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by

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Seismic Behaviour of RC Frame with URM Infill: A Case Study

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Abstract
Masonry is one of the most common types of construction materials in India. It is commonly used for infill walls in Ordinary Moment Resisting Frames. The strength based design in the code considers the masonry wall as non-structural element and its load is considered on the corresponding elements. The numerical modeling of infill is completely disregarded during analysis of the structure. To design a building realistically, the behaviour of all the primary components is needed and the load carrying capacities are required to be properly assessed. Apart from the strength based design adopted in the code of practice, the modern and advanced approach to design, the performance based design, needs a proper understanding of the behaviour of all the structural elements. Analytical modeling of infill is attempted by many researchers using equivalent strut and tie models which can idealize the behaviour of masonry infill to some extent. The effect of masonry wall on RC frame can be well understood by continuum modeling. Numerical modeling of masonry infill wall is done using Finite Element Method.
This paper attempts to simulate the nonlinear behaviour of URM infill frames with varying storeys using Applied Element Method (ARM). Two bay ordinary moment resisting frame is considered with and without infill and capacity is obtained using Nonlinear Static Pushover analysis. One, two and three storey frames with and without openings are also analyzed to understand the differences in drift ratios and the strength of the frame. It is observed that the bare frame carries lesser drift when compared to the frame with infill. This clearly proves the increase in ductility of the frame when provided with infill. In the analyses of infill frame without door and window openings, the maximum base shear of the frame decreased and the drift carrying capacity increased with the increase in number of storeys. Almost the similar behaviour is observed when the infill frame with openings is analyzed.

Keywords
Un-Reinforced Masonry (URM), Ordinary Moment Resisting Frame (OMRF), Strut and Tie models, Applied Element Method (AEM), Pushover analysis, drift ratio.

I. Introduction

India is one of such countries which are exposed to moderate to high intensity earthquakes. It has experienced devastating quakes which awakened the researchers to urgently understand the seismic behaviour of the buildings. Bhuj earthquake which occurred on 26 January 2001 had devastating effects on many buildings. The greatest damage occurred in an area of 45930 km2 which covers 22% of Gujarat state. [1] Out of many types of structures, RC moment resisting frames with masonry infill walls are one of the most common types of construction practices in India. Though the unreinforced masonry walls are constructed in practice, the design is not done for the complete modeling. These walls are neglected in the strength based design provided in standard code of practice. Understanding the seismic behaviour of Unreinforced Masonry Infill wall (URM) when the frame is with different configurations is the main objective of this paper.

Several methods have been proposed in the literature for modeling masonry infills, such as, equivalent diagonal strut method, equivalent frame method, finite element method with masonry wall discritized into several elements etc. The equivalent strut models are reliably accurate in estimating the initial stiffness and lateral strength of masonry infilled RC moment resisting frames [2]. On the other hand, these models are not completely reliable as they cannot model the behavioral effects of the infill on the frame. The main drawback of these models is that they cannot generate the interaction of the infill frames and the columns adjacent to them. Apart from this, these models can be used only for static loads (both linear and nonlinear analysis) and cannot be used for cyclic or dynamic loads because the reversal of the forces takes place and the behaviour of the infill changes with the change in the loading pattern. This cannot be modeled using single diagonal strut provided in one diagonal. Though this problem of dynamic analysis is solved using diagonal strut in both the directions, it neglects the previous drawback. Apart from this, these models cannot be used if there are substantial openings in the infill.
The above mentioned drawbacks will be solved when the RC frame with infills is modeled using Finite Element continuum modeling. This modeling can model the actual behaviour except that it cannot develop the proper interaction of the wall- column interface. Hence a new method called Applied Element method developed by Hatem (1998) is used to model the frames. These micro modeling techniques use the basic CAP model for generating the yield surfaces wherein two masonry components are modeled separately and the interface elements are used as potential crack, slip or crushing planes. This interface cap model captures all the masonry failure mechanisms. This assumption leads to a robust type of modeling, capable of following the complete load path of the structure until total degradation of stiffness [3].

II. Applied Element Method

In AEM, structure is assumed to be virtually divided into small square elements each of which is connected by pairs of normal and shear springs set at contact locations with adjacent elements. These springs bear the constitutive properties of the domain material in the respective area of representations. Global stiffness of structure is built up with all element stiffness contributed by that of springs around corresponding element. Global matrix equation is solved for three degrees of freedom of these elements for 2D problem. Stress and strain are defined based on displacement of spring end points of element edges [4].
Anisotropy of the masonry is accounted by considering masonry as a two phase material with brick units and mortar joint set in a regular interval. Structure is discritized such that each brick unit is represented by a set of square elements where mortar joints lie in their corresponding contact edges (see fig. 1).

