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STRENGTH AND STIFFNESS PREDICTION OF MASONRY INFILL PANELS

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SUMMARY

Masonry infill panels in framed structures have been long known to affect strength, stiffness and ductility of the composite structure. In seismic areas, ignoring the composite action is not always on the safe side, since the interaction between the panel and the frame under lateral loads dramatically changes the stiffness and the dynamic characteristics of the composite structure and hence its response to seismic loads. This study presents a simple method of estimating the stiffness and the lateral load capacity of concrete masonry-infilled steel frames failing in corner crushing mode, as well as the internal forces in the steel frame members. In this method, each masonry panel is replaced by three struts with force-deformation characteristics based on the orthotropic behaviour of the masonry infill panels. The method can be easily computerized and included in non-linear analysis and design of three-dimensional infilled frame structures.

Key Words: Analytical Modelling, Concrete Masonry, Infill Walls, In-Plane Strength, Lateral Stiffness, Steel Frames.

INTRODUCTION

The current study aims to present a simple method of predicting the stiffness as well as the ultimate load capacity of concrete masonry-infilled steel frames (CMISF). The method is easy enough to be included in the design or the analysis of such systems using the available resources in typical design offices. The technique can be used to produce design aids and to develop a conceptual approach for the analysis and design of such composite systems.

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Based on the knowledge gained from both analytical and experimental studies during the last five decades, different failure modes of masonry infilled frames can be categorized into five distinct modes, namely:

1. Corner crushing mode (CC mode), represents crushing of the infill in at least one of its loaded corners, as shown in Fig. 1-a. This mode is usually associated with infill of weak masonry blocks surrounded by a frame with weak joints and strong members.
2. Sliding shear mode (SS mode), represents horizontal sliding shear failure through bed joints of a masonry infill, as shown in Fig. 1-b. This mode is associated with infill of weak mortar joints and strong frame.
3. Diagonal compression mode (DC mode), represents crushing of the infill within its central region, as shown in Fig. 1-c. This mode is associated with a relatively slender infill, where failure results from out-of-plane buckling instability of the infill.
4. Diagonal cracking mode (DK mode), in the form of a crack connecting the two loaded corners, as shown in Fig. 1-d. This mode is associated with weak frame or frame with weak joints and strong members infilled with a rather strong infill.
5. Frame failure mode (FF mode), in the form of plastic hinges in the columns or the beam-column connection, as shown in Fig. 1-e. This mode is also associated with weak frame or frame with weak joints and strong members infilled with a rather strong infill.

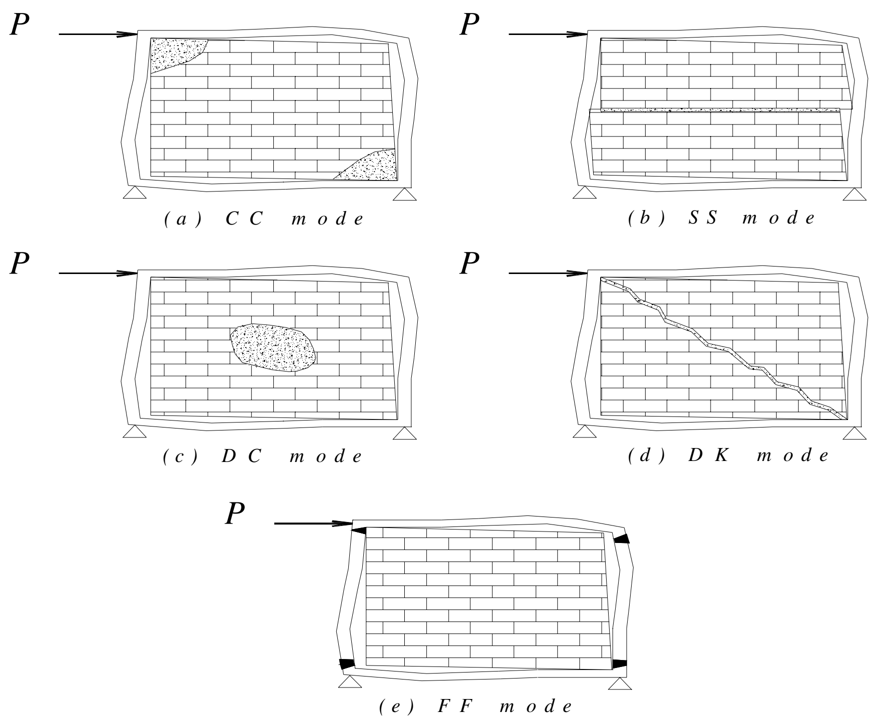

Fig. 1. Different Failure Modes of Masonry Infilled Frames:
a) **C**orner **C**rushing Mode; b) **S**liding **S**hear Mode; c) **D**iagonal **C**ompression Mode
d) **D**iagonal Crac**k**ing Mode; and e) **F**rame **F**ailure Mode

