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Seismic behavior of soft storey mid-rise steel frames with randomly distributed masonry infill

Shahriar Quayyum, M. Shahria Alam* and Ahmad Rteil

School of Engineering-Okanagan, The University of British Columbia, 3333 University Way, Kelowna, BC V1V 1V7, Canada

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Abstract. In this study, the effect of presence and distribution of masonry infill walls on the mid-rise steel frame structures having soft ground storey was evaluated by implementing finite element (FE) methods. Masonry infill walls were distributed randomly in the upper storey keeping the ground storey open without any infill walls, thus generating the worst case scenario for seismic events. It was observed from the analysis that there was an increase in the seismic design forces, moments and base shear in presence of randomly distributed masonry infill walls which underlines that these design values need to be amplified when designing a mid-rise soft ground storey steel frame with randomly distributed masonry infill. In addition, it was found that the overstrength related force modification factor increased and the ductility related force modification factor decreased with the increase in the amount of masonry infilled bays and panels. These must be accounted for in the design of mid-rise steel frames. Based on the FE analysis results on two mid-rise steel frames, design equations were proposed for determining the over strength and the ductility related force modification factors. However, it was recommended that these equations to be generalized for other steel frame structure systems based on an extensive analysis.

Keywords: seismic behavior; soft ground storey; mid-rise steel frame; masonry infill

1. Introduction

Masonry infill walls are often considered as non-structural elements in structures since they are usually used as partitions. Hence, the influence of the masonry infill walls on the structural behavior of the frame is often ignored (Mehrabi and Shing 1997). However, reported experimental and analytical works have shown that the frame and the infill walls interact and alternate the response of the structure, especially when the structure is subjected to lateral loads. Hence, evaluation of stresses in the frames subjected to lateral loads, neglecting the presence of infills, can lead to underestimation of stresses at the structural elements, especially in columns (Papia 1988). It was found that the lateral stiffness of the structure increases with presence of masonry infill walls which is neglected in the conventional design practice (Stafford-Smith 1962, 1966, Liauw and Kwan 1985, Moghaddam *et al.* 2006, Kaltakci *et al.* 2007). This could result in a smaller seismic code based lateral loads (Memari *et al.* 1999). In addition, the discontinuation of masonry infill walls at the ground storey can develop stiffness irregularity in the structure which can

*Corresponding author, Assistant Professor, E-mail: shahria.alam@ubc.ca

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generate soft storey at the ground storey level. The development of soft storey can lead to catastrophic failure of the whole structure as was evident from some previous major earthquakes, e.g., 1971 San Fernando, 1985 Mexico, 1994 Northridge, 1995 Hyogoken-Nanbu, 1995 Kobe, 1998 Adana Ceyhan, 2001 Bhuj, and 2003 Greece earthquakes (Ruiz and Diedrich 1989, Humar *et al.* 2001, Karakostas *et al.* 2005, Ghobarah *et al.* 2006). Therefore, it is essential to assess the vulnerability of soft storey steel buildings in presence of masonry infill walls in the upper storeys.

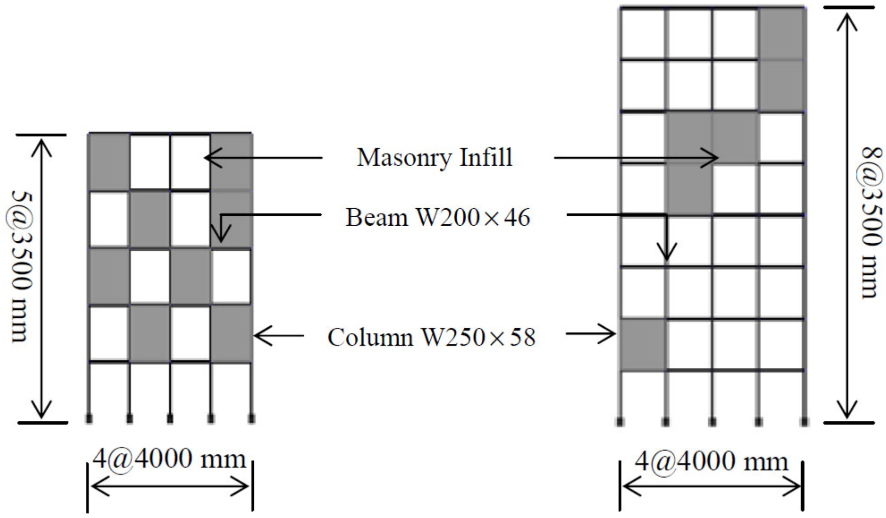
Many researchers addressed the problem from different points of view. The seismic behavior of masonry infilled steel and concrete frames was investigated by a number of researchers both experimentally and analytically (Alam *et al.* 2009, Dawe and Seah 1989, Dawe *et al.* 1989, Mosalam *et al.* 1997, Flanagan and Bennett 1999, Aliaari 2005, Aliaari and Memari 2005, Mohebkhah *et al.* 2008). The general conclusion from these studies was that during analysis and design of structures, it is necessary to take into account the additional stiffness and load carrying capacity provided by masonry infill, for realistic and sometimes economical designs. In addition, it was noted that buildings with open ground storey perform poorly during earthquakes and hence, they should be designed with proper attention to the presence and distribution of masonry infills incorporated in the building frames so that a definite guideline can be provided to assist design engineers. Although the effect of the presence of masonry infill on the seismic performance of steel structures has been addressed in different existing literature both analytically and experimentally, little research has been carried out on the effect of the random nature of infill distribution in soft storey steel frame structures and also, how the presence of masonry infill changes the ductility and overstrength related force modification factors. The present study deals with the effect of presence and the random nature of infill distribution on the seismic behavior of soft storey mid-rise steel frames including its effect on the ductility and overstrength related force modification factors and proposes design equations to calculate the force modification factors.

2. Computational modeling

2.1 Modeling of reference steel frames

In this study, the seismic performance of two mid-rise (five storey and eight storey) reference steel frames infilled with masonry panels was investigated by using finite element (FE) analysis. Mid-rise frames were selected in this study, since such type of building frames are highly vulnerable to earthquakes (Bariola 1992, Loulelis *et al*. 2012). The frames were designed and detailed in accordance with the Canadian Institute of Steel Construction guideline (CISC 2006), assuming that it is located in the southwestern corner of the province of the British Columbia, Canada (Seismic site classification type “C”) on very dense soil and soft rock with un-drained shear strength of more than 100 kPa. A moderate level of ductility was assumed for the design of the moment frames. A typical elevation of a five storey reference steel frame with 50% randomly distributed infill is shown in Fig. 1.

Nonlinear static pushover analysis was performed on masonry infilled steel frames using a FE package SeismoStruct (2010). The FE program considers both geometric and material nonlinearities for predicting large displacement behavior of structures. 3D beam elements were used for modeling the beams and the columns where the nonlinear uniaxial stress-strain response of the individual fibers was integrated to obtain the sectional stress-strain state of the elements. In order to achieve this, the section was subdivided based on the spread of material inelasticity within



Five storey frame
Eight storey frame

Fig. 1 Elevation of reference steel frames with randomly distributed infill

Table 1 Properties of the frame members used in the reference steel frames (Dawe *et al.* 2001)

Properties	Beam	Column
Section	W200×46	W250×58
Cross-sectional area, A (mm²)	5860	7420
Moment of inertia, I (mm$^{4)}$	45.5×10^6	18.8×10^6
Modulus of elasticity, E (MPa)	200,000	200,000
Plastic moment capacity with respect to axis indicated, M_{pl} (kN-m)	148.8	84.9
Maximum shear capacity V_{pl} (kN)	850	1000
Maximum axial load P_{pl} (kN)	1760	2200

the member length and cross-section. A bilinear kinematic strain hardening model was adopted to represent the steel. Table 1 shows the member properties for the frame.

2.2 Modeling of masonry infill panels

The nonlinear response of the masonry infill panels was modeled by using a plane stress infill panel element developed by Crisafulli (1997). The infill panel element is a four node bilinear element where six strut members (three along each diagonal direction) are used to represent each panel (Fig. 2). Among the three struts along each diagonal direction, two struts are designated to carry axial loads across the corners, whereas, the third strut is designated to transfer shear in between the panel. The activation of the shear strut is influenced by the displacement of the panel since this strut acts across the compression diagonal only. The nonlinear response of the axial and shear struts was modeled by adopting the hysteresis (Fig. 3) and bilinear (Fig. 4) model proposed by Crisafulli (1997). Also as can be observed in Fig. 2, four internal and four dummy nodes are incorporated to represent contact between the infill panel and the frame. The actual points of contact were established by the internal four nodes, whilst the dummy nodes account for the contact length between the infill panel and the frame. The exterior four nodes take all the internal forces when loads are applied.