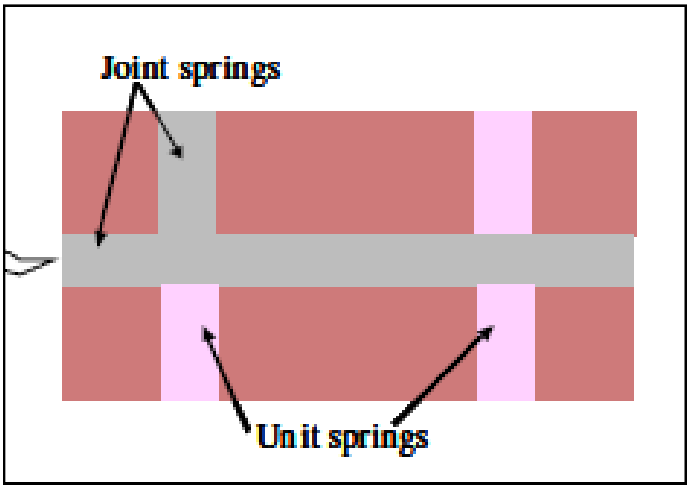

Fig. 1: Brick Masonry Discretization

Springs that lie within one unit of brick are termed as 'unit springs' and they are assigned to structural properties of brick. Springs those accommodate mortar joints are treated as 'joint springs' and are defined by equivalent properties based on respective portion of unit and mortar thickness.

The initial elastic stiffness values of unit and joint springs are given by equations below

$$K_{nunit} = \frac{E_u\ t\ d}{a};\quad K_{njoint} = \frac{E_u\ E_m t\ d}{E_u \text{ x } t_h + E_m(a-t_h)} \tag{1}$$

$$K_{sunit} = \frac{G_u\ t\ d}{a};\quad K_{sjoint} = \frac{G_u\ G_m t\ d}{G_u \text{ x } t_h + G_m(a-t_h)} \tag{2}$$

Where E_u and E_m and G_u and G_m are Young's and shear modulus for brick unit and mortar, respectively. Thickness of wall and mortar thickness are denoted by 't' and 't_h' respectively. Dimension of element size is represented by 'a' and the fraction part of element size that each spring represents is 'd'. While assembling the spring stiffness for global matrix generation, contribution of all springs around the structural element are added up irrespective to the type of spring. In the sense, for global solution of problem, there is no distinction of different phase of material but only their corresponding contribution to the stiffness system.

A. Material Models

Material model used was a composite model that takes into account the brick and mortar with their respective constitutive relation with elastic and plastic behaviour of hardening and softening. Brick springs were assumed to follow principal stress failure criteria with linear elastic behaviour [5]. Once there is splitting of brick reaching elastic limit, normal and shear stress are assumed not to transfer through cracked surface in tensile state. The brick spring's failure criterion is based on a failure envelope given by Equation

$$\frac{f_b}{f'_b} + \frac{f_t}{f'_t} = 1$$

Where f_b and f_t are the principal compression and tensile stresses, respectively, and f'_b and f'_t are the uniaxial compression and tensile strengths, respectively.

Coulomb's friction surface with tension cut-off is used as yield surface after which softening of cohesion and maximum tension takes place in exponential form as a function of fracture energy values and state variables of damage. The cohesion and bond values are constant till the stress first time when stress exceeds the respective failure envelopes. Fig. 1 and 2 shows the degradation scheme of bond and cohesion. Failure modes that come from joint participation of unit and mortar in high compressive stress is considered by liberalized compression cap as shown in fig. 3. The effective masonry compressive stress used for cap mode follows hardening and softening law as shown in fig. 4. The tension cut-off, f_1, and the sliding along joints, f_2, exhibit softening behavior whereas the compression cap experiences hardening at first and then softening. The failure surfaces used in this study derived from Lourenço, (1997), with some simplification are as given in Equations. (Fig. 4)

$$f_1(\sigma, K_1) = \sigma - f_t\ \exp\left(-\frac{f_t}{G_f^I}K_1\right) \tag{4}$$

$$f_2(\sigma, K_2) = |\tau| + \sigma\ \tan(\phi_1) - c\exp\left(-\frac{c}{G_f^{II}}K_2\right) \tag{5}$$

$$f_3(\sigma, K_3) = |\tau| + \sigma\ \tan(\phi_2)\ \{(\sigma_3(K_3) - \sigma)\} \tag{6}$$

