDY

The study of soft ground floor with structurally active infill has been performed under limited scope. Other variables and parameters may include for further study before applying in practical field of civil engineering. Advancement of present study is recommended here to compare the result as a factor of safety. The following fields related to this study can be considered for further analysis;

- The model was considered to be linearly elastic. To be more realistic with the results a finite element analysis with non linear material properties can be performed.
- Equivalent strut model is used in this study to represent the infill. The behavior of structure using other models of infills can be studied..

- The asymmetric building frames can be studied under the variables considered for symmetric frames.
- Presence of openings in infill like window or door can be considered in analysis.
- Torsional effect has not considered in this investigation. Hence, further study can be performed considering this torsional effect.

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ABBREVIATIONS

ESFM	-	Equivalent Static Force Method.
RSM	-	Response Spectrum Method.
FE	-	Finite Element
RC	-	Reinforced Concrete
BNBC	-	Bangladesh National Building Code
E	-	Young's Modulus of Elasticity
K	-	Stiffness of the structure
f_m'	-	masonry prism strength
V_m	-	maximum lateral force
u_m	-	Displacement corresponding to the lateral force
A_d	-	Area of equivalent diagonal strut
L_d	-	Length of equivalent diagonal strut
ρ	-	Density
σ	-	Poisson's ratio
V	-	Design base shear
Z	-	Seismic zone coefficient
I	-	Coefficient of structural importance
C	-	Numerical coefficient
R	-	Response modification factor for structural system
W	-	Total seismic Dead load
T	-	Time period of natural vibration
S	-	Seismic zone coefficient
DL	-	Dead Load
LL	-	Live load
PW	-	Partition wall
MI	-	Masonry Infill

Appendix-A

Variation of Base Shear of 6, 12 and 15 storied frame with different number of span.

Table 4.4: Base shear variation of 6 storied with 8 spanned frame for random application of Infilll (in different percentage) with soft ground floor

Upper floor Infilll percentage	**20%**	**40%**	**60%**	**80%**
Base shear values by RSM for different patterns of Infilll application (KN)	1044	1086	1259	1358
	944	1185	1270	1357
	887	1182	1272	1353
	1067	1131	1263	1358
	998	1111	1274	1356
	1004	1132	1271	1357
	972	1151	1272	1351
	898	1170	1268	1359
	917	1138	1272	1357
	981	1188	1270	1358
	909	1182	1273	1357
	1071	1180	1266	1354
	1079	1099	1272	1353
	849	1180	1270	1350
	795	1178	1267	1357
Average Base shear by RSM (KN)	961	1153	1269	1356
Maxima (KN)	1079	1188	1274	1359
Minima (KN)	795	1086	1259	1350
Standard deviation	82.61	33.29	3.95	2.70
Base shear by ESFM (KN)	666	702	739	775
Modification Factor	1.44	1.64	1.72	1.75

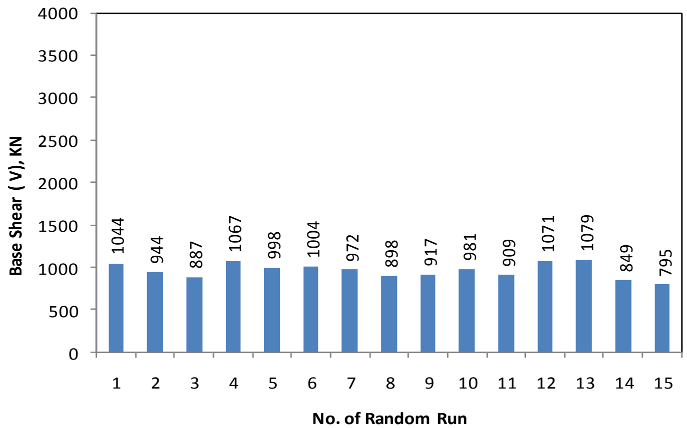


Fig.4.66 Variation of Base Shear for 20% Infilll 6 Floor8 Spanned Frame

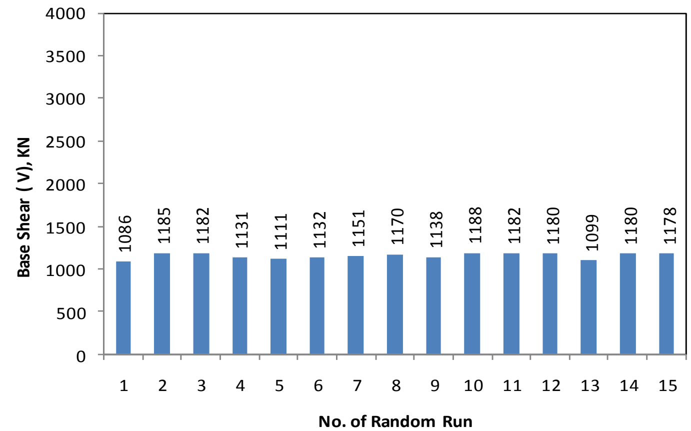


Fig.4. 67 Variation of Base Shear for 40% Infilll 6 Floor8 Spanned Frame

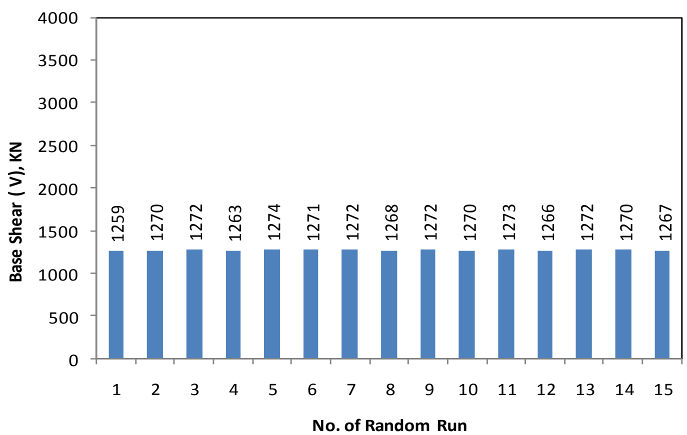


Fig.4.68 Variation of Base Shear for 60% Infilll 6 Floor8 Spanned Frame

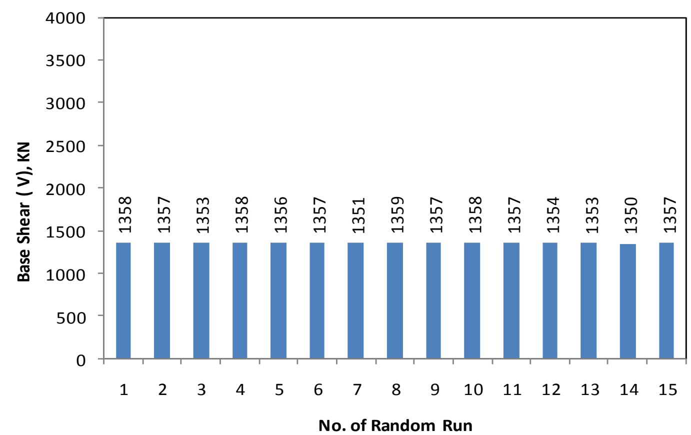


Fig.4.69 Variation of Base Shear for 80% Infilll 6 Floor8 Spanned Frame

Table 4.5: Base shear variation of 12 storied with 8 spanned frame for random application of Infilll (in different percentage) with soft ground floor

Upper floor Infilll percentage	**20%**	**40%**	**60%**	**80%**
	1722	2357	2768	3476
Base shear values by RSM for different patterns of Infilll application (KN)	1674	2551	3066	3412
	1697	2305	2922	3334
	1925	2401	2764	3394
	1712	2318	3085	3498
	1776	2477	2966	3347
	1692	2404	2806	3260
	1630	2460	2987	3497
	1822	2235	3007	3378
	1667	2386	2850	3476
	1863	2248	2870	3412
	1783	2239	3147	3335
	1728	2502	2901	3365
	1621	2170	2996	3421
	1776	2345	2915	3435
Average Base shear by RSM (KN)	1739	2360	2937	3403
Maxima (KN)	1925	2551	3147	3498
Minima (KN)	1621	2235	2764	3260
Standard deviation	82.41	106.01	110.78	65.90
Base shear by ESFM (KN)	950	1005	1062	1116
Modification Factor	1.83	2.35	2.77	3.05

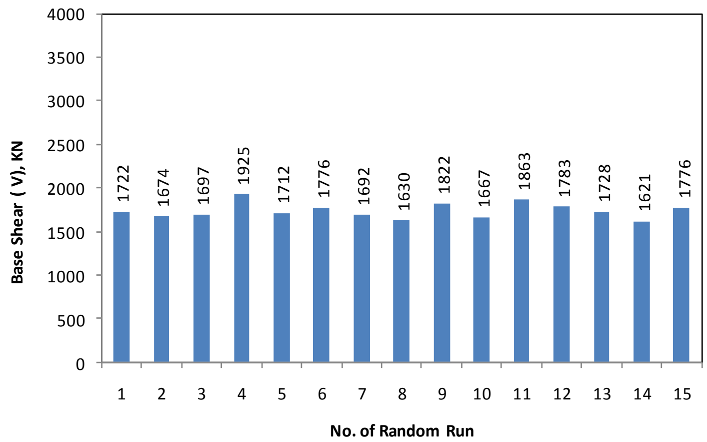


Fig.4.70 Variation of Base Shear for 20% Infilll 12 Floor 8 Spanned Frame

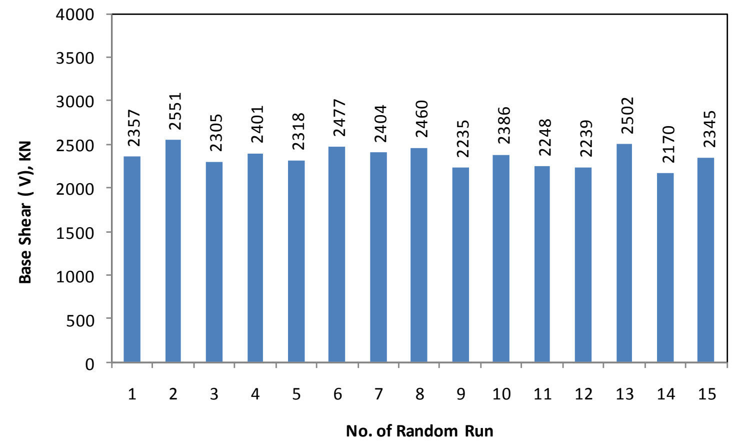


Fig.4.71 Variation of Base Shear for 40% Infilll 12 Floor8 Spanned Frame

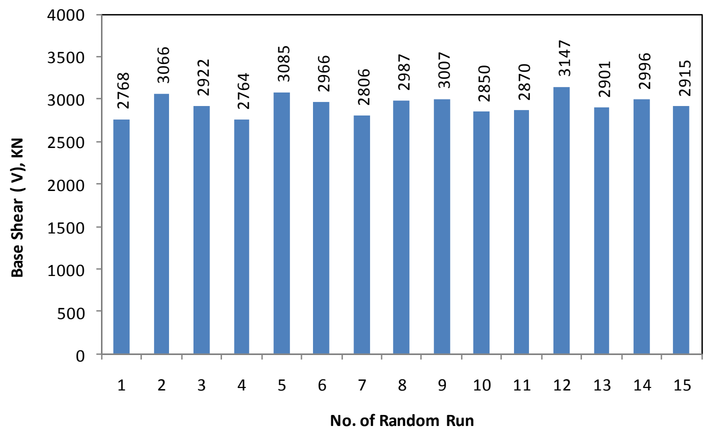


Fig.4.72 Variation of Base Shear for 60% Infilll 12 Floor8 Spanned Frame

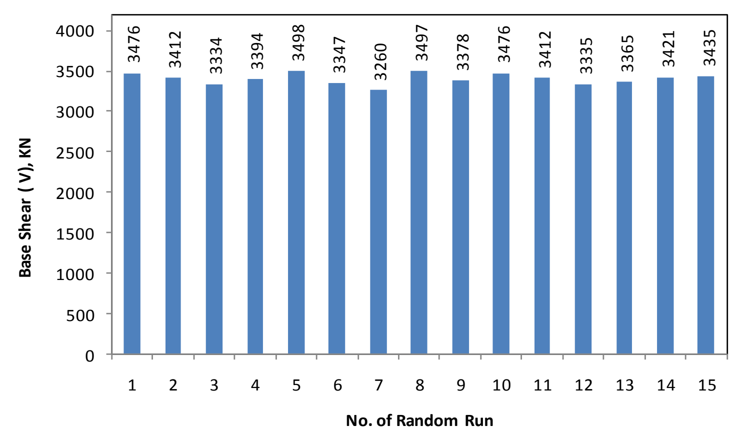


Fig.4.73 Variation of Base Shear for 80% Infilll 12 Floor8 Spanned Frame

Table 4.6: Base shear variation of 15 storied with 8 spanned frame for random application of Infilll (in different percentage) with soft ground floor

Upper floor Infilll percentage	**20%**	**40%**	**60%**	**80%**
Base shear values by RSM for different patterns of Infilll application (KN)	1952	2881	3489	3985
	1856	2826	3361	4038
	1951	2851	3478	3994
	2052	2613	3419	3964
	2044	2800	3395	4091
	2091	2870	3497	4097
	2074	2784	3449	4085
	2011	2792	3324	4114
	2006	2835	3370	4068
	1970	2628	3537	4072
	1939	2832	3475	4150
	2128	2926	3481	4052
	2041	2690	3350	4068
	1810	2706	3381	4049
	2071	2669	3518	4084
Average Base shear by RSM (KN)	2000	2780	3435	4061
Maxima (KN)	2091	2926	3537	4150
Minima (KN)	1810	2669	3324	3964
Standard deviation	84.78	93.08	65.23	48.08
Base shear by ESFM (KN)	1083	1150	1214	1280
Modification Factor	1.85	2.42	2.83	3.17

