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EFFECT OF INFILL STIFFNESS ON SEISMIC PERFORMANCE OF MULTI-STOREY RC FRAMED BUILDINGS IN INDIA

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SUMMARY

The typical multi-storey construction in India comprises reinforced concrete (RC) frames with brick masonry infills. Unreinforced masonry infill wall panels may not contribute towards resisting gravity loads, but contribute significantly, in terms of enhanced stiffness and strength under earthquake (or wind) induced lateral loading. However, in practice, the infill stiffness is commonly ignored in frame analysis, resulting in an under-estimation of stiffness and natural frequency. Also, the infills have energy dissipation characteristics that contribute to improved seismic resistance. It is instructive to study the implications of the common practice of ignoring the infill stiffness with regard to performance under seismic loading.

Two typical existing buildings located in moderate seismic zones of India (as per IS: 1893-2002[1]) are identified. Features like plan irregularity and vertical irregularity (soft storey) are found in one of the buildings, while the other is fairly symmetric. Infills were modelled using the equivalent strut approach. Static analysis (for gravity and lateral loads), response spectrum analysis and non-linear pushover analysis (assigning the hinge properties to beams and column sections) were performed. It is observed that the seismic demand at the soft storey level is significantly large when infill stiffness is considered, with larger base shear and larger displacements. This effect, however, is not found to be significant in the symmetric building (without soft storey). Seismic performance was compared in the pushover analysis for the two cases. The results are described in detail in this paper.

INTRODUCTION

A large number of buildings in India are constructed with masonry infills for functional and architectural reasons. Masonry infills are normally considered as non-structural elements and their stiffness contributions are generally ignored in practice. However, infill walls tend to interact with the frame when the structure is subjected to lateral loads, and also exhibit energy-dissipation characteristics under seismic loading. Masonry walls contribute to the stiffness of the infill under the action of lateral load. The term ‘infilled frame’ is used to denote a composite structure formed by the combination of a moment resisting

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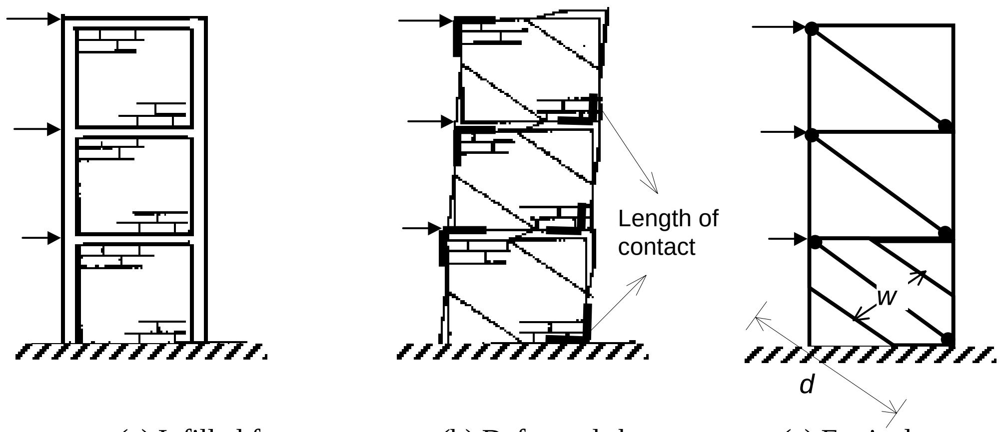
plane frame and infill walls. The infill may be integral or non-integral depending on the connectivity of the infill to the frame. In the case of buildings under consideration, integral connection is assumed. The composite behaviour of an infilled frame imparts lateral stiffness and strength to the building. The typical behaviour of an infilled frame subjected to lateral load is illustrated in Figures 1 (a) and (b). Two building case studies are presented here.