It is worth mentioning that only the first two modes, the CC and the SS modes, are of practical importance (Comite [1]) since the third mode is very rare to occur and requires a high slenderness ratio of the infill to result in out-of-plane buckling of the infill under in-plane loading. This is hardly the case when practical panel dimensions are used, and the panel thickness is designed to satisfy the acoustic isolation and fire protection requirements. The fourth mode should not be considered a failure mode, due to the fact that the wall still carries more load after it cracks. The fifth mode, although might be worth considering in the case of reinforced concrete frames, yet when it comes to steel frames infilled with un-reinforced hollow masonry blocks, this mode hardly occurs. The study conducted herein models the CC mode only, which is the most common mode of failure. In order to determine the governing failure mode, the capacity of the infill panels obtained by the proposed method should be compared to the capacity under SS mode which may be estimated using the method suggested by Paulay and Priestley [2].

DEVELOPMENT OF CMISF MODEL

Subjecting a bare masonry panel to a diagonal loading usually results in a sudden failure initiated by a stepped crack along the loaded diagonal, dividing the panel into two separate parts and immediately leading to the collapse of the specimen due to lack of confinement (El-Dakhakhni et al. [3]). Unlike the unconfined panel, as soon as a diagonal crack develops within an infilled panel (usually at a much lower load and deflection levels than ultimate) the panel finds itself confined within the surrounding frame and bearing against it over contact lengths, as shown in Fig. 2-a. The contact lengths provide enough confinement to prevent failure and allowing the panel to carry more load until the existing diagonal crack continues to widen and new cracks appear leading, eventually, to ultimate failure. To model this behaviour it is rational to consider the panel to be composed of two diagonal regions, as shown in Fig. 2. One region connects the top beam to the leeward column and the other connects the windward column to the lower beam. As reported by many researchers, (Reflak and Fajfar [4], Saneinejad and Hobbs [5], Mosalam et al. [6], [7], [8] and Buonopane and White [9]), the bending moments and shearing forces in the frame members cannot be replicated using a single diagonal strut (although has been used frequently) connecting the two loaded corners. Based on the above, it is suggested that, at least two additional off-diagonal struts located at the points of maximum field moments in the beams and the columns are required to reproduce these moments as shown in Fig. 2-b. Furthermore, since the load transfer from the frame members to the infill depends on the contact length which, in turn, is affected by the stiffness and the deflected shape of the frame members, the use of a multi-strut model will allow for the interaction between different panels in multi-story buildings. This is due to the fact that some beams (and/or columns) will be loaded from the upper and lower panels (or left and right panels) at different locations within the span (or height), which will affect their deflected shape and hence the panel's strains, and consequently changing the failure load.