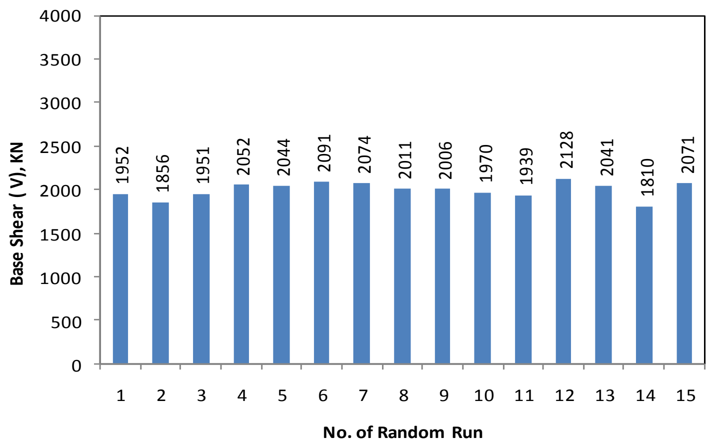


Fig.4.74 Variation of Base Shear for 20% Infilll 15 Floor 8 Spanned Frame

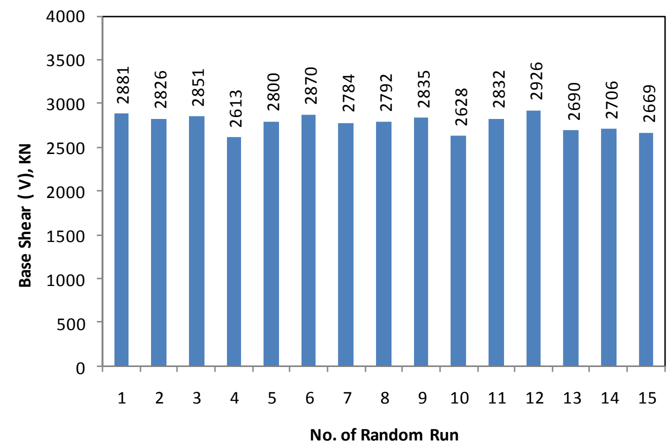


Fig.4.75 Variation of Base Shear for 40% Infilll 15 Floor 8 Spanned Frame

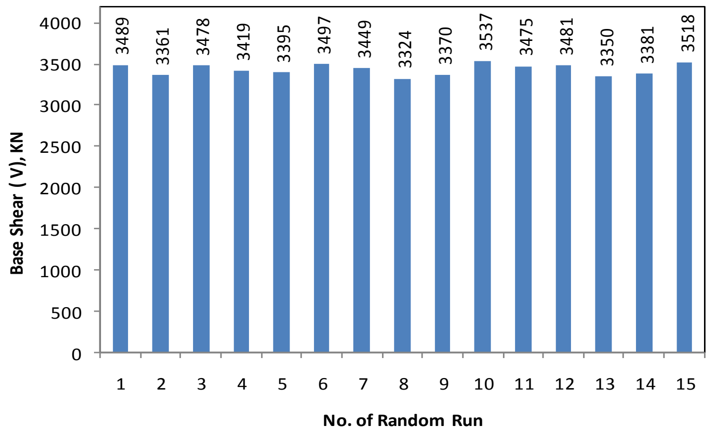


Fig.4.76 Variation of Base Shear for 60% Infilll 15 Floor 8 Spanned Frame

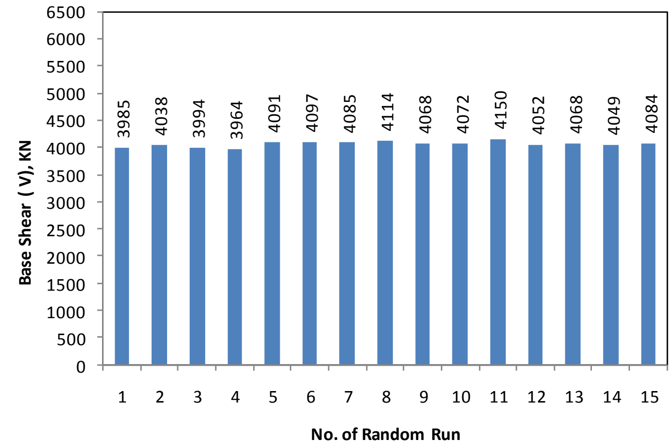


Fig.4.77 Variation of Base Shear for 80% Infilll 15 Floor 8 Spanned Fram

Appendix-B

Effect of Variation of Infilll Percentage on Story Sway

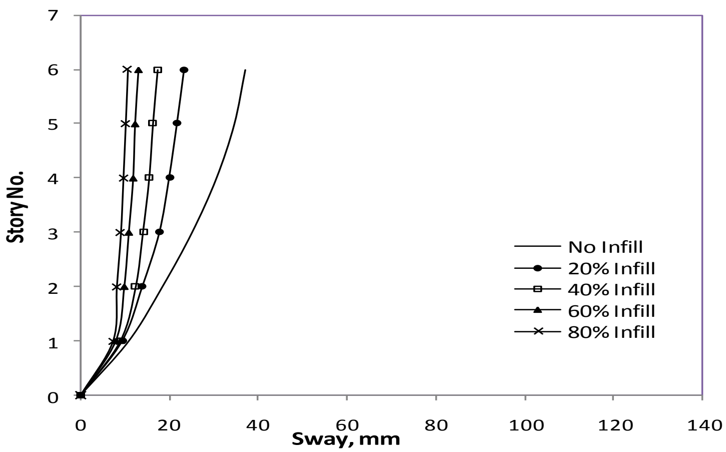


Fig. 4.78 Story Sway of 6 storied and 2 spanned Frame by ESFM

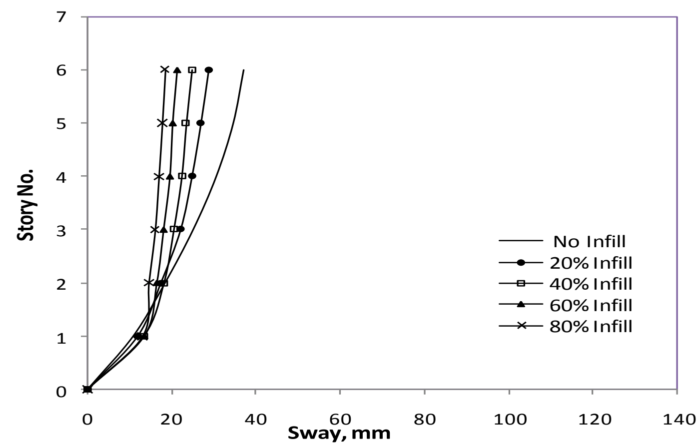


Fig. 4.79 Story Sway of 6 storied and 2 spanned Frame by RSM

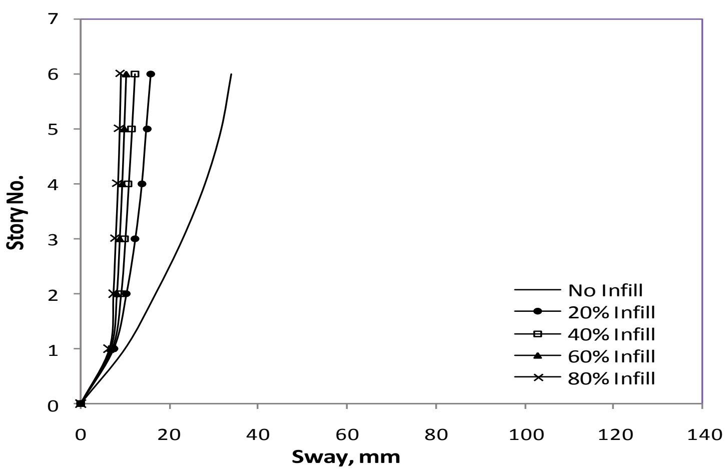


Fig. 4.80 Story Sway of 6 storied and 6 spanned Frame by ESFM

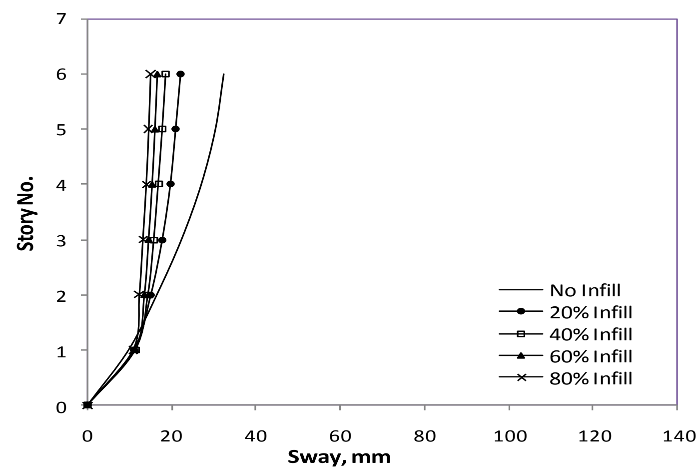


Fig. 4.81 Story Sway of 6 storied and 6 spanned Frame by RSM

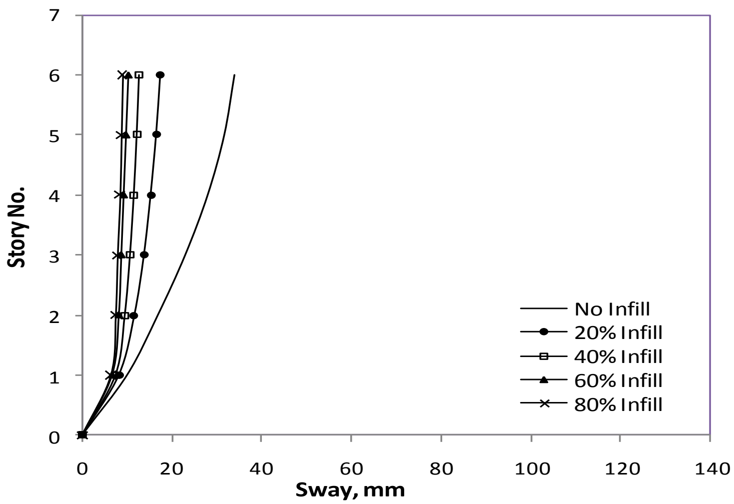


Fig. 4.82 Story Sway of 6 storied and 8 spanned Frame by ESFM

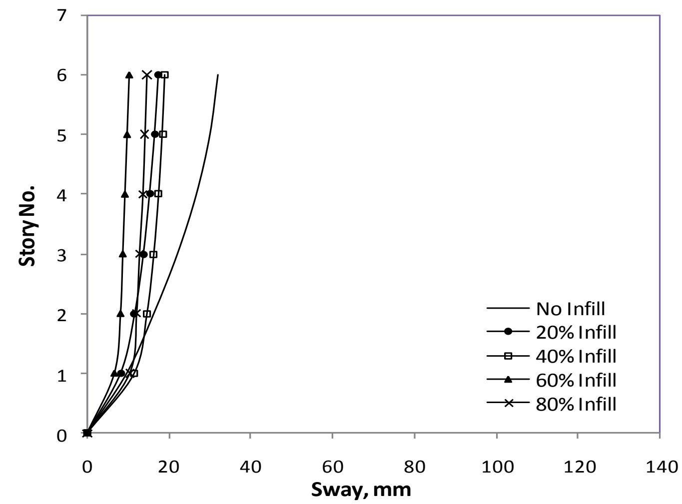


Fig. 4.83 Story Sway of 6 storied and 8 spanned Frame by RSM

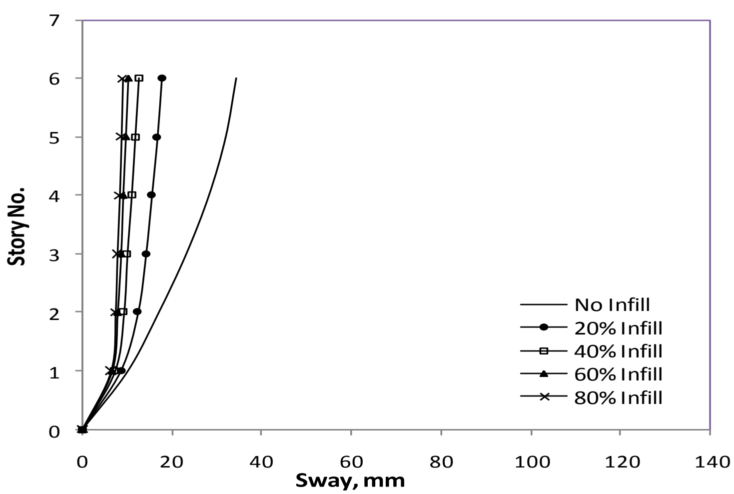


Fig. 4.84 Story Sway of 6 storied and 10 spanned Frame by ESFM

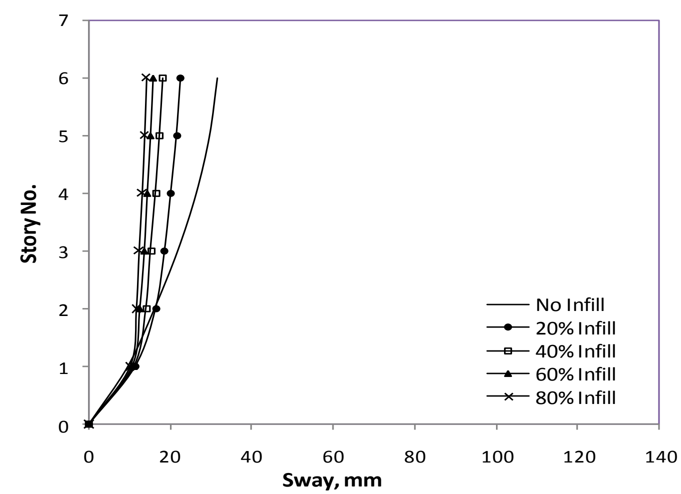


Fig. 4.85 Story Sway of 6 storied and 10 spanned Frame by RSM

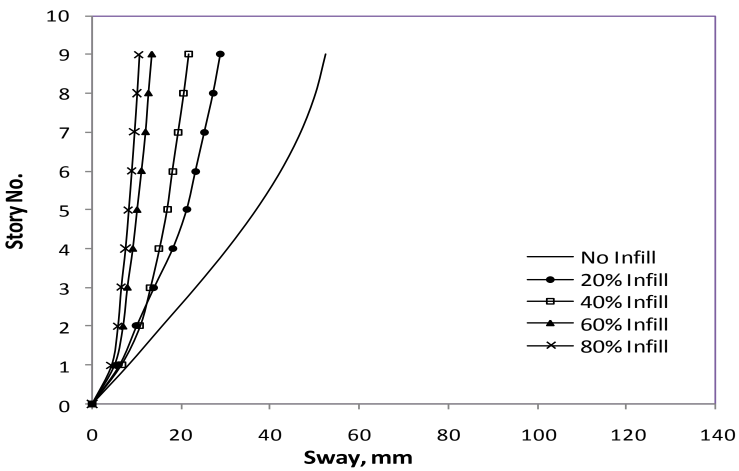


Fig. 4.86 Story Sway of 9 storied and 2 spanned Frame by ESFM

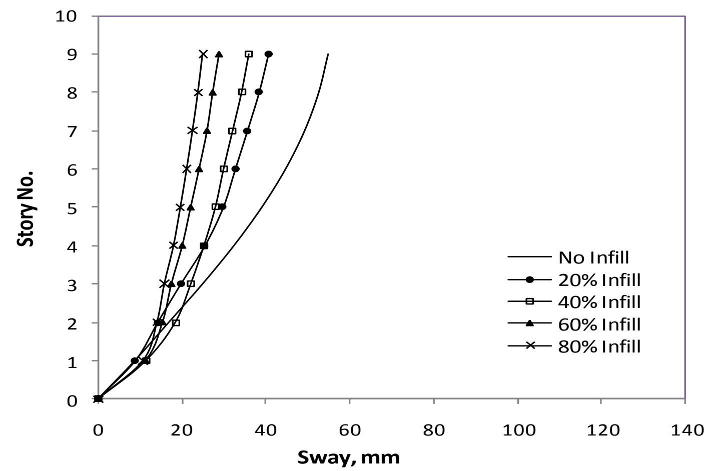


Fig. 4.87 Story Sway of 9 storied and 2 spanned Frame by RSM

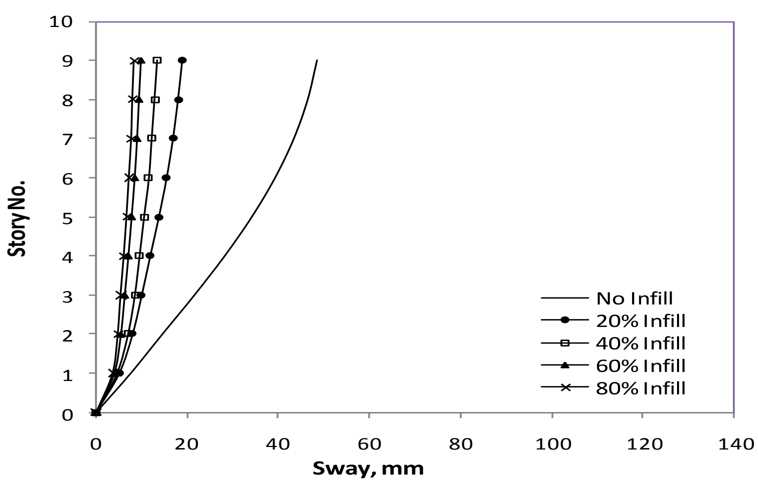


Fig. 4.88 Story Sway of 9 storied and 6 spanned Frame by ESFM

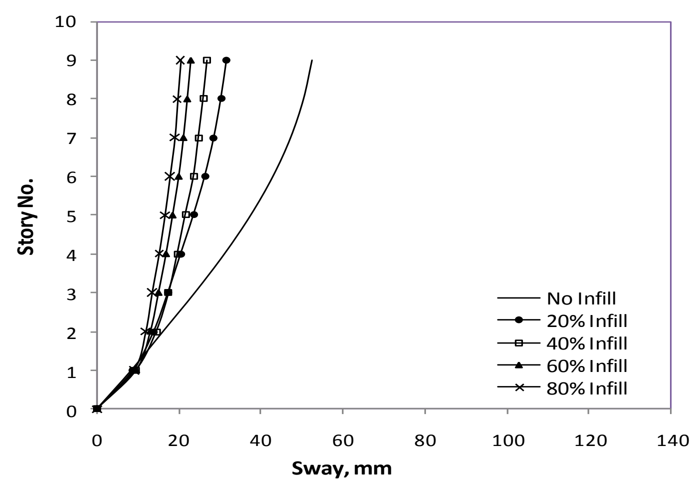


Fig. 4.89 Story Sway of 9 storied and 6 spanned Frame by RSM

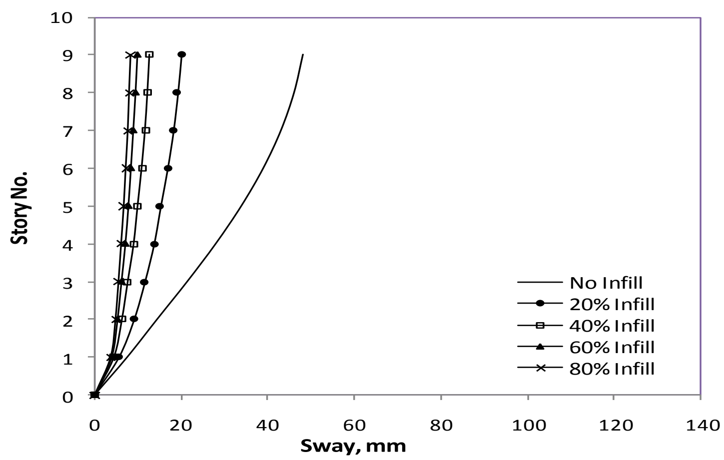


Fig. 4.90 Story Sway of 9 storied and 8 spanned Frame by ESFM

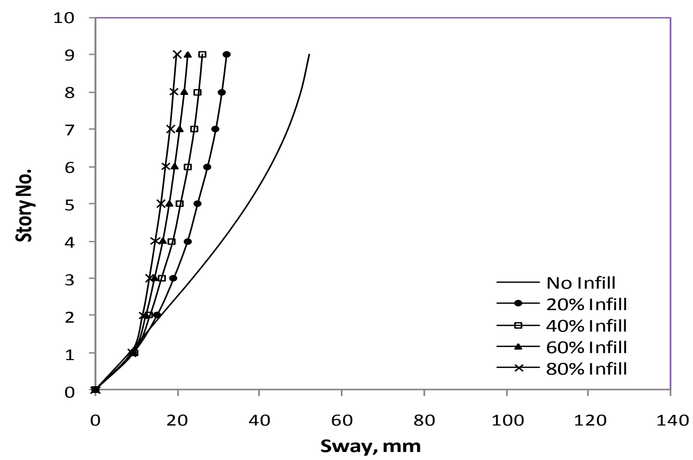


Fig. 4.91 Story Sway of 9 storied and 8 spanned Frame by RSM

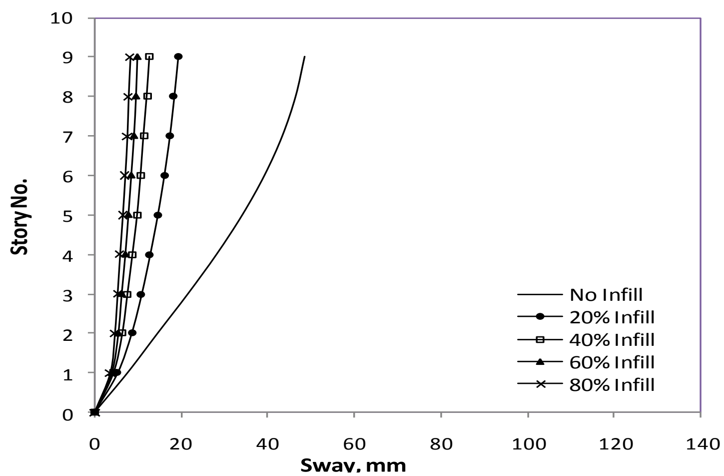


Fig. 4.92 Story Sway of 9 storied and 10 spanned Frame by ESFM

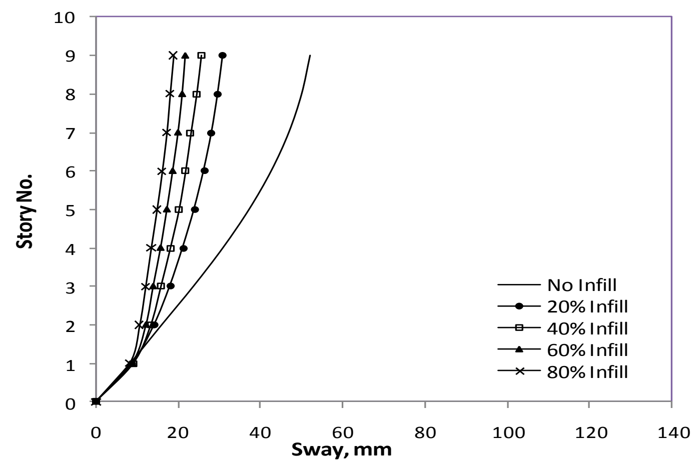


Fig. 4.93 Story Sway of 9 storied and 10 spanned Frame by RSM

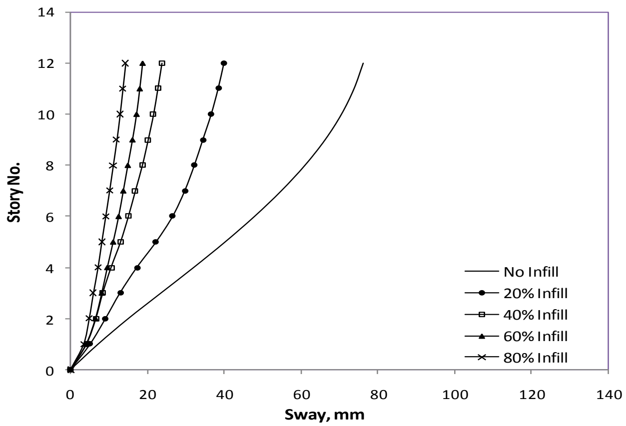


Fig. 4.94 Story Sways of 12 storied and 2 spanned Frame by ESFM

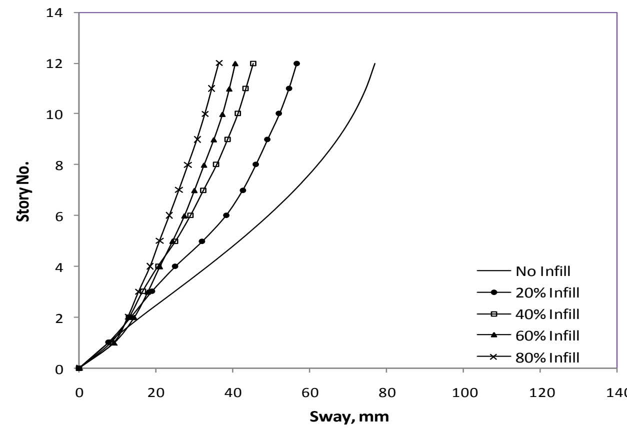


Fig. 4.95 Story Sway for of 12 storied and 2 spanned Frame by RSM

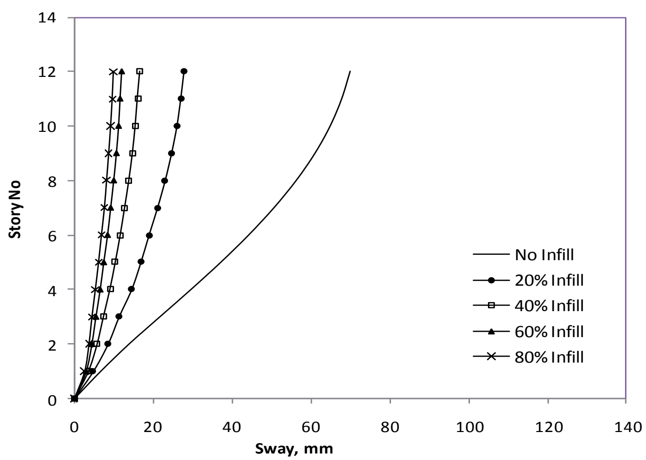


Fig. 4.96 Story Sway of 12 storied and 6 spanned Frame by ESFM

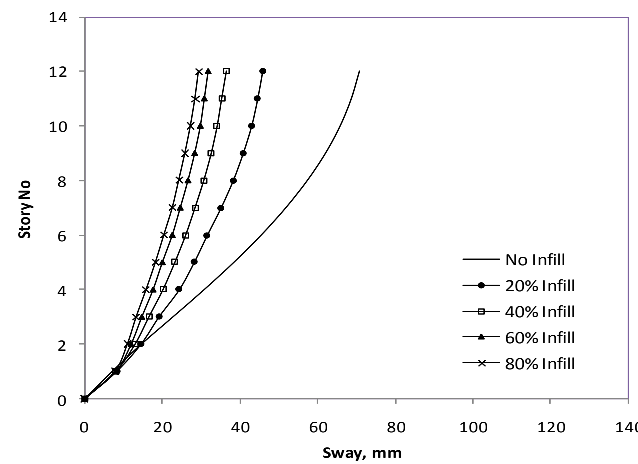


Fig. 4.97 Story Sway of 12 storied and 6 spanned Frame by RSM

