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PREDICTING SHEAR FAILURE IN COLUMNS OF MASONRY INFILLED RC FRAMES USING MACRO-MODELING APPROACH

Syed Humayun Basha(1), Hemant B. Kaushik(2)

(1)Doctoral Student, Dept of Civil Engineering, Indian Institute of Technology Guwahati, India, humayunbashasyed@gmail.com
(2)Associate Professor, Dept of Civil Engineering, Indian Institute of Technology Guwahati, India, hemantbk@iitg.ac.in

Abstract

Masonry infilled reinforced concrete (RC) frames are known to have high seismic vulnerability under strong ground motions due to the detrimental effect of infill. One of the most potential adverse effects of infill is that it may shear off weak columns, specifically ground storey exterior columns where shear force demand is higher in addition to the effect of infill. Therefore, determination of shear demand on columns due to the detrimental effect of infill is of primary importance for proper design that results in ductile flexural failure. In the current study, macro-modeling technique using equivalent diagonal strut was improved to capture the component (shear) failure as well the global response of the structure. The analytical model was calibrated from the experimental results of bare frame and infilled frame carried out by the authors. To validate the proposed modeling technique, masonry infilled frames from the past experimental studies were chosen and non-linear pushover analyses were carried out. From the results, it was observed that the improved modelling technique can be successfully used to predict the shear failure of RC columns, and hence, can be used in evaluating the realistic shear demand on RC columns.

Keywords: shear failure; masonry infill; RC frame; macro-modelling; equivalent diagonal strut

1. Introduction

Both the positive and negative effects of presence of masonry infills in reinforced concrete (RC) frames have been demonstrated from various past earthquakes [1-3]. The beneficial effects include enhanced lateral strength, stiffness, and energy dissipation; on the other hand, infills are potentially detrimental inducing captive column effect, soft-storey effect, torsion, and out-of-plane collapse if the infills are irregularly distributed in plan and elevation. From the past studies [4-6], it was observed that under lateral loading the shear demand on RC columns of masonry infilled RC frames is very high, due to the interaction between frame and infill, for which the columns are generally not designed. It was observed that the shear failure of columns may be either due to (i) the interaction of columns with strong and stiff infills which may shear-off weak columns in ground storey where contact is only on one-side, or (ii) the column is in contact with infill over a partial height creating captive column effect by decreasing the effective length of the column that has to resist the entire inter-storey drift. Therefore, determination of shear demand on columns due to the detrimental effect of infill is of primary importance for proper design that results in ductile flexural failure.

Analytical modelling of RC frames has been well established in the literature, but modelling the effect of frame-infill interaction has been a topic of research for some time due to the intricacies involved in behavior of infill under lateral loading. A lot of analytical models were proposed in the past ranging from the simplistic macro-modelling technique using single strut [7, 8], multi strut [9, 10] to more complicated micro-modelling techniques [11]. A comprehensive review of the various modelling techniques used in the past has been amalgamated in the literature [12, 13]. The detailed behavior of the infilled frames under lateral loading may be easily captured by employing micro-modelling approach but the major inefficacy of the micro-modelling techniques was the high computational effort required for large building structures, which was found to be unacceptable for practical engineering purposes. Most of the past analytical studies resorted to macro-modelling techniques especially modelling infill as equivalent diagonal strut because of the ease in idealizing the parameters involved in modelling. The global behavior of the infilled frames (ultimate deformation/ base shear) can be easily evaluated by performing the nonlinear static pushover analysis and it has been proven successful in the past studies. But the major limitation of modeling the masonry as equivalent diagonal strut lies in its inability to evaluate the internal forces in columns and beams and to capture the local shear failure of the members near the joints. This may pose a serious threat to the safety of the frame members as the under-designed columns may fail in brittle shear mode due to their interaction with the masonry infill walls under lateral forces. In the recent years, it was observed that more emphasis was given to upgrade the suitability of macro-models to capture the component failure. A few analytical studies by D'Ayala et al. [14], Celarec and Dolšek [15], and Cavaleri and Di Trapani [16] were carried out to evaluate the shear failure of the columns by simplified modelling techniques. As a step forward, in the current article equivalent diagonal strut model was further improved to capture the component failure as well the global response of the structure. To evaluate the proposed improvements, infilled frames from the comprehensive experimental study carried out by Basha and Kaushik [17] were considered. Initially effectiveness of modelling infill as equivalent diagonal strut was investigated followed by the recommendations to capture the local shear failure of columns by accounting for the additional interaction effect of infill. Later the improved modelling technique was validated with the past experimental studies [4, 18].