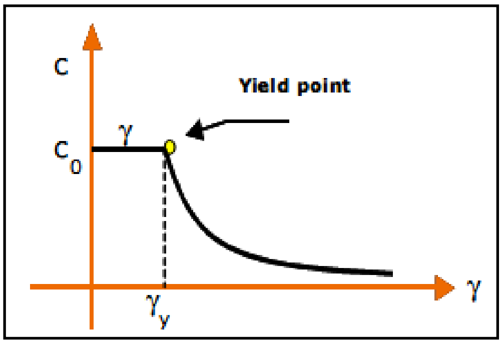

Fig. 2: Cohesion Degradation

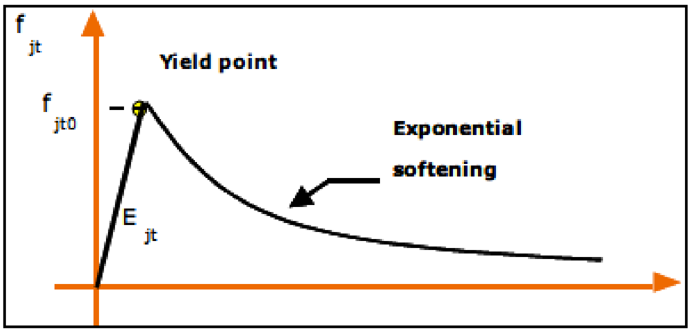

Fig. 3: Bond Degradation

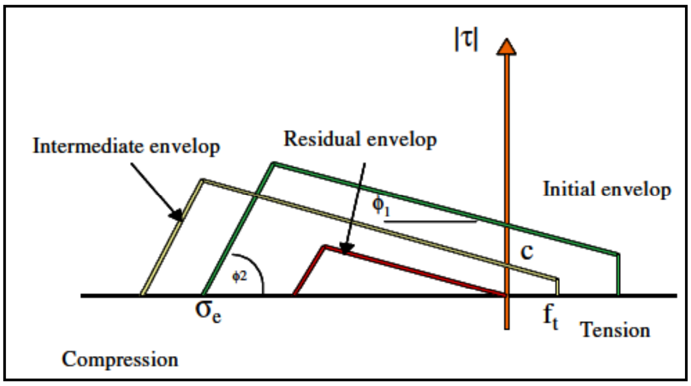

Fig. 4: Failure Criteria for Joint Spring

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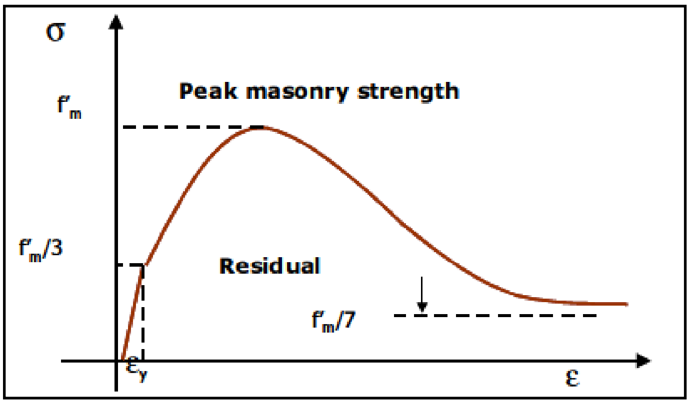


Fig. 5: Hardening and Softening Applied for Joint Spring in Compression Cap

In above equations, K_1, K_2 and K_3 are hardening and softening parameters for tension, shear and compression behavior respectively. G_{If} and G_{IIf} is fracture energy in tension and shear respectively.

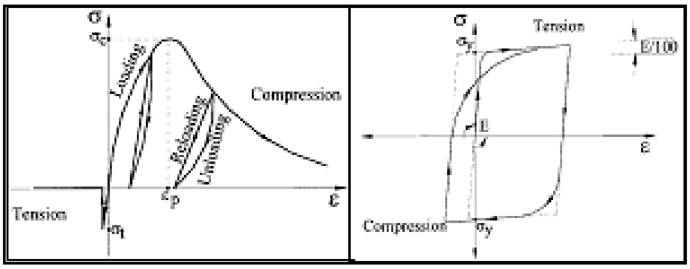


Fig. 6: Material Models for Concrete and Steel

As a material modeling of concrete under compression condition, Maekawa compression model [Okamura and Maekawa, 1991], as shown in fig. 6 is adopted. The tangent modulus is calculated according to the strain at the spring location and whether the spring is in loading or unloading process. Once the peak compression stresses are reached, stiffness is assumed to be minimum value (1% of initial value) to avoid singular stiffness matrix. The difference between calculated spring stress and stress corresponding to the strain at the spring location are redistributed each increment. Till the cracking point stresses are assumed to be proportional to strains and after that stiffness is assumed as minimum value (1% of initial value) to avoid having a singular stiffness matrix. For reinforcement, bilinear stress strain relation is assumed. After yield of reinforcement, steel spring stiffness is assumed as 1% of the initial stiffness. No model is used, up to this stage, for cut of reinforcement because the behaviour of the structure becomes mainly dynamic behaviour and the static stiffness matrix becomes singular.