Typical cyclic response of the masonry infill panels used in the model due to axial stresses is shown in Fig. 3 which includes the effect of the inner loops. It can be observed that the successive inner loops increase the reloading strain and do not affect the plastic deformation and remain inside the cycle defined for the complete unloading and reloading curves. The former can exhibit change in direction of its concavity depending on the starting point of the loading curve, while the latter show no inflection point. Cracking is the most dominant aspect of the behavior of the analytical infill panel model. It is considered that cracking occurs, when the stresses reach a failure surface. Once cracking develops, a yield surface is introduced to model the opening cracks. The model uses the smeared approach to simulate the effect of cracking. In order to avoid mesh

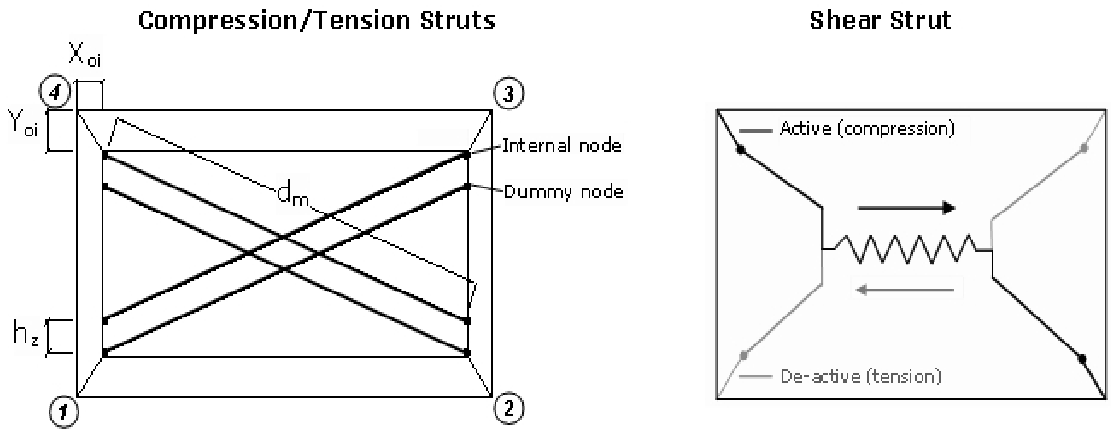


Fig. 2 Modeling of masonry infill panels (Crisafulli 1997)

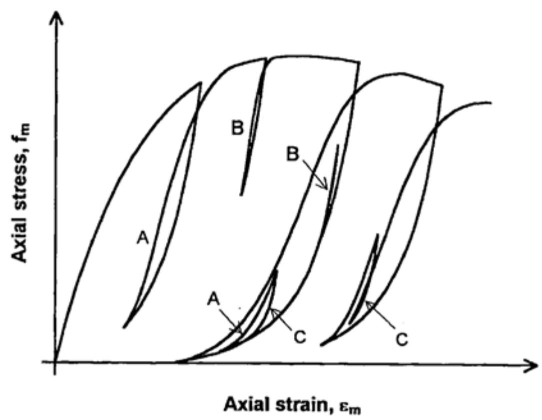


Fig. 3 Typical cyclic responses with small cycle hysteresis for masonry infill panels (Crisafulli 1997)

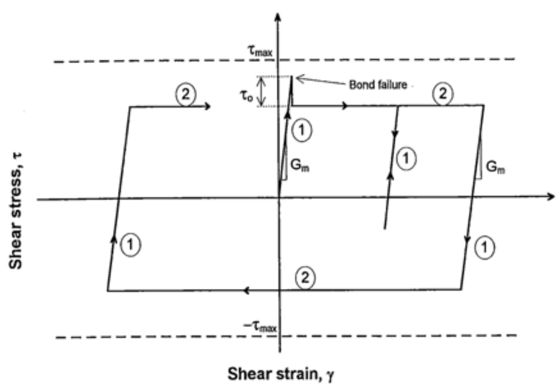


Fig. 4 Analytical response for cyclic shear response of mortar joints (Crisafulli 1997)

sensitivity and convergence problems, stress softening was modeled with a nonlinear curve from the point of maximum tensile stress to zero, at a strain five times greater than the strain at the maximum tensile stress. The response of the material in compression was modeled using elasto-plastic theory. Associated flow and isotropic hardening were used in the model. The adopted model is capable of representing the shear behavior when bond failure happens along the mortar joints. It is assumed that the behavior of the latter is linear elastic while the shear strength is not reached. Unloading and reloading are also in the elastic range. The cyclic response of masonry in shear was represented by two hysteresis rules and included the axial load in the masonry as a variable in the shear strength (Fig. 4). The shear strength is evaluated following a bond-friction mechanism, consisting of a frictional component and the bond strength τ_0 (elastic response-rule 1). The former depends on the coefficient of friction μ and the compressive stress perpendicular to the mortar joints f_n.

$$\tau_m = \tau_0 + \mu|f_n| \le \tau_{\max} \quad \text{if } f_n < 0 \tag{1a}$$

$$\tau_m = \tau_0 \quad \text{if } f_n \ge 0 \tag{1b}$$

Where $\tau_{\max}$ represents an upper limit for the shear strength. When the shear strength is reached, the bond between brick and mortar is destroyed and cracks appear in the affected region. In this phase, one part of the infill panel slides, with respect to the other part and only the frictional mechanism remains (sliding-rule 2). Consequently the shear strength is given by Eq. (2).

$$\tau_m = \mu|f_n| \le \tau_{\max} \quad \text{if } f_n < 0 \tag{2a}$$

$$\tau_m = 0 \quad \text{if } f_n \ge 0 \tag{2b}$$

It is assumed that the unloading and reloading after the bond failure follows a linear relationship. This process can be represented by rule 1, using Eq. (1). The reloading line increases the shear stress until the shear strength is reached and sliding starts again. The properties of the masonry infill panels used are summarized in Table 2. The definition of the input parameters of the masonry infill panels are presented in Appendix. A.

Table 2 Properties of masonry infill panel (Crisafulli 1997)

Properties	Values	Properties	Values
Young Modulus, E_m (kPa)	3500000	Shear bond strength, τ_0 (kPa)	300
Compressive Strength, f_m (kPa)	3500	Friction co-efficient, μ	0.7
Tensile strength, f_t (kPa)	575	Maximum shear resistance, $\tau_{\max}$ (kPa)	600
Strain at maximum stress, ε_m	0.0012	Reduction shear factor, α_s	1.5
Ultimate strain, ε_u	0.024	Thickness of infill panel (mm)	250
Closing strain, ε_{cl}	0.03		
Starting unloading stiffness factor, g_u	1.7		
Strain reloading factor, a_r	0.2		

2.3 Distribution of masonry infill panels

In practical cases, the amount and the distribution of the masonry infill walls will vary from floor to floor following architectural design and presence of openings and windows and the distribution of infill is usually random in nature. Therefore, in this study, the masonry infill walls were distributed randomly in the upper storeys, while keeping the ground storey open without any masonry infill walls simulating a soft ground storey. The amount of masonry infill considered in this study varied between 10%-100% of the frame panels excluding the ground storey panels. For every percentage of infill (except for 100% infill), the structural response was averaged for ten different arrangements (distributions) and these ten infill distributions were randomly chosen. For example, the five storey reference building frame shown in Fig. 1 has five stories with four bays in each storey. Thus, the total number of frame panels is 20 (5×4) and excluding ground storey panels, it is 16. To provide 20% infill, we need 3 panels. These 3 panels were chosen randomly and were modeled with diagonal struts. Ten separate analyses were performed with ten random choices of these 3 panels representing 20% infilled panels. For other amounts of masonry infills (except for 100% infill), the analyses were performed in a similar way. In this paper, unless otherwise stated, percent (%) infill refers to the number of infill panels out of total panels in a frame except the ground storey panels. For instance a 100% infill means all the panels will be filled with walls except the ground storey panels, which means that always there will be soft storey at the ground level.