2. Description of frames

To understand the lateral load behavior and failure mechanisms of infilled frames, an experimental study was undertaken by Basha and Kaushik [17] in which half-scale RC frame specimens designed using the current Indian seismic standards were tested. The half scale model used represented an exterior ground-story frame of a two-story office building in Assam, which is one of the most seismic-prone regions in India. The prototype structure considered in the study was designed for the lateral forces corresponding to the highest seismic zone in India. Three types of frames: bare frame (BF), infilled frame with fly ash bricks using low strength bars for reinforcement (IF-FB1), and infilled frame with fly ash bricks using high strength deformed bars for reinforcement (IF-FB2) were considered. The reinforcement details and the material properties of the tested specimens are given in Table 1.

Table 1 - Properties of the frame specimens

Material properties (MPa)			RC frame Parameters		
Compressive strength of concrete cube	24		Column	115×175 mm	
Elastic modulus of concrete	24,600		Beam	115×175 mm	
Compressive strength of fly ash bricks	5.7		Column longitudinal reinforcemnt	4-12φ at corners* 2-8φ at middle	
Compressive strength of 1:4 grade mortar cubes	17.3		Column transverse reinforcement	6φ-3 legged stirrups at 90 mm c/c till 500 mm from the face of top and bottom beam; and remaining 110 mm c/c	
Compressive strength of fly ash brick masonry prisms	3.9		Beam longitudinal reinforcemnt	At support	At mid-span
				2-10φ at bottom 3-8φ at top	2-10φ at bottom 2-8φ at top
Yield strength of reinforcing bars	BF/IF-FB1	IF-FB2	Beam transverse reinforcement	6φ-2legged stirrups at 90 mm c/c till 500 mm from the face of columns; and remaining 110 mm c/c	
	265 (6φ) 460 (12φ)	520 (6φ) 530 (12φ)			

* 4-12φ represents 4 bars of 12 mm diameter

The frames were tested under slow cyclic displacement controlled loading using servo controlled hydraulic actuator and the response was recorded using the data acquisition system. It was reported that the infilled frames were significantly stiffer (7-10 times) and stronger (1.6-2.5 times), and dissipated more energy (1-2.3 times) than the corresponding bare frames. It was observed that though strong frame-weak infill configuration was used in the study, columns of the infilled frame failed in shear mode as shown in Fig. 1, unlike the past studies [4, 5]. To assess the possible reasons for the shear failure of columns in masonry infilled frames, analytical study is carried out and discussed in detail in the following sections.

3. Analytical modelling of frames

The analytical study was carried out using the nonlinear finite element software SAP 2000 [19]. Column and beam elements were modeled as 2-noded frame elements with column ends fixed at the bottom with six degrees of freedom at each node. However, only three degrees of freedom were considered due to the plane frame analysis as out-of-plane action was restricted. In the experimental study, no damage was observed in the top beam and the slab due to the stiffening (T-beam) action provided by the slab. Therefore, linear elastic shell elements were used to model the RC slab. Centre line modelling technique was used to model the different components of the frame.

(a)

(b)

(c)

Fig. 1 - Failure mechanisms of : (a) bare frame ; and infilled frames (b) IF-FB1; (c) IF-FB2.

In addition to the elastic material properties, nonlinear material properties are required in pushover analysis. In SAP 2000, nonlinearity in frame elements was modeled using lumped plasticity approach at specified hinge locations. Plastic hinge length and location was determined based on the crack pattern observed during the experimental study. It was observed that hinges in columns were formed at a distance 90 mm from the top face of the bottom beam and bottom face of the top beam, respectively. The plastic hinge length was found to be approximately half of the depth of the column. As the beam-column joints were assumed to be semi rigid, the plastic hinges were formed at a distance $l_p/2$ from the face of the beam and columns as shown in Fig. 2.

Flexural hinge properties in columns were defined using axial force-bending moment interaction (*P-M*) as the failure envelope, and bending moment-rotation (*M-θ*) was used as the corresponding load deformation relation for defining flexural hinge properties in beams and columns. Force controlled shear hinge properties involve specification of shear capacity of the sections. This ensures that the shear hinges fail in a brittle manner such that the hinge loses its load carrying capacity as soon as the shear capacity is reached. Nonlinear material properties were used to develop the plastic hinge properties of the frame elements. Mander confined concrete model [20] was used to define the concrete stress-strain characteristics to define the flexural hinges. The nonlinear model for rebars was defined from the uniaxial stress-strain curves obtained from the tension tests conducted in the laboratory. The ultimate strain in steel was found to be about 16% and the ultimate tensile strength was about 1.3 times the yield stress. The derived plastic properties were lumped at specified hinge locations (Fig. 2).