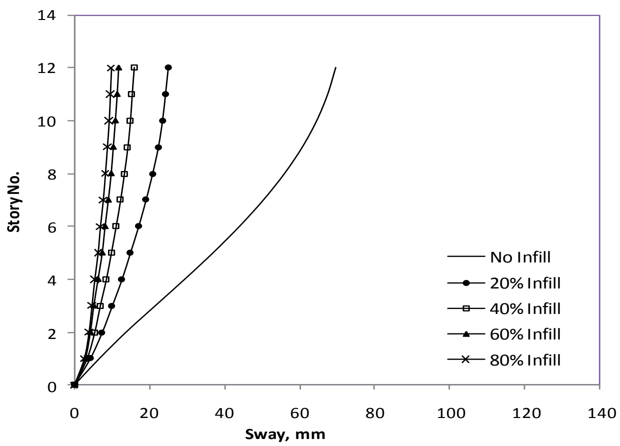


Fig. 4.98 Story Sway of 12 storied and 8 spanned Frame by ESFM

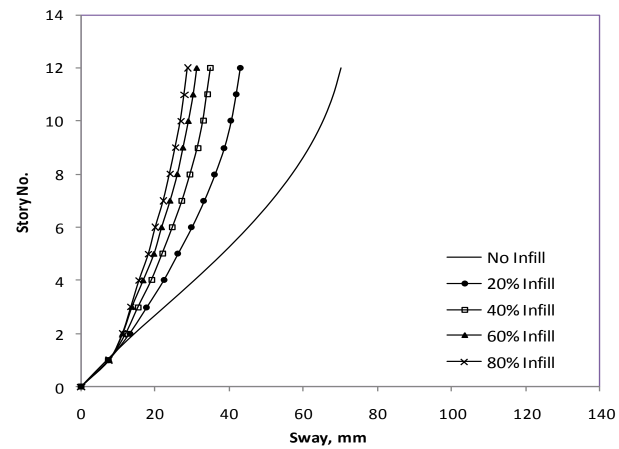


Fig. 4.99 Story Sway of 12 storied and 8 spanned Frame by RSM

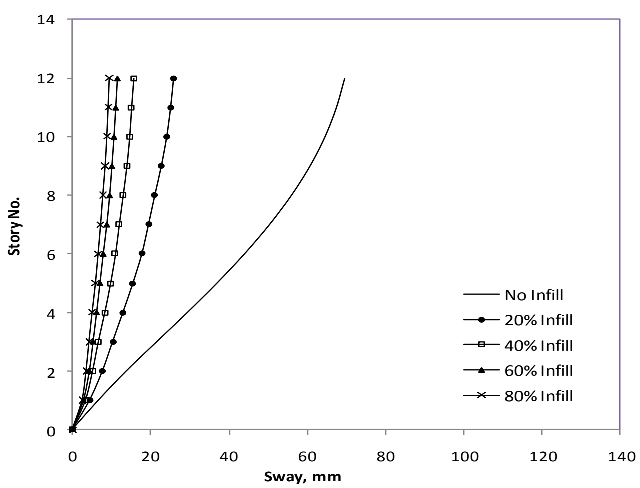


Fig. 4.100 Story Sway of 12 storied and 10 spanned Frame by ESFM

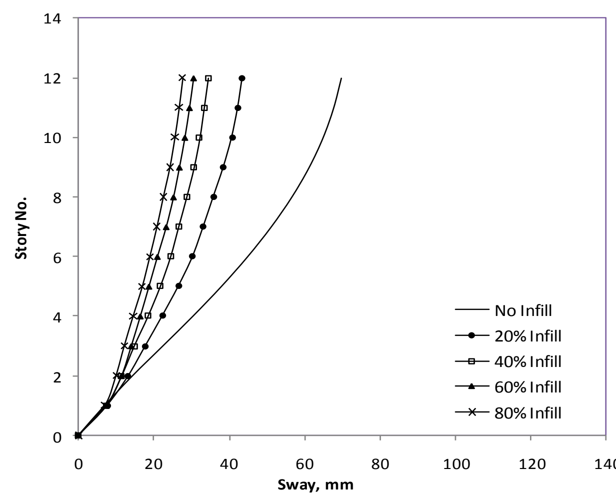


Fig. 4.101 Story Sway of 12 storied and 10 spanned Frame by RSM

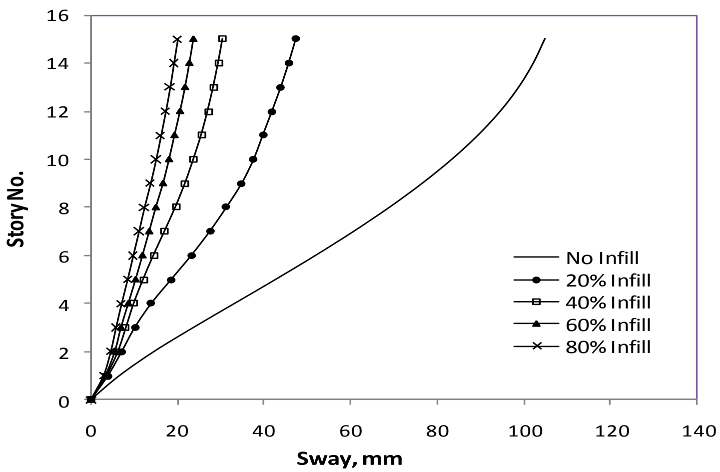


Fig. 4.102 Story Sway of 15 storied and 2 spanned Frame by ESFM

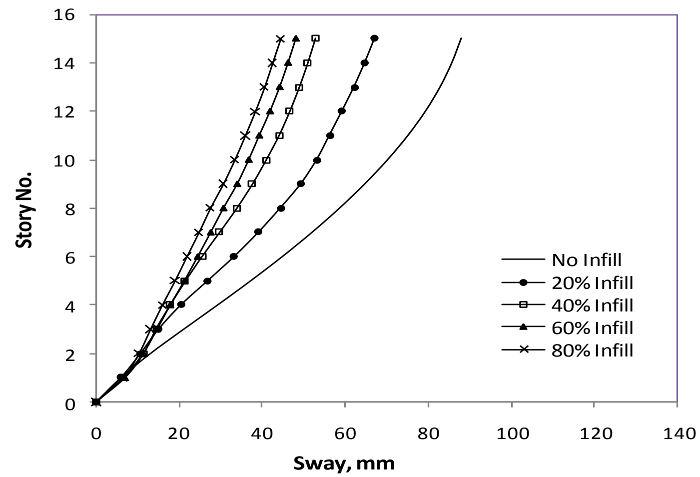


Fig. 4.103 Story Sway of 15 storied and 2 spanned Frame by RSM

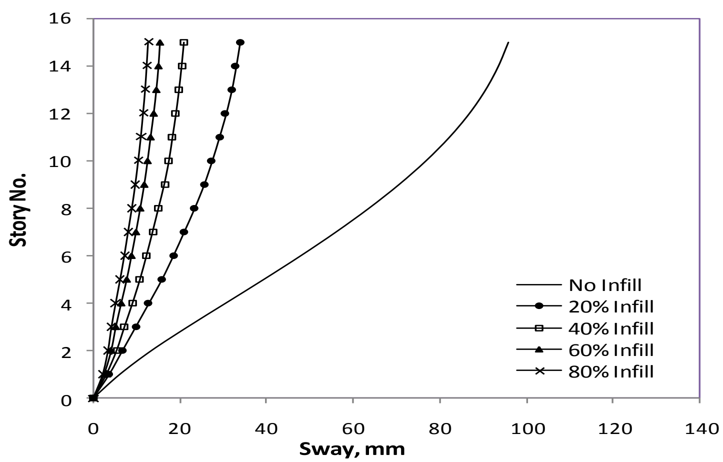


Fig. 4.104 Story Sway of 15 storied and 6 spanned Frame by ESFM

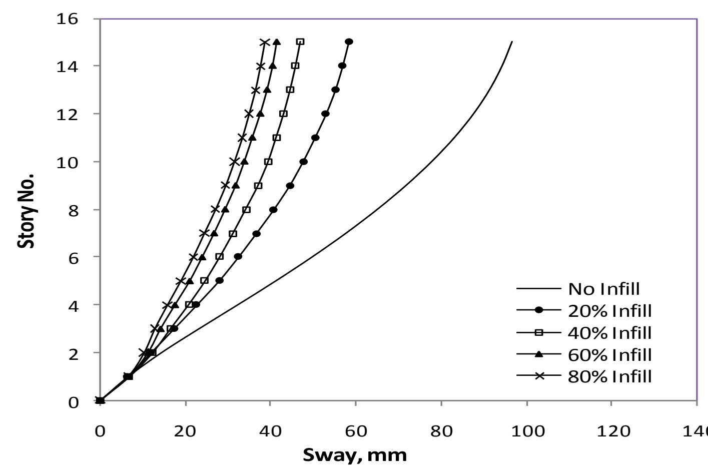


Fig. 4.105 Story Sway of 15 storied and 6 spanned Frame by RSM

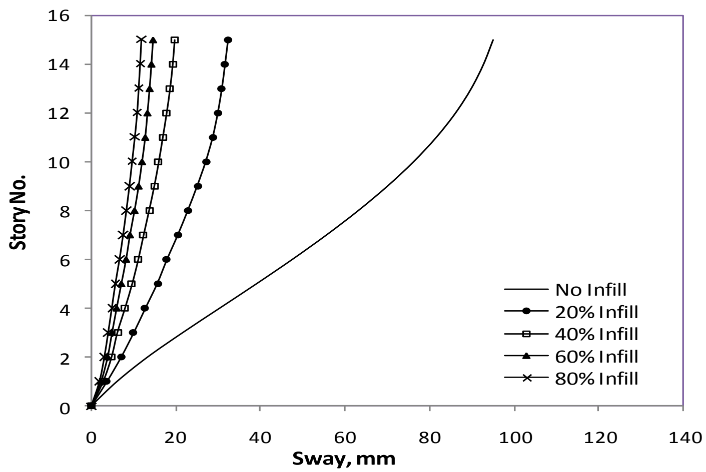


Fig. 4.106 Story Sway of 15 storied and 8 spanned Frame by ESFM

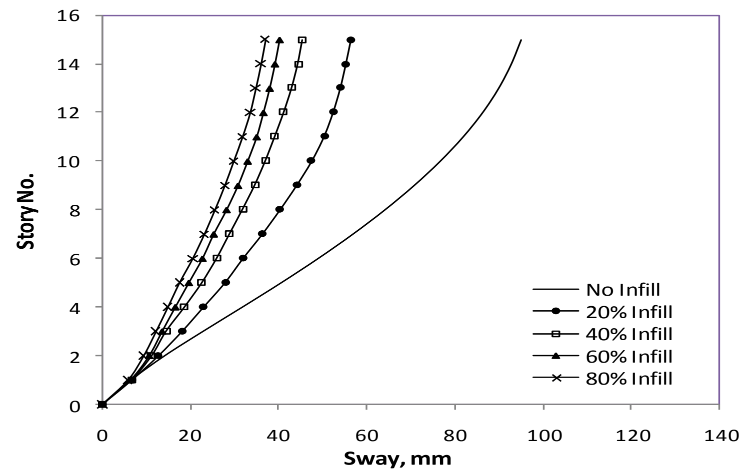


Fig. 4.107 Story Sway of 15 storied and 8 spanned Frame by RSM

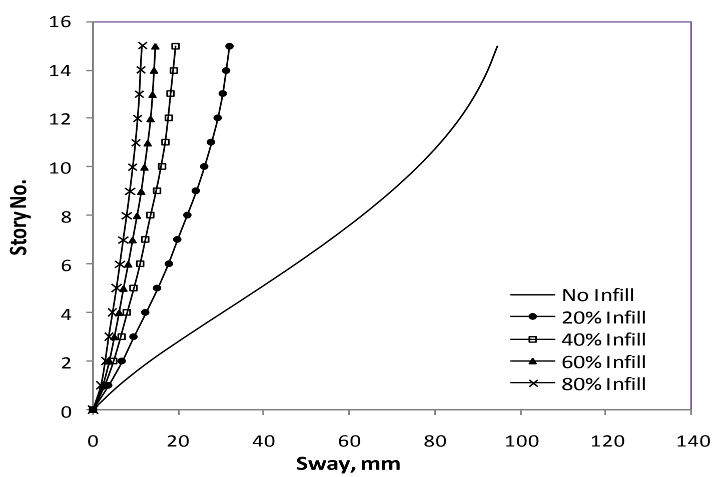


Fig. 4.108 Story Sway of 15 storied and 10 spanned Frame by ESFM

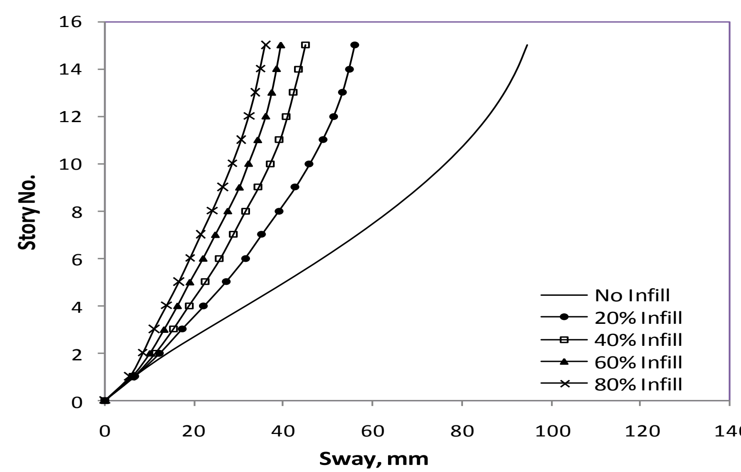


Fig. 4.109 Story Sway of 15 storied and 10 spanned Frame by RSM

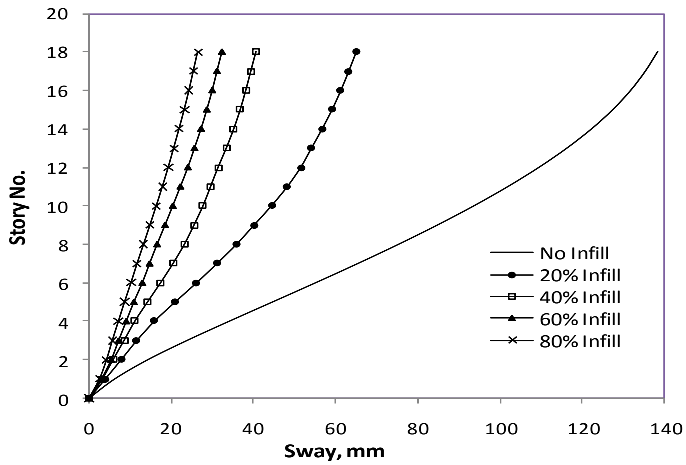


Fig. 4.110 Story Sway of 18 storied and 2 spanned Frame by ESFM

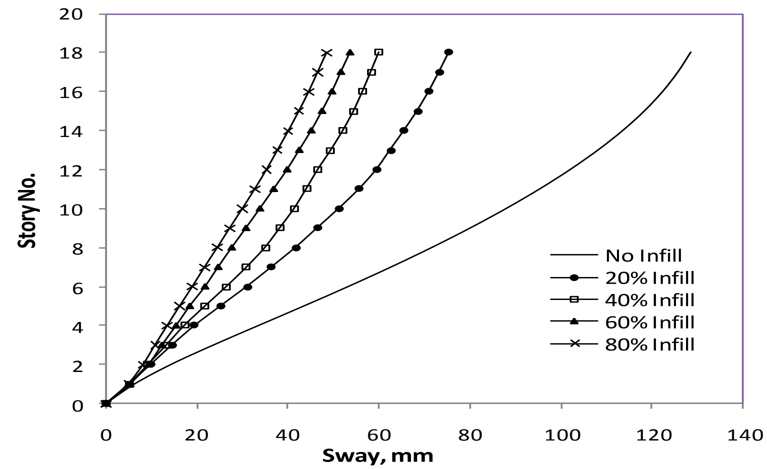


Fig. 4.111 Story Sway of 18 storied and 2 spanned Frame by RSM

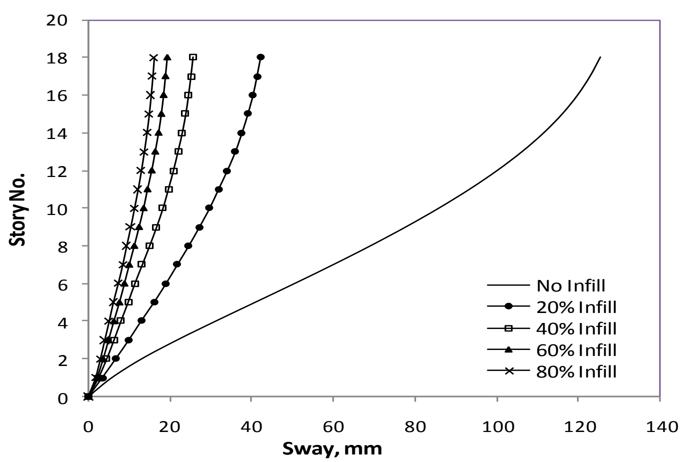


Fig. 4.112 Story Sway of 18 storied and 6 spanned Frame by ESFM

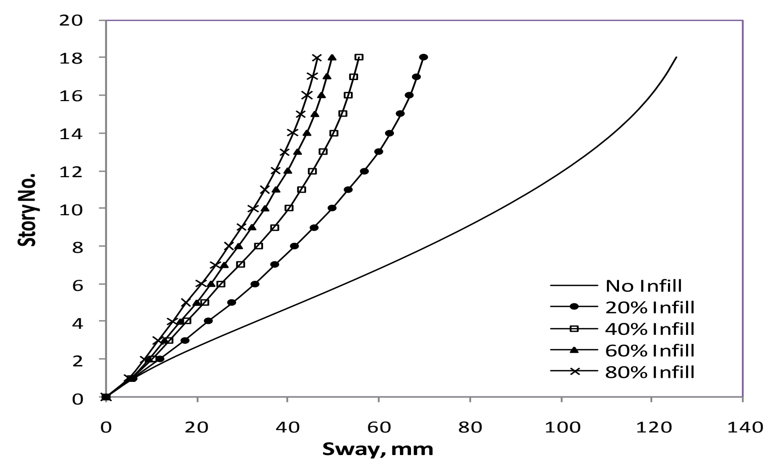


Fig. 4.113 Story Sway of 18 storied and 6 spanned Frame by RSM

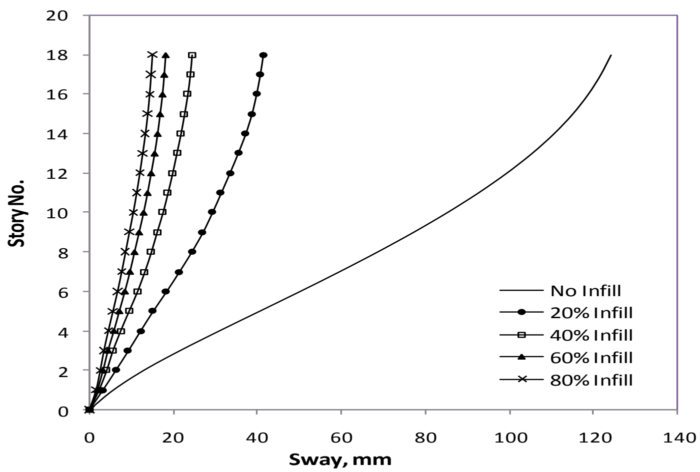


Fig. 4.114 Story Sway of 18 storied and 8 spanned Frame by ESFM

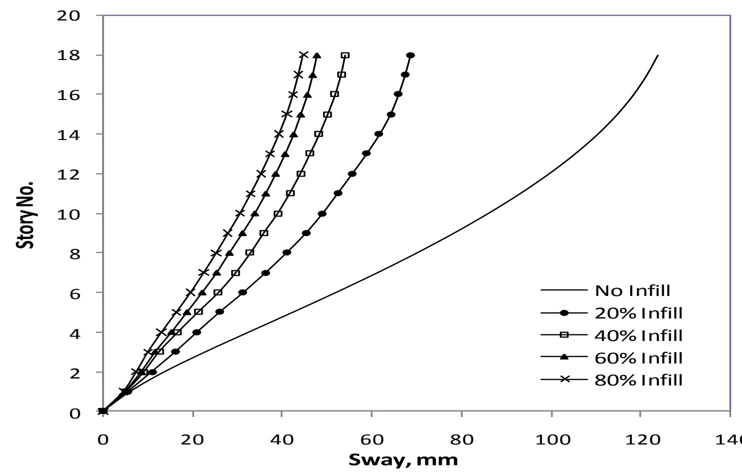


Fig. 4.115 Story Sway of 18 storied and 8 spanned Frame by RSM

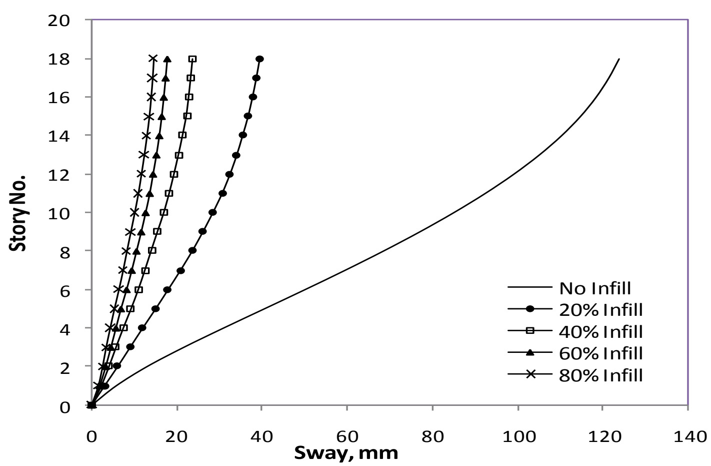


Fig. 4.116 Story Sway of 18 storied and 10 spanned Frame by ESFM

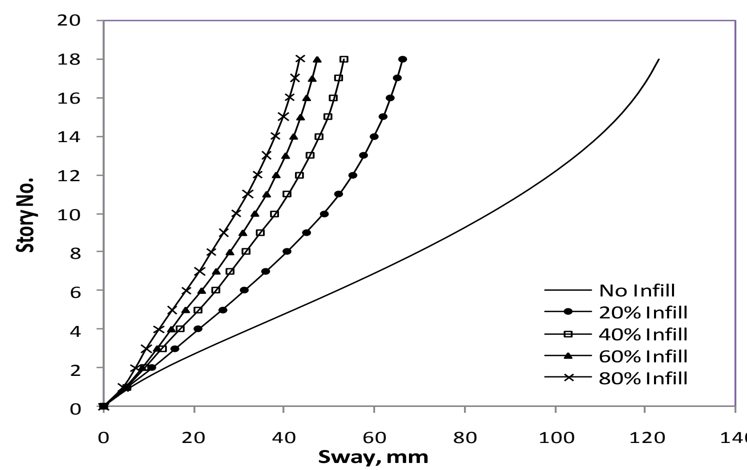


Fig. 4. 117 Story Sway of 18 storied and 10 spanned Frame by RSM

<u>Appendix- C</u>

Effect of Span Length & Infilll Percentage on Story Drift

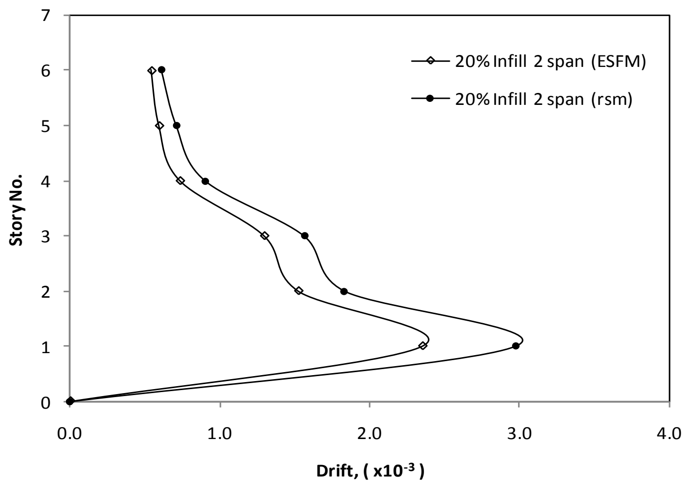


Fig.4.118 Comparison of Drift ESFM vs RSM for 20% Infill 6 Floor and 2 spanned Frame

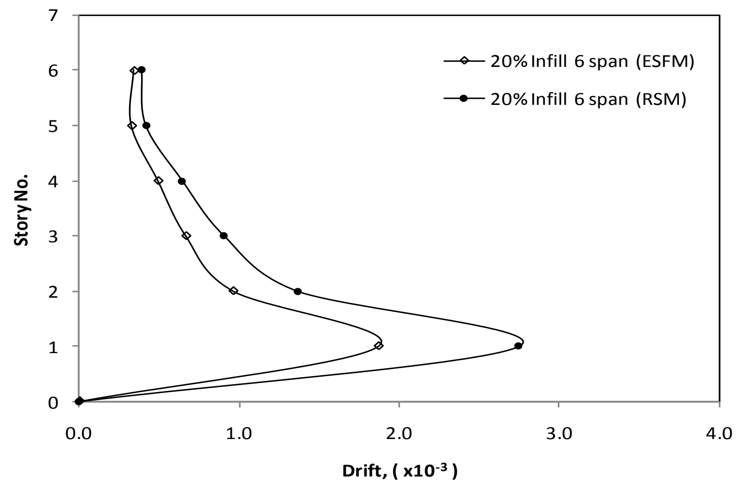


Fig.4.119 Comparison of Drift ESFM vs RSM for 20% Infill 6 Floor and 6 spanned Frame

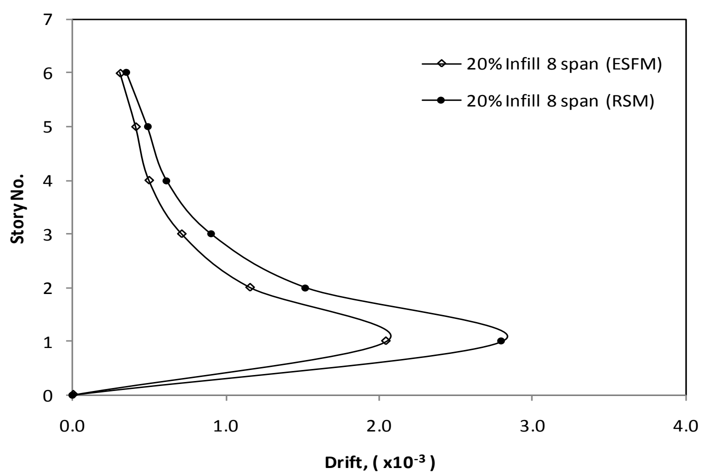


Fig.4.120 Comparison of Drift ESFM vs RSM for 20% Infill 6 Floor and 8 spanned Frame