2.4 Analysis methods

The reference steel frames were subjected to gravity loading as well as seismic loading. The seismic design guidelines of the National Building Code of Canada (NBCC 2005) were followed for calculating the seismic loads. The seismic response of the reference steel frames was studied by using nonlinear finite element (FE) analysis package SeismoStruct (2010). Comparisons of the seismic structural responses were made for different amount and distribution of masonry infill. The strength and ductility of the reference steel frames were investigated through static nonlinear pushover analyses. In the nonlinear pushover analysis the whole structure was pushed to evaluate the seismic performance of the reference steel frames using inverse triangular load distribution pattern until the roof displacement reaches a target value of 1 m. This type of load distribution is very similar to the equivalent lateral load distribution as suggested by FEMA-356 and is well suited for structures deforming primarily in the first mode. Since the seismic response of superstructure shows the first order mode only, and the effect of higher order modes are comparatively small, the study reported herein adopted the inverse triangular load distribution. Moreover, it has been shown by Akkar and Metin (2007) that inverse triangular lateral loading shows conservativeness over the uniform lateral loading pattern. However, the percentage of infill did not have any bearing on the choice of the load distribution. The lateral load pattern was distributed along the height of the frames in such a way that each floor was subjected to a concentrated force. The displacements were applied incrementally and each load increment was divided into 1000 time steps. Pushover curve can be generated at each step based on the base shear and the roof displacement and the pushover curve can be used as a measure of the capacity and the ductility of the structure during earthquake.

3. Results and discussion

3.1 Modal analysis

3.1.1 Natural period

The National Building Code of Canada (NBCC 2005) specifies the expression for calculating the fundamental period of vibration of a steel building as in Eq. (3)

$$T = 0.085(h_n)^{\frac{3}{4}} \tag{3}$$

where h_n is the height of the building above the base in meters. To get a conservative estimate of the base shear, Eq. (3) is calibrated to yield a lower value of the fundamental period than the actual period by 10-20% (Amanat and Hoque 2006). For the no infill case, the analysis revealed that the natural period of the building was approximately double the value predicted by the code equation (Fig. 5). As the amount of infilled panel increased, the value of the natural period obtained from the modal analysis decreased and it converged with the value obtained from the code equation (Fig. 5). This indicates the necessity of incorporating the interaction of masonry infill with the bonding frame for a better dynamic analysis of steel structures. In addition, decrease of natural period with increased amount of infill indicates increase in the stiffness of the frame, which validates again that the presence of masonry infill in the frame increases the stiffness and the lateral strength of the frame.

3.2 Nonlinear static pushover analysis

3.2.1 Pushover curves

Figs. 6 and 7 show the variation of the storey displacement, the pushover curves and the column shear of a five and an eight storey reference steel frame, respectively for ten random

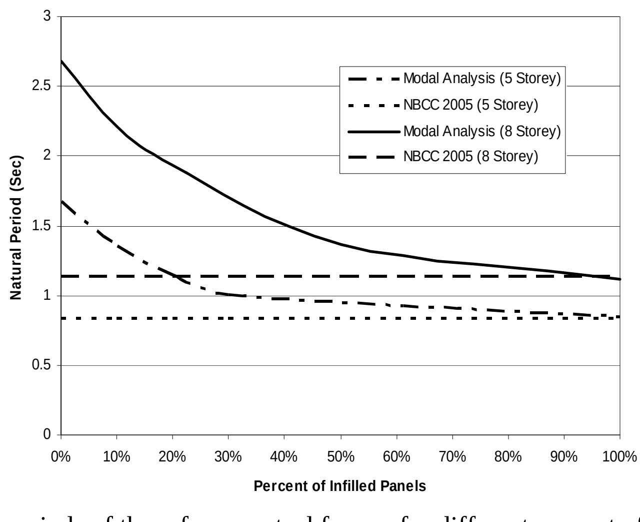
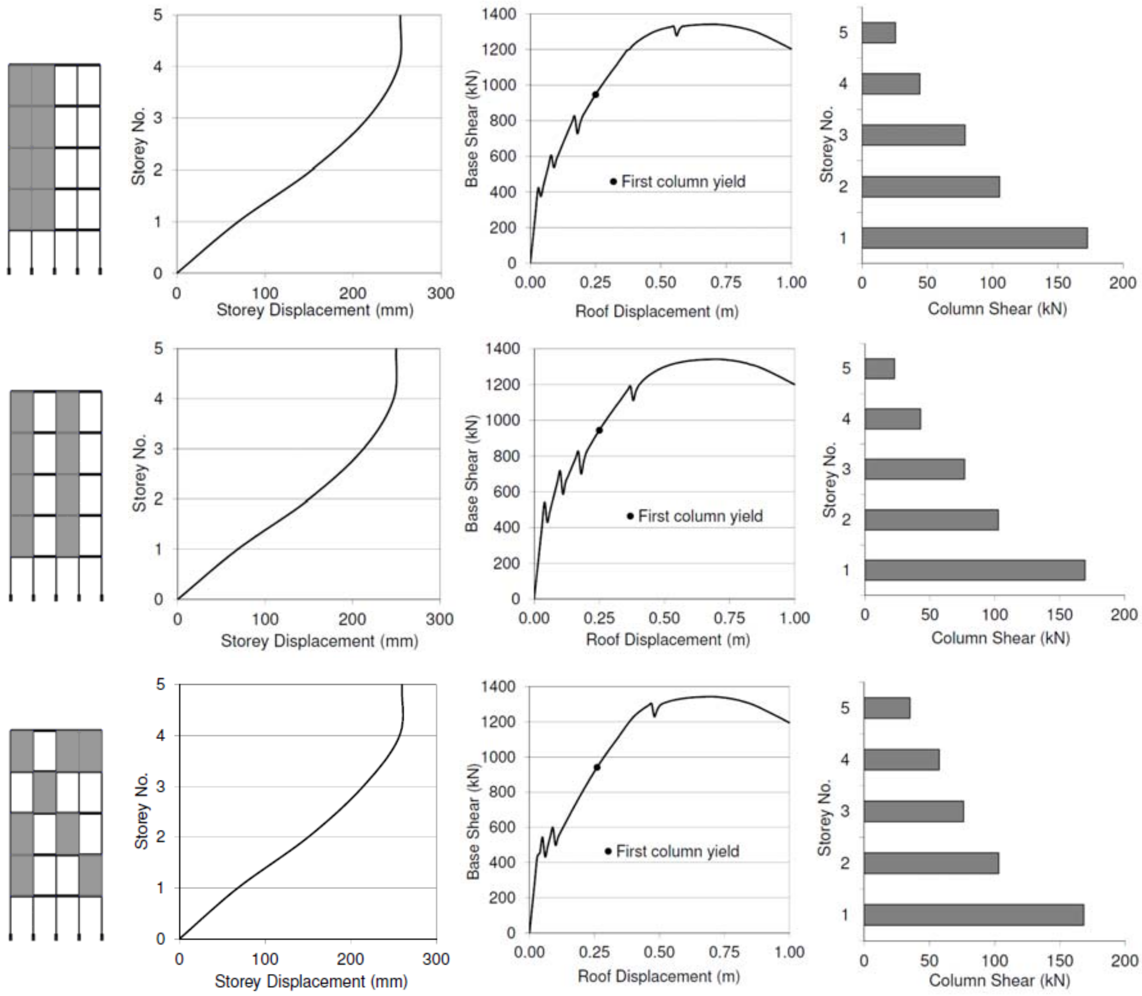


Fig. 5 Natural periods of the reference steel frames for different amount of masonry infill

distributions of 50% infills. In the pushover curves, the initial spikes indicated initiation of cracks in the masonry infill. It was observed that for the same amount of infill, the reference frames showed different behavior for different arrangements of infills. For example, in case of a five storey reference steel frame with 50% infill, the maximum and the minimum lateral deflection of the reference steel frame were 300 mm and 180 mm at first column yield, for ten different arrangements of infill (Fig. 6). This is 66.7% increase in the lateral deflection due to the change in the arrangement of infill. The base shear capacity at first column yield also varied between 940 kN to 1090 kN for ten different arrangements of 50% infill, which indicates about 16% increase in the base shear capacity (Fig. 6).