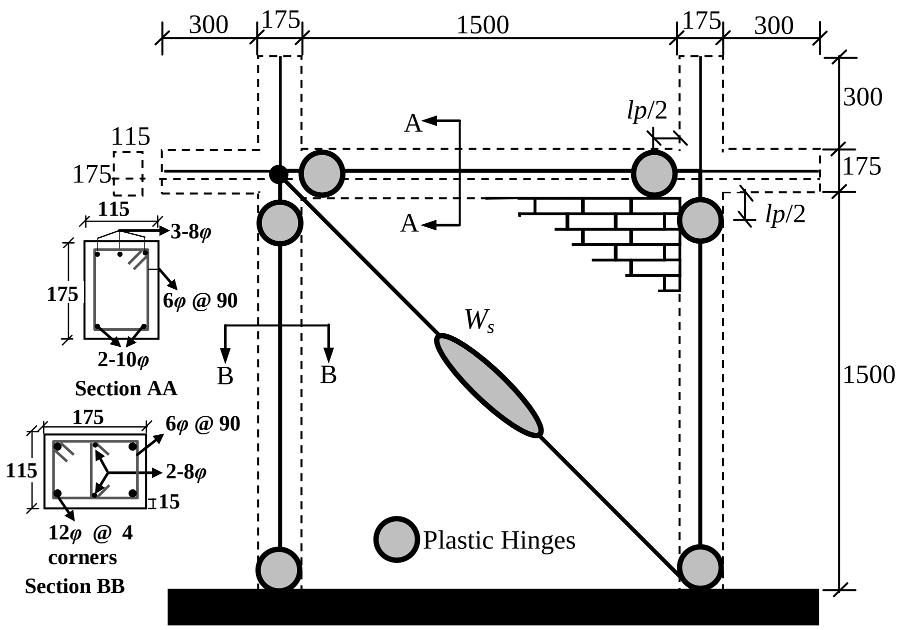


Fig. 2 - Details of analytical model and plastic hinge locations.

Accurate analytical modeling of masonry infills has been a distant task till date due to a number of interacting parameters involved and many possible failure modes needed to be evaluated with high degree of uncertainty [13]. In order to evaluate the overall global response of the structure in terms of lateral strength and stiffness, masonry may be idealized as equivalent diagonal compressive strut. The basic parameter that affects the strength and stiffness of the strut is their equivalent width which depends on the relative infill-frame stiffness. Various formulations are proposed in the literature to calculate the relative stiffness parameter. ASCE 41 [21] recommends using the equation proposed by Mainstone [22] as it provides lower bound value for the width of the strut, and found to be popular over the years and has been implemented by many researchers. Width of the diagonal strut calculated using Mainstone's equation was found to be approximately one-tenth of the diagonal length of the infill panel. The strut was modeled as compression element and moment releases were provided at both ends of the diagonal strut such that the transfer of bending moment from RC elements was prevented. As the equivalent strut was modelled as compression only element, axial hinge was assumed to develop in the centre of the masonry infill. Nonlinear compressive stress-strain curves of masonry prisms were assigned to model the hinge properties of masonry infill [23]. In the current article, infill wall was constructed using fly ash bricks with 1:4 intermediate mortar grade. The nonlinear stress-strain characteristics of fly ash brick masonry prisms for various grades of mortar has been determined in the past exhaustive experimental study carried out by Basha and Kaushik [24], and the average stress-strain curve with major control points is shown in Fig. 3. Axial hinges were assumed to be developed at the center of the strut and the length of the hinge was assumed to be about three-fourth of the diagonal length of the infill wall.

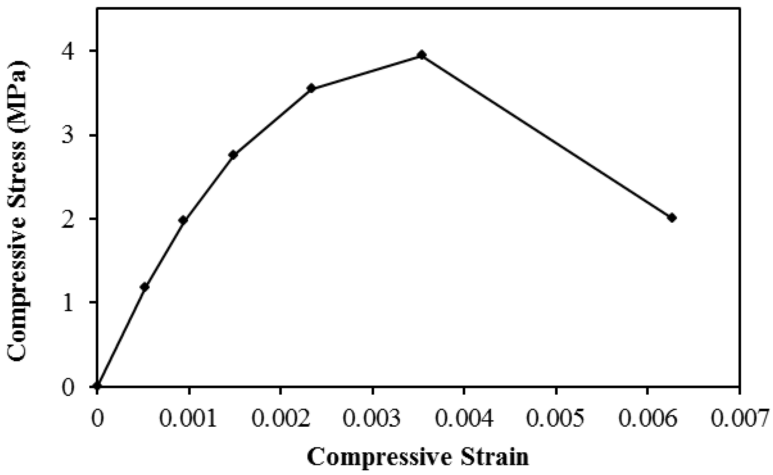


Fig. 3 - Average compressive stress strain curve for 1:4 fly ash brick masonry prism [24].

