A RATIONAL APPROACH TO ANALYTICAL MODELING OF MASONRY INFILLS IN REINFORCED CONCRETE FRAME BUILDINGS

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ABSTRACT :

Masonry infills, which generally have high stiffness and strength, play a crucial role in lateral load response of reinforced concrete (RC) frame buildings. Geographically, there is a large variation in material properties of masonry. Moreover, masonry behaves in a highly nonlinear manner. Therefore, a generalized yet rational model is required for masonry infills that can efficiently incorporate its linear and nonlinear material properties and common failure modes in RC members and masonry infills under the action of lateral forces. Interestingly, national codes of most of the countries do not specify modeling procedures for such structural systems. On the other hand, several analytical models for masonry infills are available in literature, for example, equivalent diagonal strut models, finite element models, etc. Therefore, a comparative study was carried out considering different analytical models for masonry infills using experimental results for nonlinear material properties of masonry infills. Analytical models considered in the present study include single-strut model, 3-strut model, and finite element models. By linear and non-linear analyses, it was observed that the 3-strut model can estimate force resultants in RC members with sufficient accuracy, in addition to modeling the local failures in infills and in beams and columns due to interaction between masonry infills and RC frame. It was also observed that the single-strut model can be effectively used in cases where masonry infill walls are discontinued in the first-storey to generate parking space (for example, soft or open storey buildings).

KEYWORDS: Reinforced concrete frames; masonry infill walls; nonlinear behaviour of masonry; analytical modeling; pushover analysis.

1. INTRODUCTION

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. On the other hand, the present national codes do not specify any method for analytical modeling of masonry infills in reinforced concrete (RC) buildings[1]. Generally, the finite element method requires more computing power and time than that required by the equivalent diagonal strut methods, which have been found to be sufficiently accurate in estimating the initial stiffness and lateral strength of masonry-infilled RC frame buildings[2-5]. Masonry infills can be conveniently modeled using a single diagonal strut, two struts, three struts, and similarly more number of struts[6-9]. Although many analytical models for masonry infills are available in literature, guidelines on selection of a mathematical model are not available in literature. Several researchers in the past have raised concerns on use of 1-strut model for masonry infills[5,6,10,11]. In case of frames with full height masonry infills, FEMA[12] recommends to evaluate the effect of strut compression forces applied to the column and beam, eccentric from the beam-column joint. Obviously, the complicacy and effort involved in these methods increases with the increase in number of elements (diagonal struts) used in modeling infills. There are limitations associated with the simple diagonal strut methods as well as more complicated finite element methods for masonry infills. In the present study, linear and non-linear analyses of a single-storey, single-bay masonry-infilled RC frame was carried out by modeling masonry infills using five different modeling techniques, and the results compared to arrive at a rational modeling scheme for masonry infilled RC frames.

2. LINEAR STATIC ANALYSES

A comparative study was carried out to assess a suitable model for masonry infills in RC frames. Suitability of a model is judged depending upon several factors, namely, time required and effort involved in modeling, ability to model lateral stiffness and strength of infilled frame, and ability to model failure modes in not only infills but also in RC members of the frame. A single-bay, single-storey frame of a multi-storey RC building was considered in the comparative study (Fig. 1). Modeling techniques and assumptions considered in the present study are discussed in the following.