On the other hand, in case of an eight storey reference steel frame with 50% infill, the maximum and the minimum lateral deflection of the reference steel frame were 430 mm and 350 mm, respectively at first column yield for ten different arrangements of infill (Fig. 7). This is 23% increase in the lateral deflection due to the change in the arrangement of infill. The base shear



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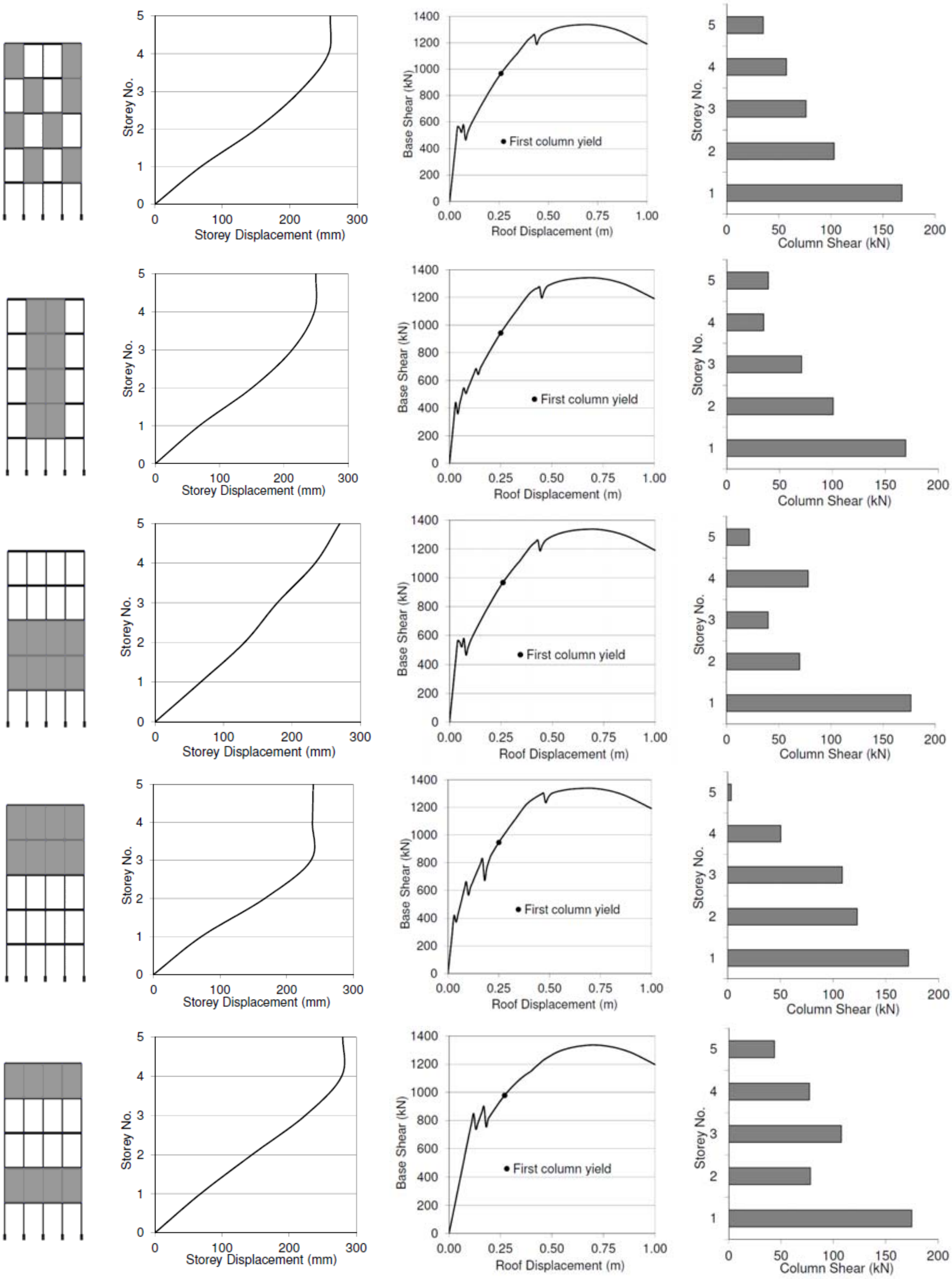

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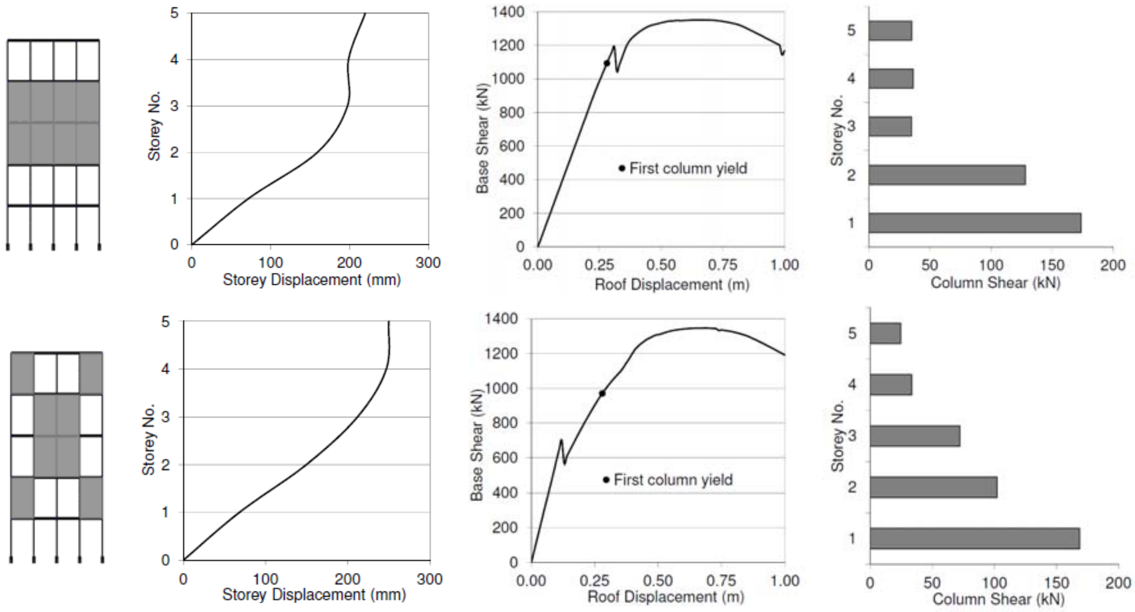


Fig. 6 Storey displacement, pushover curves and storey shear of a five storey reference steel frame with different distributions of 50% masonry infills

capacity at first column yield also varied between 853 kN to 913 kN for ten different arrangements of 50% infill, which indicates about 7% increase in the base shear capacity (Fig. 7). It was noticeable from Figs. 6 and 7 that with different arrangements of the same amount of masonry infill, the lateral displacement profile and the storey shear distribution changed for each case. This was attributed to the change in stiffness of different storey with change in the distribution of the masonry infills. This indicates the uncertainty associated with the distribution of masonry infill and it can be anticipated that there may be some cases when the column shear or bending moment will be under estimated due to neglecting the critical distribution of masonry infill.

In this study, ten different location arrangements were chosen for each percentage of infill and the average result was reported for comparing the effect of the amount of infill on the behavior of the reference steel frames. For example, Fig. 8 shows ten pushover curves for a five (Fig. 8(a)) and an eight (Fig. 8(b)) storey reference steel frame with ten different distributions of 50% infills along with the average pushover curve. The average line was drawn through the middle of all the pushover curves and this average value of ten different arrangements will be used for comparing the effect of the amount of infill on the reference steel frames. The lateral load-displacement (pushover curves) behavior of a five and an eight storey reference steel frames are shown in Fig. 9 and Fig. 10, respectively. The black points on the curves indicate the first column yield. For each percent of masonry infill, ten distributions were considered and the average values were reported (except 100% infill). It was observed that the base shear capacity of the reference steel frames increased with the increase in the amount of infill. The increase in the base shear capacity (up to yield point) was in the range of 20%–25% for 100% masonry infilled reference steel frames in comparison to the bare frame. This happened due to the contribution of the masonry infills in the lateral load resisting mechanism of the frames.