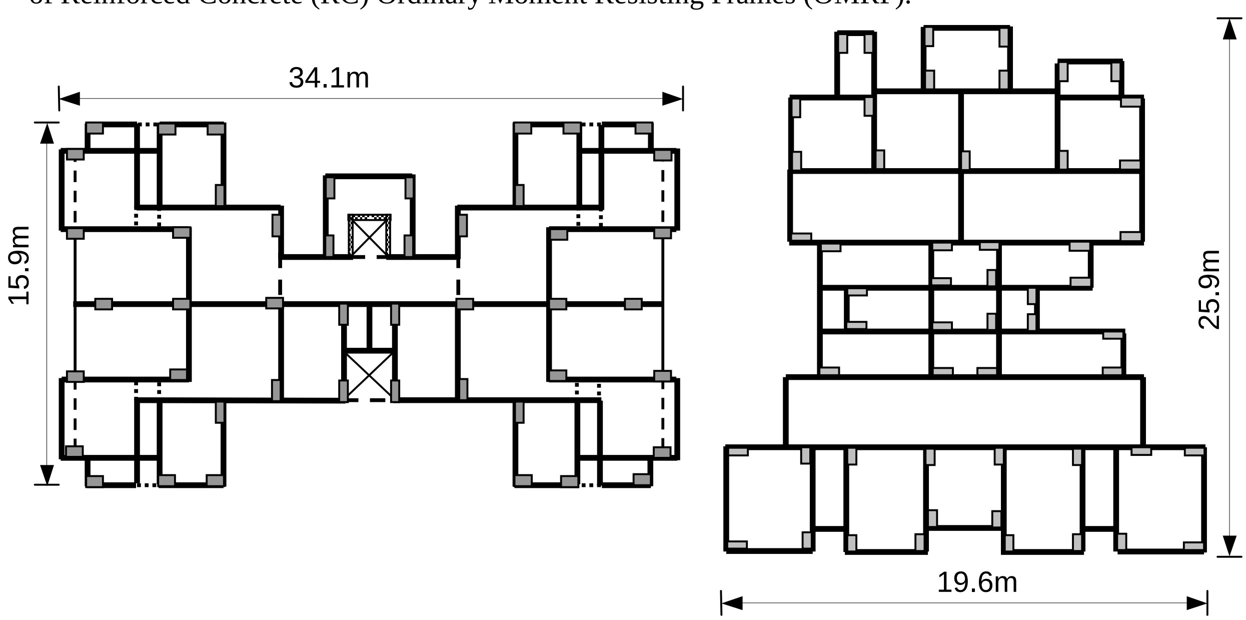


(a) Infilled frame (b) Deformed shape (c) Equivalent strut

Figure 1: Behaviour of Infilled Frames (Govindan, 1986 [2])

BUILDING DESCRIPTION

Two typical buildings, Building A1 and Building A2 (Figure 2), were selected in the moderate seismic zone (Zone III) of India. Building A2 has features like plan irregularity and vertical irregularity whereas Building A1 is fairly symmetric in plan and in elevation. Building A1 is a Ground + 7 storey building (25.4 m high) and Building A2 is a Ground + 4 storey building (16.8 m high). Both the buildings are made of Reinforced Concrete (RC) Ordinary Moment Resisting Frames (OMRF).



(a) Building A1 (b) Building A2

Figure 2: Typical floor plans

In both the buildings, the columns are supported on isolated footings with plinth beams. The concrete slab is 150 mm thick at each floor level. The brick wall thicknesses are 230 mm for external walls and 120 mm for internal walls. Imposed load is taken as 2 kN/m^2 for all floors. The grade of concrete and steel used in both the buildings are M20 and Fe 415 respectively.

STRUCTURAL MODELLING

The two buildings are modelled and analysed for static, response spectrum and pushover analyses, using the finite element package SAP2000. The analytical models of the buildings include all components that influence the mass, strength and stiffness. The non-structural elements and components that do not significantly influence the building behaviour were not modelled. The floor slabs are assumed to act as diaphragms, which ensure integral action of all the vertical lateral load-resisting elements. Beams and columns were modelled as frame elements with the centrelines joined at nodes. Rigid offsets were provided from the nodes to the faces of the columns or beams. The stiffness for columns and beams were taken as $0.7EI_g$, accounting for the cracking in the members and the contribution of flanges in the beams.

The weight of the slab was distributed to the surrounding beams as per IS 456: 2000 [4], Clause 24.5. The mass of the slab was lumped at the centre of mass location at each floor level. This was located at the design eccentricity (based on IS 1893:2002 [1]) from the calculated centre of stiffness. Design lateral forces at each storey level were applied at the centre of mass locations independently in two horizontal directions (*X*- and *Y*- directions).