It should be emphasized that some other failure phenomena, like buckling of reinforcement and spalling of concrete cover, are not considered in the analysis in this analysis. However, the shear transfer and shear softening are approximately considered in the analysis. For more details about material models used and the results in case of monotonic loading conditions, refer to [Meguro and Tagel-din, 2001].

B. Failure Criteria

One of the main problems associated with the use of elements having three degrees of freedom is the modeling of diagonal cracking. Applying Mohr-Coulomb's failure criteria calculated from normal and shear springs, not based on principal stresses, has some problems. When the structure is really composed of individual elements, such as granular material or brick masonry buildings, Mohr-Coulomb's failure criteria is reasonable. However, when we use elements by dividing the structure virtually, which are not really composed of elements, for convenience of numerical simulation, adopting Mohr-Coulomb's failure criterion leads to inaccurate simulation of fracture behaviour of the structure. The idea of the technique is how to use the calculated stresses around each element to detect the occurrence of cracks. To determine the principal stresses at each spring location, the following technique is used. Referring to fig. 7 the shear and normal stress components (τ and $\sigma 1$) at point (A) are determined from the normal and shear springs attached at the contact point location.

The secondary stress ($\sigma 2$) can be calculated by Eq. (7) from normal stresses in points (B) and (C), as shown in Figure 7

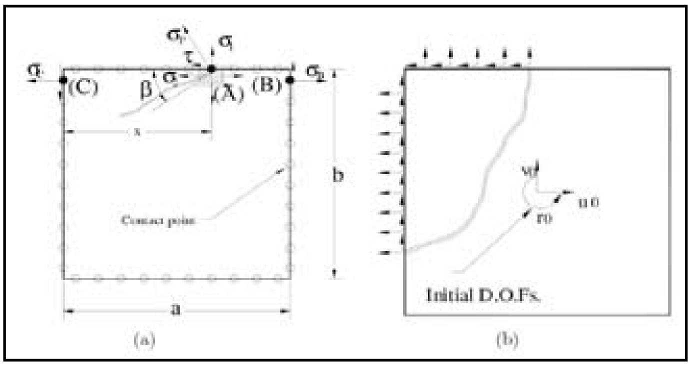


Fig. 7: Failure Criteria for Springs

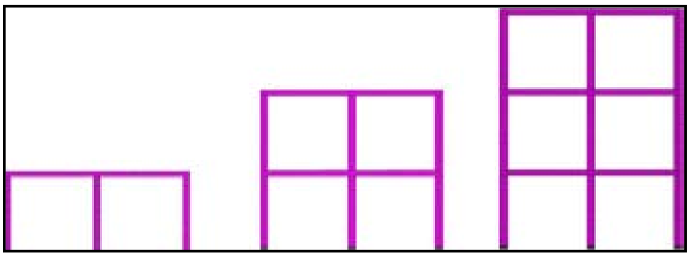
$$\sigma_2 = \frac{x}{a}\sigma_B + \frac{(a-x)}{a}\sigma_C \tag{7}$$

$$\sigma_p = \left(\frac{\sigma_1+\sigma_2}{2}\right) + \sqrt{\left(\frac{\sigma_1-\sigma_2}{2}\right)^2 + (\tau)^2} \tag{8}$$

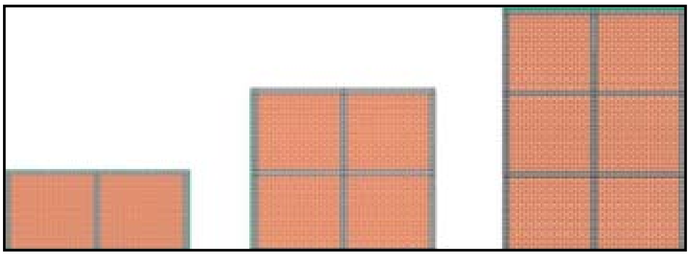
The value of principal stress (σp) is compared with the tension resistance of the studied material. When σp exceeds the critical value of tension resistance, the normal and shear spring forces are redistributed in the next increment by applying the normal and shear spring forces in the reverse direction. These redistributed forces are transferred to the element centre as a force and moment, and then these redistributed forces are applied to the structure in the next increment. The redistribution of spring forces at the crack location is very important for following the proper crack propagation. For the normal spring, the whole force value is redistributed to have zero tension stress at the crack faces. Although shear springs at the location of tension cracking might have some resistance after cracking due to the effect of friction and interlocking between the crack faces, the shear stiffness is assumed to be zero after the crack occurrence. The zero value of shear stress means that the crack direction is coincident with the direction of the element edge. In shear dominant zones, the crack direction is mainly dominant by shear stress value. This technique is simple and has the advantage that no special treatment is required representing the cracking. In cases when the shear stresses are not dominant, like case of slender frames, the angle (β) tends to be zero. This indicates that the crack is parallel to the element edge and hence, high accuracy is expected.