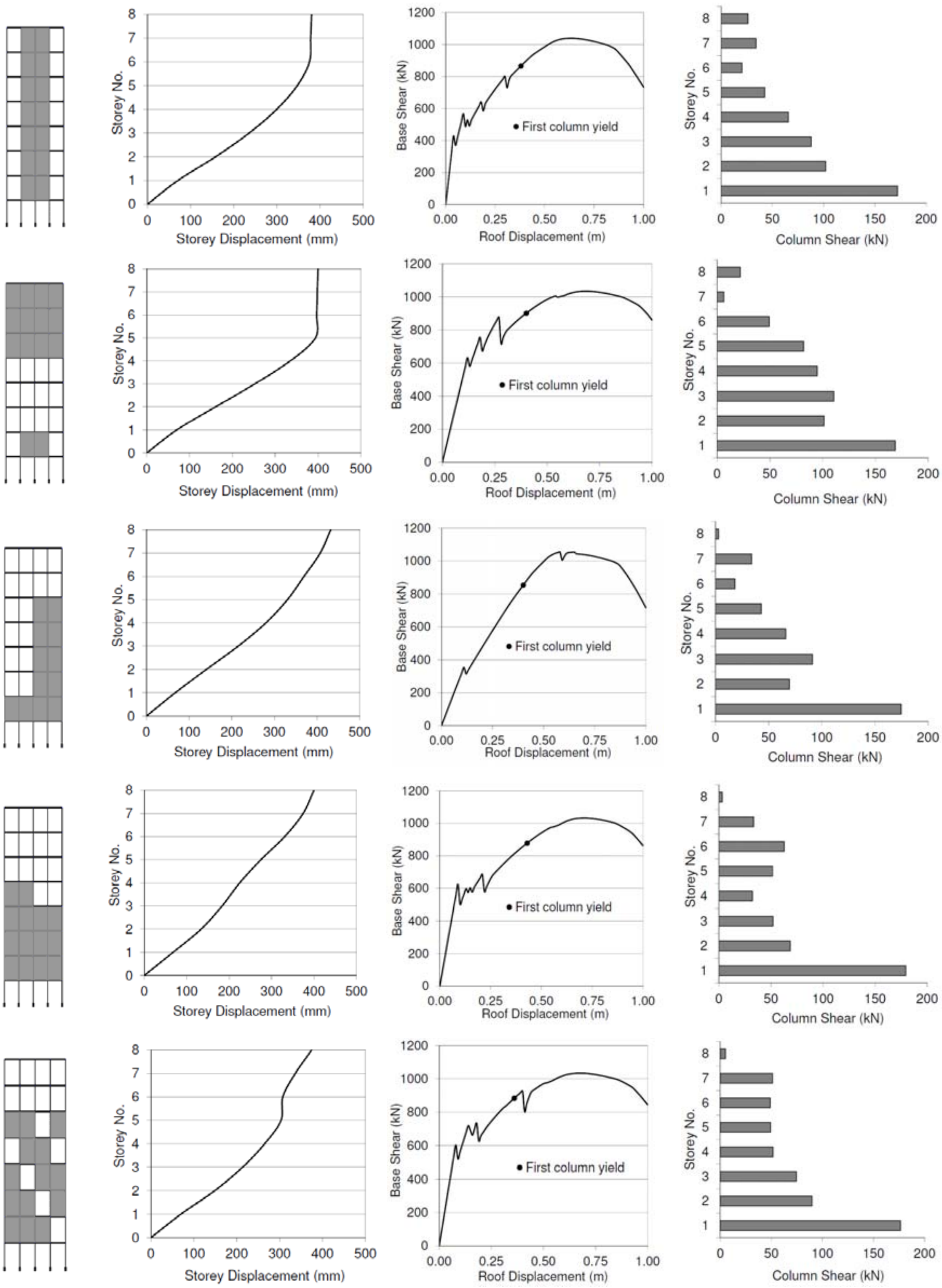


Fig. 7 Continued

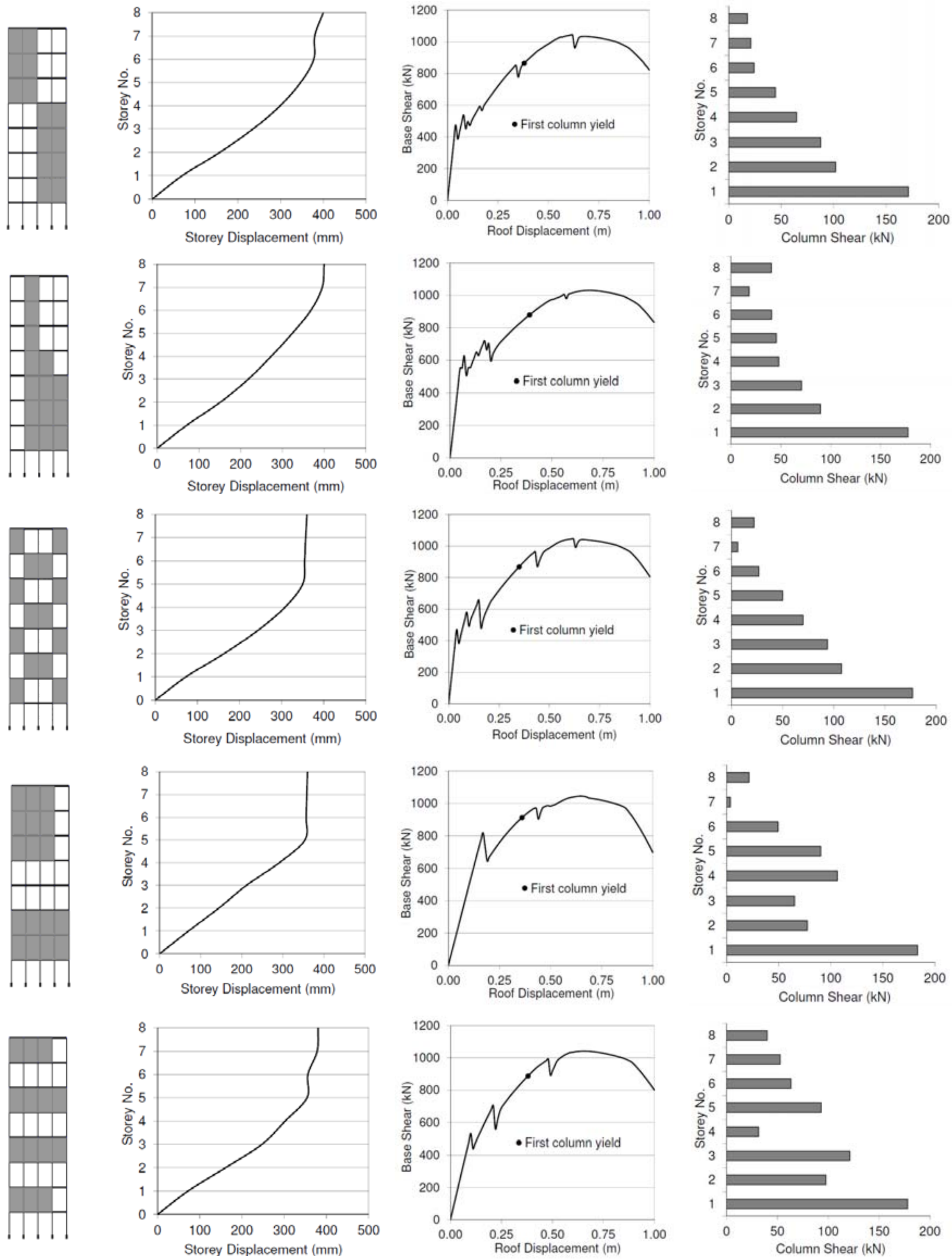
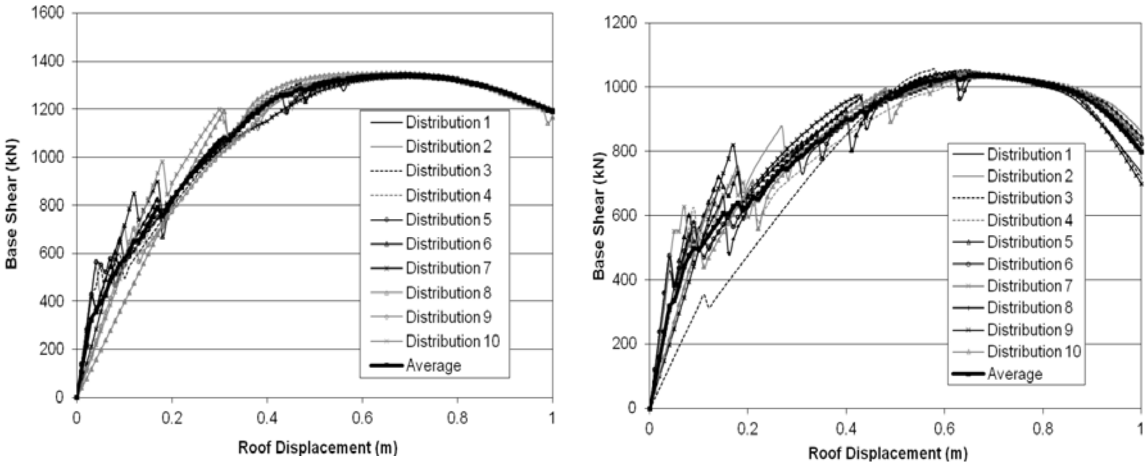


Fig. 7 Storey displacement, pushover curves and storey shear of an eight storey reference steel frame with different distributions of 50% masonry infills



(a) Five storey frame (b) Eight storey frame

Fig. 8 Pushover curves of reference steel frames for ten different distributions of 50% infill along with the average line