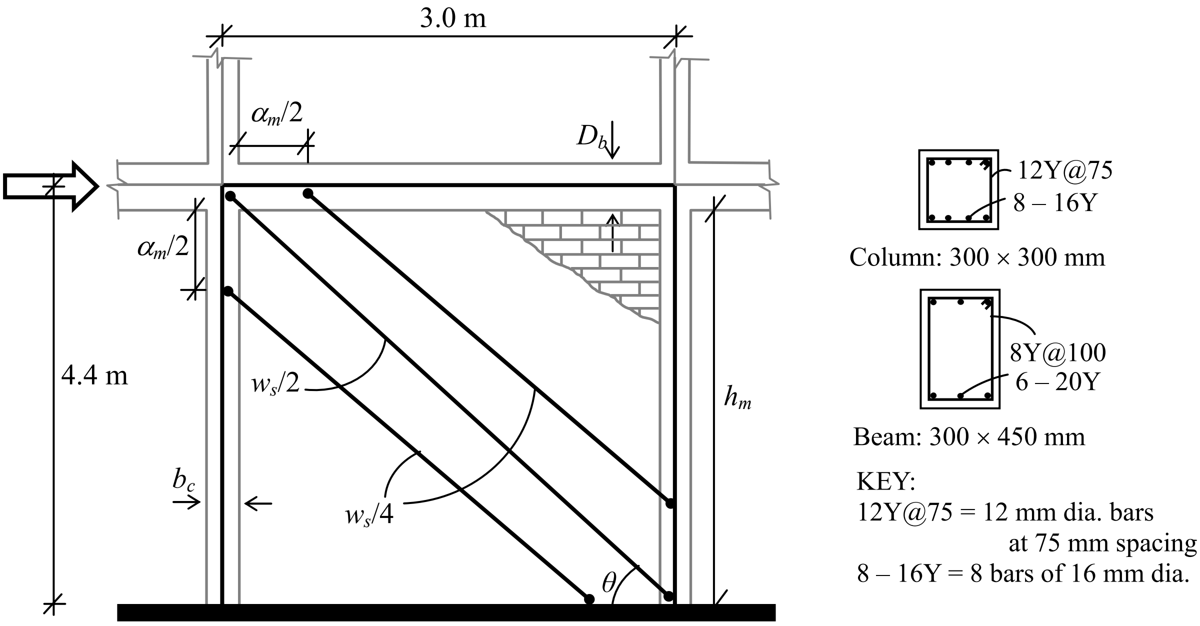

Figure 1 Example frame considered in the comparative study and details of 3-strut model for masonry infills

2.1 Analytical Modeling

The frame was assumed to be fixed at the bottom, and columns and beams of the frame were modeled using two-noded frame or beam elements. Grades of concrete and steel considered were M25 (cube strength f_{ck} of 25 MPa, and modulus of elasticity E_c of 25000 MPa) and Fe415 (yield strength f_y of 415 MPa), respectively. Masonry infill walls were modeled as (1) equivalent diagonal struts (1-strut and 3-strut) using two-noded beam elements, and (2) finite elements using 3-noded and 4-noded shell elements. Out of the several equivalent diagonal strut models available in the literature, only two models were chosen for the present study (1-strut model and 3-strut model) because of their simplicity. Transfer of bending moments from RC frame to masonry was prevented by specifying moment releases at both ends of the struts. In case of finite element modeling (using shell elements), only membrane action in shell elements was considered, and bending action ignored. Using results of the experimental study carried out by Kaushik et al.[13], modulus of elasticity of masonry E_m was taken as $550 f_m^{'}$ in MPa, where $f_m^{'}$ is the compressive prism strength of masonry in MPa.

Width of equivalent diagonal struts (w_s) can be found out using a number of expressions given by different researchers but it has been shown in the past that the value of one-third to one-fifth of the diagonal length of masonry infill wall may be conservatively assumed[2-4]. It has also been shown that width equal to one-third of the diagonal length represents the upper bound value[3]. An average value of one-fourth of the diagonal length of infill (d_w) was used in the present study as also suggested by Paulay and Priestley[4]. In case of the model with 3 struts, width of central diagonal strut was taken as one-eighth of the diagonal length of wall, and width of off-diagonal struts as half the width of the diagonal strut. In this way, total width of all equivalent struts considered was one-fourth of the diagonal length of wall. In 3-strut model, location of equivalent struts is an important parameter. Out of the three struts, the off-diagonal struts were connected to the columns at the centre of the distance α_m known as the vertical length of contact between infill and column (Fig. 1) suggested in the literature[3,4] as:

$$\alpha_m = \frac{\pi}{2}\sqrt[4]{\frac{4E_c I_c h_m}{E_m t \sin(2\theta)}} \quad \text{(mm)} \tag{1}$$

where E_c and E_m are modulus of elasticity of column and masonry material in MPa, respectively, I_c is moment of inertia of column section in mm^4, h_m and t are height and thickness of masonry infill wall in mm, respectively, and θ is angle in degrees of inclination of the equivalent diagonal strut with the horizontal. It was observed in the present study that varying the length of contact between infill and beam does not change the analyses results considerably. Therefore, the horizontal length of contact between infill and beam was considered same as the vertical contact length between infill and column.

Linear analyses of the frame were carried out using SAP2000[14] considering six different models including the bare frame model in which strength and stiffness of masonry infills was not considered. Under the effect of increasing lateral forces, contact area between masonry infill wall and RC frame reduces. To study the effect of reduction in contact area between RC members and masonry infills, three finite element models were analyzed by considering different contact areas between RC frame and infill. All the models studied are discussed below and are shown in Fig. 2. Self weight of full masonry wall was considered in all the models. The left and right columns of the frame are referred as "L" and "R", respectively. Results of the linear static analyses are discussed and compared in the next section.