Staircases and water tanks were not modelled for their stiffness but their masses were considered in the static and dynamic analyses. The design spectrum for medium soil as specified in IS 1893:2002 [1] was used for the analyses. The effect of soil-structure interaction was ignored in the analyses. The columns were assumed to be hinged at the level of the bottom of the base slabs of respective isolated footings.

The first ten modes were considered for the dynamic analysis (response spectrum method), which gives more than 99% mass participation in both the horizontal directions. The SRSS method of modal combination was used for analysis.

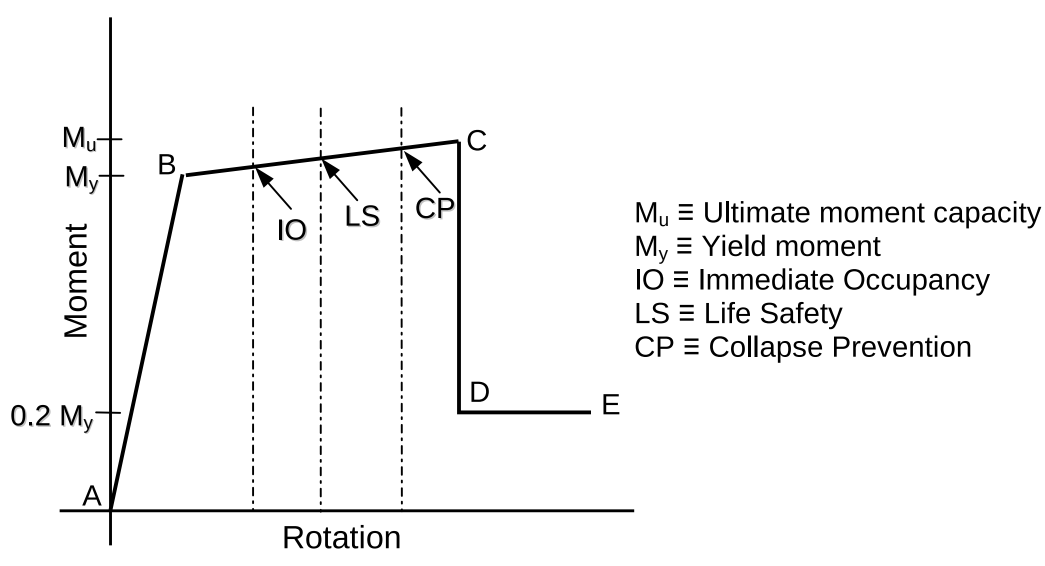


Figure 3: Typical moment-rotation relations for plastic hinges.

For pushover analysis, beams and columns were modelled with concentrated plastic hinges for flexure and shear at the column and beam faces, respectively. Beams have both moment (M3) and shear (V2) hinges, whereas columns have axial load and biaxial moment (PMM) hinges and shear hinges in two directions (V2 and V3). The normalised moment-rotation relations for the hinges were obtained from IS 456:2000 [4] (Figure 3)

The moment-rotation relations were taken as symmetric in the positive and negative sides of the bending moment axis. The default ACI 318-95 interaction surface (with $\phi = 1$) was considered for the column hinge.

Modelling of masonry infill

In the case of an infill wall located in a lateral load resisting frame the stiffness and strength contribution of the infill are considered by modelling the infill as an equivalent compression strut (Smith [5]).

Because of its simplicity, several investigators have recommended the equivalent strut concept. In the present analysis, a trussed frame model is considered. This type of model does not neglect the bending moment in beams and columns. Rigid joints connect the beams and columns, but pin joints at the beam-to-column junctions connect the equivalent struts.

Infill parameters (effective width, elastic modulus and strength) are calculated using the method recommended by Smith [5]. The length of the strut is given by the diagonal distance *d* of the panel (Figure 1c) and its thickness is given by the thickness of the infill wall. The estimation of width *w* of the strut is given below. The initial elastic modulus of the strut E_i is equated to E_m the elastic modulus of masonry. As per UBC (1997), E_m is given as $750f_m$, where f_m is the compressive stress of masonry in MPa. The effective width was found to depend on the relative stiffness of the infill to the frame, the magnitude of the diagonal load and the aspect ratio of the infilled panel.