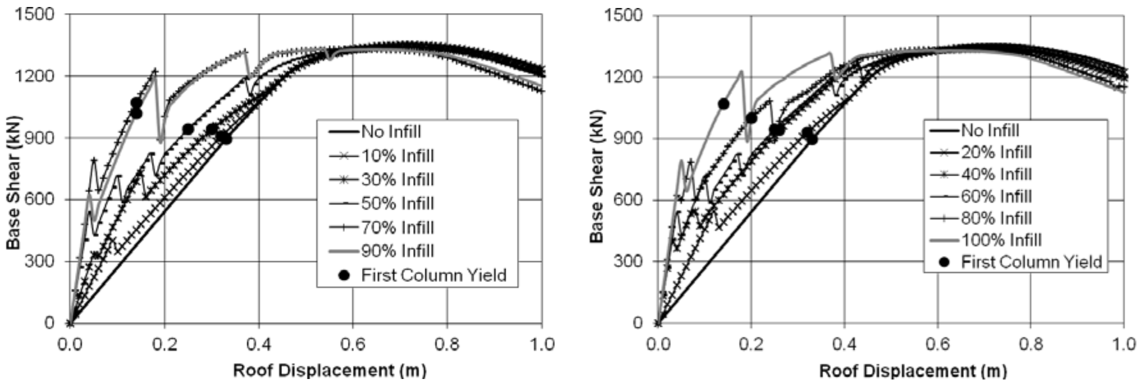


Fig. 9 Pushover curves of a five storey reference steel frame for different amount of masonry infill

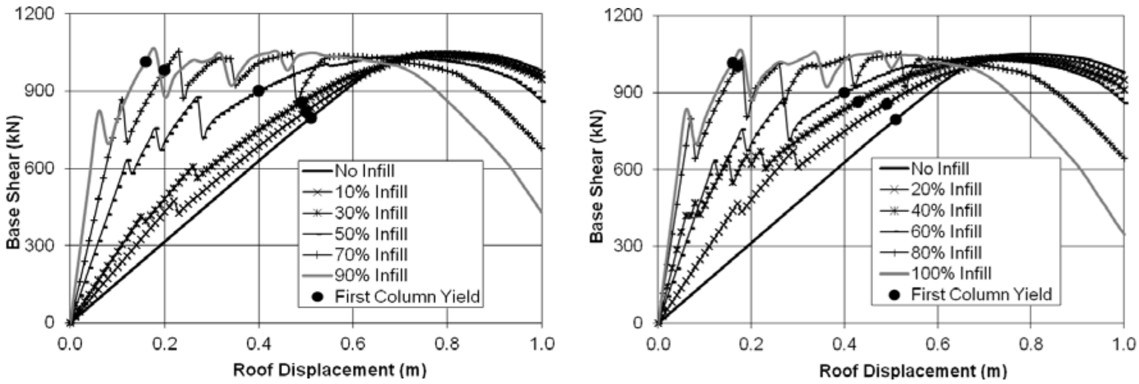


Fig. 10 Pushover curves of an eight storey reference steel frame for different amount of masonry infill

Table 3 Average base shear and roof displacement at first column yield

% of Infill	5 Storey Frame		8 Storey Frame	
	First column yield base shear (kN)	Roof displacement at first column yield (mm)	First column yield base shear (kN)	Roof displacement at first column yield (mm)
0	879	324	797	510
10	896	315	822	500
20	906	294	831	490
30	922	287	856	470
40	933	259	860	425
50	956	255	901	400
60	986	253	908	335
70	1031	245	974	270
80	1043	190	982	170
90	1052	140	987	150
100	1072	135	988	150

On the other hand, the lateral deflection at first column yield decreased with increase in the amount of infill (Table 3). When the amount of masonry infill was increased keeping the ground storey open, the stress at the ground storey columns increased, which eventually caused the ground storey columns to yield. Therefore, the more the masonry infills, the less the lateral deflection the columns can take before yielding. It was observed that for a five storey reference steel frame with 100% infills, the ground storey columns yielded at a roof displacement of 135 mm, whereas for the bare frame model the columns yielded at 324 mm of roof displacement (Table 3). In the case of an eight storey reference steel frame with 100% infills, the ground storey columns yielded at a roof displacement of 150 mm, whereas for the bare frame model the columns yielded at 510 mm of roof displacement (Table 3).

It becomes quite obvious that although with the increase in the amount of masonry infills, the lateral strength and the stiffness of the structure increases, but due to the presence of soft ground storey, the ground storey columns yielded at a much lower roof displacement in comparison to the bare frame. This decrease in the roof displacement, due to the presence of soft ground storey, must be accounted for during the design of steel structures since the conventional design procedure does not account for such effect.

3.2.2 Storey displacement

Figs. 11 and 12 show the storey displacement at first column yield along the height of a five and an eight storey reference steel frame, respectively with different infill conditions. For both the reference steel frames, the lateral deflection was the highest for frame with 0% infill and it reduced as the percent of infill was increased due to the increased stiffness of the storey. Displacement profiles had a sudden change of slope at the ground storey level in the presence of the infill walls (Figs. 11 and 12). This abrupt change in the slope of the profile was due to the stiffness irregularity between the soft ground storey with no infill and the upper storeys which had infill walls. For the bare frame, the storey displacement increased with the height of the building gradually (Fig. 11 and 12). On the contrary, in the presence of the infill walls, the increase in storey displacement was large at the bottom storey, and above that the storey displacement was almost negligible (Fig. 11 and 12).

This is because in a bare frame, each floor drifts with respect to the neighboring floors as a result of the independent mass in each floor. On the contrary, for frames with infill, the drift of each floor is restricted relative to the adjacent floors. As a result, the mass of the upper floors act together as a unified body and causes significant increase in the lateral displacement at the bottom storey. As a result, the more the infill walls in the upper storeys, the more is the lateral deflection at the bottom storey.

3.2.3 Column shear

The distribution of the shear force in a typical exterior column of the reference steel frames along the building height at the first column yield is shown in Fig. 13. It can be inferred from this figure that for bare frames, the column shear was gradually distributed in each storey with the largest value of shear force occurring at the ground storey columns. But when infill was present,

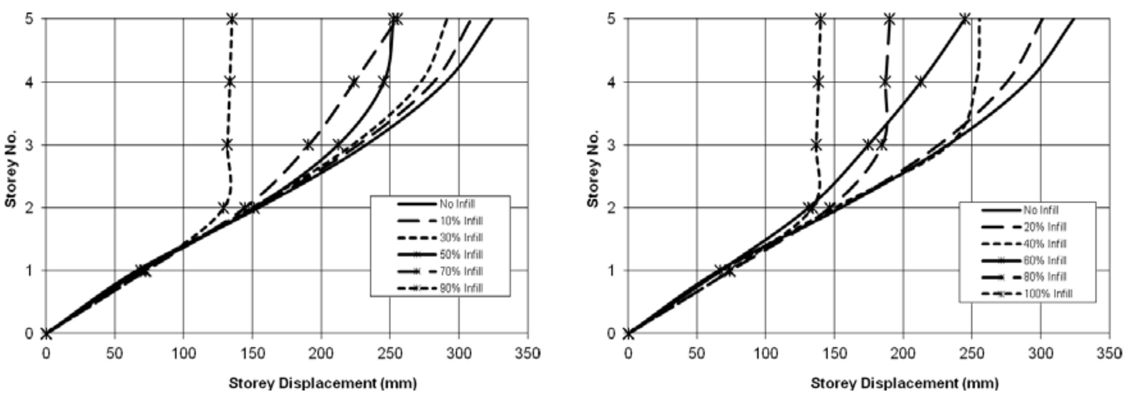


Fig. 11 Storey displacement of a five storey reference steel frame for different amount of masonry infill at first column yield