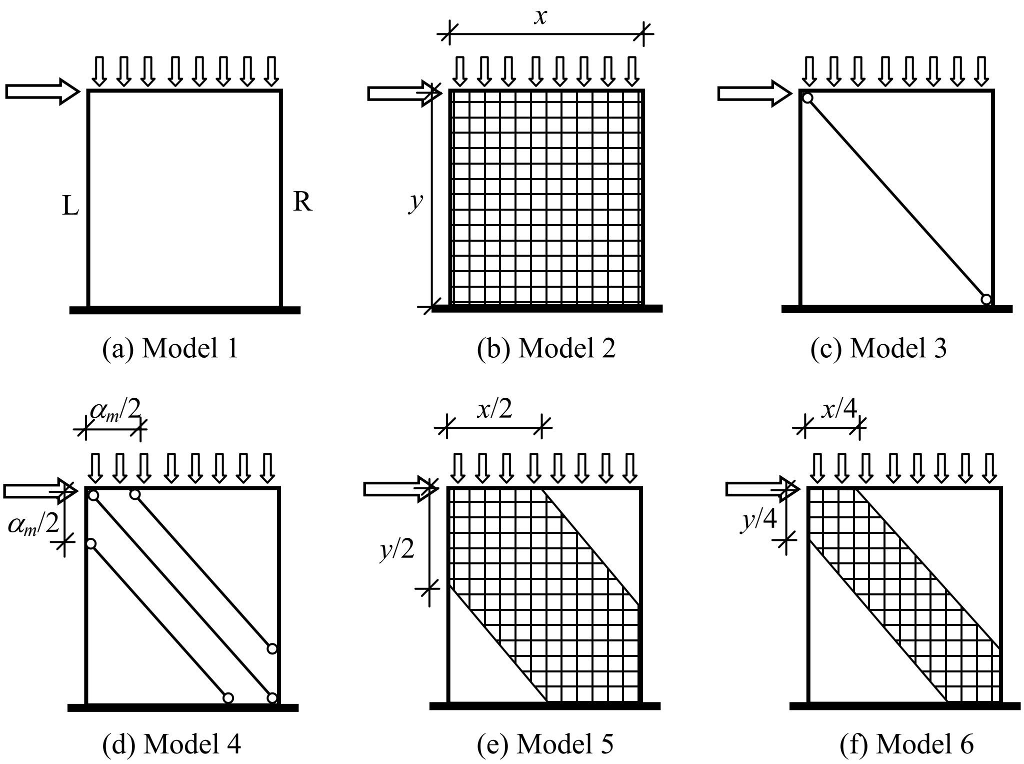

Figure 2 Different analytical models studied: (a) Model 1 - bare frame, (b) Model 2 - full infill wall modeled using finite element model, (c) Model 3 - infill wall modeled using a single diagonal strut, (d) Model 4 - infill wall modeled using three diagonal struts, (e) Model 5 - partial infill wall modeled using finite elements such that only half of length of beam and column was in contact with the wall, and (f) Model 6 - partial infill wall modeled using finite elements such that only quarter of length of beam and column was in contact with the wall

2.2.Results of linear static analysis

Linear static analyses of the six models were carried out under the action of vertical dead loads (DL) and lateral earthquake loads (EQ), and corresponding maximum force resultants in columns and beam of the frame obtained for the load case 1.5(DL+EQ) are compared in this section. It was observed that force resultants, most noticeably the bending moment, reduce considerably when stiffness of infill was considered in the analytical model, because most of the lateral forces were then transferred to the infill wall as axial forces. With increase in lateral forces on the frame, area of wall in contact with RC frame reduces because of separation of wall from RC frame near the tension-diagonal joints. In other words, effective lateral stiffness of wall, and therefore the effective width of equivalent diagonal strut reduce with increase in the lateral forces. Effect of this on RC columns may be considered similar to creation of short columns because of varying unsupported length of columns under increasing lateral forces[15]. Therefore, Model 2 (full shell) and Model 3 (1-strut) may not adequately capture the actual behaviour of the masonry-infilled RC frame because these models consider full stiffness of infill wall irrespective of the extent of lateral forces.