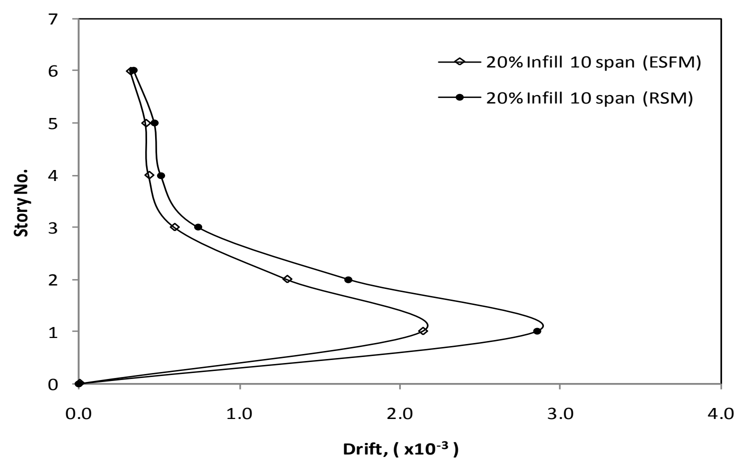


Fig.4.121 Comparison of Drift ESFM vs RSM for 20% Infill 6 Floor and 10 spanned Frame

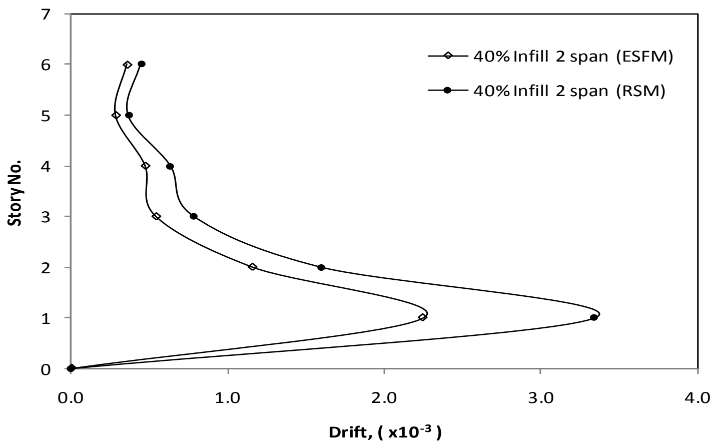


Fig.4.122 Comparison of Drift ESFM vs RSM for 40% Infill 6 Floor and 2 spanned Frame

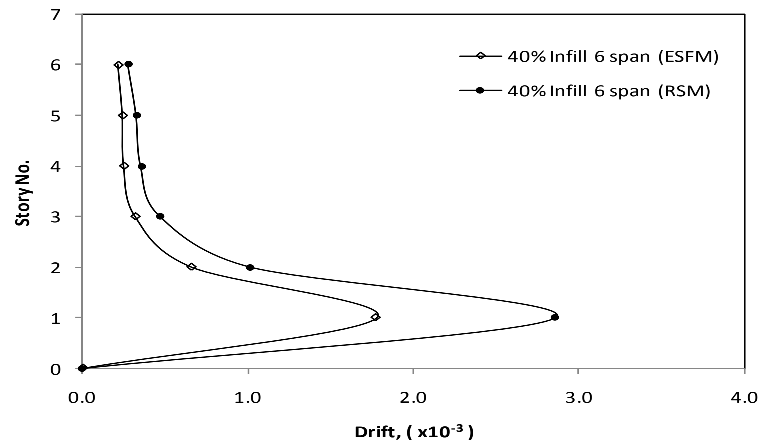


Fig.4.123 Comparison of Drift ESFM vs RSM for 40% Infill 6 Floor and 6 spanned Frame

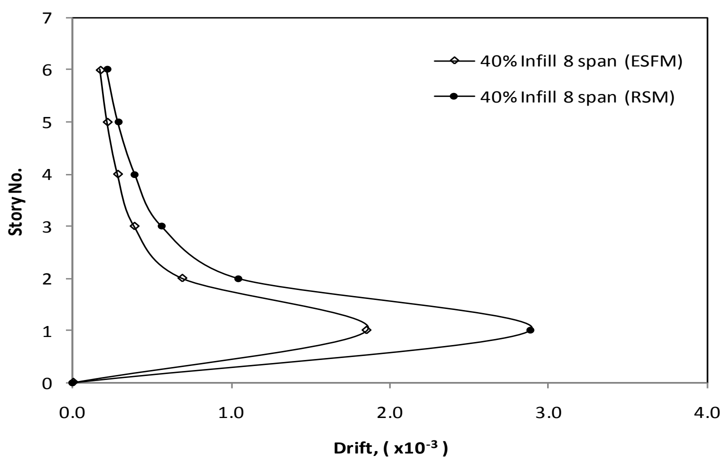


Fig.4.124 Comparison of Drift ESFM vs RSM for 40% Infill 6 Floor and 8 spanned Frame

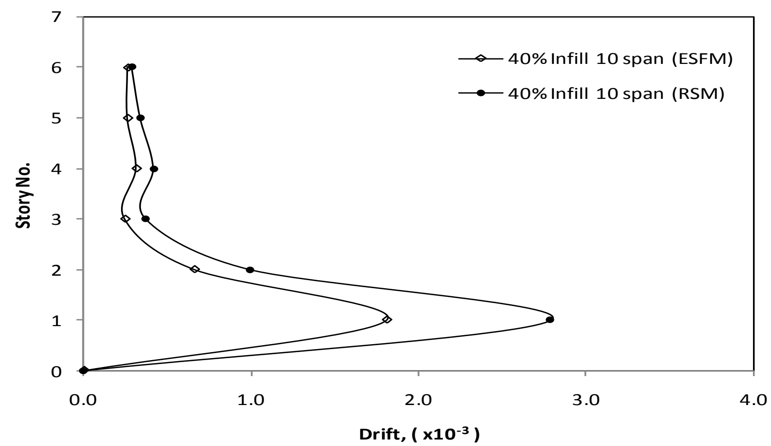


Fig.4.125 Comparison of Drift ESFM vs RSM for 40% Infill 6 Floor and 10 spanned Frame

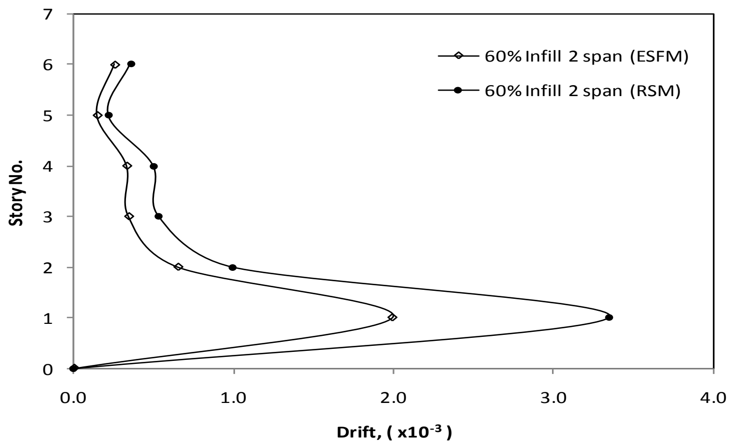


Fig.4.126 Comparison of Drift ESFM vs RSM for 60% Infill 6 Floor and 2 spanned Frame

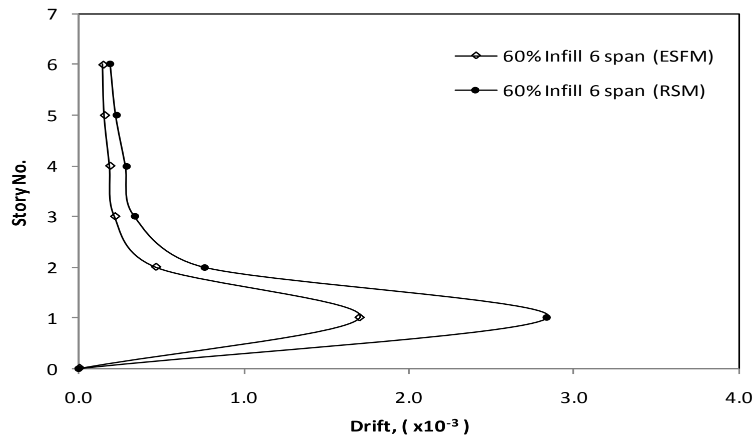


Fig.4.127 Comparison of Drift ESFM vs RSM for 60% Infill 6 Floor and 6 spanned Frame

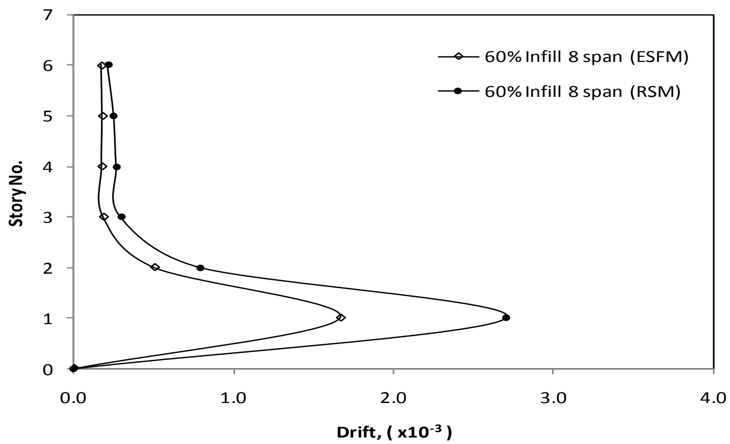


Fig.4.128 Comparison of Drift ESFM vs RSM for 60% Infill 6 Floor and 8 spanned Frame

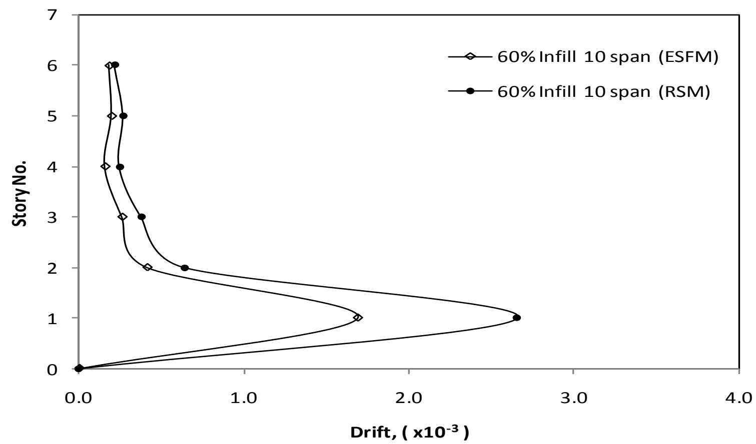


Fig.4.129 Comparison of Drift ESFM vs RSM for 60% Infill 6 Floor and 10 spanned Frame

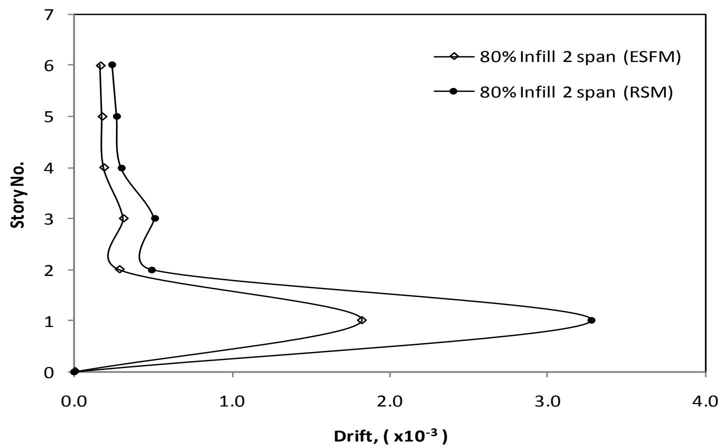


Fig.4.130 Comparison of Drift ESFM vs RSM for 80% Infill 6 Floor and 2 spanned Frame

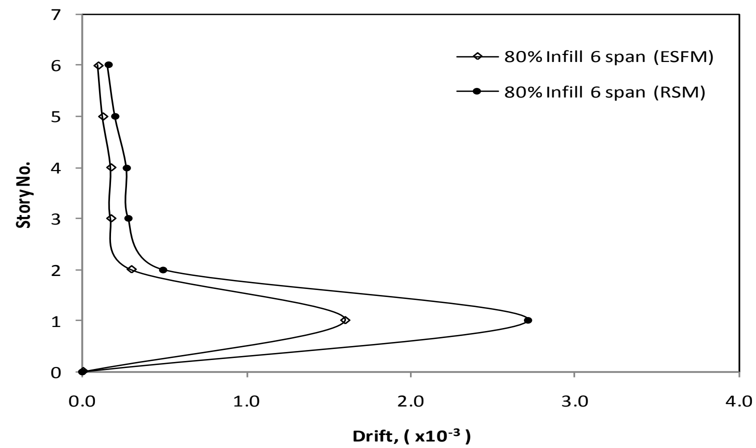


Fig.4.131 Comparison of Drift ESFM vs RSM for 80% Infill 6 Floor and 6 spanned Frame

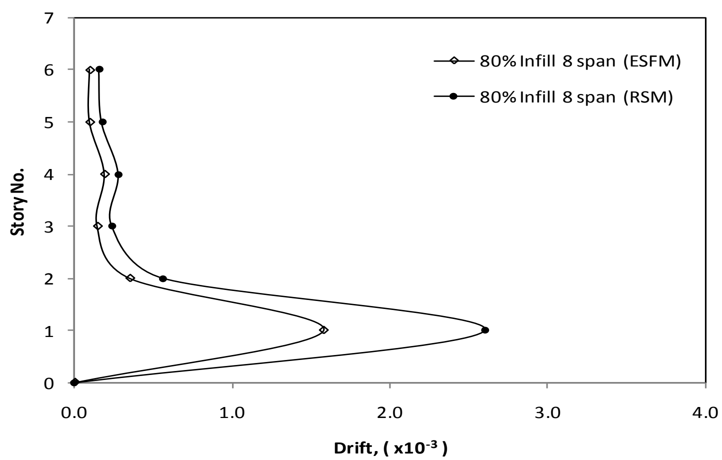


Fig.4.132 Comparison of Drift ESFM vs RSM for 80% Infill 6 Floor and 8 spanned Frame

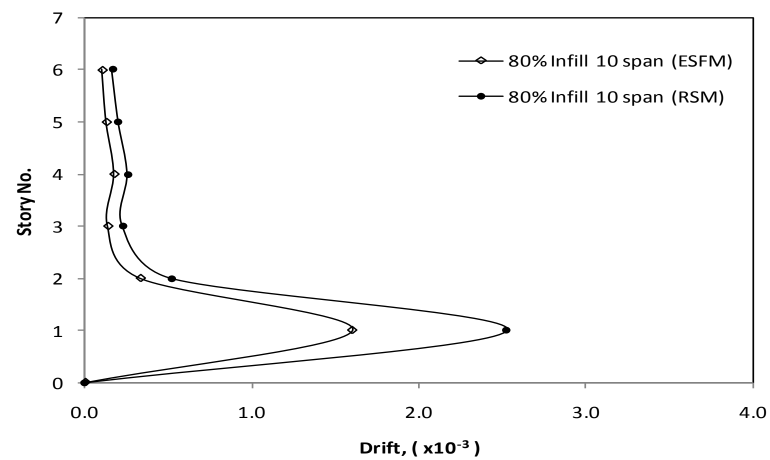


Fig.4.133 Comparison of Drift ESFM vs RSM for 80% Infill 6 Floor and 10 spanned Frame

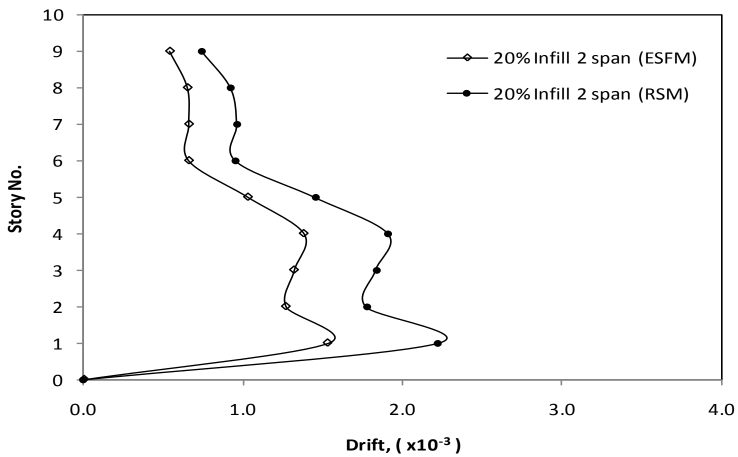


Fig.4.134 Comparison of Drift ESFM vs RSM for 20% Infill 9 Floor and 2 spanned Frame

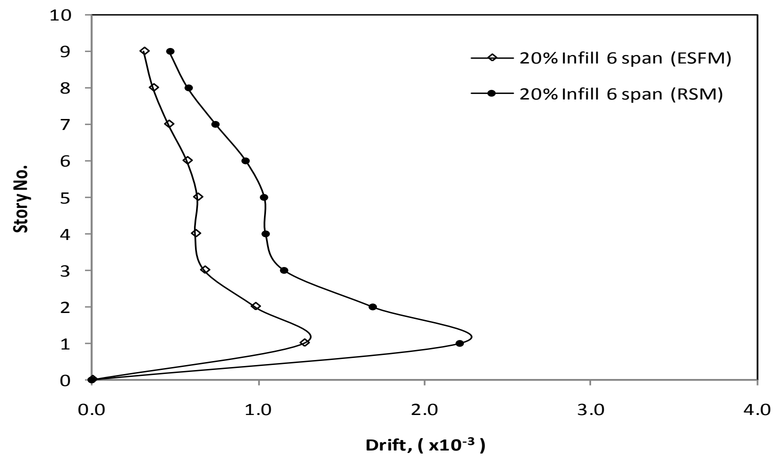


Fig.4.135 Comparison of Drift ESFM vs RSM for 20% Infill 9 Floor and 6 spanned Frame

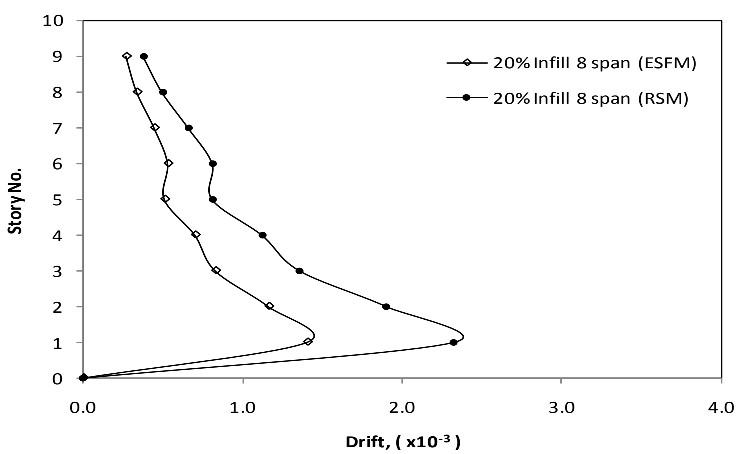


Fig.4.136 Comparison of Drift ESFM vs RSM for 20% Infill 9 Floor and 8 spanned Frame

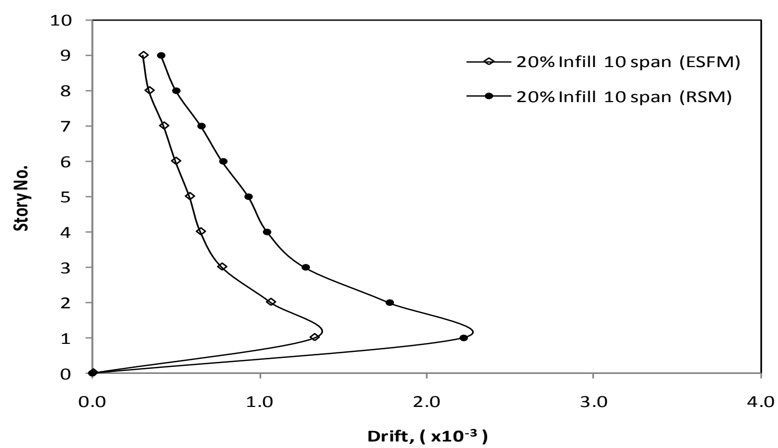


Fig.4.137 Comparison of Drift ESFM vs RSM for 20% Infill 9 Floor and 10 spanned Frame

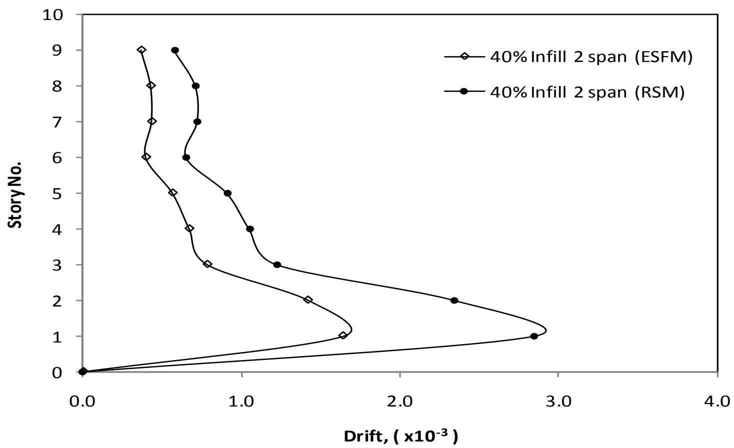


Fig.4.138 Comparison of Drift ESFM vs RSM for 40% Infill 9 Floor and 2 spanned Frame

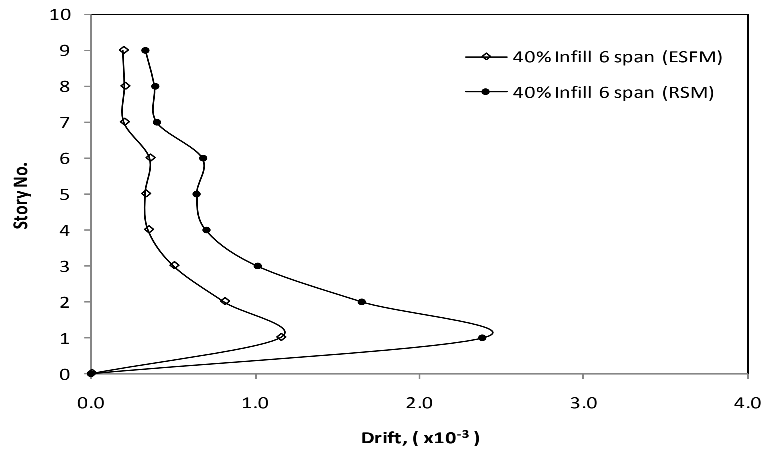


Fig.4.139 Comparison of Drift ESFM vs RSM for 40% Infill 9 Floor and 6 spanned Frame