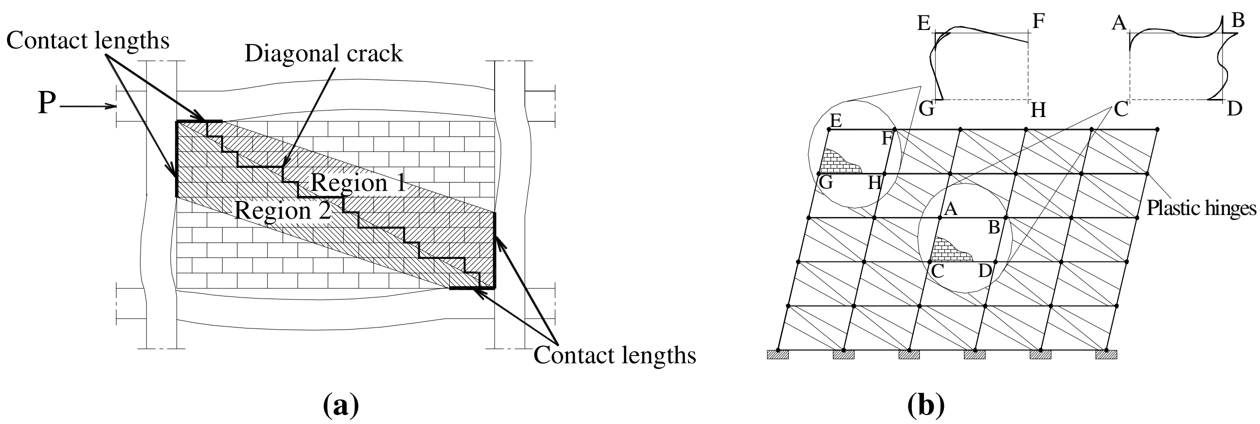

Fig. 2. The Infill Panels Behaviour: a) Separation into Two Diagonal Regions; and b) Resulting Bending Moment Diagrams for a Different Bays in Multi-Story Infilled Frame Building

Steel frame model

The steel frame members were modeled elastic beam elements connected by non-linear rotational spring elements at the beam-column joints. The concentration of non-linearity in the frame joints only is based on the fact that due to the limited infill ductility and thus limited frame deformation at the peak load except at the loaded corners, the maximum field moments as well as the bending moments at the unloaded joints are lower than that at the loaded joints and has been found to be, at most, 20% of the plastic moment capacity of the section (Saneinejad and Hobbs [5]).

Using elastic frame elements requires the area and the moment of inertia of the member section as well as Young's modulus of the steel to be the only required input properties for the frame sections to form the stiffness matrix; this eliminates the need to modify the stiffness matrix as well as the iteration process to account for the non-linear behaviour of the steel frame. The use of elastic elements is justified based on the earlier discussion on the steel frame geometrical model.

The ultimate moment capacity of the non-linear rotational spring, representing the beam-column joint, is defined as the minimum of the column's, the beam's or the connection's ultimate capacity, M_{pj} , which will be referred to as the plastic moment capacity of the joint. The rotational stiffness of the spring can be calibrated so that the lateral stiffness of the frame model matches that of the actual bare frame, which can be obtained experimentally or using simple elastic analysis or, in case of semi-rigidly connected members, using available data on modeling semi-rigid connections (Chen and Lui [10]). The joint behaviour is shown in Fig. 3-a, where, ϕ_{el} is the maximum elastic rotation that the joint can undergo without yielding; ϕ_{pl} is the maximum plastic rotation before the joint undergoes moment reduction below M_{pj}; and ϕ_{ult} is the maximum plastic rotation beyond which the joint cannot sustain any moment.

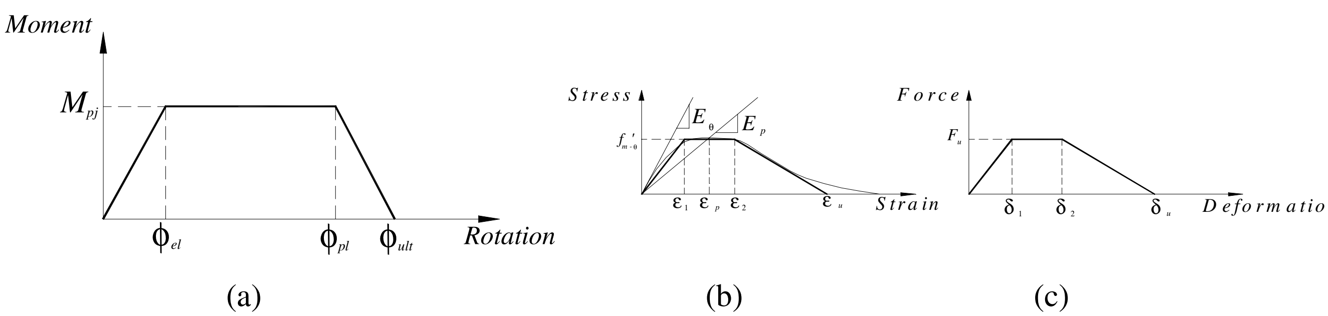

Fig. 3. Proposed Infilled Frame Components Behaviour: a) The Beam-Column Connection: b) Simplified Tri-Linear Stress-Strain Relation of the Concrete Masonry; and c) Typical Force-Deformation Relation for Struts