In reality, under increasing lateral loads Model 2 (full infill) should first transform into Model 5, and then into Model 6. Interestingly, a consistent increase in force resultants was observed in columns when 3-strut model was used as compared to the 1-strut model (Fig. 3). This increase in force resultants of columns in 3-strut model was primarily due to partial contact between the ductile RC frame and relatively stiffer masonry infills. Force resultants in columns obtained using 3-strut model were found to be matching more closely with that obtained using partial shell models (Models 5 and 6). Therefore, the 3-strut model appears to be more accurately representing the behaviour of masonry infills under lateral forces. Similar increase in axial forces was also observed in the beam in case of 3-strut model as compared to the 1-strut model (Fig. 4). On the other hand, a slight reduction was observed in shear forces and bending moments in beam when 3-strut model was used as compared to the 1-strut model (Fig. 4). However, the force resultants in beam obtained using 3-strut model were also found to be more closely matching with that obtained using the partial shell models. This further supports the case of using a 3-strut model for masonry infills.

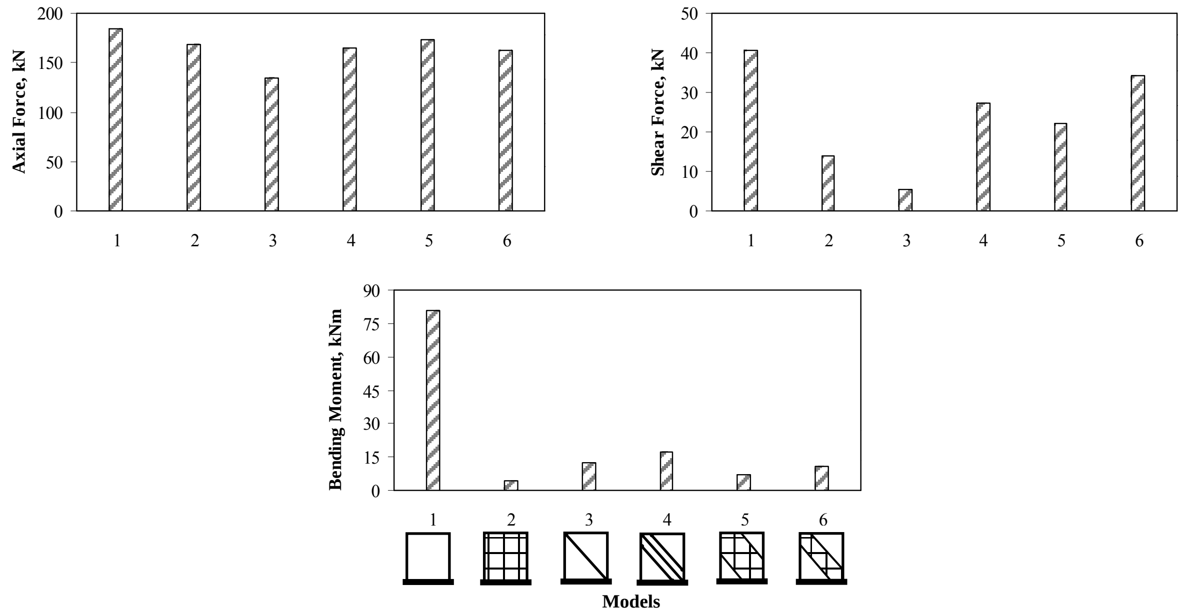

Figure 3 Comparison of maximum force resultants in columns for load case 1.5(DL+EQ) obtained using the six modeling schemes