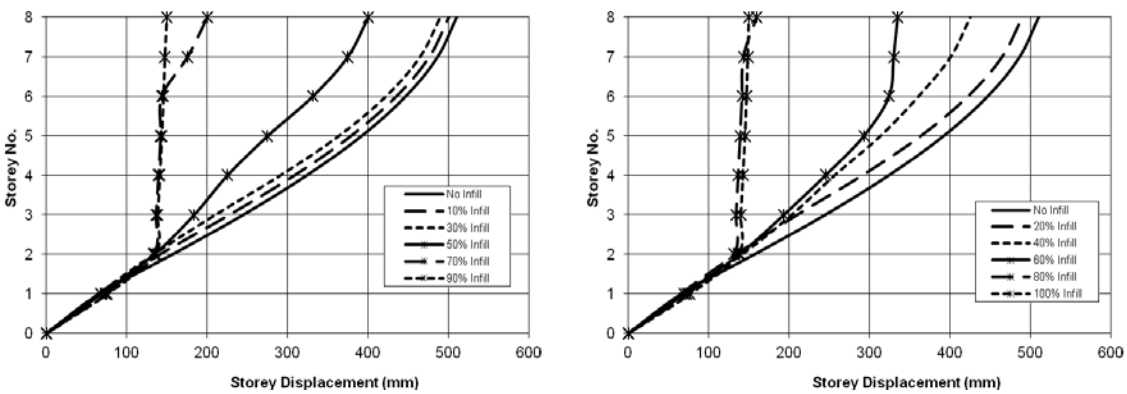


Fig. 12 Storey displacement of an eight storey reference steel frame for different amount of masonry infill at first column yield

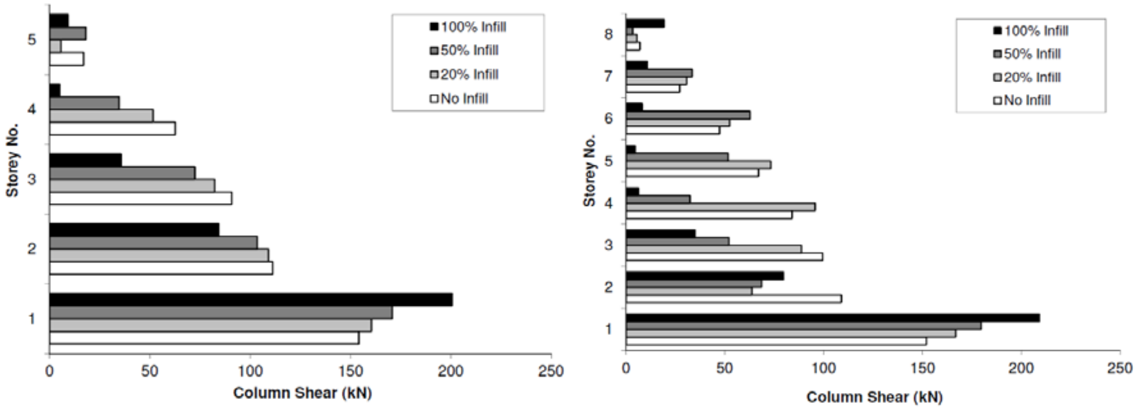


Fig. 13 Shear force in a typical exterior column of the reference steel frames for different amount of masonry infill at first column yield

the column shear force near the ground storey (soft storey) had a sharp increase compared to the shear force of the bare frame column (Fig. 13). This increase in ground storey shear was 30% and 38% for five storey and eight storey reference steel frames respectively, with 100% masonry infill, compared to the bare frame model. In addition, it was observed that in presence of masonry infill, the distribution of column shear is concentrated at the bottom storey with very small amount of column shear distributed in the upper storeys (Fig. 13). This happened due to the same reason for which the displacement had a sharp increase at the bottom storey i.e. in presence of masonry infill the mass of upper floors act as a unified body which causes a sharp increase in the column shear at the bottom storey. For a bare frame, the horizontal shear is distributed in each floor because of the relative drift between adjacent floors. However, a higher estimation of storey drift or column shear will inevitably lead to higher bending moment, which must be taken into account during analysis and design of the structure.

3.2.4 Stiffness and ductility

Table 4 shows the stiffness and the ductility of the reference steel frames for different amount of masonry infills. It was found that with the increase in the amount of infill, the stiffness of the reference steel frames increased, as expected. When the frames had 100% infill, the increase in stiffness was about 5.8 and 8.9 times the stiffness of the bare frame for five storey and eight storey reference steel frames, respectively. Therefore, an increase in the amount of infill made the frames stiffer in comparison to the bare frame, which eventually decreased the ductility of the frames. It was observed that the ductility of the reference steel frames for 100% infills decreased by 11% and 22% in comparison to the bare frames for five and eight storey reference steel frames, respectively. This manifests that the presence of infill will change the lateral load behavior of the structures and hence, their presence should be taken into consideration while designing the structures.

Table 4 also shows the ductility (R_d) and overstrength (R_0) related force modification factors for the reference steel frames for different amount of infills. According to NBCC 2005, the ductility related force modification factor, R_d, reflects the capability of a structure to dissipate energy through inelastic behavior and the overstrength related force modification factor, R_0, accounts for the dependable portion of reserve strength in a structure. In this study, these force modification

Table 4 Stiffness and ductility of the reference steel frames for different amount of masonry infills

% Infill	5 Storey frame				8 Storey frame			
	Stiffness (kN/m)	Ductility related force modification factor, $R_d = \frac{V_e}{V_y}$	Over strength related force modification factor, $R_o = \frac{V_y}{V_s}$	$R_o R_d$	Stiffness (kN/m)	Ductility related force modification factor, $R_d = \frac{V_e}{V_y}$	Over strength related force modification factor, $R_o = \frac{V_y}{V_s}$	$R_o R_d$
0	2715	2.07	1.07	2.22	1562	1.55	1.00	1.55
10	4457	2.04	1.09	2.22	2147	1.53	1.02	1.56
20	6662	2.03	1.16	2.35	2738	1.51	1.03	1.55
30	6795	2.02	1.17	2.37	2741	1.49	1.04	1.55
40	7065	2.01	1.29	2.59	4278	1.47	1.12	1.65
50	13517	2.00	1.29	2.59	5266	1.45	1.16	1.68
60	15365	1.98	1.30	2.57	6658	1.44	1.31	1.89
70	15452	1.95	1.34	2.61	7868	1.40	1.56	2.19
80	15570	1.92	1.70	3.26	11381	1.41	2.41	3.41
90	15624	1.87	2.29	4.29	13710	1.30	2.67	3.47
100	15852	1.84	2.38	4.37	13914	1.20	2.67	3.20

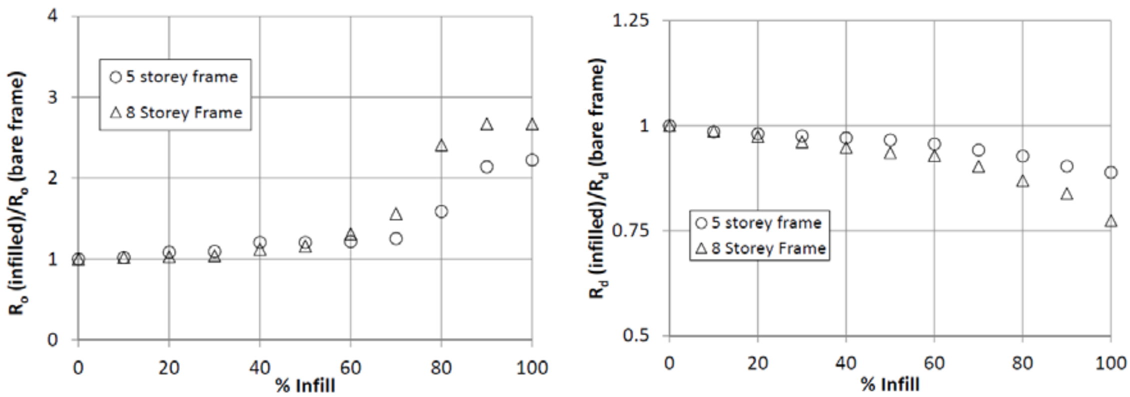


Fig. 14 Variation of overstrength and ductlity related force modification factors with the amount of infill

factors were obtained through bi-linear idealizing of the pushover curves. The force modification factors showed the same trend as was previously obtained. The overstrength related force modification factor started to increase with increase in the amount of infill due to the additional load carrying capacity provided by the presence of masonry infill, whereas the ductility related force modification factor decreased with increase in the amount of infill signifying decrease in the ductility of the frames. In Fig. 14, R_0 and R_d for the reference steel frames were plotted against the amount of infill, where R_0 and R_d values were normalized by their respective values for the bare frame. The normalized R_0 value showed an up-going trend when plotted against the percent of infill. This indicated as the amount of infill was increased, the frame's strength increased compared to the bare frame. On the other hand, the normalized R_d value showed a declining trend when plotted against the percent of infill indicating reduction in ductility of the frame with the amount of infill compared to the bare frame.