4. Results of nonlinear static analysis

The comparison of experimental lateral load response of infilled frames with their analytical pushover (base shear-drift) response is shown in Fig. 4. The analytical model of bare frame without infill walls was first calibrated with the experimental results of a bare frame. It can be observed that both the experimental and analytical results, in terms of initial stiffness and the lateral load capacity, matched very well. Experimental results showed that the failure of bare frame was primarily due to the formation of flexural hinges in columns and no damage occurred in the beam due to the stiffening action provided by the slab. Similar observation was made from the analytical model as well, as flexural hinges were concentrated only in columns and no hinges were formed in beams. The maximum lateral load recorded during the experiment was 42 kN and is found to be in good relation with analytically determined capacity of bare frame. The failure of the bare frame was largely flexural as the moment carrying capacity of columns was reached at the collapse state. Therefore, it can be ascertained that global response of the bare frame can be evaluated analytically by modeling the frame elements as 2-noded elements and modeling the non-linear deformational properties by lumped plasticity approach. The same analytical model was then updated to include the effects of masonry infills and the updated model was calibrated with the experimental results.

Fig. 4 the shows the initial stiffness and the lateral load carrying capacity of both the experimental and analytical models of infilled frames (IF-FB1 and IF-FB2) matched quite well. Initially the lateral force was resisted by the masonry infills due to their large initial stiffness which attracts large forces. As the lateral displacement increased, failure in infill was observed followed by attainment of the lateral load carrying capacity of the infilled frame; later on the behavior of the infill frames followed the profile of the bare frame at larger drift levels. Degradation in lateral load carrying capacity of the analytical models was found to be very gradual and in good agreement with the experimental results which generally is not observed in case of analytical models of infilled frames where abrupt reduction in lateral strength was observed when infill fails. Therefore, from the analytical model it was ascertained that the global lateral load behavior of the infilled frame can be simulated using the equivalent diagonal strut model. Initially, the moment capacity of the flexural hinges was attained followed by axial hinge in the centre of the infill panel before the lateral load carrying capacity was reached in case of both IF-FB1 and IF-FB2. No shear hinges were formed in the analytical model, as the maximum shear force demand observed in the column was found to be far less than the shear capacity of the column. But, from the experimental study, it was reported that the diagonal shear cracks were formed in infilled frame at a lower drift level due to the excessive shear demand on the column due to the effect of infill. Even though the equivalent diagonal strut model was able to simulate the lateral load response of the infilled frame, but the model was not able to capture the local component failures of the infilled frame.

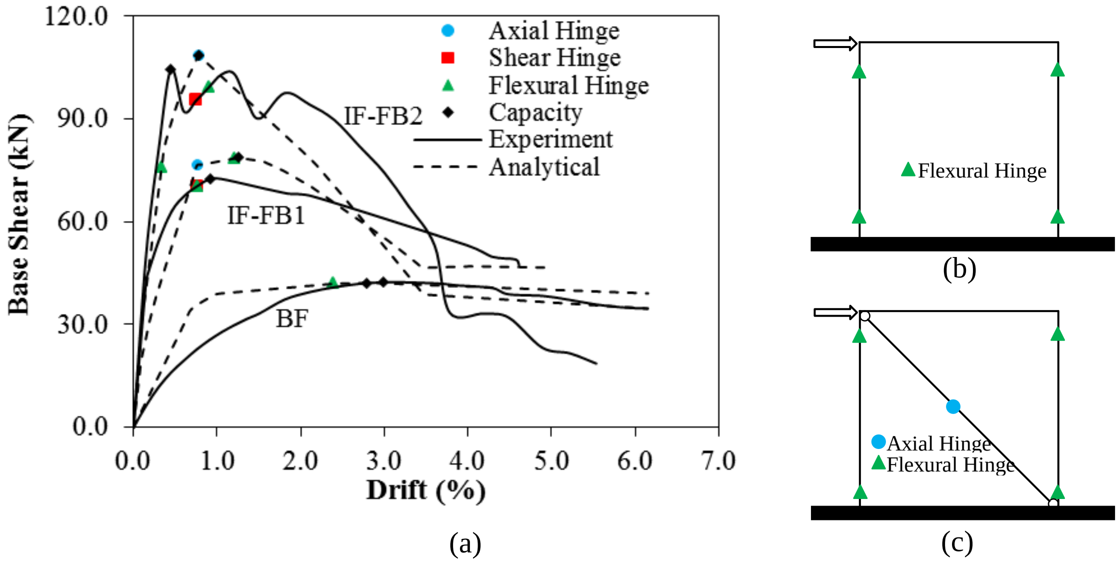


Fig. 4 - Comparison of: (a) pushover curves for bare and infilled frames (IF-FB1 and IF-FB2); (b) and (c) formation of plastic hinges for bare and infilled frames using equivalent diagonal strut model.