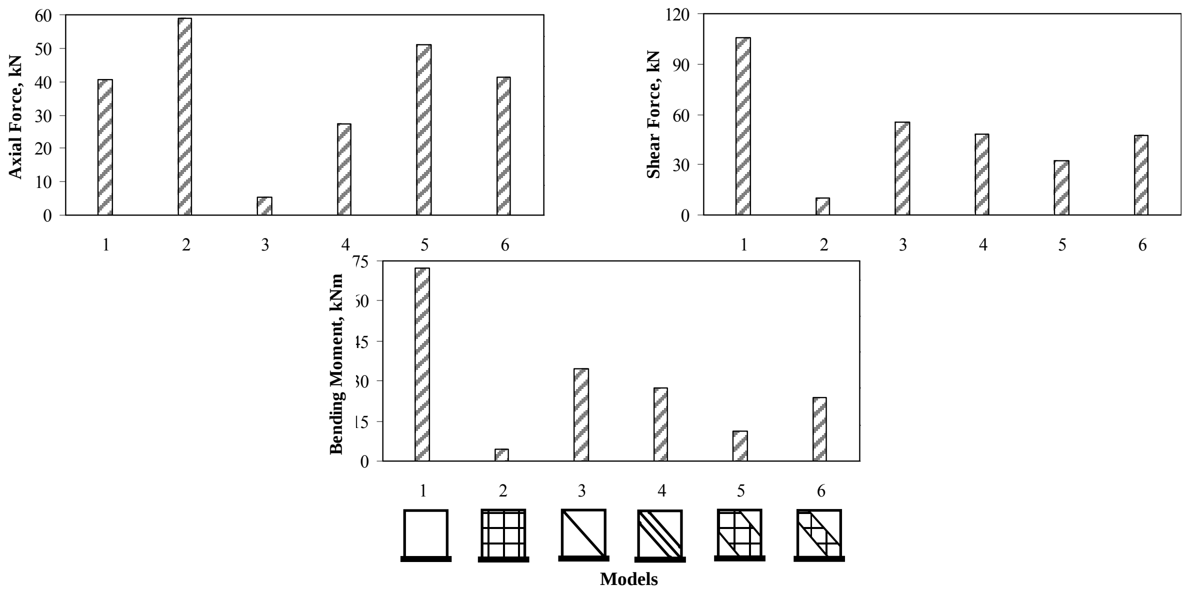

Figure 4 Comparison of maximum force resultants in beam for load case 1.5(DL+EQ) obtained using the six modeling schemes

Results of linear static analyses indicated that the RC frame members may be subjected to significant increase in force resultants due to presence of partial masonry infills; this is in contrast to the behaviour of fully-infilled frame. Force resultants in RC columns obtained using the 1-strut models were found to be less than those obtained using 3-strut models and partial shell models. Therefore, the 1-strut model underestimated the design lateral forces for masonry-infilled RC frames. Also, force resultants in RC beam and columns in case of 3-strut model were observed to be matching more closely with those obtained using the partial shell models, which more accurately represent the lateral load behaviour of masonry infills. As a result, the 3-strut model appears to be a better choice than the other models because of its simplicity over the finite element models, and its effectiveness in predicting the realistic force resultants in RC frame elements. In order to substantiate these results further, non-linear static pushover analyses were carried out for: 1-strut model and 3-strut model.

3. NONLINEAR STATIC PUSHOVER ANALYSIS

Non-linear static pushover analysis involves pushing structures laterally until a pre-specified lateral force or displacement is reached. In the present study, SAP2000[14] was used for displacement-controlled pushover analyses of Model 1 – bare frame, Model 3 – 1-strut model, and Model 4 – 3-strut model to understand difference in their behaviour in non-linear range. Non-linear material properties for different structural members are required to be specified in pushover analysis in addition to the elastic material properties discussed in linear analysis. In SAP2000, non-linearity is not distributed along length of members; instead, lumped plasticity is modeled at desired locations on members. Modeling of non-linear material properties in RC and masonry members of the frame is discussed in the following.

3.1. Modeling of RC members

Plasticity in RC members was assumed to be lumped at probable locations. Plastic hinges were assumed to form at a distance equal to half the average plastic hinge length l_p from the member ends; l_p was calculated using the following expression[4]:

$$l_p = 0.08L + 0.022 d_b f_y \qquad \text{(m)} \tag{2}$$

where L is length of member in m, d_b is diameter of longitudinal steel in m, and f_y is yield strength of longitudinal steel in MPa. Plastic hinges that generally develop in RC members are those corresponding to flexure and shear. Flexural hinge properties involve axial force – bending moment interaction (*P-M*) as failure envelope and bending moment – rotation (M-θ) as corresponding load deformation relation. On the other hand, shear hinge properties involve shear force – shear deformation relation (V-Δ).

P-M interaction properties for RC columns were developed using the work of Dasgupta[16] who used the stress-strain model for confined concrete developed by Razvi and Saatcioglu[17], and the stress-strain curves for reinforcement bars were obtained experimentally (ultimate strain ≈ 14.5%, ultimate stress ≈ $1.25f_y$). Strength and deformation properties for M-θ model were calculated using a simplified method in which bending moment diagram of a member under lateral forces are assumed to vary linearly[18,19]. And if the point of contra-flexure is in the mid-span, a member fixed at both ends can be replaced by an equivalent cantilever of half span with a concentrated load at its tip. The M-θ relationship for the linear distribution of moments was obtained using the moment-curvature relationship of the section. The ultimate rotation values given in FEMA[12] were simplified and taken as 1.5 times to 2.0 times the rotation corresponding to the maximum moment capacity for the section. Strength characteristics of the shear hinge (V-Δ) model for RC members were calculated using the relevant Indian Standards. Shear failure of RC members was considered to take place when shear strength of section was reached, and the failure was assumed to be force-controlled because of the associated brittleness. Typical hinge properties for RC members are shown in Fig. 5.