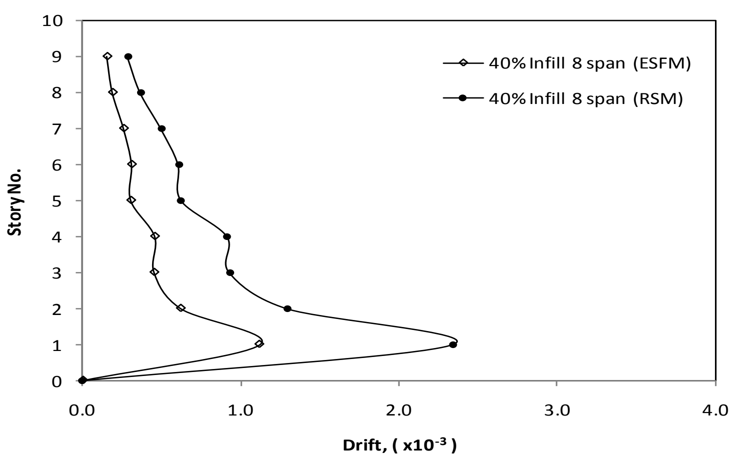


Fig.4.140 Comparison of Drift ESFM vs RSM for 40% Infill 9 Floor and 8 spanned Frame

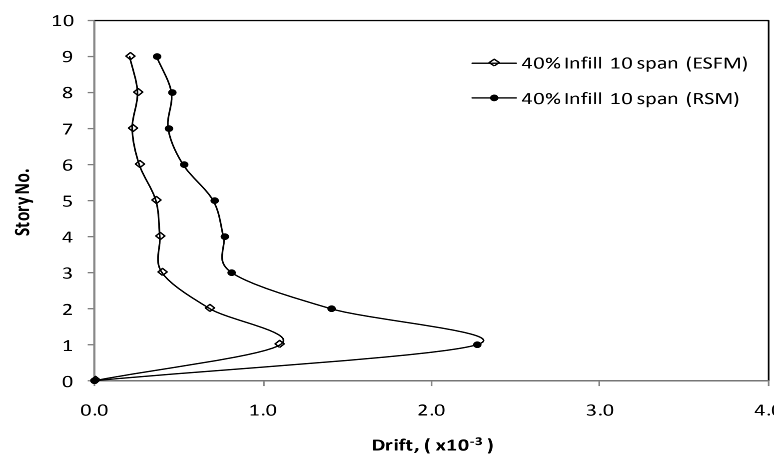


Fig.4.141 Comparison of Drift ESFM vs RSM for 40% Infill 9 Floor and 10 spanned Frame

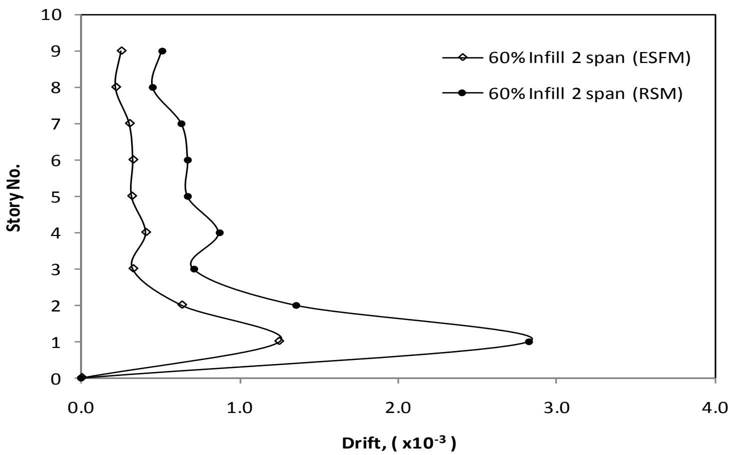


Fig.4.142 Comparison of Drift ESFM vs RSM for 60% Infill 9 Floor and 2 spanned Frame

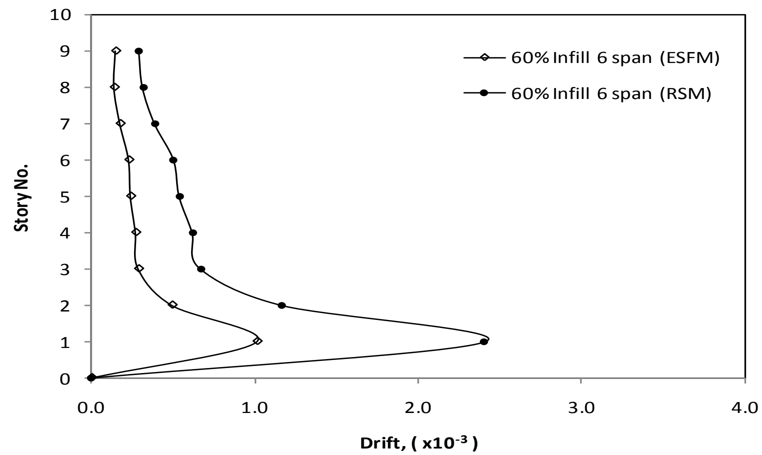


Fig.4.143 Comparison of Drift ESFM vs RSM for 60% Infill 9 Floor and 6 spanned Frame

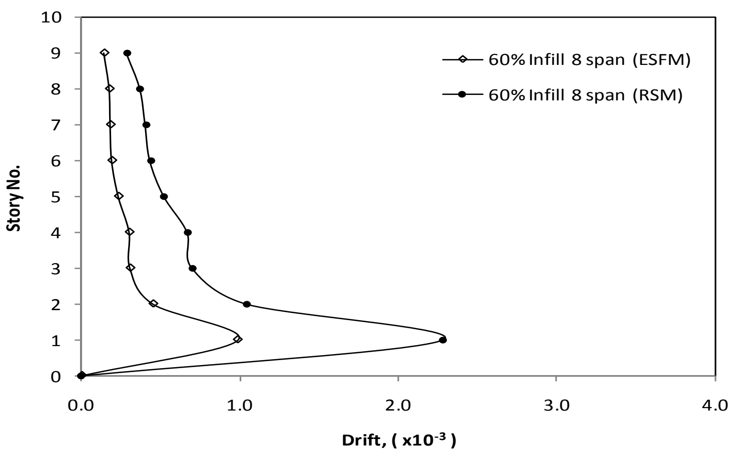


Fig.4.144 Comparison of Drift ESFM vs RSM for 60% Infill 9 Floor and 8 spanned Frame

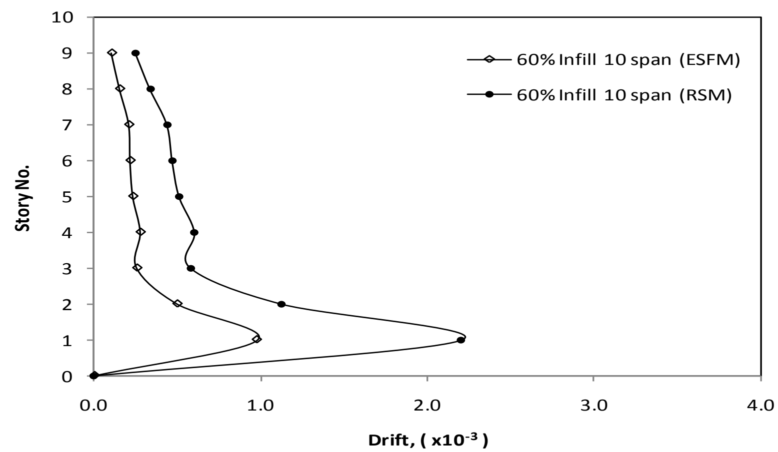


Fig.4.145 Comparison of Drift ESFM vs RSM for 60% Infill 9 Floor and 10 spanned Frame

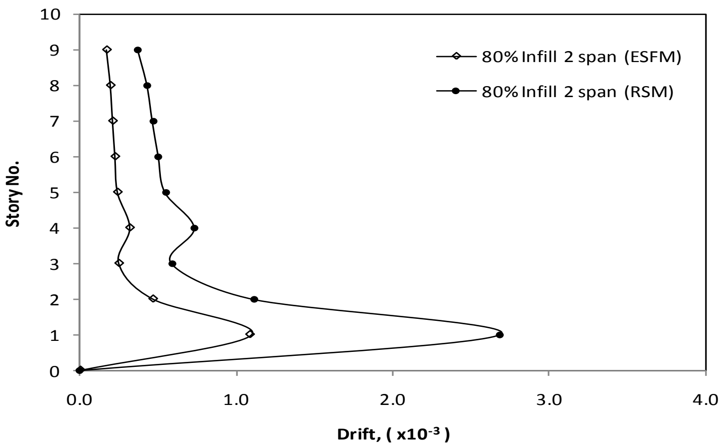


Fig.4.146 Comparison of Drift ESFM vs RSM for 80% Infill 9 Floor and 2 spanned Frame

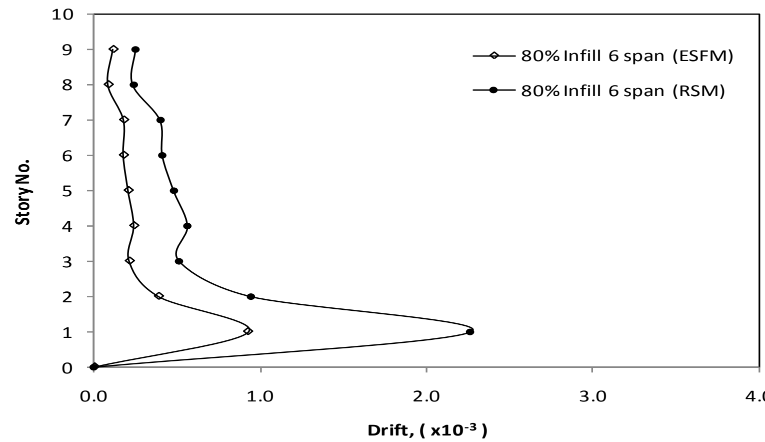


Fig.4.147 Comparison of Drift ESFM vs RSM for 80% Infill 9 Floor and 6 spanned Frame

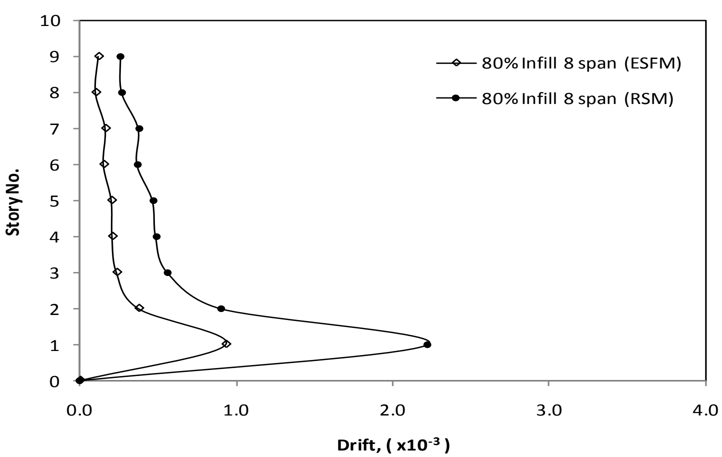


Fig.4.148 Comparison of Drift ESFM vs RSM for 80% Infill 9 Floor and 8 spanned Frame

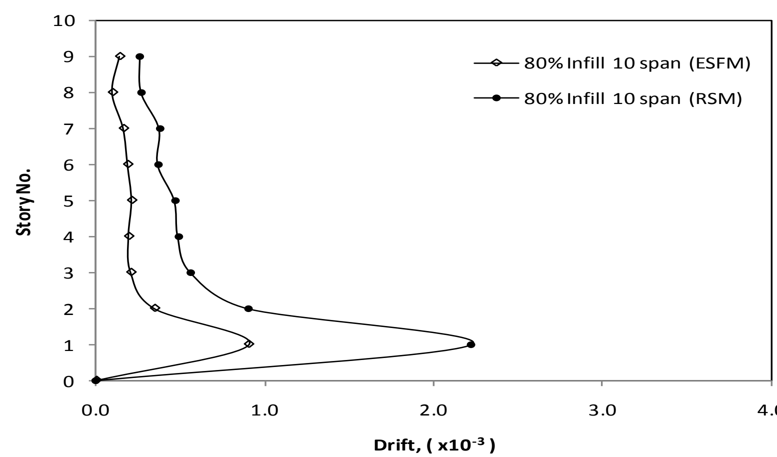
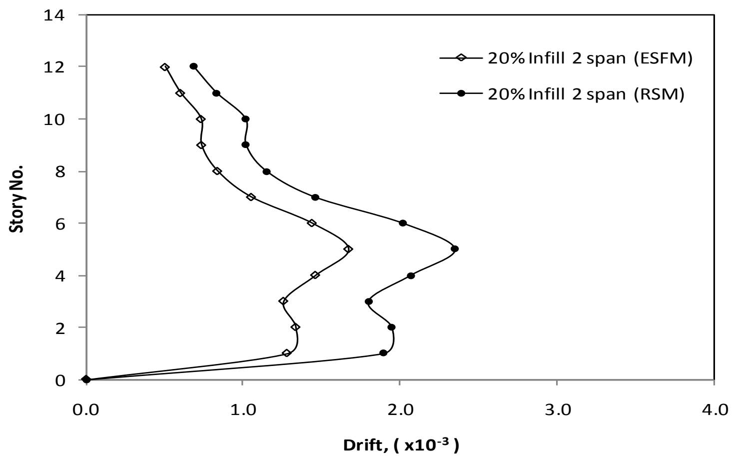


Fig.4.149 Comparison of Drift ESFM vs RSM for 80% Infill 9 Floor and 10 spanned Frame



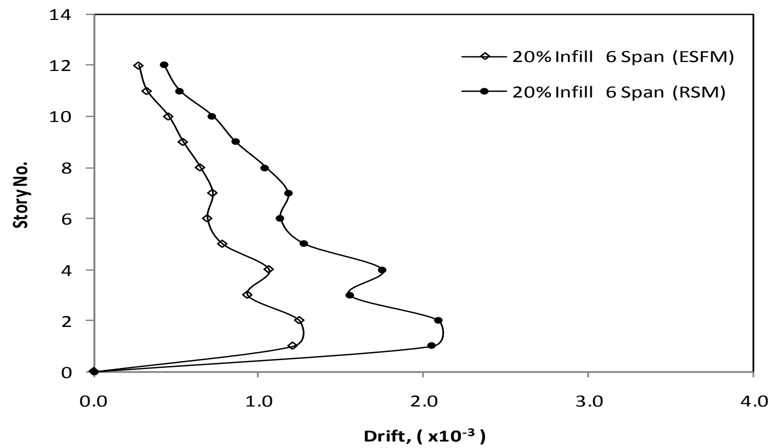
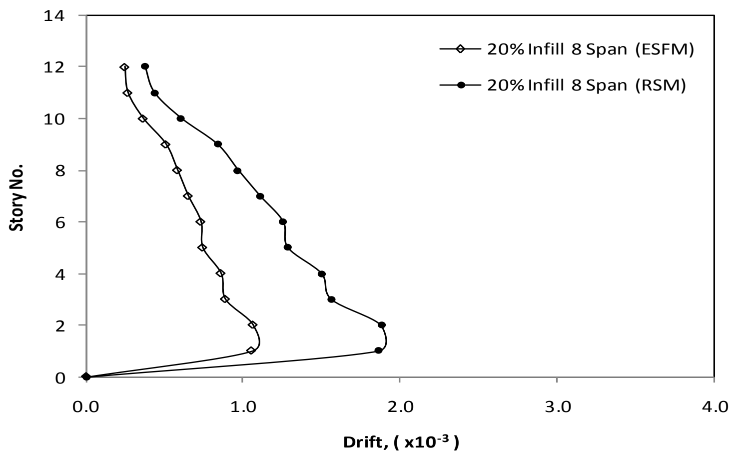


Fig.4.150 Comparison of Drift ESFM vs RSM for 20% Infill 12 Floor and 2 spanned Frame

Fig.4.151 Comparison of Drift ESFM vs RSM for 20% Infill 12 Floor and 6 spanned Frame



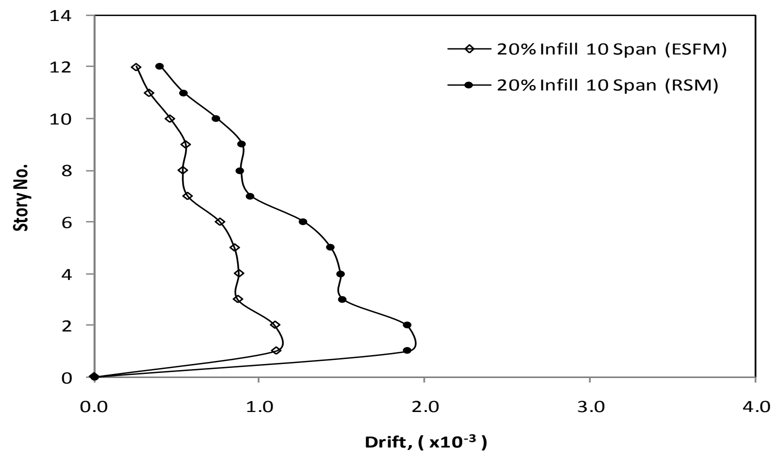


Fig.4.152 Comparison of Drift ESFM vs RSM for 20% Infill 12 Floor and 8 spanned Frame

Fig.4.153 Comparison of Drift ESFM vs RSM for 20% Infill 12 Floor and 10 spanned Frame

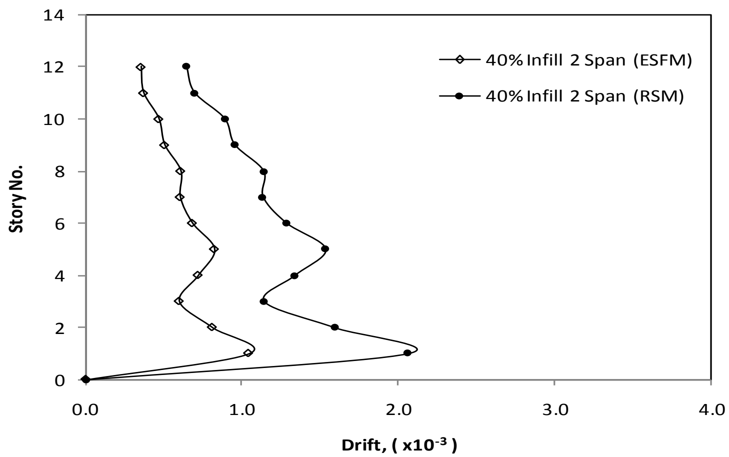


Fig.4.154 Comparison of Drift ESFM vs RSM for 40% Infill 12 Floor and 2 spanned Frame

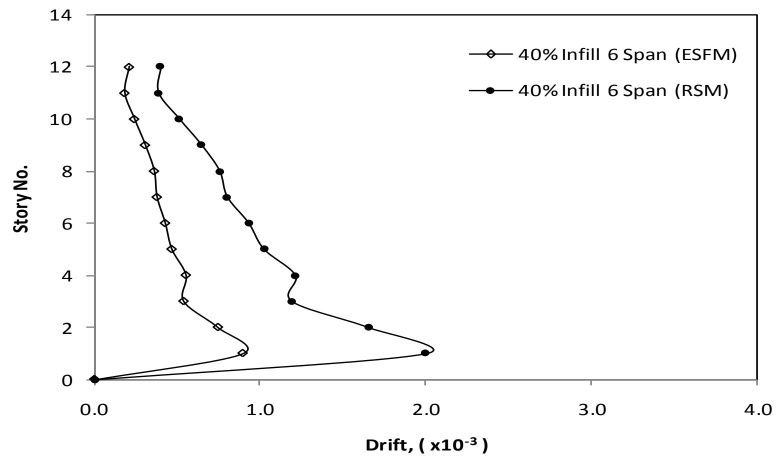


Fig.4.155 Comparison of Drift ESFM vs RSM for 40% Infill 12 Floor and 6 spanned Frame

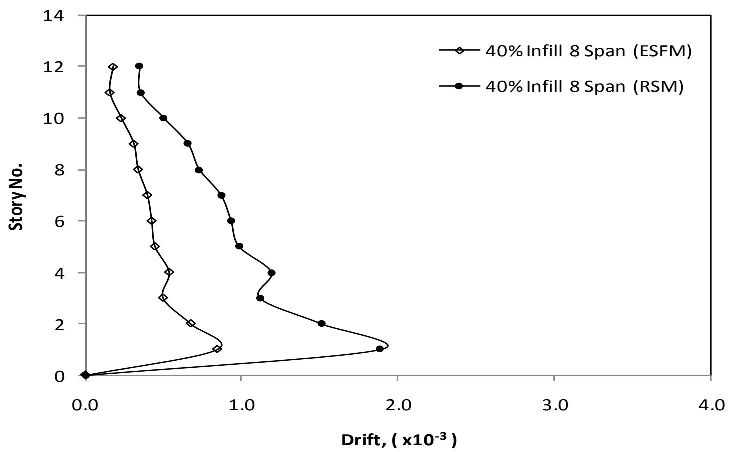


Fig.4.156 Comparison of Drift ESFM vs RSM for 40% Infill 12 Floor and 8 spanned Frame

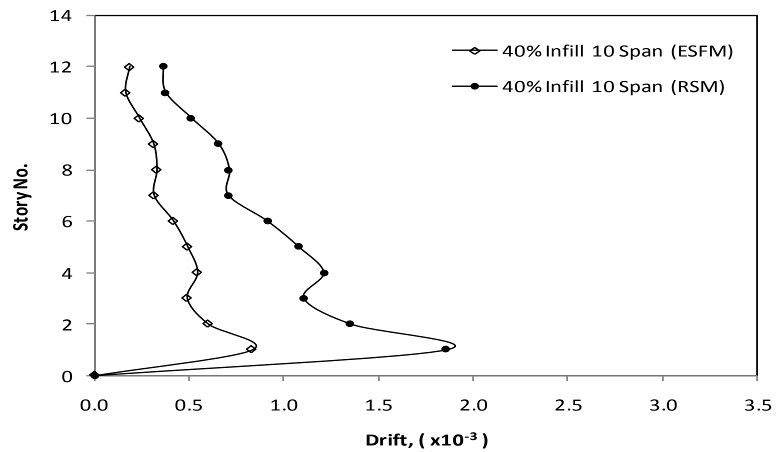


Fig.4.157 Comparison of Drift ESFM vs RSM for 40% Infill 12 Floor and 10 spanned Frame