III. Modeling

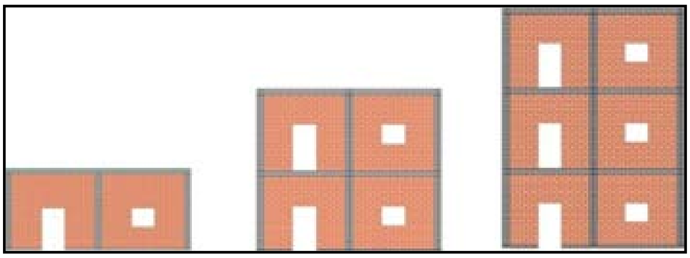
Three types of ordinary moment resisting frames are modeled. i) OMR bare frames with one, two and three storeys, (Figure 8a) ii) OMR frame with brick masonry infill walls with one, two and three storeys, (Fig. 8(b) (iii) OMR frames with brick masonry infill walls having door and window openings with one, two and three storeys (Fig. 8(c)). The structural details and material properties are given in Table 1 and Table 2 respectively.



(a)



(b)



(c)

Fig. 8: Models Considered

Table 1: Material Properties

Material Properties	
Compressive strength of concrete	20 N/mm²
Yield stress in steel	250 N/mm²
Poisson's ratio of concrete	0.2
Compressive strength of brick masonry	20 N/mm²
Tensile strength of brick masonry	8.5 N/mm²
Friction angle in mortar	0.75
Cohesion	350 kN/m²

Table 2 Structural Details

Structural Details	
Bay width	3.0 m
Height	3.99 m
No of bays	2
Column size	0.3 x 0.3 m
Beam size	0.3 x 0.23 m
Brick size	0.2 x 0.1 x 0.07 m
Main Reinforcement	
Column	8 no's – 12 Y
Beams	4 no's – 12 Y
Reinforcement	
Columns	8 Y @ 150 mm c/c
Beams	8 Y @ 150 mm c/c
Door size	0.91 x 2.03 m
Window size	0.9 x 0.9 m

IV. Analysis and Results

Nonlinear Static Pushover analysis is a method to obtain the capacity of a structure. The main objectives of pushover analysis are to obtain the maximum shear strength of the structure, to obtain the monotonic displacements and global ductility capacity of the structure and to estimate the concentration of damage that can be expected during the nonlinear seismic response [6]. In this analysis, the dynamic loads which are induced during an earthquake are idealized as static forces in different increments of time. The structure is analyzed statically in each time step separately. A set of continuous static analysis are used to represent the dynamic behaviour of earthquake. Pushover curve is a plot between lateral roof displacement and maximum base shear. This analysis describes the three design parameters of the structure, stiffness strength and ductility. The slope of the curve at any particular point represents the stiffness of the structure at that point whereas the difference between the displacement at the peak base shear and maximum displacement capacity is the ductility and the maximum base shear obtained is called the strength.

The pushover analysis is done on all the frames modeled (a, b & c), by applying a controlled displacement at the top of the particular frame. For the frames with more than one storey are assumed as single degree of freedom systems and the point of control is considered to be at the top of the frame. The pushover curves obtained are plotted against the base shear, obtained by adding the forces at the bottom fixed mesh elements, and the drift ratio obtained at the top of the frame. The collapse state of the frame is assumed to be obtained when the global strength of the structure is reached to 20% of the maximum strength of the frame. (Mehmet Inel, Hayri Baytan Ozmen) [7].

A. OMR Bare Frame

Figs. 9, 10, 11 show the pushover curves of bare frames for one, two and three storeys respectively. In all these analyses, it is clearly observed that the pushover curves show sudden drop downs at certain displacements. This is due to the failure of a set of members which reduces the global capacity of the frame considerably. The increase in the strength after sudden drop down can be accounted for the redundancy of the frame.

By closely observing the figures, it is known that the maximum strength of the frame increased with increase in number of storeys. This effect can be accounted for the redundancy which is present in the frame. Redistribution of moments and forces takes place when a particular element reaches the maximum yield. On the other hand, the drift ratio on the horizontal axis decreased with increase in number of storeys. The maximum strength carrying capacity of the structure is able sustain higher lateral loads without substantial displacements.