5. Upgradation of analytical model

From the above results it is clear that the equivalent diagonal strut model was unable to capture the shear demand on the columns of the infilled frames due to the effect of infill. For this reason, to evaluate the shear demand on the column due to the effect of masonry infill, the existing analytical model was improved by considering additional shear forces on the columns of the infilled frames to simulate the effect of infill (*F*). Initially, bare frame was modelled as described in the previous section, later the infill was modelled as equivalent diagonal strut. The contact length of the masonry infill with the column was calculated as the full vertical width of the diagonal strut of the infill using Eq. (1) and Eq. (2), so as to apply the effect of infill (strut force) along the contact length (l_c) as shown in Fig. 5.

$$l_c = \frac{w_s}{\cos\theta_c} \tag{1}$$

$$\tan\theta_c = \frac{\left[h - \left(w_s / \cos\theta_c\right)\right]}{h} \tag{2}$$

According to Flanagan and Bennet [25], Al-chaar et al. [18], and ASCE 41 [21], the effect of the infill (strut force) is determined based on the strength of the strut. The component of these forces in the direction of the equivalent strut was used to estimate the effect of infill (*F*). But, it has to be noted that the failure of the infill panel depends on various aspects such as compressive and tensile strength of brick and mortar bed joints, amount of confining effect provided by the surrounding RC frames, aspect ratio, slenderness ratio, type of bond employed in the construction of infill wall, opening ratio, relative stiffness of frame and infill, etc. In the current study, the effect of infill was evaluated based on the failure modes observed in the previously tested infilled frame specimens [17]. The infilled frames considered were constructed in running bond and infill was failed by observing bed joint sliding cracks, corner crushing, and diagonal cracking with cracks passing through bed and head mortar joints and often passed through bricks by splitting the masonry units vertically (Fig. 1). In the current article, it was assumed that the shear failure of the column due to the effect of infill is expected only when the infill exerts maximum force onto the column. In order to evaluate this maximum force exerted by the infill on to the columns, the maximum strength of masonry infill against two failure modes (crushing and shear) was considered.

1) Crushing mode of failure

The masonry infill crushing strength (R_{cr}) is the compressive load that the equivalent masonry strut (f'_m) can carry before the masonry is crushed and is calculated using Eq. (3) as

$$R_{cr} = f'_m w_s t_w \tag{3}$$

2) Shear mode of failure

The capacity of masonry under shear forces was evaluated from the diagonal compression strength (V_t) of masonry wallettes which generally is a representative of the shear strength of the masonry panel. Shear strength (R_s) is calculated using Eq. (4) as

$$R_s = V_t l_w t_w \tag{4}$$

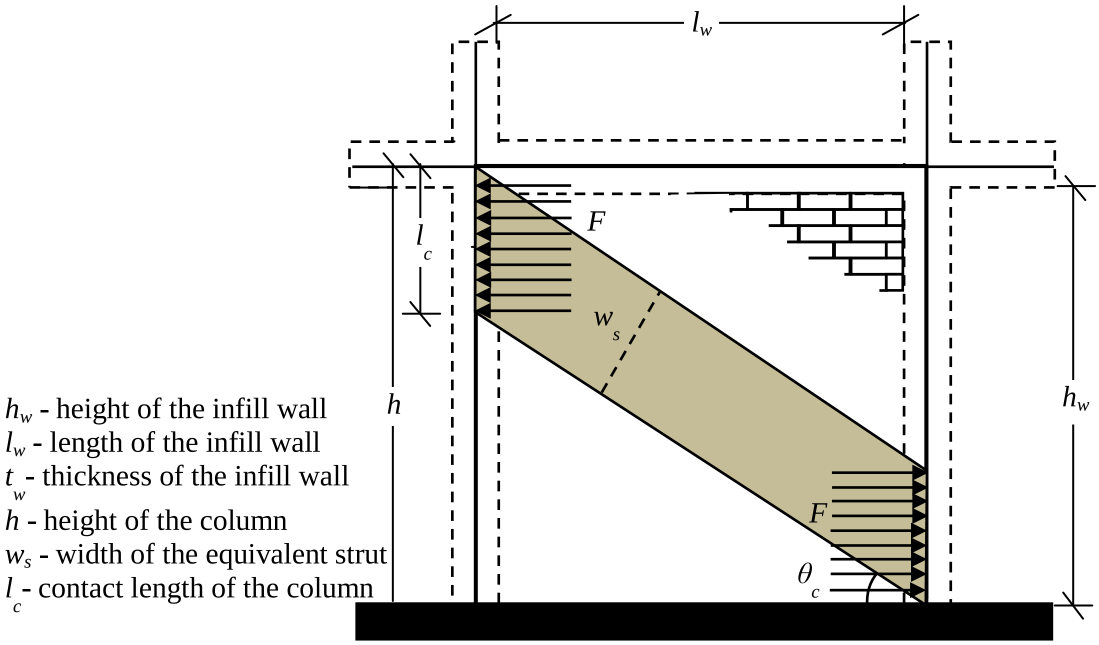
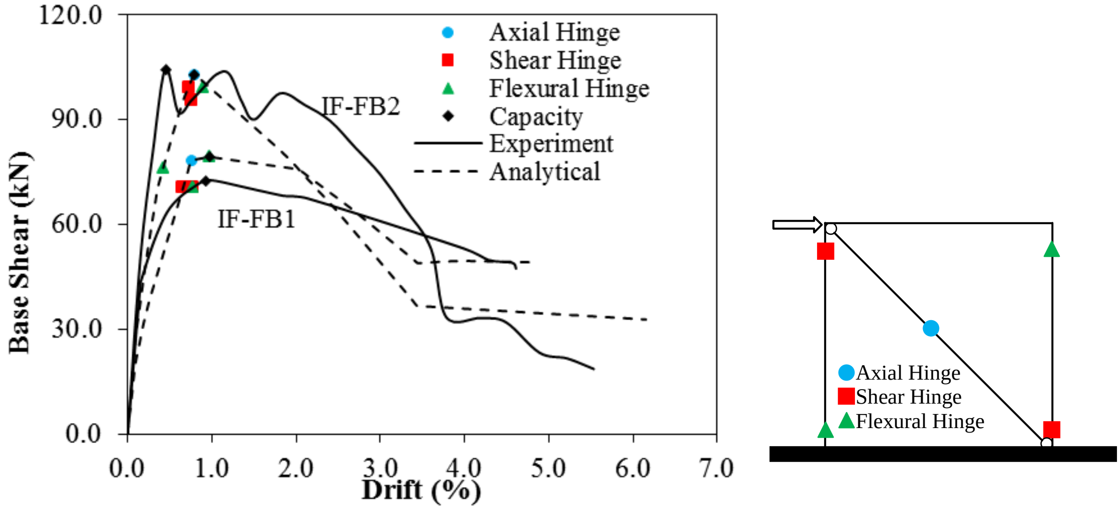