Infill pane model

El-Dakhakhni et al. [3] showed that for steel frame members infilled with masonry panel, the points of maximum field moment developed within the frame members lie approximately at the end of the contact lengths, and are located at distances from the beam-column connection given by

$$\alpha_c h = \sqrt{\frac{2\,(M_{pj} + 0.2\,M_{pc})}{t\,f'_{m-0}}} \leq 0.4h \qquad (1)$$

$$\alpha_b l = \sqrt{\frac{2\,(M_{pj} + 0.2\;M_{pb})}{t\,f'_{m-90}}} \leq \;0.4l \tag{2}$$

where, α_c is the ratio of the column contact length to the height of the column and α_b is the ratio of the beam contact length to the span of the beam; h is the column height and l is the beam span. M_{pj} is the minimum of the column's, the beam's or the connection's plastic moment capacity, referred to as the plastic moment capacity of the joint; M_{pc} and M_{pb} are the column and the beam plastic moment capacities respectively; f_{m-0} and f_{m-90} are the compressive strength of the masonry panel parallel and normal to the bed joint respectively; and finally t is the wall thickness.
Assuming that the equivalent uniformly loaded diagonal region of the panel to be of area equal to A, where A is to be given later, hence each region of the panel shown in Fig. 2-a will be of area $=A/2$. Furthermore, assuming uniform contact stress distribution along the contact areas, each region will be replaced by two struts, each of area $A_1=1/2$x$(A/2)=A/4$, located at the beginning and the end of the contact length. Combining the two struts connecting the loaded corners, from the two regions, into one strut of area $A_2=2$x$A_1= A/2$ results in representing the whole panel by three struts, an upper strut connecting the upper beam with the leeward column with area $A_1=A/4$, a middle strut connecting the two loaded corners with area $A_2=A/2$, and finally a lower strut of area $A_1=A/4$ connecting the windward column with the lower beam, where $A=2A_1+A_2$. The proposed model for a typical CMISF is shown in Fig.4.

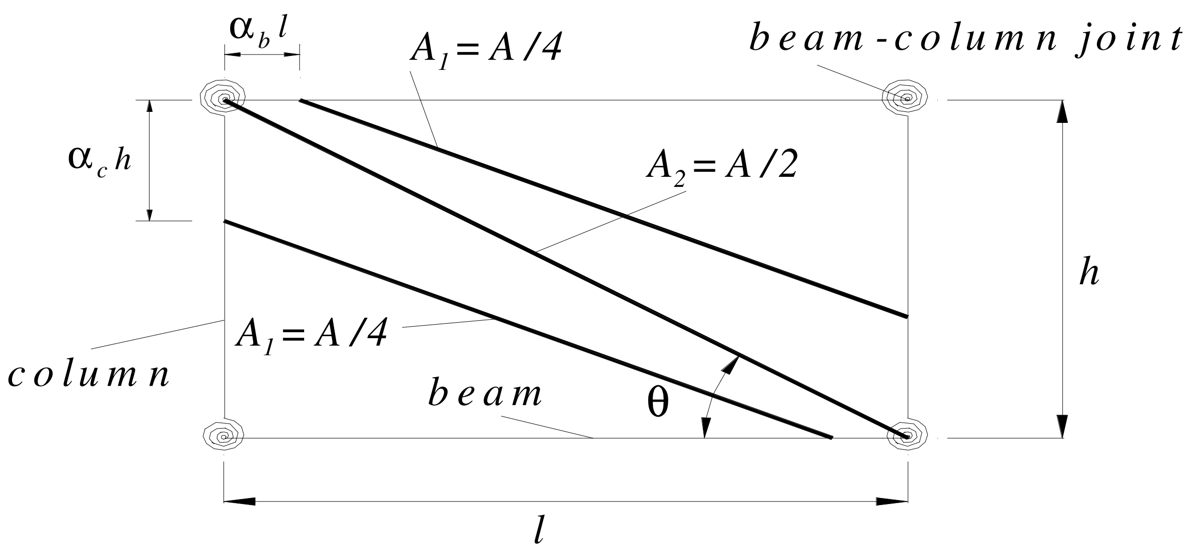

Fig. 4. The Proposed CMISF Model

El-Dakhakhni et al. [3] suggested that the total diagonal struts area, A, is to be calculated by

$$A = \frac{(1-\alpha_c)\,\alpha_c\,h\;t}{\cos\;\theta} \tag{3}$$

Due to the fact that the panel behaves as if it was diagonally loaded, constitutive relations of orthotropic plates (Shames and Cozzarelli [11]) and axes transformation matrix, are used to obtain the Young's modulus, E_θ, of the panel in the diagonal direction using the following equation