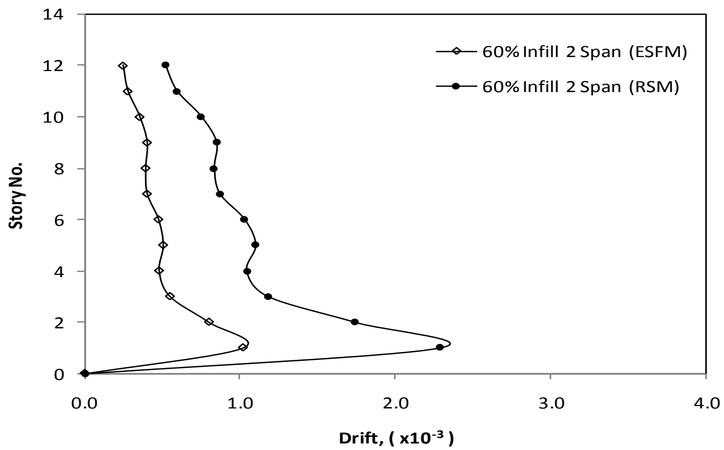


Fig.4.158 Comparison of Drift ESFM vs RSM for 60% Infill 12 Floor and 2 spanned Frame

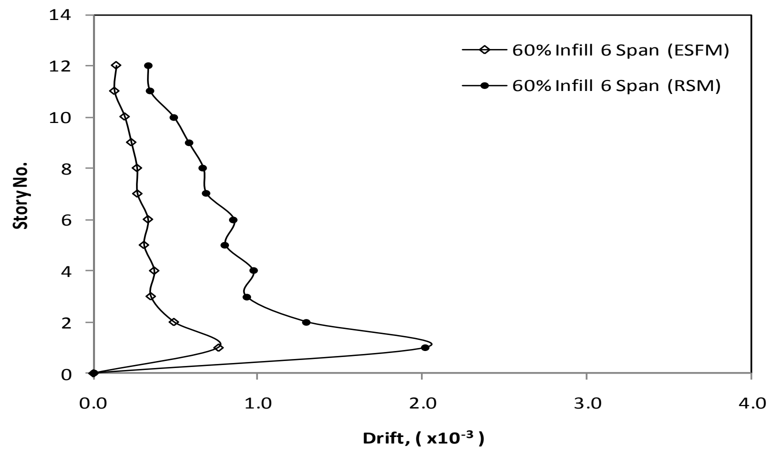


Fig.4.159 Comparison of Drift ESFM vs RSM for 60% Infill 12 Floor and 6 spanned Frame

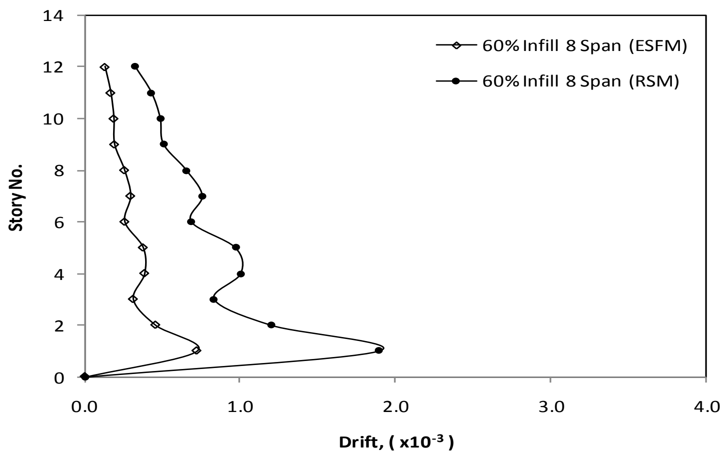


Fig.4.160 Comparison of Drift ESFM vs RSM for 60% Infill 12 Floor and 8 spanned Frame

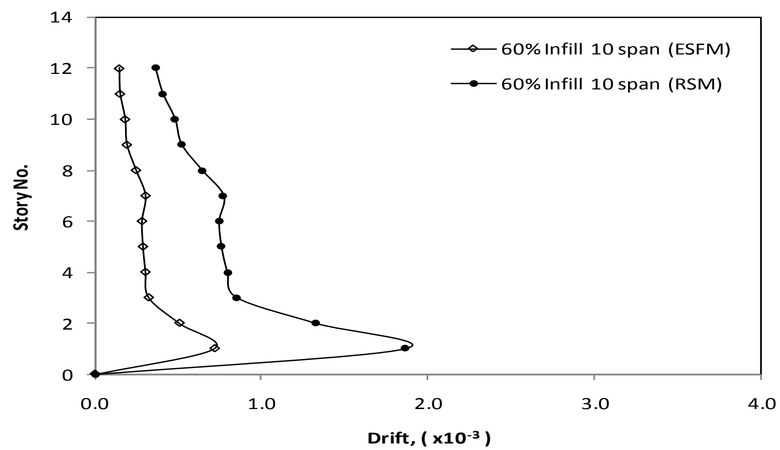


Fig.4.161 Comparison of Drift ESFM vs RSM for 60% Infill 12 Floor and 10 spanned Frame

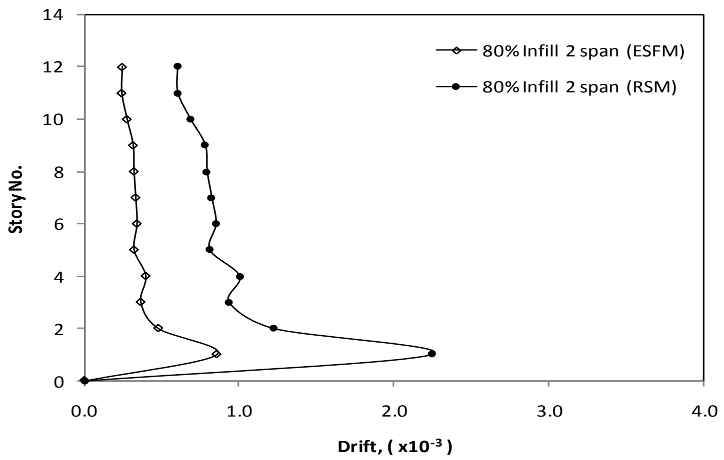


Fig.4.162 Comparison of Drift ESFM vs RSM for 80% Infill 12 Floor and 2 spanned Frame

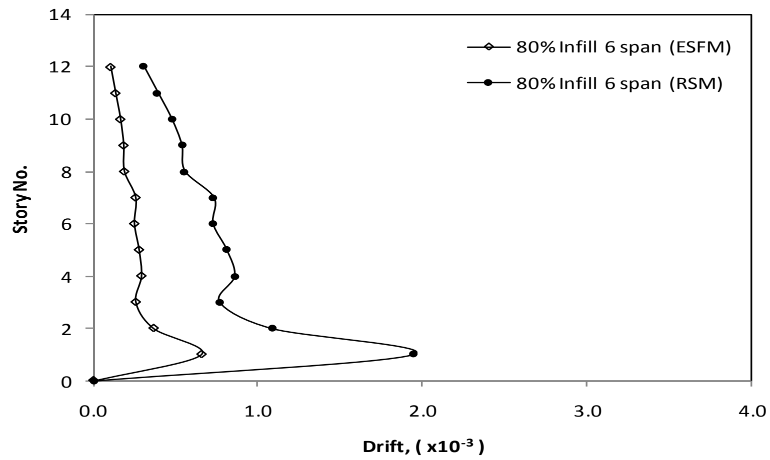


Fig.4.163 Comparison of Drift ESFM vs RSM for 80% Infill 12 Floor and 6 spanned Frame

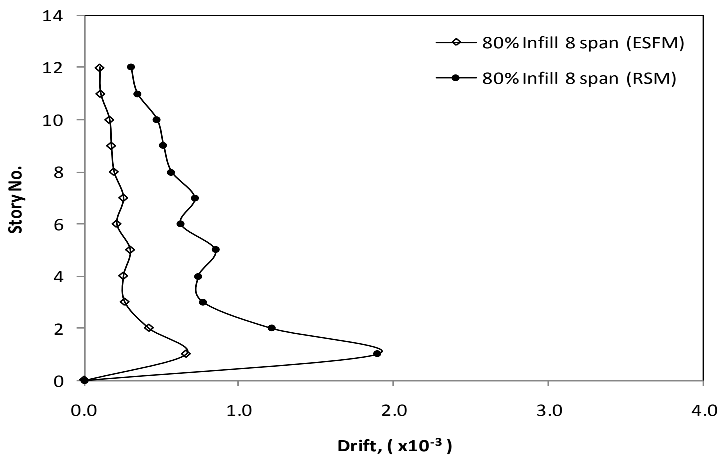


Fig.4.164 Comparison of Drift ESFM vs RSM for 80% Infill 12 Floor and 8 spanned Frame

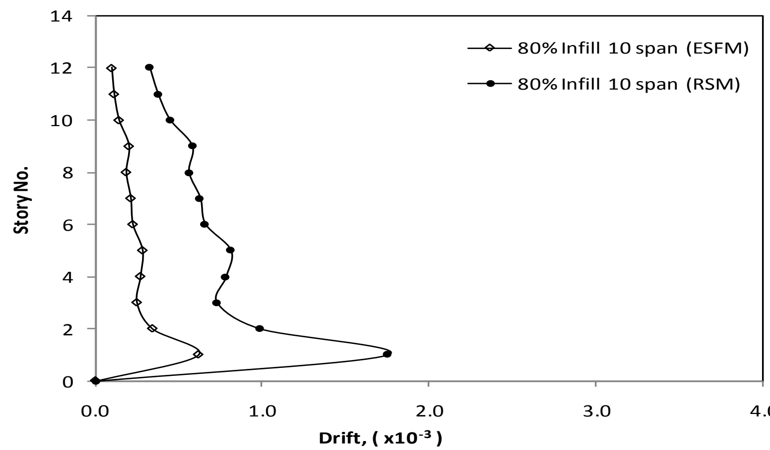


Fig.4.165 Comparison of Drift ESFM vs RSM for 80% Infill 12 Floor and 10 spanned Frame

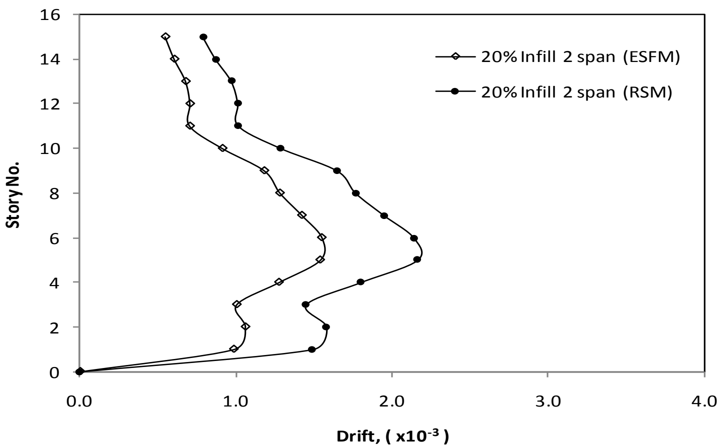


Fig.4.166 Comparison of Drift ESFM vs RSM for 20% Infill 15 Floor and 2 spanned Frame

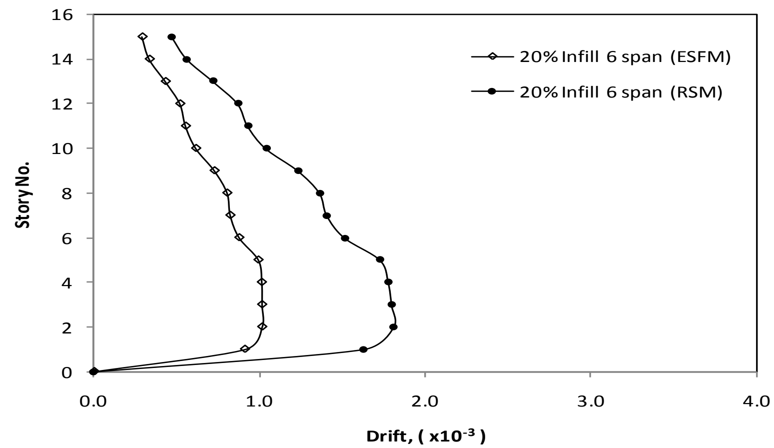


Fig.4.167 Comparison of Drift ESFM vs RSM for 20% Infill 15 Floor and 6 spanned Frame

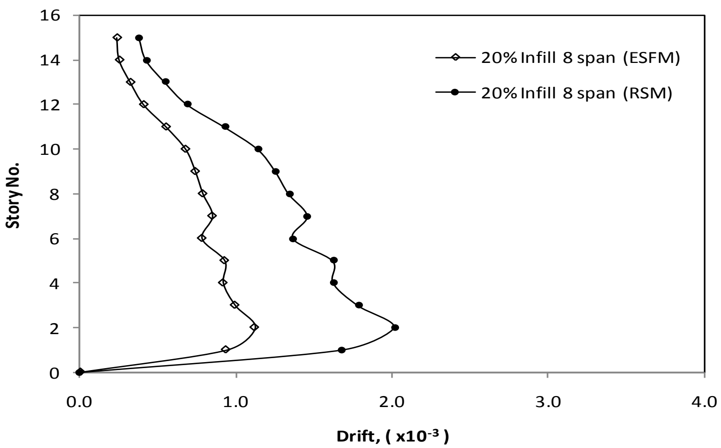


Fig.4.168 Comparison of Drift ESFM vs RSM for 20% Infill 15 Floor and 8 spanned Frame

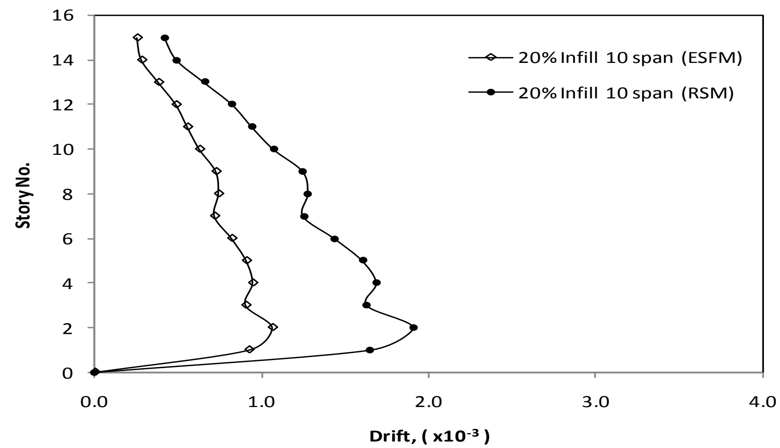


Fig.4.169 Comparison of Drift ESFM vs RSM for 20% Infill 15 Floor and 10 spanned Frame

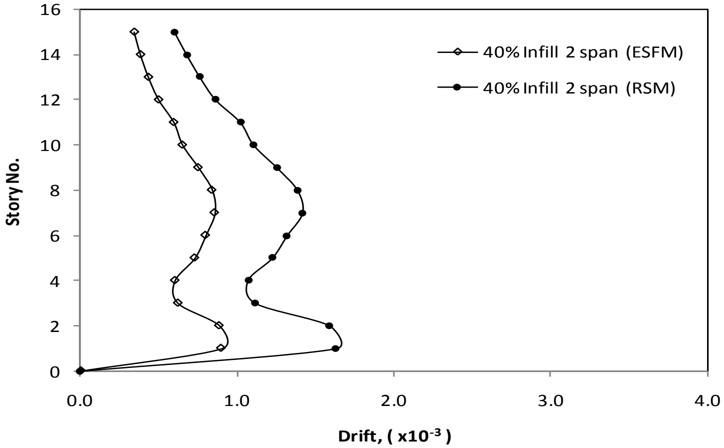


Fig.4.170 Comparison of Drift ESFM vs RSM for 40% Infill 15 Floor and 2 spanned Frame

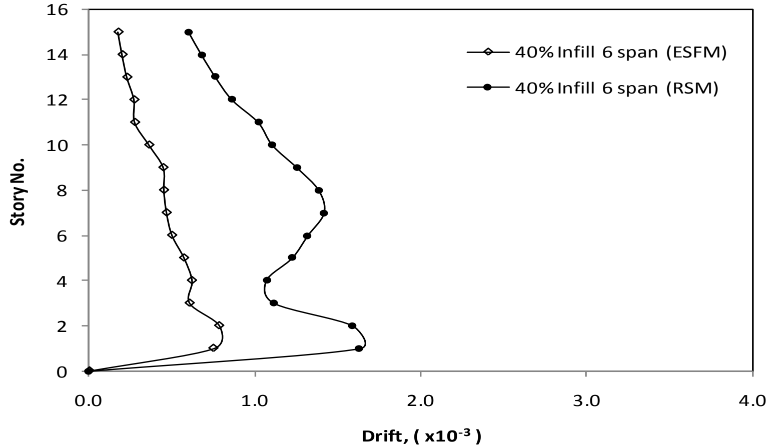


Fig.4.171 Comparison of Drift ESFM vs RSM for 40% Infill 15 Floor and 6 spanned Frame

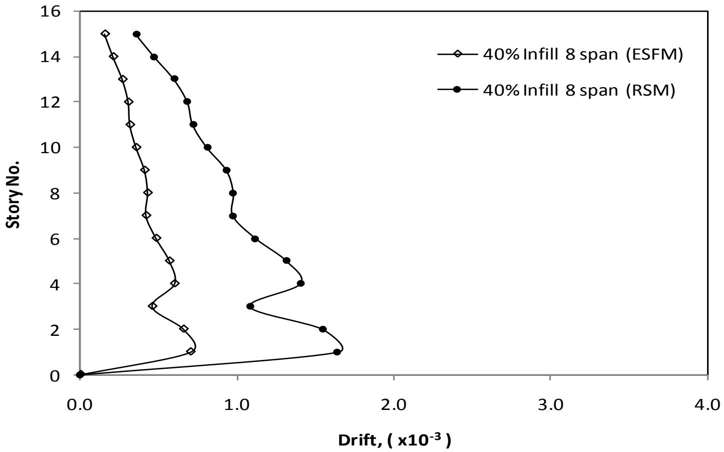


Fig.4.172 Comparison of Drift ESFM vs RSM for 40% Infill 15 Floor and 8 spanned Frame

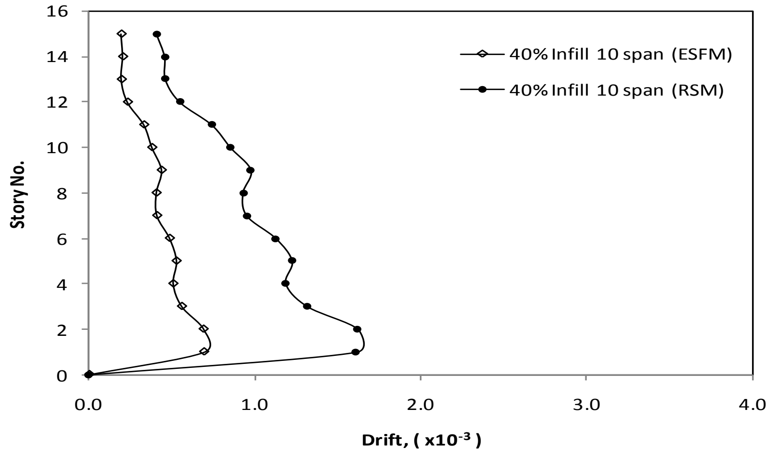


Fig.4.173 Comparison of Drift ESFM vs RSM for 40% Infill 15 Floor and 10 spanned Frame

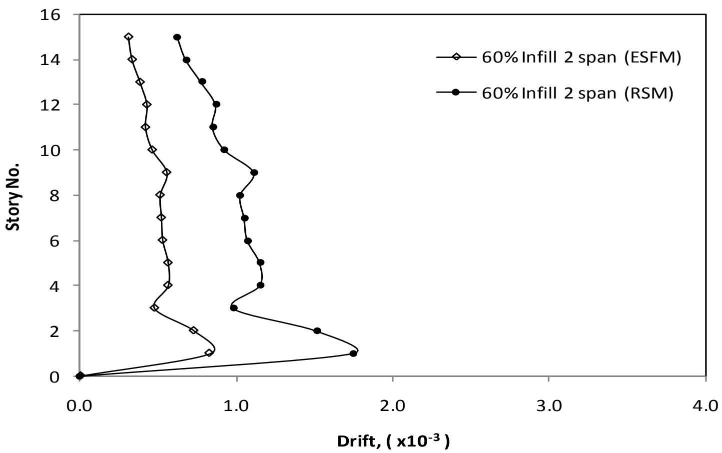


Fig.4.174 Comparison of Drift ESFM vs RSM for 60% Infill 15 Floor and 2 spanned Frame

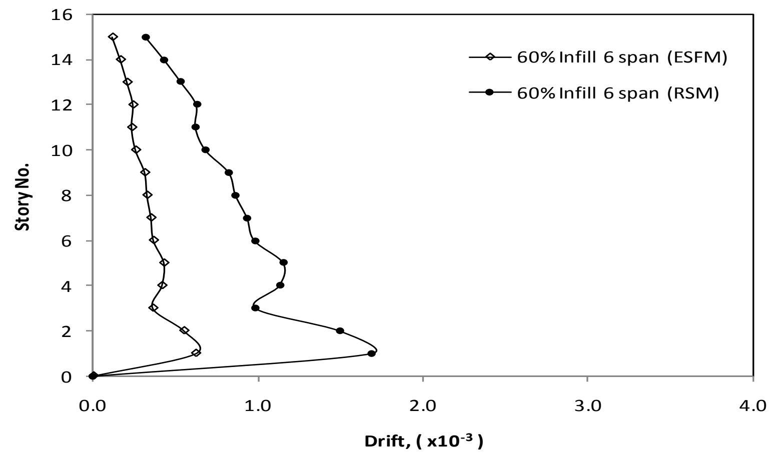


Fig.4.175 Comparison of Drift ESFM vs RSM for 60% Infill 15 Floor and 6 spanned Frame

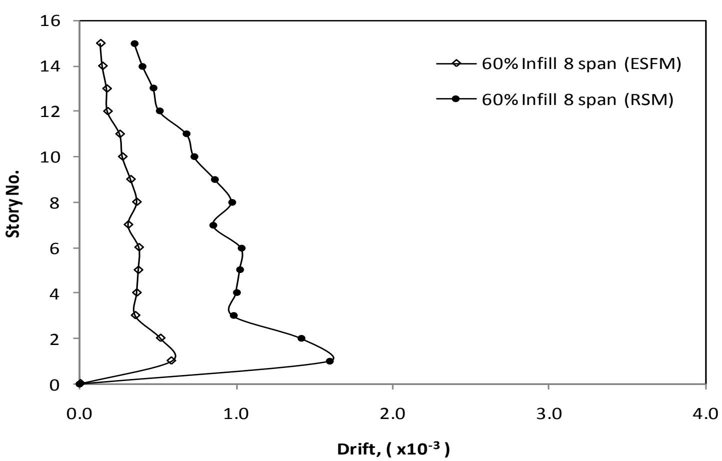


Fig.4.176 Comparison of Drift ESFM vs RSM for 60% Infill 15 Floor and 8 spanned Frame