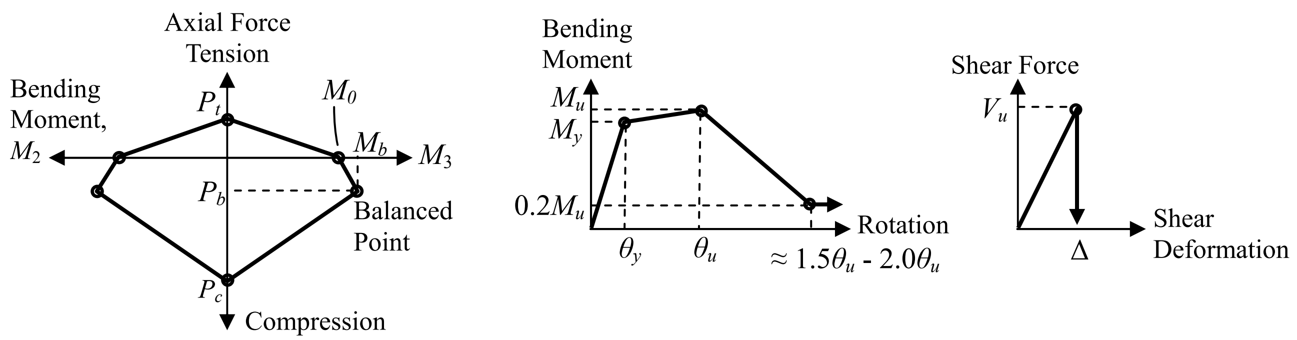

Figure 5 Typical hinge properties assigned to RC members of the frame

3.2. Modeling of masonry infills

In the model used in non-linear pushover analyses, masonry infills were modeled using compressive diagonal struts along the loaded diagonals. Stress-strain curves for masonry under compression were assigned as axial hinge properties to the struts. The non-linear stress-strain curves were obtained in an experimental study by Kaushik et al.[13] in which four different bricks and three different mortar grades were used to construct masonry prisms (5 bricks high). Average compressive strength of the burnt clay bricks used in the study was 21 MPa with coefficient of variation (COV) of about 30%. On the other hand, average compressive strength of the three grades (Cement : Lime : Sand by volume) of mortar cubes of 50 mm size was 3 MPa with about 20% COV (1:0:6 grade), 20 MPa with about 8% COV (1:0:3 grade), and 15 MPa with about 6% COV (1:0.5:4.5 grade). The tests were carried out as per the relevant ASTM Standards.

Stress-strain curves for masonry prisms were obtained by averaging the test data from seven specimens of each combination of bricks and mortar (total 84 specimens). In order to capture non-linear characteristics of the stress-strain curves, displacement controlled servo-hydraulic actuators were used to apply compressive forces on masonry prisms and corresponding compressive deformations were recorded using Epsilon extensometers. The average stress-strain curves for masonry prisms obtained for the three grades of mortar are shown in Fig. 6.

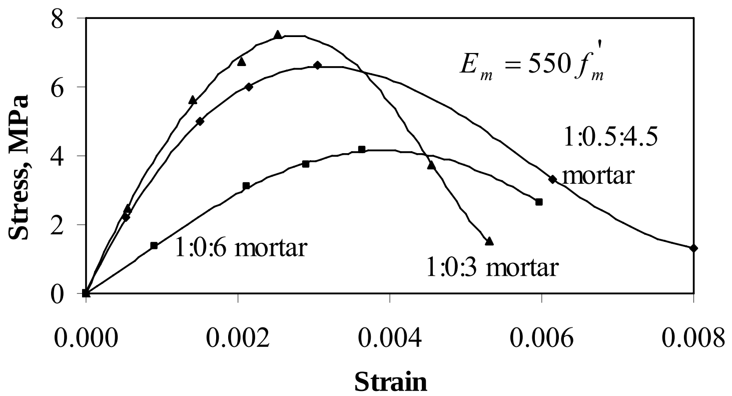

Figure 6 Non-linear stress-strain curves for masonry prisms under compression

The average compressive strength of masonry prisms obtained in the study for the three grades of mortar was 4.1 MPa with COV of about 25% (1:0:6 mortar grade), 7.5 MPa with COV of about 20% (1:0:3 mortar grade), and 6.6 MPa with COV of about 20% (1:0.5:4.5 mortar grade). In the present comparative study, stress-strain curve corresponding to the mortar grade 1:0:3 was used to model non-linear material properties of the struts. Plastic hinges in masonry infills were assumed to develop at center of the struts, and length of plastic hinge was considered as half to three-fourth of the strut length.