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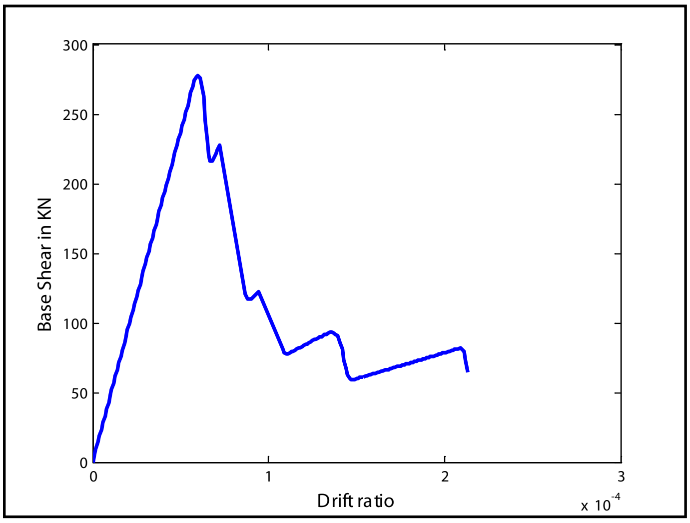

Fig. 9: Bare frame with 2 bays and 1 storey

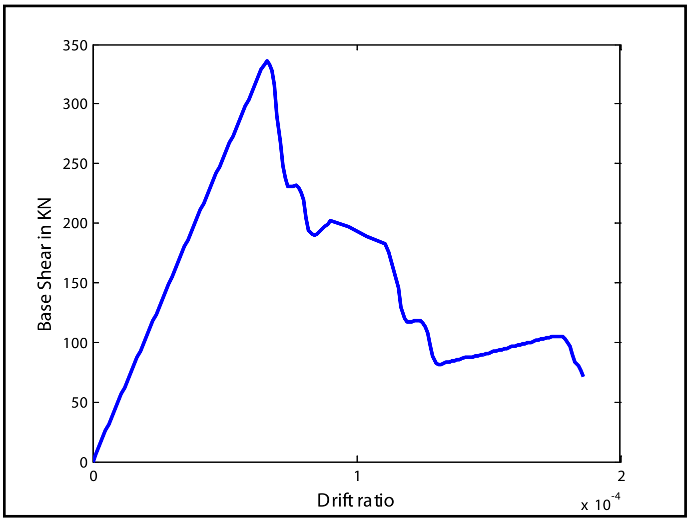

Fig. 10: Bare frame with 2 bays and 2 storey

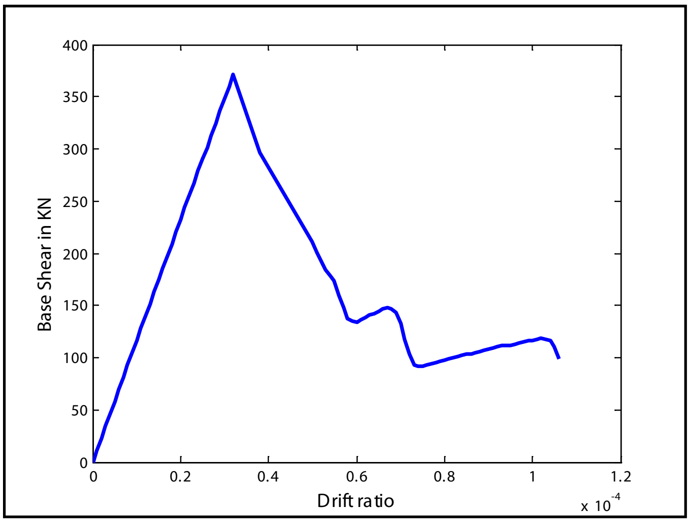

Fig. 11: Bare frame with 2 bays and 3 storey

B. OMR frames with URM Infill Walls

Capacity curves of OMR frames with URM infill walls for one, two and three storeys are shown in figs. 12, 13 and 14 respectively. These pushover curves are smoothened ones of the original curves, which represent the actual behaviour of the frames which is jagged. As it is discussed, the method of modeling is done using virtual springs for which the failure criteria is defined. It is a known fact that the masonry infill wall which is bounded by the RC columns, apply impact loads on the columns which causes additional shear stresses on the columns. Due to the increased shear stresses suddenly, the springs at the interface between the column and wall reaches the failure envelope and the energy stored in the spring is released immediately. Hence in order to obtain the proper behaviour of the frame, the pushover curves are needed to be smoothened.

Analyzing the capacity curves of the three frames, it is known that the maximum base shear is decreased with increase in the number of storeys whereas a slight increase in the drift ratio is observed. This behaviour of the frames can be accounted for the increase in ductility of the frames by decreasing the maximum load carrying capacity.