Fig. 5 - Schematic representation of infilled frame considering the effect of infill.

From the experimental study [24], it was found that the compressive strength of masonry prisms and shear strength of fly ash brick masonry wallettes was approximately 3.9 MPa and 0.14 MPa, respectively. The force exerted by the infill in crushing was significantly higher compared to the shear force. Both ASCE 41 [21] and Al-chaar et al. [18] recommend considering the strength of infill to be the minimum of crushing and shear, but according to Flanagan and Bennett [25] the final failure mode of infill is crushing even though other modes of failure were observed during lateral loading. In the current article, to evaluate the maximum shear demand on the columns, the force corresponding to the maximum of two failure modes was considered, i.e., the crushing strength of the infill. The obtained horizontal component of the infill force is applied as uniformly distributed load along the contact length (l_c) of the column (Fig. 5).

5.1 Evaluation of the upgraded model

The upgraded methodology was evaluated by carrying out nonlinear static analysis of the previously tested infilled frame specimens IF-FB1 and IF-FB2. The details of the analytical model have been discussed in detail in section 3. The contact length of the column was calculated based on the geometric calculations by placing full vertical component of the width of the strut along the column (Fig. 5). Later the horizontal component of the axial force in the strut was applied as uniformly distributed load along the contact length of the column on either end of the diagonal strut and analysis was carried out. Fig. 6 shows the comparison of response of both the analytical model and experimental envelope curves for both the infilled frames. From the experimental study, it was observed that the diagonal shear cracks were formed in the columns at a drift level of about 0.77% in both the infilled frames at lateral loads of 71 kN (IF-FB1) and 95.5 kN (IF-FB2), respectively. The upgraded analytical model also predicted formation of shear hinges in the columns of the frames. The shear capacity of the columns in frames IF-FB1 and IF-FB2 was reached at a drift level of about 0.66% (70.6 kN), 0.72% (99.4 kN), respectively, as shown in Fig. 6. From the results, it is observed that the analytical model successfully captured the initiation of shear failure in the columns due to the effect of infill. On the other hand, the shear demand on the columns in the equivalent diagonal strut model without considering the effect of infill was found to be far less than the shear capacity of the column section. The capacity of the infill (axial hinges in masonry strut) reached simultaneously or just prior to reaching the lateral strength of the infilled frame. From the analysis, it was found that the proposed analytical model was able to successfully capture not only the global response of the infilled frame system but also the shear failure of columns. The validation of the upgraded modeling technique is discussed in the following section.



(a) (b)

Fig. 6 - Comparison of pushover curves and plastic hinge formation for infilled frames IF-FB1 and IF-FB2 obtained using the upgraded analytical model with that obtained experimentally.

5.2 Validation of upgraded model

To validate the upgraded modelling technique, two frames from the past experimental studies were chosen: non-ductile frame infilled with clay brick masonry [18], and weak frame with solid infill [4]. The sectional and material details of the considered infilled frames are given in Table 2 and Table 3, respectively. From the experimental study, it was reported that both the frames observed shear mode of failure under in-plane lateral loads.