However, from Fig. 14, it is evident that both the normalized R_0 and R_d values varied with the number of storeys. As the number of storey increased, the normalized R_0 value increased and the normalized R_d value decreased. It must be noted that the variation of the normalized R_0 and R_d values with the amount of infill was nonlinear. Therefore, a nonlinear regression was performed on the normalized R_0 and R_d values, number of storey and the percent of infill and the following equations were obtained for determining R_0 and R_d.

$$R_0 = R_{0(bare\ frame)} - 0.03pn + 0.3p^2n \tag{4}$$

$$R_d = R_{d(bare\ frame)} - 0.006pn - 0.02p^2n \tag{5}$$

where p is the amount of infill in fraction between 0 and 1, and n is the number of storey. Fig. 15 shows the comparison of Eqs. (4) and (5) with the FE analysis results. It can be observed that these equations can predict the variation R_0 and R_d values with the amount of infill very well. However, these equations were developed based on the analysis on only two reference steel frames having 5 and 8 storey and four bays. This study intends to present a method and sample for the two steel frame case studies and it is recommended that similar equations should be developed based on extensive analyses of different steel frames having different height and bay width. The results of the FE analysis and the developed equations underlines that the overstrength and the ductility related force modification factors change with change in the amount of masonry infill and these need to be accounted for in the design of steel frames.

3.3 Observations

It was observed that the lateral behavior of mid-rise steel frames is significantly influenced by the presence and the random distribution of masonry infills. But it may not be always possible to

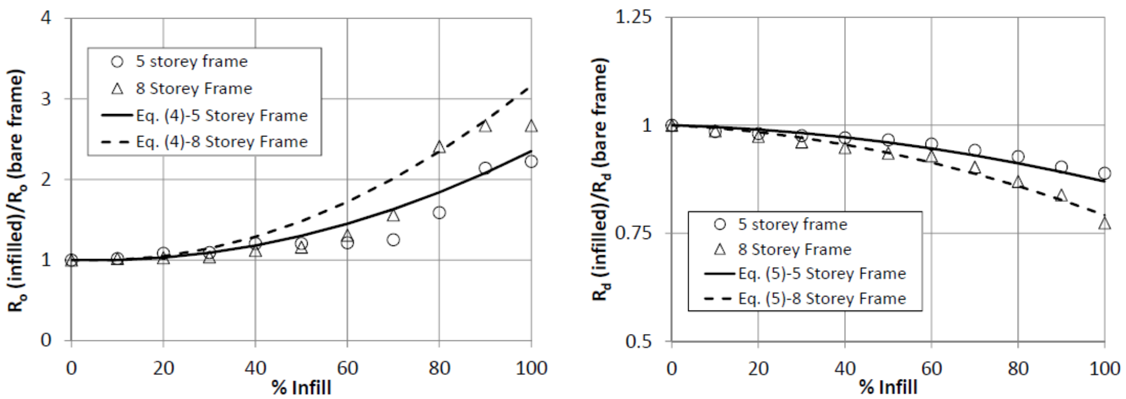


Fig. 15 Comparison of the proposed equations with FEA results

Table 5 Indicative amplification factors for conventional column design forces

% of infill	Five storey reference steel frame		Eight storey reference steel frame		Recommended amplification factor
	Column shear (kN)	Amplification factor	Column shear (kN)	Amplification factor	
0	154.1	1.0	151.9	1.0	1.0
10	158.3	1.03	158.5	1.04	1.10
20	164.5	1.07	166.8	1.10	1.10
30	166.1	1.08	169.6	1.12	1.15
40	166.8	1.08	169.6	1.12	1.15
50	172.6	1.12	179.6	1.18	1.20
60	178.8	1.16	181.7	1.20	1.20
70	191.0	1.24	206.5	1.36	1.40
80	197.2	1.28	206.8	1.36	1.40
90	198.4	1.29	207	1.36	1.40
100	200.6	1.30	209.1	1.38	1.40

find the particular distribution of masonry infill which gives the maximum design forces. This paper suggests that the studied structural responses might be calculated from a conventional analysis, but amplification due to the presence and the distribution of masonry infill must be accounted for. Thus, conventional results might be amplified by some appropriate factors to take care of the randomness in infill arrangement (distributions) for a safer design. Table 5 shows the amplification factors for the reference steel frames for different amount of masonry infills. The amplification factors were determined based on the peak responses of column shear. It can be seen that when there was 100% infill in the frame structure, the design forces needed to be multiplied by 1.4 to account for the presence and the randomness of infill distribution. This necessitates the incorporation of an amplification factor for column shear or a particular stress resultant during the seismic design of a steel structure with masonry infills. The design codes might include amplification factors for column shear or any particular stress resultant based on similar analyses on a large number of structures with variable parameters and distribution of masonry infill. The amplification factors need not be constant; these may be influenced by a number of parameters including the height of the frame, number of bays and bay width, number of storeys, etc.

4. Conclusions

The effect of the presence and the distribution of masonry infill walls on mid-rise steel frames with soft storey were investigated by performing nonlinear finite element analysis. The results of the analysis lead to the following conclusions.

- The distribution of the masonry infill in the upper storeys plays an important role on the values of the column shear force, the storey drift and the base shear, when the ground storey is open without any masonry infill. It was observed that for the same amount of masonry infill, the distribution can increase the column shear and the storey drift by 10%–20%. This suggests that the distribution of masonry infill should be taken into account when designing a steel structure.

• Modal analysis of a bare frame produces a natural period of the structure two times the NBCC (2005) code equation, but with increase in the amount of the masonry infill, the modal value tends to converge with the code value, which indicates that for better dynamic analysis of steel structures, the presence of masonry infill walls should be included in the analysis.

• The base shear capacity of a steel frame increases with increase in the amount of masonry infill due the contribution of the masonry infill in the lateral load carrying capacity of the structure. The displacement capacity at first column yield decreases with increase in the amount of infill, which is attributed to the sudden change in the stiffness of the structure due to the presence of soft storey at the ground storey level, and with the increase in the amount of masonry infill in the upper storey the change in stiffness becomes even larger and makes the condition more vulnerable. As a result, in presence of soft ground storey, the ground storey columns yield first.

• When the frame has a soft bottom storey and infilled upper storeys, there is a sudden increase in the column shear and the storey drift at the bottom storey level. This increase in the column shear was found to be between 30-38% for mid-rise steel frames. Sudden increase in the storey drift and the column shear will lead to sudden increase in the column bending moment. Thus, there is a possibility of underestimating these forces during the design of multi-storey buildings with open ground storey and masonry infilled upper storeys using conventional design procedure, which may lead to an unsafe structure. Hence, this paper recommended that the column shear needs to be amplified by a factor to account for the effect of the sudden increase in column shear and it was observed that for a mid-rise steel frame with four bays the amplification factor can be as much as 1.4. The design codes might include these amplification factors based on a large number of analyses on structures with variable parameters and infill distributions.

• The amount of masonry infill affects the overstrength and the ductility related force modification factors for steel frames. The overstrength related force modification factor increases and the ductility related force modification factor decreases with increase in the amount of infill. This is attributed to the additional load carrying capacity provided by the masonry infill which eventually decreases the ductility of the steel frames. Two design equations were proposed for mid-rise steel frames to determine the overstrength and the ductility related force modification factors, which showed good agreement with the FEA results. However, in order to develop generalized equations for the force modification factors, an extensive analysis should be performed on a large number of steel frames with different heights and bays, and infill distributions.

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Appendix A. Strut and shear curve input parameters of the masonry infill panel

Input Parameters	Definition
Initial young modulus, E_m	This parameter represents the initial slope of the stress-strain curve
Compressive strength, f_m	Compressive strength refers to the diagonal capacity of the infill panel, that is, it refers to the capacity of the masonry in the direction of the principal stress, which, typically, is assumed to coincide with the diagonal that links two opposite corner nodes
Tensile strength, f_t	The tensile strength represents the tensile strength of the masonry or the bond-strength of the interface between frame and infill panel
Strain at maximum stress, ε_m	This parameter represents the strain at maximum stress and influences, via the modification of the secant stiffness, the ascending branch of the stress-strain curve
Ultimate strain, ε_u	This strain is used to control the descending branch of the stress-strain curve, modelled with a parabola so as to obtain better control of the strut's response
Closing strain, ε_{cl}	Closing strain presents the limiting strain at which cracks are closed partially and compressive stresses are resisted
Starting unloading stiffness factor, g_u	This parameter controls the slope of the unloading branch of the envelope curve
Strain reloading factor, a_r	It defines the point on the strength envelope, where the reloading curve reach the strength envelope
Maximum shear resistance, $\tau_{\max}$	This is the largest shear stress that may be mobilised by the infill panel
Reduction shear factor, α_s	This empirical parameter represents the ratio between the maximum shear stress and the average stress in the panel