The relative stiffness of the infill to the frame is expressed in terms of a parameter λ.

$$\lambda = \sqrt[4]{\frac{E_i t \sin 2\theta}{4 E_c I_c h'}} \tag{1}$$

Here, E_i is initial elastic modulus of the infill material, E_c is elastic modulus of the concrete in column, *h* is height of column between centrelines of beams, *h'* is clear height of infill wall, I_c is moment of inertia of each column, *l* is length of beam between centrelines of columns, *t* is thickness of infill wall, and $\theta = \tan^{-1}\left(h/l\right)$ is the slope of the infill diagonal to the horizontal.

The properties quantifying the behaviour of the equivalent strut are all expressed as functions of λh, a non-dimensional quantity, either mathematically or graphically. The stiffness and strength of the equivalent strut depend not only on the infill dimensions and material properties, but also on the lengths of contact of the infill at the frame elements (Smith [5]). Variations in the column stiffness can influence the mode of failure and lateral stiffness of the infill. It was found that the length of contact at the column (α_c) is more important than at the beam (α_b).

The length of contact at the column (α_c) at the compression diagonal corner is calculated using the following formula.

$$\frac{\alpha_c}{h} = \frac{\pi}{2\lambda h} \tag{2}$$

As the lengths of contact change with increasing load, the effective width of the equivalent strut (*w*) is not constant, but decreases. The width can be expressed as *w/d*, where *d* is the length of the strut between the beam-to-column joint nodes. Curves for *w/d* are plotted as a function of λh for various panel aspect ratios (Smith [5]). The reduction of *w* with increasing load includes the reduction of the elastic modulus of the infill.

From the curves, the following relationships were derived using a regression analysis (Govindan [2]).

$$w/d = \begin{cases} 0.580\,(l'/h')^{-0.445}(\lambda h)^{-0.335(l'/h')^{0.064}} & : R/R_C = 0 \\ w/d = 0.286(l'/h')^{-0.18}(\lambda h)^{-0.202(l'/h')^{0.041}} & : R/R_C = 1 \end{cases} \quad (3)$$

For intermediate values of R/R_c, between 0 and 1 the value of *w/d* can be interpolated linearly.

Figure 4 shows a typical force-deformation relation for the axial hinge in strut. The force-deformation relationship in compression is obtained from literature (Reinhorn [6]). It is a linear curve till the peak stress is reached followed by a plastic behaviour. For higher strains, the stress is assumed to drop with increasing strain to a small fraction of the peak value and thereafter the stress remains almost constant. The tensile strength of the masonry infill was neglected for the analysis. It provides a rational basis for estimating the lateral strength and stiffness of the infilled frames as well as the infill diagonal-cracking load.