Table 2 - Details of non-ductile frame infilled with clay bricks [18]

Material	Properties	RC frame parameters	
Concrete: Cube Compressive Strength	38.4 MPa	Bay width	2032 mm
Concrete: Elastic Modulus	29,992 MPa	Bay height	1524 mm
Mortar: Cube Compressive Strength	10.6 MPa	Column depth	203 mm
Masonry: Compressive Prism Strength	26.7 MPa	Column width	127 mm
Reinforcing Bars: Yield Stress	338.5 MPa	Beam depth	197 mm
Reinforcing Bars: Elastic Modulus	2,00,000 MPa	Beam width	127 mm
		Column longitudinal reinforcement	4-10φ
		Column ties	4.88φ at 152 mm c/c
		Beam longitudinal reinforcement	4-10φ (top), 2-10φ (bottom)
		Beam ties	4.88φ at 76 mm c/c

Table 3 - Details of weak frame infilled with solid concrete bricks [4]

Material	Properties	RC frame parameters	
Concrete: Cube Compressive Strength	20.8 MPa	Bay width	2134 mm
Concrete: Elastic Modulus	18,052 MPa	Bay height	1422 mm
Mortar: Cube Compressive Strength	17.6 MPa	Column depth	178 mm
Masonry: Compressive Prism Strength	13.8 MPa	Column width	178 mm
Reinforcing Bars: Yield Stress	420.3 MPa	Beam depth	229 mm
Reinforcing Bars: Elastic Modulus	2,00,000 MPa	Beam width	153 mm
		Column longitudinal reinforcement	8-12.7φ
		Column ties	6.35φ at 63.5 mm c/c
		Beam longitudinal reinforcement	4-15.87φ
		Beam ties	6.35φ at 76.2 mm c/c

Both non-ductile frame infilled with clay brick masonry [18] and weak frame with solid infill [4] were half scale models designed such that the beam-column joints had inadequate longitudinal and transverse reinforcement, and the design of the both the frames ignored the contribution of infill. Initially, bare RC frame was modelled using the given material and sectional properties, and later infill was modelled as the equivalent diagonal strut, with width of the equivalent diagonal strut calculated from Mainstone's [22] equation. Nonlinear static analysis was carried out by assigning flexure, shear, and axial hinges to the respective elements. Axial hinge in the equivalent diagonal strut was evaluated using the nonlinear stress-strain model developed by Kaushik et al. [26] as shown in Fig. 7. The calibration of the analytical model was carried out by varying the properties of infill so as to match the experimental lateral load response of the infilled frame. Once the infilled frame model was calibrated, the effect of infill along the contact length of the column was modelled and the nonlinear analysis was again carried out.

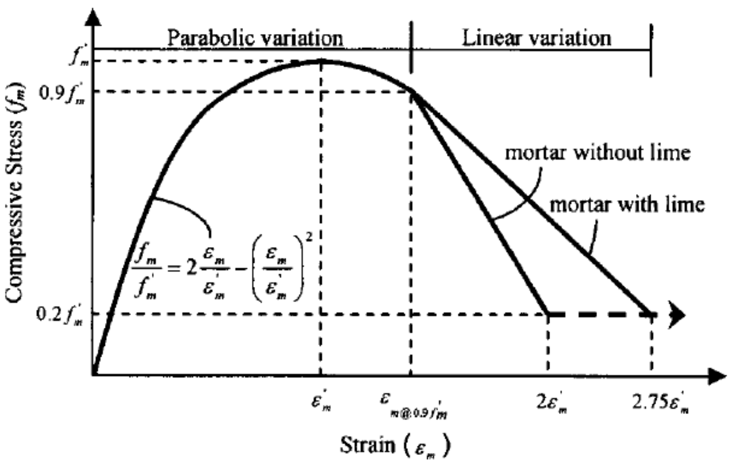
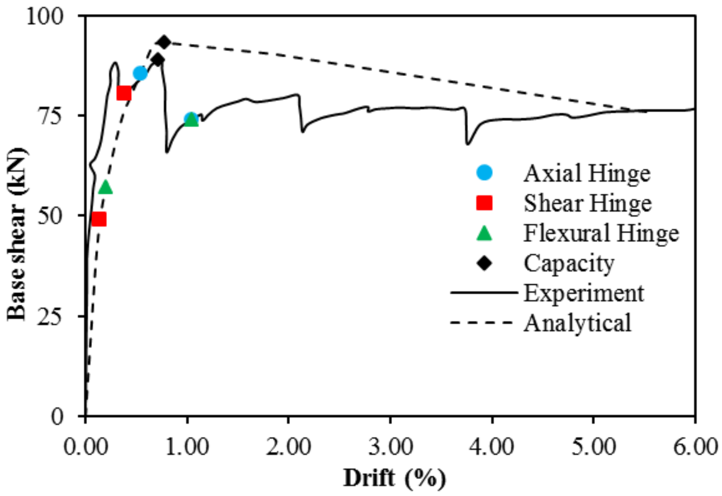
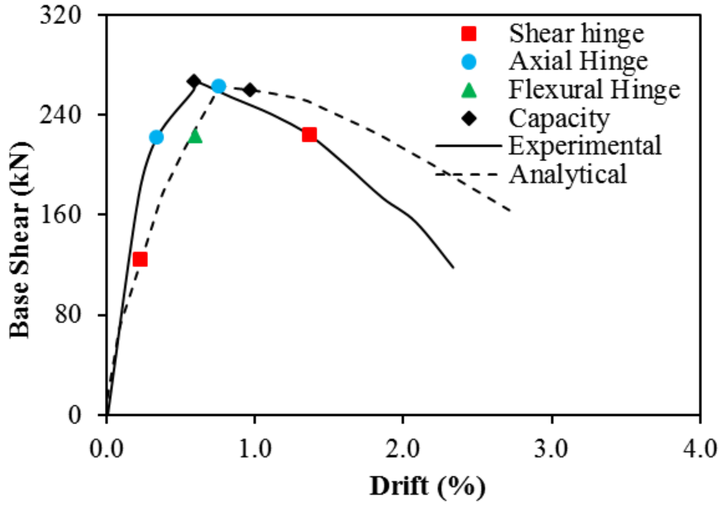