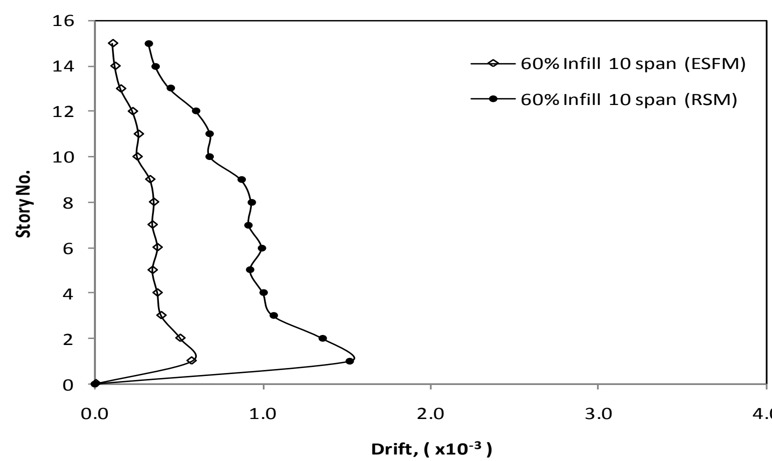


Fig.4.177 Comparison of Drift ESFM vs RSM for 60% Infill 15 Floor and 10 spanned Frame

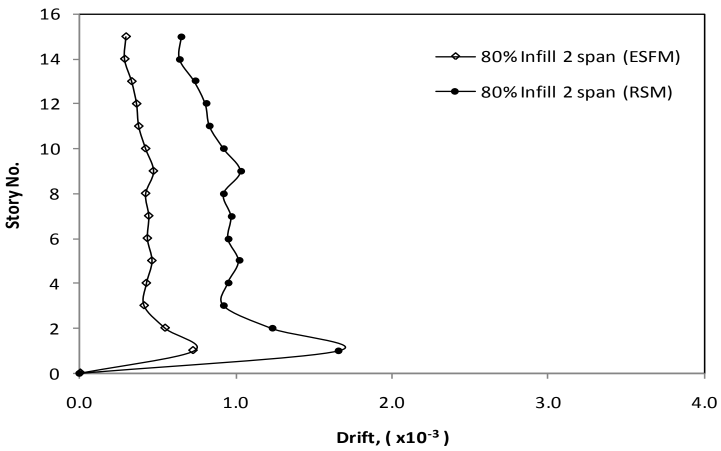


Fig.4.178 Comparison of Drift ESFM vs RSM for 80% Infill 15 Floor and 2 spanned Frame

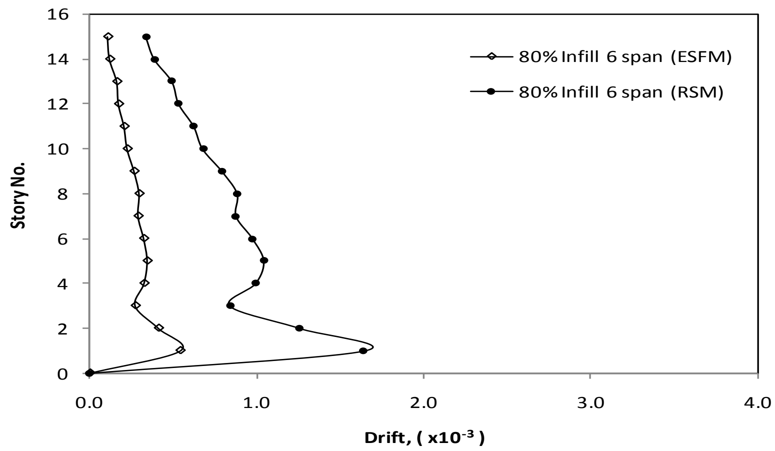


Fig.4.179 Comparison of Drift ESFM vs RSM for 80% Infill 15 Floor and 6 spanned Frame

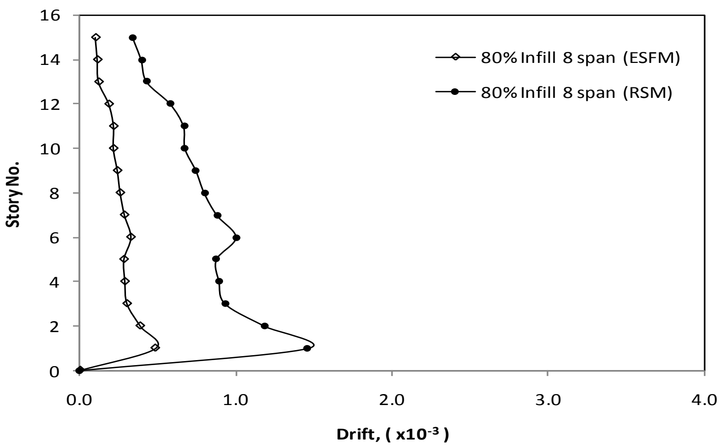


Fig.4.180 Comparison of Drift ESFM vs RSM for 80% Infill 15 Floor and 8 spanned Frame

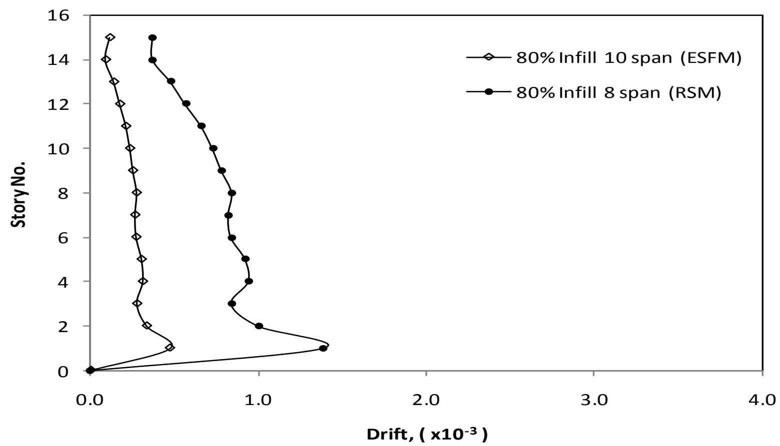


Fig.4.181 Comparison of Drift ESFM vs RSM for 80% Infill 15 Floor and 10 spanned Frame

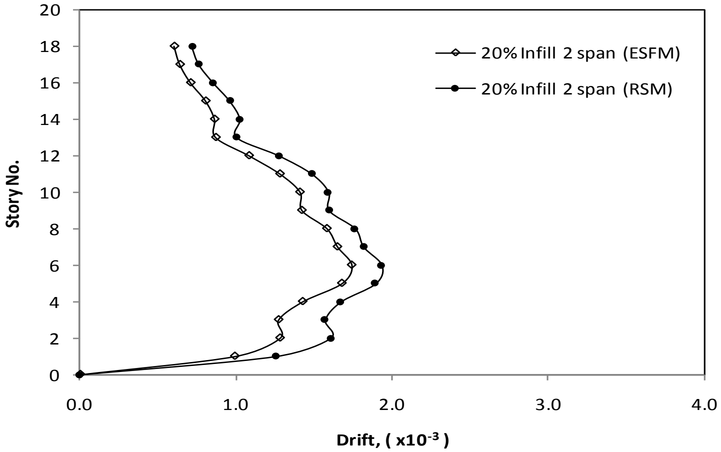


Fig.4.182 Comparison of Drift ESFM vs RSM for 20% Infill 18 Floor and 2 spanned Frame

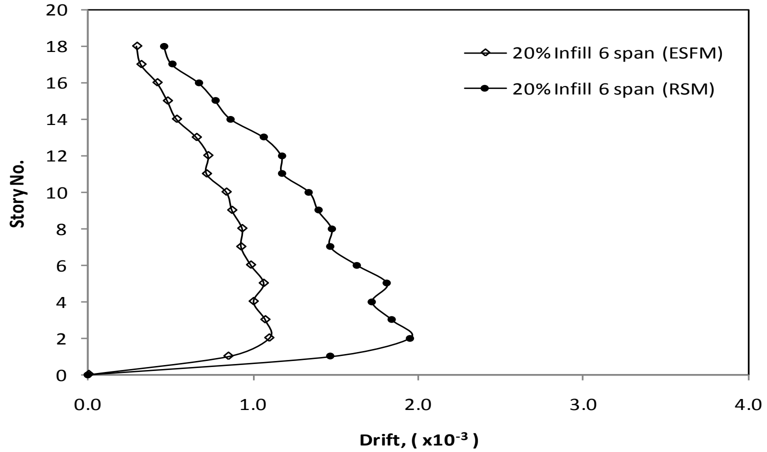


Fig.4.183 Comparison of Drift ESFM vs RSM for 20% Infill 18 Floor and 6 spanned Frame

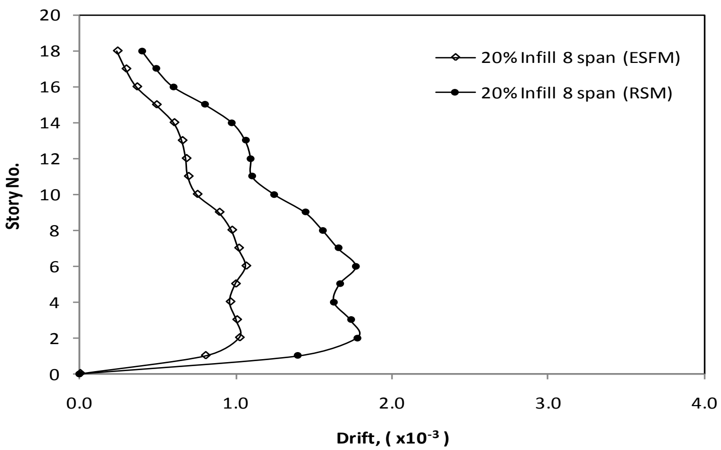


Fig.4.184 Comparison of Drift ESFM vs RSM for 20% Infill 18 Floor and 8 spanned Frame

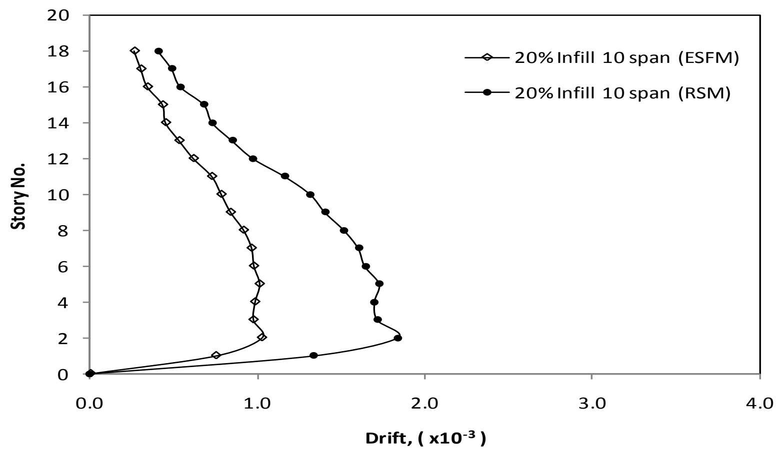


Fig.4.185 Comparison of Drift ESFM vs RSM for 20% Infill 18 Floor and 10 spanned Frame

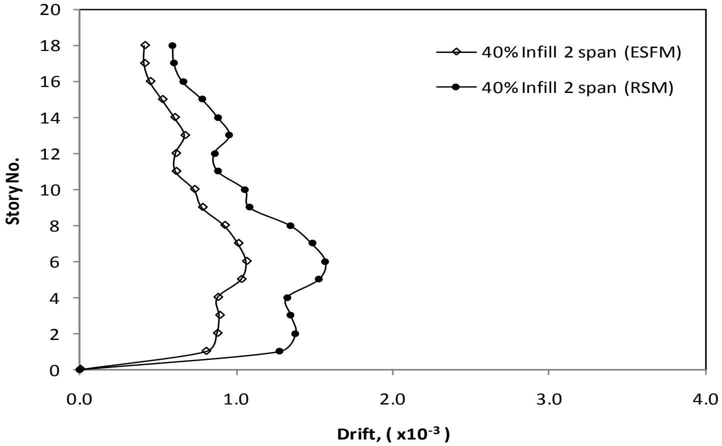


Fig.4.186 Comparison of Drift ESFM vs RSM for 40% Infill 18 Floor and 2 spanned Frame

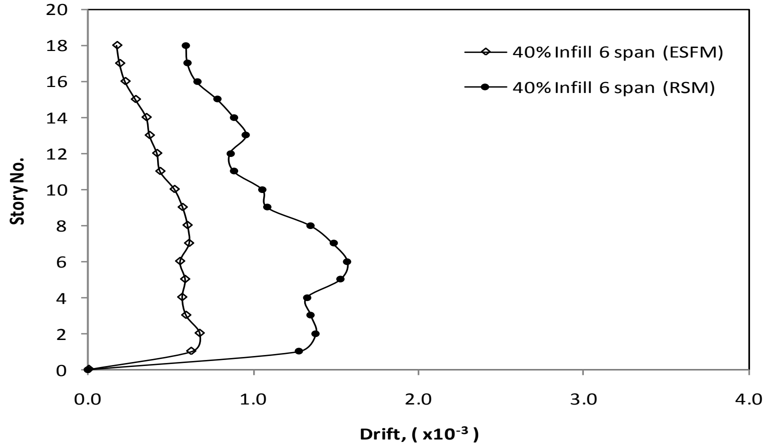


Fig.4.187 Comparison of Drift ESFM vs RSM for 40% Infill 18 Floor and 6 spanned Frame

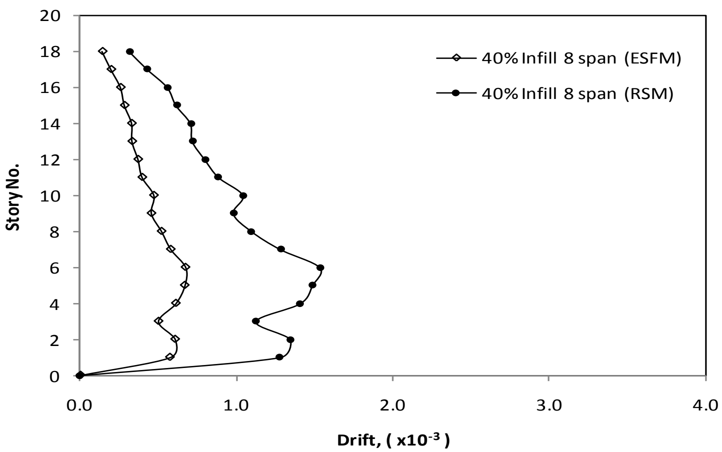


Fig.4.188 Comparison of Drift ESFM vs RSM for 40% Infill 18 Floor and 8 spanned Frame

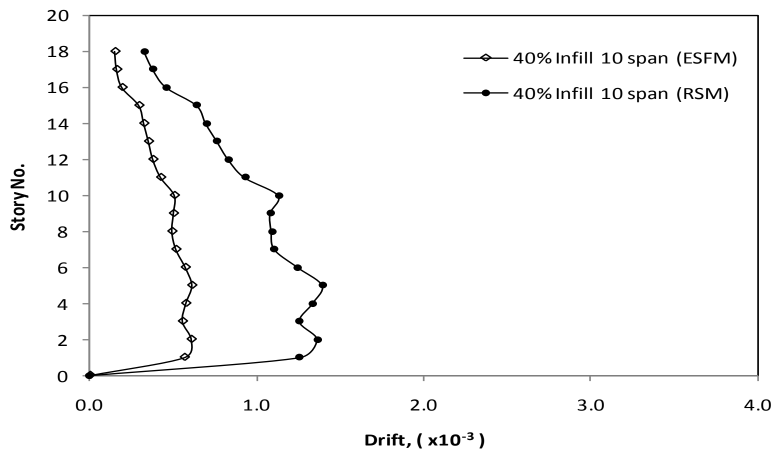


Fig.4.189 Comparison of Drift ESFM vs RSM for 40% Infill 18 Floor and 10 spanned Frame

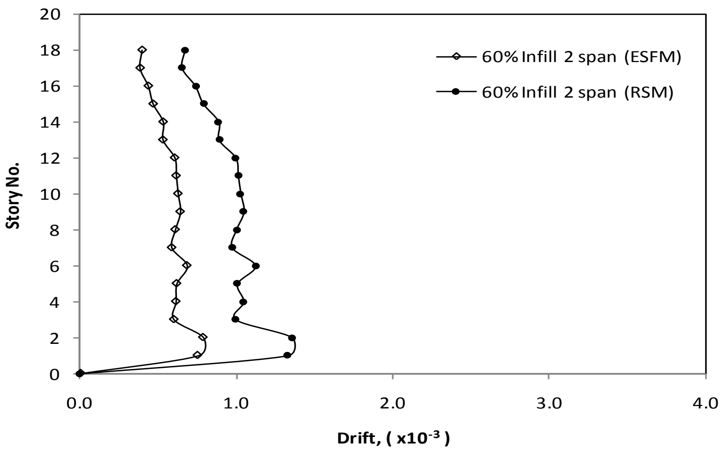


Fig.4.190 Comparison of Drift ESFM vs RSM for 60% Infill 18 Floor and 2 spanned Frame

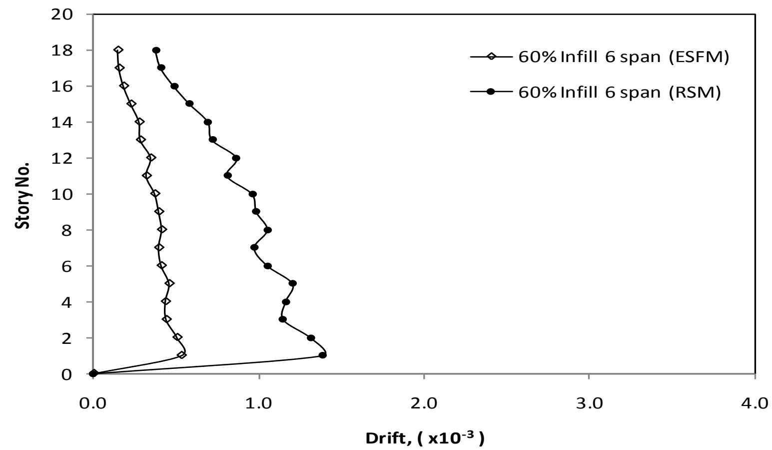


Fig.4.191 Comparison of Drift ESFM vs RSM for 60% Infill 18 Floor and 6 spanned Frame

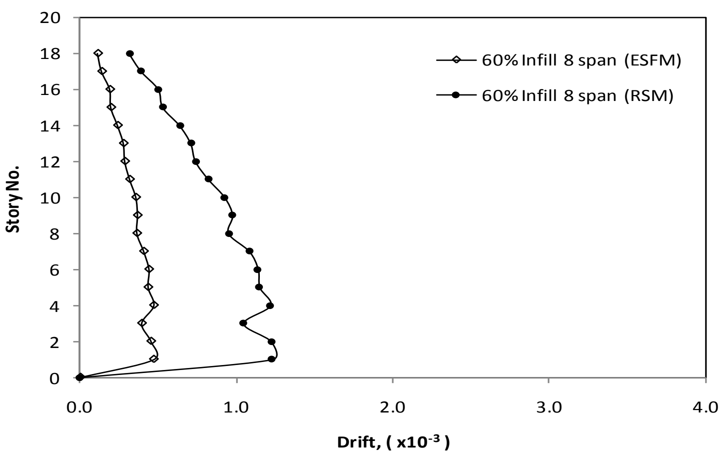


Fig.4.192 Comparison of Drift ESFM vs RSM for 60% Infill 18 Floor and 8 spanned Frame

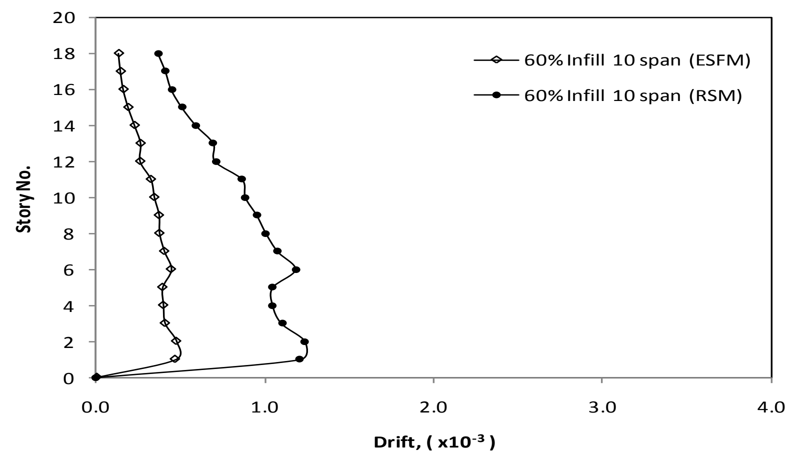


Fig.4.193 Comparison of Drift ESFM vs RSM for 60% Infill 18 Floor and 10 spanned Frame

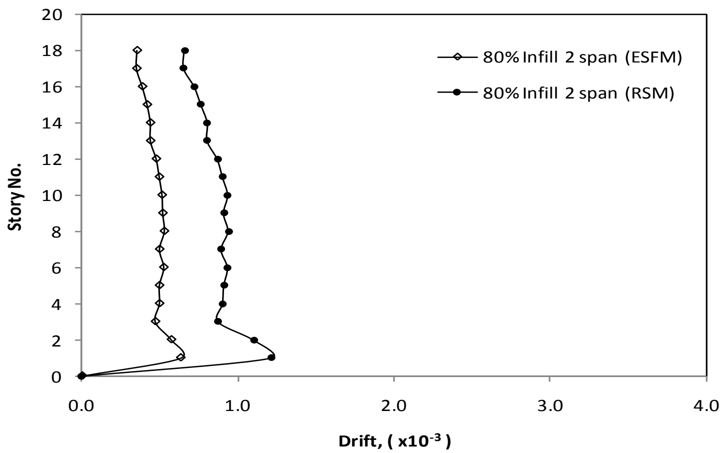


Fig.4.194 Comparison of Drift ESFM vs RSM for 80% Infill 18 Floor and 2 spanned Frame

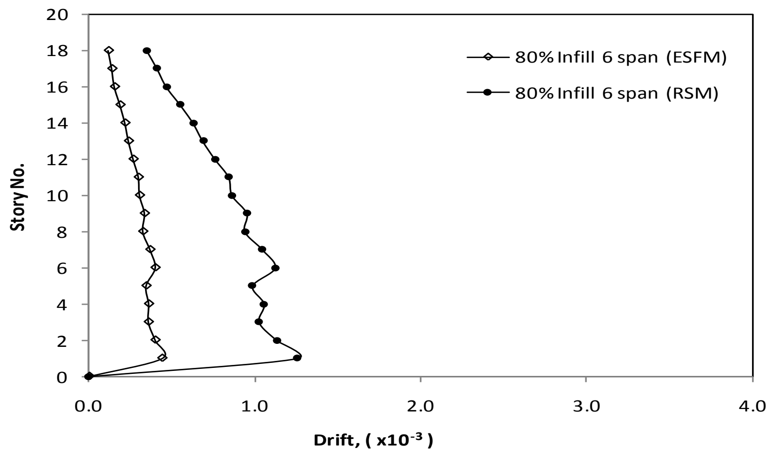


Fig.4.195 Comparison of Drift ESFM vs RSM for 80% Infill 18 Floor and 6 spanned Frame