3.3. Results of Non-Linear Static Pushover Analyses

Pushover curves obtained by non-linear static analyses of the three models are shown in Fig. 7. For better comparison between different models, base shear and lateral displacement obtained by pushover analyses of the three models are normalized with respect to those obtained for the 1-strut model. As expected, initial stiffness of masonry-infilled frame was found to be significantly large as compared to the bare frame, and initial stiffness exhibited by Models 3 and 4 matched quite well. Most of the initial base shear was resisted by masonry infills because of their large initial stiffness; therefore, lateral strength of models with infills was found to be much higher than that of the bare frame.

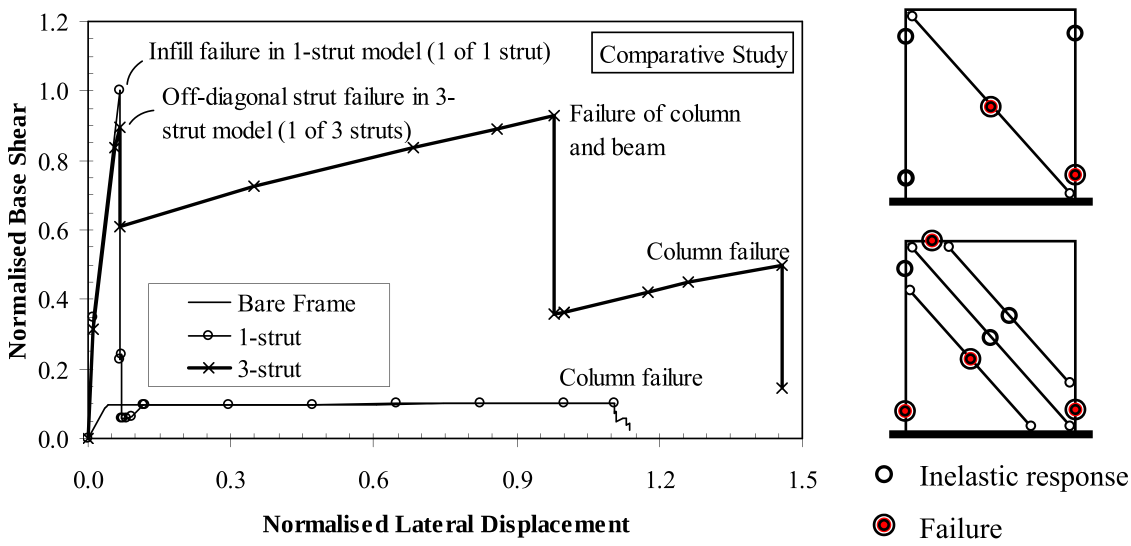

Figure 7 Pushover curves and plastic hinge formation in Models 1, 3, and 4

In Model 3, the single strut carried large amount of axial force, and therefore abrupt reduction in lateral strength of the frame was observed after failure of infill wall (modeled using a single strut). After failure of the strut, behaviour of Model 3 became similar to the bare frame model. This characteristic behaviour of 1-strut model is far from realistic performance of masonry-infilled frames in which infill walls do not fail abruptly. In comparison, lateral strength of Model 4 (3-strut model) was found to be only about 10% less than that of 1-strut model. In 3-strut model, presence of off-diagonal struts was responsible for development of large shear forces in columns of the frame. Moreover, lateral strength of the frame did not reduce drastically because of progressive (one-by-one) failure of the three struts. Flexural and shear failure of beam and columns resulted into failure of the RC frame.

Clearly, use of 3-strut model appears to be a more realistic and rational choice because of presence of off-diagonal struts, which distribute the stiffness of infills to a larger area, and therefore enforce gradual failure of infills. Moreover, unlike the 1-strut model, the 3-strut model was also found to model the local failures in RC beams and columns.

4. CONCLUSIONS

In a comparative analytical study involving various analytical models for masonry infills in a single-storey, single bay RC frame, the 1-strut model was found to be predicting the global behaviour (initial stiffness and ultimate failure load) of the system with reasonable accuracy. On the other hand, 3-strut model was found to be estimating the force resultants in the frame members more accurately as compared to that of a 1-strut model. Moreover, the 3-strut model was found to model local failures in the frame members in addition to the compressive failure of struts.