Fig. 7 - Idealised stress-strain curve for masonry prisms under uniaxial compression [26].

Fig. 8 shows the comparison of the base shear-lateral drift response obtained analytically (using the upgraded analytical model developed in the current study) and experimentally for the considered infilled frame specimens. It was observed that the initial stiffness and capacity of both the analytical model and the experimental envelope curves matches quite well. In case of Al-Chaar's data, it was observed that the upgraded analytical model predicted the formation of the shear hinge at a drift level 0.14% (lateral load of 49 kN); it was reported in the experimental study that the first major shear crack formed at 0.4% drift (81 kN) near the column joints (Fig. 8a). Authors did not report the initiation of the shear cracks in the columns. Similarly, in case of Mehrabi's study, the proposed analytical model predicted the formation of shear hinge at 0.2% lateral drift (124 kN), whereas, in case of the test specimen formation of major shear crack was reported at 1.4% drift (224 kN) (Fig. 8b). D'Ayala et al. [14] from their analytical study to predict the shear failure of columns also reported that the shear cracking initiated at a lower drift level prior to the formation of major shear crack and lateral load capacity of the infilled frames in the experimental study. Later, flexural hinges were formed near the top and bottom column sections of the infilled frame specimens. From the analytical results, it can be ascertained that the global response as well as the local failure mechanism can be captured satisfactorily with the upgraded analytical model proposed in the current study. The nonlinear masonry material model was developed based on the nonlinear stress-strain curve model proposed by Kaushik et al. [26] and it was observed that the results were found to be sensitive to material model as properties of masonry are region specific. A more generic masonry material model needs to be established for accurate estimation of shear demand on the columns.



(a)



(b)

Fig. 8 - Comparison of pushover curves and plastic hinge formation for: (a) non-ductile frame infilled with clay brick masonry; and (b) weak frame infilled with solid infill.

6. Summary and conclusions

Inspite of many beneficial effects, infills are potentially dangerous as the increase in lateral stiffness and strength of the infilled frames often causes increase in shear demand on the columns for which they are not generally designed. The current article addressed evaluation of the detrimental effect of infill, i.e., increased shear demand on the column due to the infill-frame interaction under lateral loading using the upgraded simplified equivalent diagonal strut model. Initially, the beneficial effect of modelling infill as equivalent diagonal strut was evaluated by considering half scale bare and infilled frame specimens from the past experimental study. Non-linear analysis was carried out using lumped plasticity approach and idealizing the infill as equivalent diagonal compressive strut. From the analysis, it was established that the global response (lateral strength and stiffness) of the infilled frame can be estimated using the aforementioned modelling technique. The major constraint of the diagonal strut model was its inability to evaluate the internal member forces (shear force and bending moment). In order to overcome this shortcoming, the diagonal strut model was upgraded by incorporating the effect of infill to account for the increased shear demand on the column due to the frame-infill interaction along the contact length of the column. The effect of infill was calculated based on the compressive crushing strength of masonry and applied as uniformly distributed load along the contact length of the column. The upgraded methodology was evaluated by carrying out nonlinear static analysis of the previously tested infilled frames IF-FB1 and IF-FB2. It was found that the upgraded model was able to capture the local shear failure of columns by forming shear hinges near the column ends, in addition to accurately predicting the global response (lateral strength and stiffness) of the frames. From the analysis, it was observed that both the initiation of shear cracks and shear hinge formation observed at approximately at the same drift level. Validation of the upgraded modelling technique was carried out using the past experimental models [18, 4]. From the analysis, it was observed that the proposed upgraded modelling technique successfully predicted both the global response and local component (shear) failure in infilled frames. Even though the model predicted the formation of shear hinges in columns, the formation of major shear cracks took place at a higher drift level in the experimental investigation. Similar observation was made in the past analytical study where it was observed that the shear stresses reached the limiting value prior to the visible formation of major shear cracks in the experimental study.

7. Acknowledgement

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