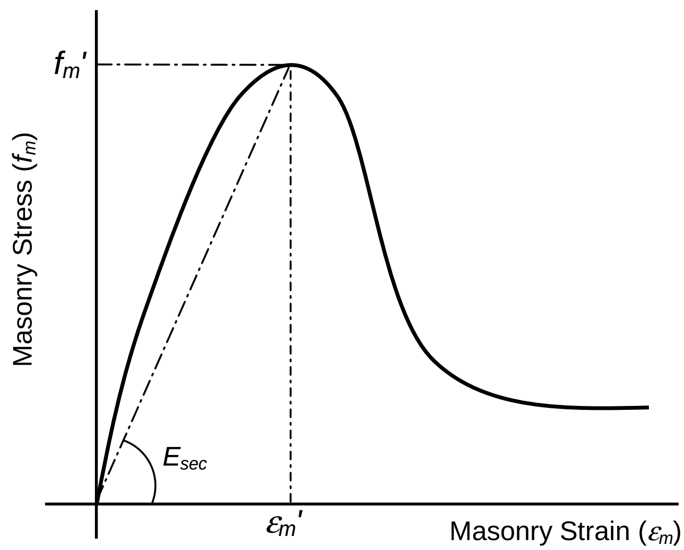
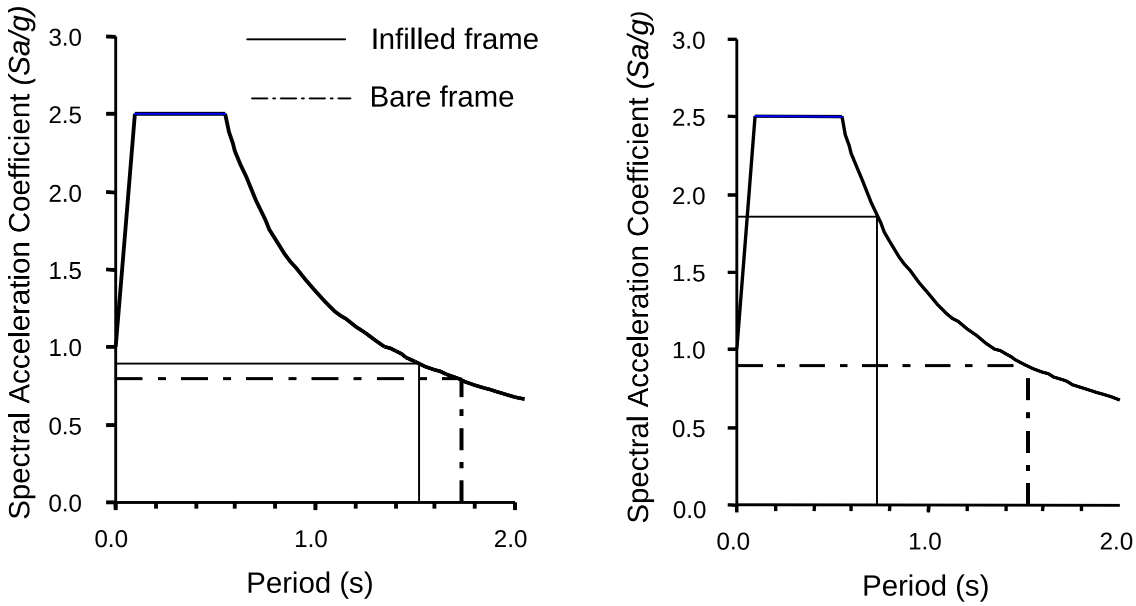


Figure 4: A typical stress-strain relation for axial hinges in equivalent struts (Reinhorn [6])

ANALYSIS RESULTS AND DISCUSSION

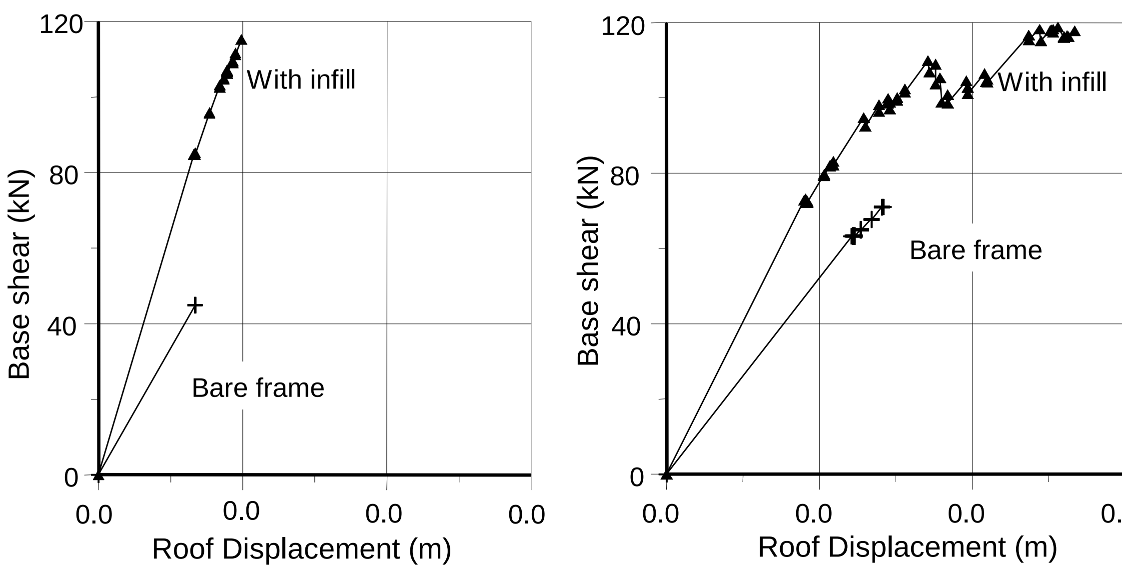
Both the buildings were analysed using equivalent static method (linear static method) and response spectrum method (linear dynamic method) according to IS 1893:2002[1]. Pushover analysis (non-linear static method) was also carried out. The pushover analysis provides an insight into the structural aspects, which control the performance during earthquakes. It also provides data on the strength and ductility of a building. The analyses were done by using the finite element analysis software, SAP2000.