Under lateral forces when strut action develops, a finite area of infill is physically connected to the beams and columns of the frame. This finite area, which can be effectively modeled using a 3-strut model, is responsible for distribution of large stiffness of infill walls to a larger area on beams and columns; this prevents abrupt failure of masonry infills under increasing lateral forces. The finite area is also responsible for local shear failure in beams and columns, and in a 3-strut model, this failure mode can be represented in a more appropriate manner. Moreover, design force resultants in RC beams and columns can be obtained more accurately when 3-strut models are used. Therefore, the 3-strut model appears to be physically more appropriate than the 1-strut model for masonry-infilled RC frames.

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REFERENCES

1. Kaushik, H.B., Rai, D.C., and Jain, S.K. (2006). Code Approaches to Seismic Design of Masonry Infilled Reinforced Concrete Frames: A State-of-the-Art Review. *Earthquake Spectra* **22:4**, 961-983.
2. Holmes, M. (1961). Steel Frames with Brickwork and Concrete Infilling. *Proceedings of Institute of Civil Engineers* **19:2**, 473-478.
3. Stafford-Smith, B., and Carter, C. (1969). A Method of Analysis of Infilled Frames. *Proceedings of the Institution of Civil Engineers* **44**, 31-48.
4. Paulay, T., and Priestley, M. J. N. (1992). Seismic Design of Reinforced Concrete and Masonry Buildings. Wiley-Interscience, New York.
5. Saneinejad, A., and Hobbs, B. (1995). Inelastic Design of Infilled Frames. *Journal of Structural Engineering* **121:4**, 634-650.
6. Thiruvengadam, V. (1985). On the Natural Frequencies of Infilled Frames. *Earthquake Engineering and Structural Dynamics* **13**, 401-419.
7. Moghaddam, H. A., and Dowling, P. J. (1987). The State of the Art in Infilled Frames. ESEE Research Report No. 87-2, Imperial College of Science and Technology, Civil Engineering Department, London.
8. Abrams, D. P., ed., "Proceedings from the NCEER Workshop on Seismic Response of Masonry Infills," Technical Report No. NCEER-94-0004, February 4-5, National Center for Earthquake Engineering Research, State University of New York, Buffalo, USA, 1994.
9. El-Dakhakhni, W. W., Elgaaly, M., and Hamid, A. A. (2003). Three-Strut Model for Concrete Masonry-Infilled Steel Frames. *Journal of Structural Engineering* **129:2**, 177–185.
10. Reflak, J., and Fajfar, P. (1991). Elastic Analysis of Infilled Frames using Substructures. Proceedings of 6th Canadian Conference on Earthquake Engineering, Toronto, Canada, pp. 285–292.
11. Buonopane, S. G., and White, R. N. (1999). Pseudodynamic Testing of Masonry Infilled Reinforced Concrete Frame. *Journal of Structural Engineering* **125:6**, 578-589.
12. FEMA (2000). Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA 356). Prepared by ASCE for Federal Emergency Management Agency, Washington, D.C., USA.

13. Kaushik, H. B., Rai, D. C., and Jain, S. K. (2007). Stress-Strain Characteristics of Clay Brick Masonry under Uniaxial Compression. *Journal of Materials in Civil Engineering* **19:9**, 728-739.
14. CSI (2004). Structural Analysis Program (SAP2000) – Advanced, Static and Dynamic Finite Element Analysis of Structures. Computers and Structures, Inc., Berkeley, USA.
15. Guevara, L. T., and García, L. E. (2005). The Captive- and Short-Column Effects. *Earthquake Spectra*, **21:1**, 141-160.
16. Dasgupta, P. (2000). Effect of Confinement on Strength and Ductility of Large RC Hollow Sections. M.Tech. Thesis, Department of Civil Engineering, Indian Institute of Technology Kanpur, Kanpur, India.
17. Razvi, S., and Saatcioglu, M. (1999). Confinement Model for High Strength Concrete. *Journal of Structural Engineering* **125:3**, 281-289.
18. Park, R., and Paulay, T. (1975). Reinforced Concrete Structures. John Wiley and Sons, New York.
19. Das, D., and Murty, C. V. R. (2004). Brick Masonry Infills in Seismic Design of RC Framed Buildings: Part 2–Behaviour. *The Indian Concrete Journal* **78:8,** 31-38.