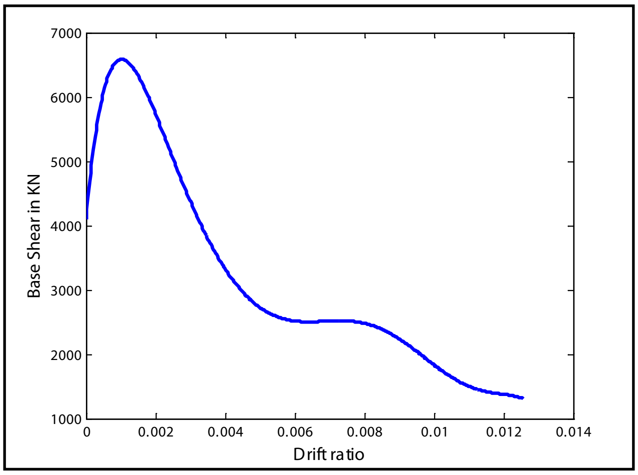

Fig. 12: Infil Frame with 2 bays and 1 storey

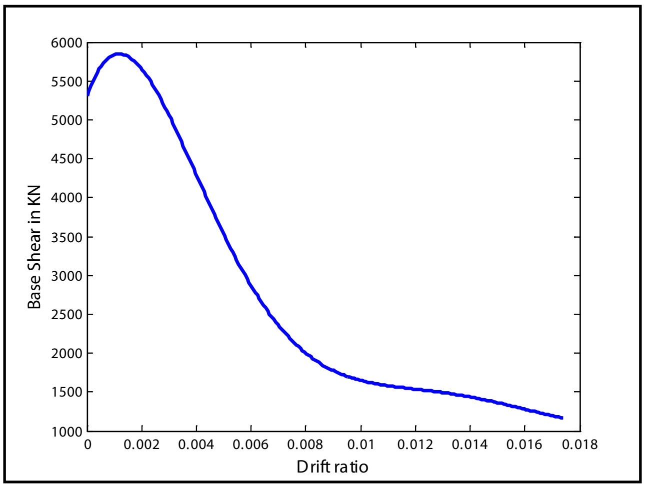

Fig. 13: Infil Frame with 2 bays and 2 storey

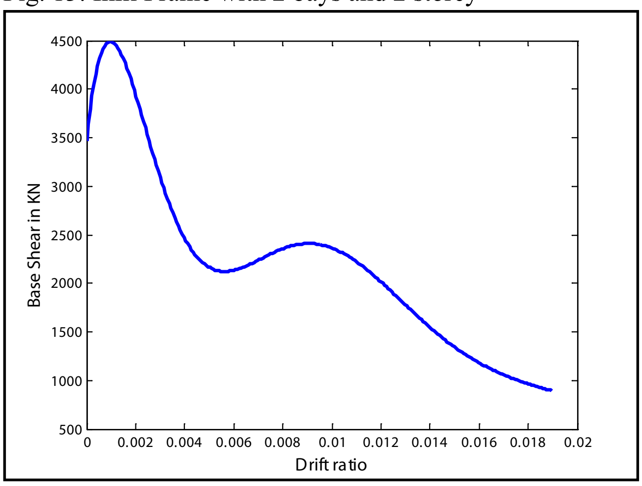

Fig. 14: Infil Frame with 2 bays and 3 storey

C. OMR frames with URM Infill Walls with openings

The capacity curves with one, two and three storey frames are shown in figs. 15, 16 and 17 respectively. As the openings are provided in the wall, the effective area resisting the lateral forces is reduced. By closely observing the obtained curves, it is found that the drift ratio is largely increased with the increase in number of storeys. Apart from this, all the curves show the brittle behaviour of the frame by sudden drop down of the peak. This is caused due to the discontinuity of the cracks that are formed in the wall at the openings.