The periods of vibration of the buildings were determined from eigenvalue analyses using SAP 2000. When infill stiffness is considered, the fundamental period of the structure reduces and the structure attracts more base shear. Figure 5 shows the fundamental period of two buildings analysed with and without considering infill stiffness and the corresponding spectral acceleration. As Building A1 has open ground storey, the difference in the fundamental period is relatively less, but for the regular building A2, there is a drastic decrease in the fundamental time period, and hence a corresponding under estimation of the seismic demand.



(a) Building A1 (b) Building A2

Figure 5: Fundamental period and corresponding spectral acceleration

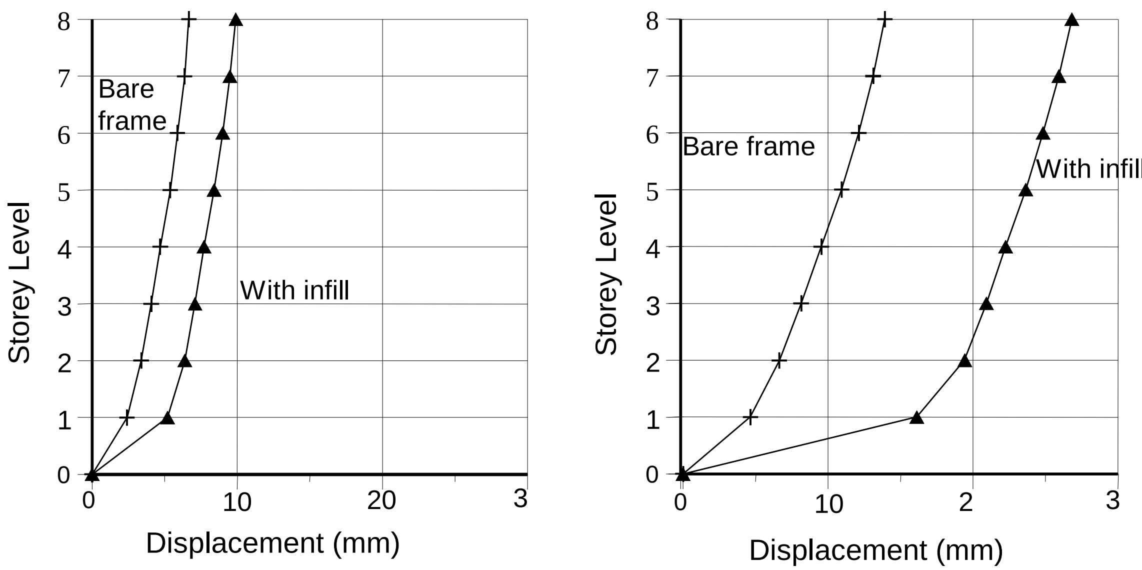


(a) X-direction (b) Y-direction

Figure 6: Base shear versus Roof displacement curve for Building A1

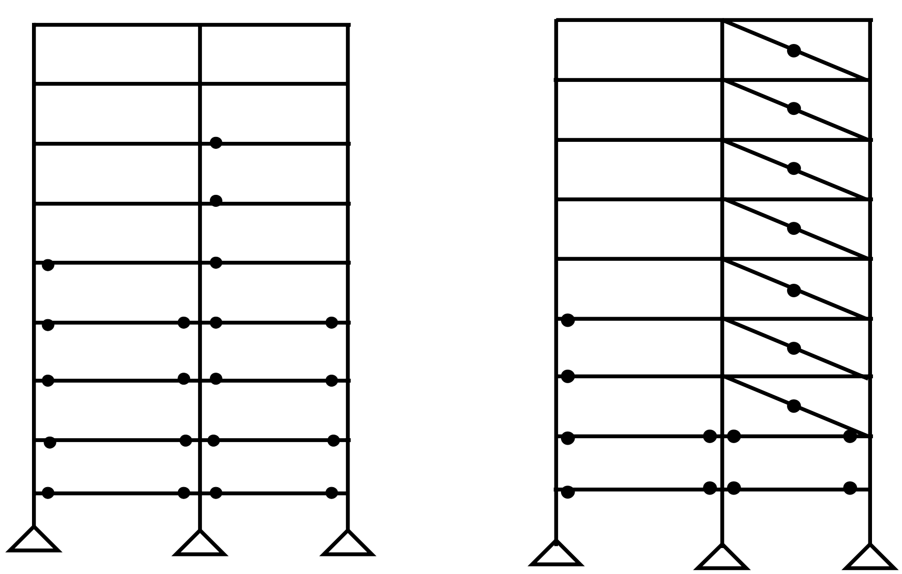
Figure 6 shows the base shear versus roof displacement curve for Building A1 in both the directions as obtained from pushover analyses. The curve shows that consideration of infill stiffness gives 150% and 64% more base shear capacity along *X* and *Y* directions, respectively. It also increases the ductility by an amount of 47% and 88% along *X* and *Y* directions, respectively.

Figure 7 shows displacement profile for Building A1 at the formation of mechanism. The building is significantly less stiff in *Y*-direction, compared to the *X*-direction, and the maximum drift in the *Y*-direction works out to 27mm. It is only 0.1% of the total height of the building and it satisfies the requirement of inelastic drift limitation of 1.2% of total height.