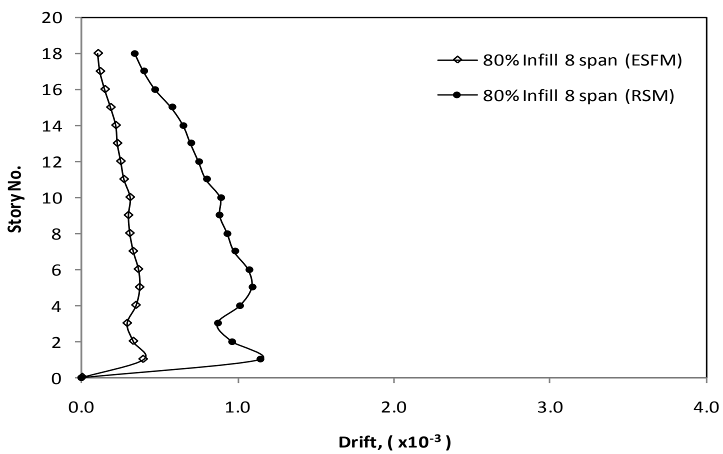


Fig.4.196 Comparison of Drift ESFM vs RSM for 80% Infill 18 Floor and 8 spanned Frame

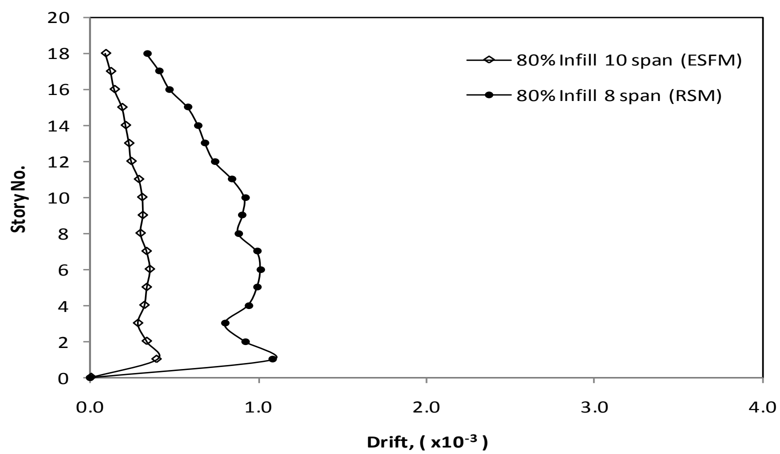


Fig.4.197 Comparison of Drift ESFM vs RSM for 80% Infill 18 Floor and 10 spanned Frame

<u>Appendix-D</u>

Effect of Variation of Slenderness (H/L) on Base shear

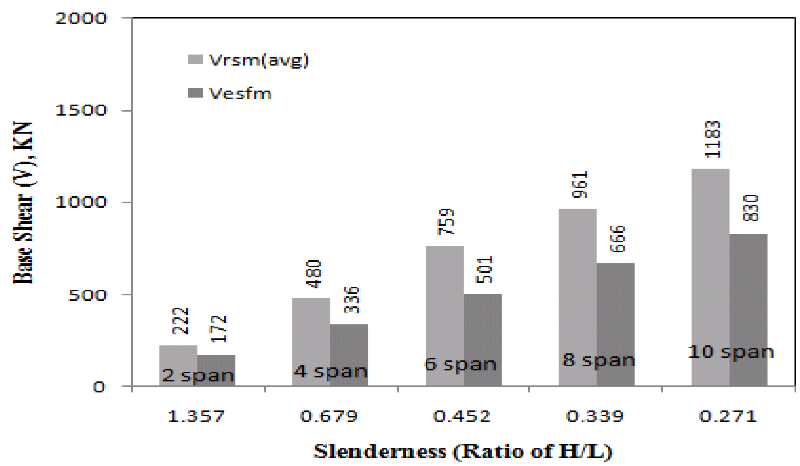


Fig.4.198 Comparison of Base Shear with Slenderness for 20% Infill 6 Floor

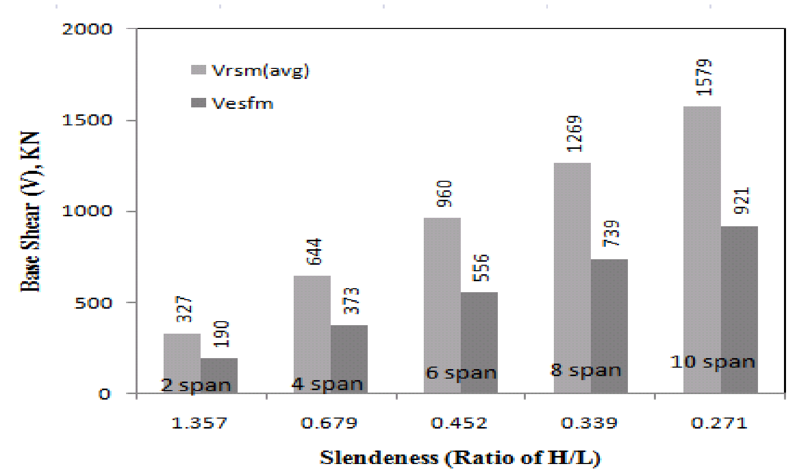


Fig.4.199 Comparison of Base Shear with Slenderness for 60% Infill 6 Floor

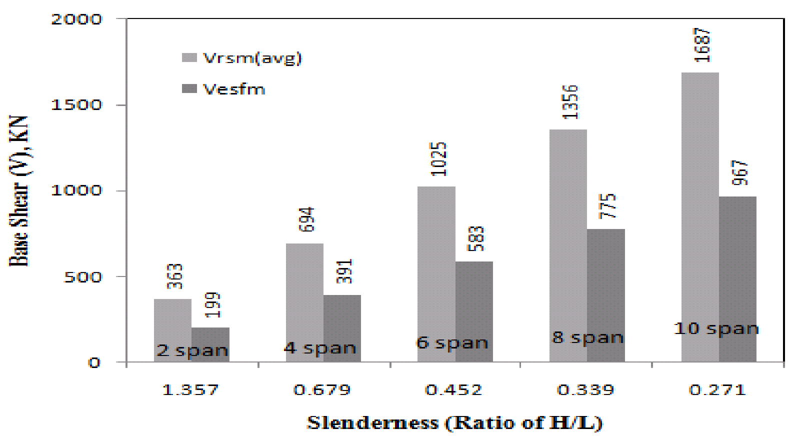


Fig.4.200 Comparison of Base Shear with Slenderness for 80% Infill 6 Floor

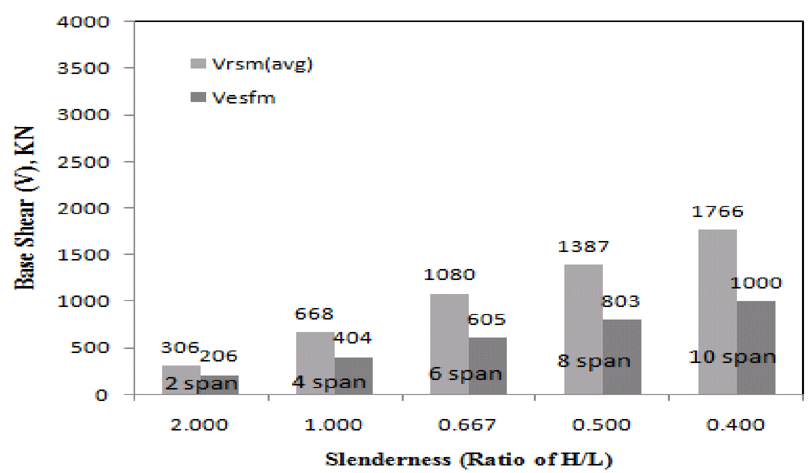


Fig.4.201 Comparison of Base Shear with Slenderness for 20% Infill 9 Floor

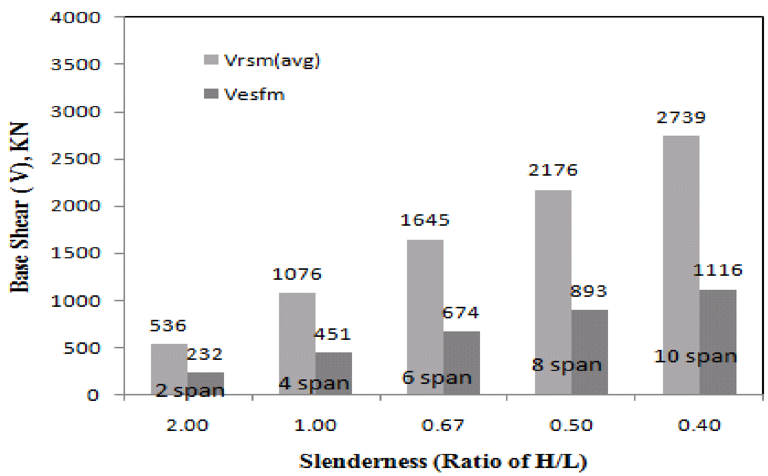


Fig.4.202 Comparison of Base Shear with Slenderness for 60% Infill 9 Floor

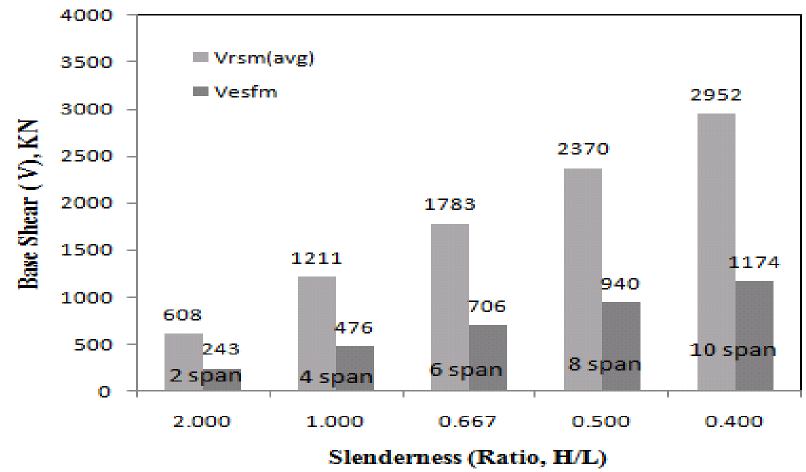


Fig.4.203 Comparison of Base Shear with Slenderness for 80% Infill 9 Floor

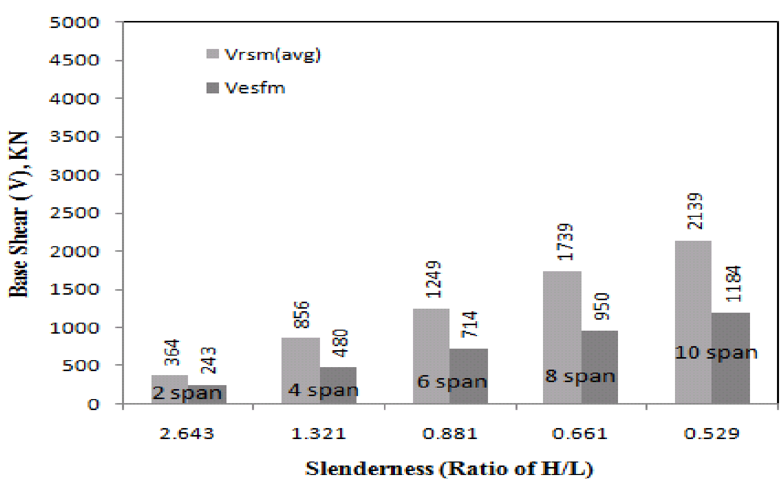


Fig.4.204 Comparison of Base Shear with Slenderness for 20% Infill 12 Floor

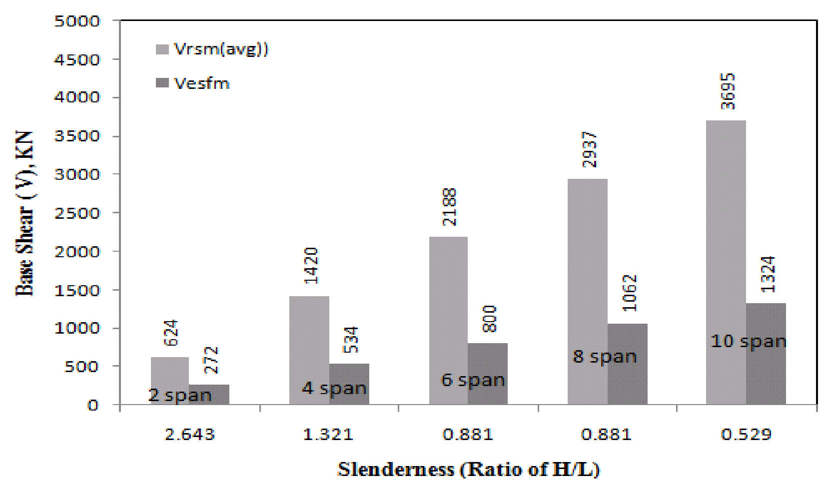


Fig.4.205 Comparison of Base Shear with Slenderness for 60% Infill 12 Floor

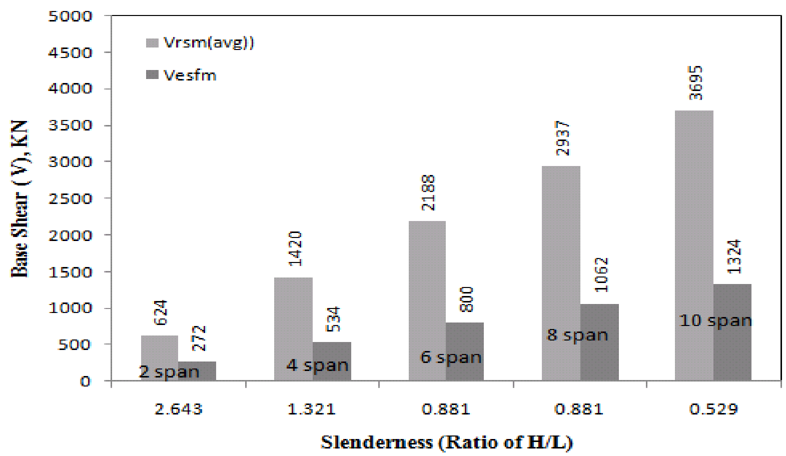


Fig.4.206 Comparison of Base Shear with Slenderness for 80% Infill 12 Floor

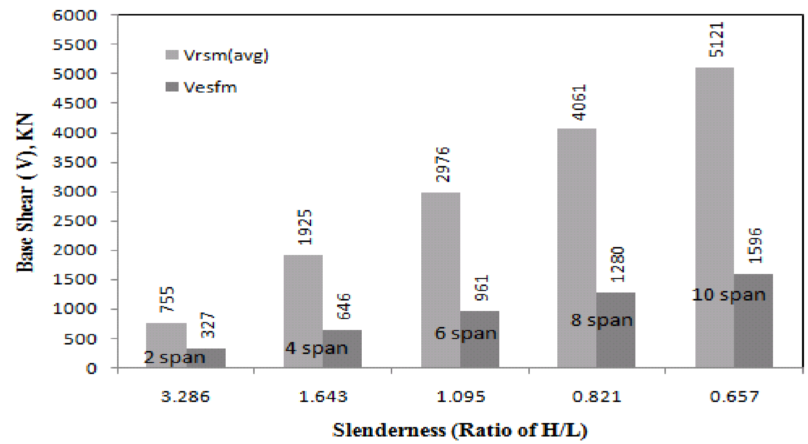


Fig.4.207 Comparison of Base Shear with Slenderness for 20% Infill 15 Floor

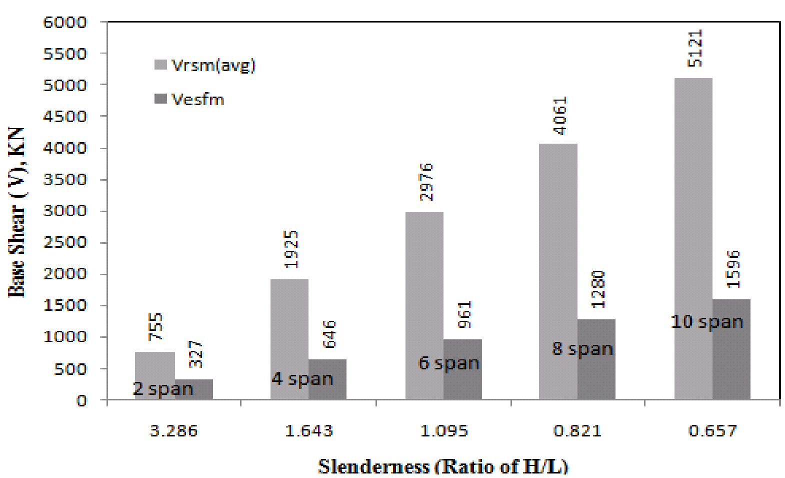


Fig.4.208 Comparison of Base Shear with Slenderness for 60% Infill 15 Floor

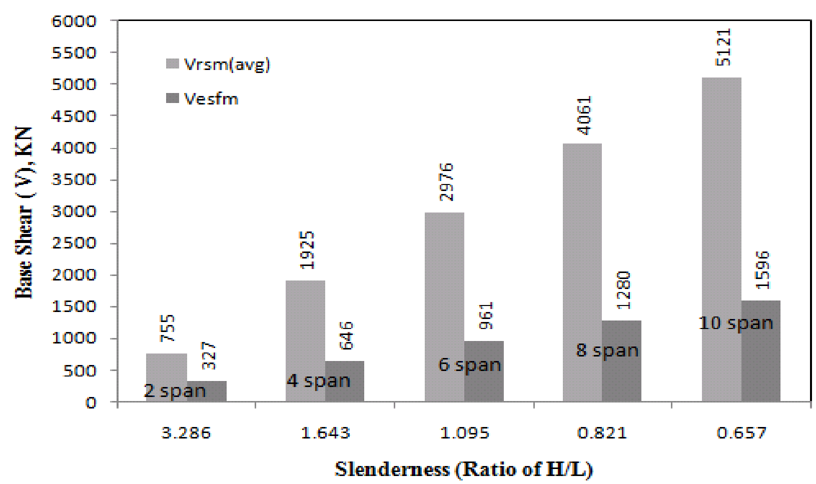


Fig.4.209 Comparison of Base Shear with Slenderness for 80% Infill 15 Floor

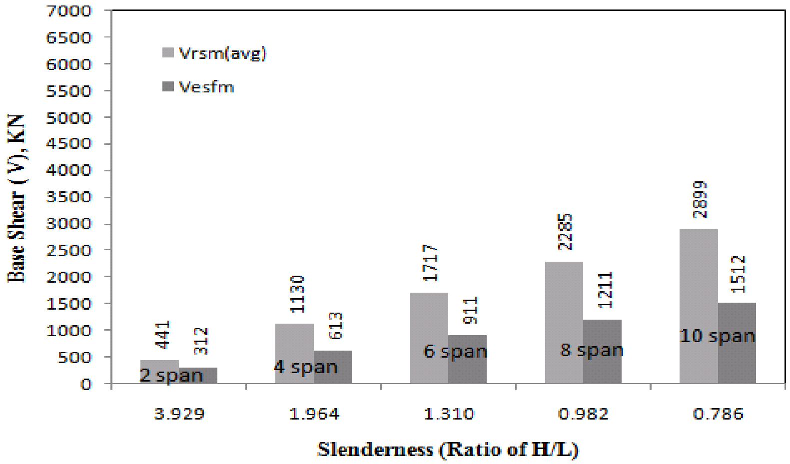


Fig.4.210 Comparison of Base Shear with Slenderness for 20% Infill 18 Floor

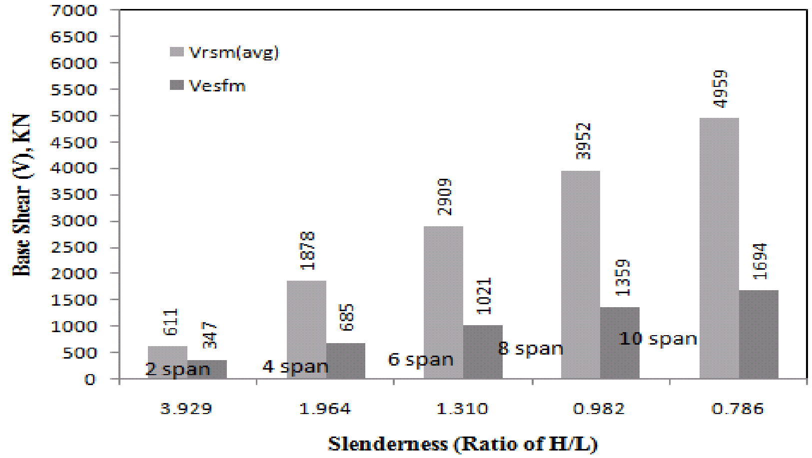


Fig.4.211 Comparison of Base Shear with Slenderness for 60% Infill 18 Floor

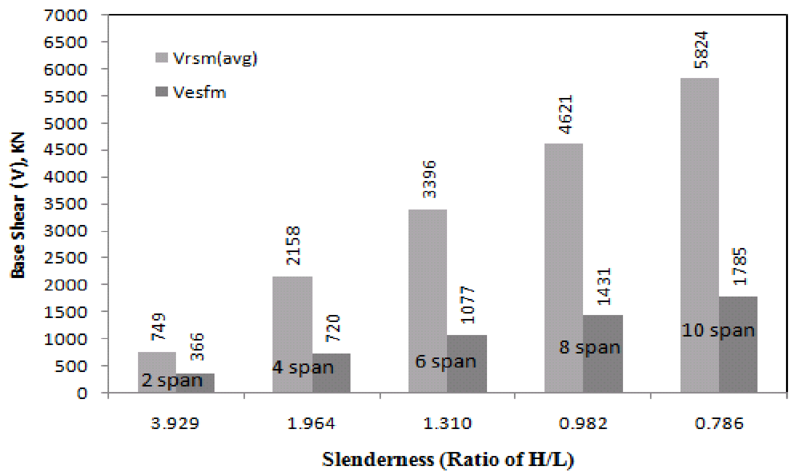


Fig.4.212 Comparison of Base Shear with Slenderness for 80% Infill 18 Floor

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