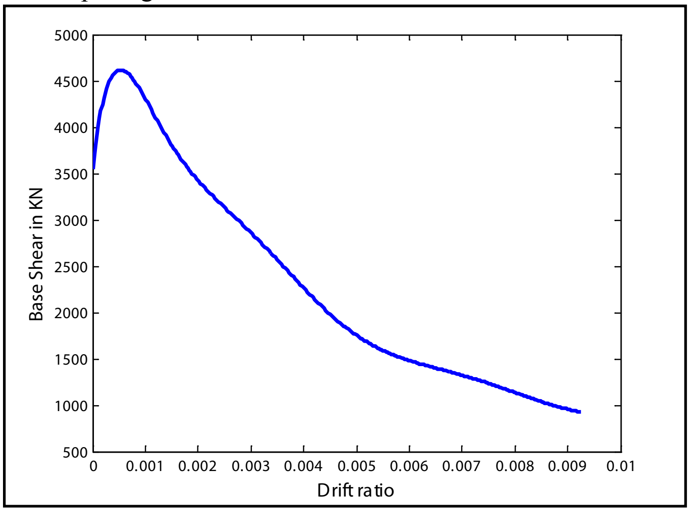

Fig. 15: 2 bays and 1 storey Infil with openings

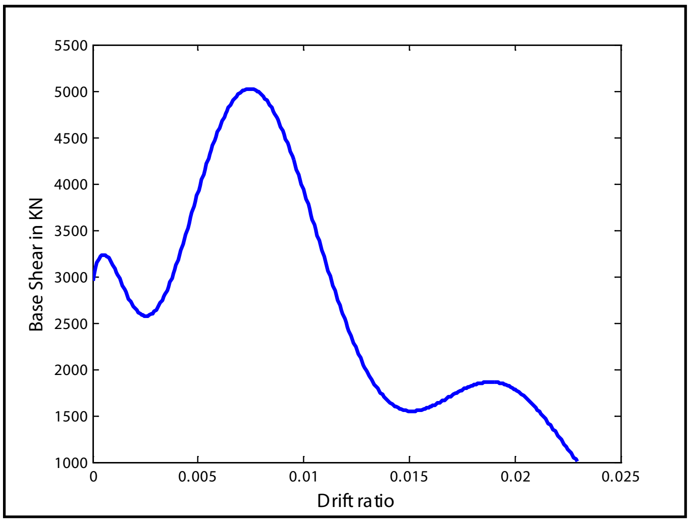

Fig. 16: 2 bays and 2 storey Infil with openings

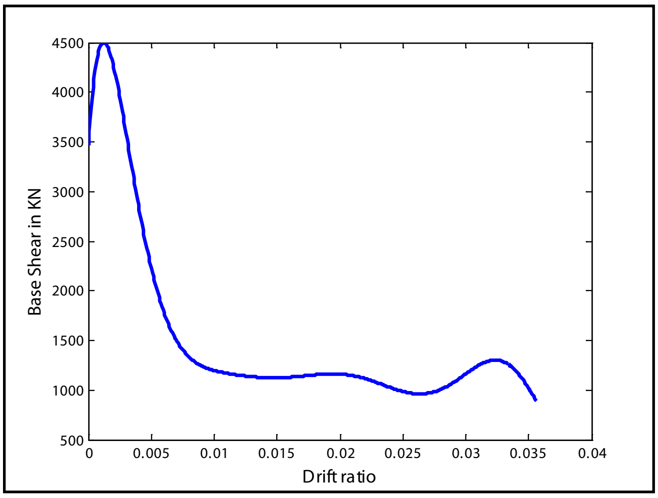

Fig. 17: 2 bays and 3 storey Infil with openings

VII. Conclusion

Three sets of 2D ordinary moment resisting frames with and without unreinforced masonry infill walls (with and without openings) are considered. Applied Element Method is used to model the frames and nonlinear static pushover analysis is carried out to obtain the capacity curves. 20% of maximum global strength is considered as the collapse state of the frame. It is observed that the strength of the frame with infill is 10 times more than the ordinary bare frame. This should be noted that the strength based design which is considering the infill walls as non structural elements needed to be revised to incorporate the effect of infill walls on the frames. Secondly, the bare frame indicated global brittle failure when compared to the frame with infill. This clearly shows that the infill wall effectively participates in resisting the lateral forces along with the RC frame. Also there is a huge variation of drift capacities between the same size of a bare frame and a frame with infill. Hence the ductility of the frame increases with the addition of the infill walls.

Thirdly, with increase in number of storeys, the strength of the bare frame increases, obviously, whereas the strength of the frame with infill decreases. This effect of infill is required to be noted and incorporated when working with different number of stories. Therefore, it can be said that the difference in behaviour of bare frame should not only be verified on a single storey but to be checked with different number of stories. Fourthly, the infilled frames with openings showed slightly different behaviour with respect to the other frames. For the same frame with one storey has a decreased strength when compared with complete wall without openings but whereas the frame with three storeys showed almost the same strength as compared to the bare frame. It is also observed that the difference in the maximum strength of the frames decreases with increase in the number of storeys. This means that the effect of openings is reducing with the number of storeys. This analysis performed on the frames with infill with openings, cannot be clearly justified only from the pushover curves. In order to understand the behaviour of these frames with openings, a different parametric study is to be carried out. Along with it stress contours are needed to be plotted on the frame in order to conclude the reasons for the behaviour of the frames with wall openings.

Apart from the present study, it is required to check the behaviour of each storey with respect to the other in order to completely understand the difference between the different stories. Also the pattern of loading that is to be applied in order to obtain the required behaviour during the earthquake is to be studied and interpreted to apply it in the design philosophy.

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