(a) *X*-direction (b) *Y*-direction

Figure 7: Displacement profile of Building A1 at the formation of mechanism

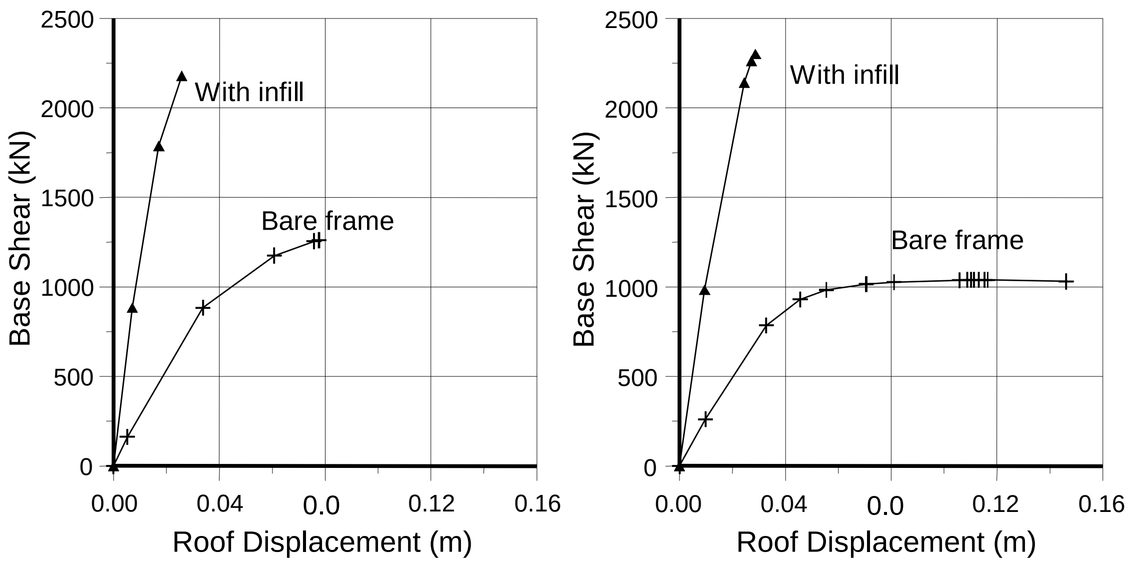


(a) Bare frame model (b) Infill frame model

Figure 8: Load Resisting Mechanism for a typical frame of Building A1

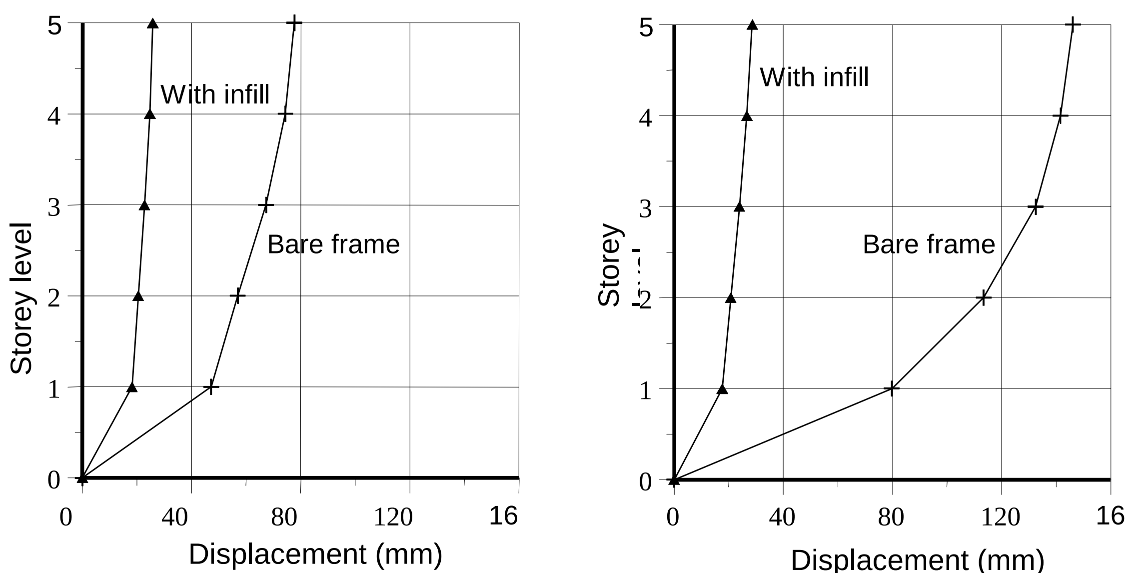
Figure 8 depicts the failure mechanism at a typical frame of Building A1. It shows that the hinge formation is concentrated in the plinth and ground levels when the infill stiffness is considered. Whereas when the infill stiffness is neglected it shows better performance as the hinges are spread over the elevation of the frame. It is also seen that bending moment in the ground floor columns are enhanced by almost 100 percent when the soft storey effect is considered, indicating that the existing buildings are vulnerable.

Figure 9 shows the base shear versus roof displacement curve for Building A2 in both the directions as obtained from pushover analyses. The curve shows that consideration of infill stiffness gives 69% and 110% more base shear capacity along *X* and *Y* directions, respectively. However the ductility in both the directions reduces considerably.



(a) *X*-direction (b) *Y*-direction

Figure 9: Base shear versus Roof displacement curve for Building A2



(a) *X*-direction (b) *Y*-direction

Figure 10: Displacement profile of Building A2 at the formation of mechanism.

Figure 10 shows displacement profile for Building A2 at the formation of mechanism. It shows that the building has almost same roof displacement in both the directions when infill stiffness is considered. But it gives fairly large displacement (0.9% of the total building height) in *Y*- direction when infill stiffness is ignored. There is no substantial increase in bending moment in the ground floor columns.

CONCLUSIONS

The influence of masonry infill on the response of multi-storeyed building under seismic loading is illustrated through typical examples. The presence of masonry infill panels modifies the structural force distribution significantly. The total storey shear force increases considerably as the stiffness of the building increases in the presence of masonry infill. Also, the bending moments in the ground floor columns increase (more than twofold), and the mode of failure is by soft storey mechanism (formation of hinges in ground floor columns)

The lateral load resisting mechanism of the masonry infilled frame essentially different from the bare frame. The bare frame acts primarily as a moment resisting frame with the formation of plastic hinges at the joints under lateral loads. In contrast, the infill frame behaves like a braced frame resisted by a truss mechanism formed by the compression in the masonry infill panel and tension in the column. The plastic hinges are confined with the joint in contact with the infill panel. It is seen that the existing buildings with open ground storey are deficient and in need of retrofit.

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