$$E_\theta = \frac{1}{\frac{1}{E_0}\cos^4\theta + \left[-\frac{2\nu_{0-90}}{E_0} + \frac{1}{G}\right]\cos^2\theta\sin^2\theta + \frac{1}{E_{90}}\sin^4\theta} \tag{4}$$

where, E_0 and E_{90} are Young's moduli in the direction parallel and normal to the bed joints respectively; $\nu_{0\text{-}90}$ is Poisson's Ratio defined as the ratio of the strain in the direction normal to the bed joints due to the strain in the direction parallel to the bed joints; and G is the shear modulus.

It was also suggested by El-Dakhakhni et al. [3] that, not only Young's modulus will change, but also the ultimate strength of the masonry wall in the θ direction, $f_{m\text{-}\theta}$. To account for this direction variation, and to relate E_θ to $f_{m\text{-}\theta}$ by the same factor relating E_{90} to $f_{m\text{-}90}$, i.e.

$$f'_{m-\theta} = \frac{E_\theta}{\alpha} \text{ where, } \alpha = \frac{E_{90}}{f'_{m-90}} \tag{5}$$

Based on non-linear FE analyses, Saneinejad and Hobbs [5] suggested that the secant stiffness of the infilled frames at the peak load to be half the initial stiffness. This might be directly interpreted into the stress strain relation for the masonry panel by assuming that the secant Young's modulus at peak load, E_p, is equal to half the initial Young's modulus, E_θ, i.e. $E_p = 0.5\,E_\theta$. As shown in Fig. 3-b, knowing E_p and f_θ it is now an easy task to determine the strain corresponding to the peak load ε_p. Instead of using the parabolic stress-strain relation shown in Fig. 3-b, it is suggested to approximate it into a *tri-linear* relation that is simpler and more practical for analysis as shown by the thick lines in the same figure. Unless more accurate data are available, the parameters in Fig. 3-b will be assumed according to the following

$$\varepsilon_1 = \varepsilon_p - 0.001 \tag{6-a}$$

$$\varepsilon_2 = \varepsilon_p + 0.001 \tag{6-b}$$

$$\varepsilon_u = 0.01 \tag{6-c}$$

Knowing the stress strain relation along with the area (from equation 3) and the length of each of the three struts (which can be easily calculated knowing the panel dimensions and the contact lengths given by equations 1 and 2) makes it possible to obtain a force-deformation relation for each strut. As shown in Fig. 3-c, by simply multiplying the strains ε_1, ε_2 and ε_u by the length of each strut resulting in obtaining δ_1, δ_2 and δ_u respectively. Also multiplying the stress, $f_{m\text{-}\theta}$, by the area of each strut results in obtaining F_u for each strut. In fact assuming that E_θ and $f_{m\text{-}\theta}$ are the same for all struts and neglecting the minor difference in the inclination angle between the middle strut and both the upper and the lower strut, will result in finding only two distinct force-deformation relations, one for the upper and lower struts and another for the middle strut.

MODEL VERIFICATION

The suggested method was used to model five CMISF specimens. Four of the specimens were tested at the University of New Brunswick under monotonic racking load by Yong [12], McBride [13], Amos [14], and Richardson [15]. The Fifth specimen was tested in Cornell University by Mosalam et al. [6] under quasi-static displacement controlled loading. The first four specimens are identical single panel CMISF with different masonry strength. The load-deflection relation of the model for specimen WB2 tested by Yong [12], utilizing the proposed technique is shown in Fig. 5-a along with test results for comparison. The figure shows the capabilities of the proposed method to predict both the stiffness and ultimate load capacity up to failure. The model appears to overestimate the ultimate capacity by about 9% and acceptably estimates the average stiffness up to failure. The load-deflection relations for specimen WA3 (McBride [13]), and specimen WC7 (Amos [14]) are shown in Figs. 5-b and 5-c respectively. In which the model closely approximate the stiffness of the CMISF up to failure and overestimate the strength of specimen WA3 by 14 %, and underestimate that of specimen WC7 by 3%. The fourth specimen WD7, tested by Richardson [15], was loaded up to 60 mm. The load-deflection relations for the bare and the infilled frame model are shown in Fig. 5-d along with test results for comparison. Again the proposed model is efficient in duplicating the test results up to failure. The model underestimated the failure load by 10% and the experimental test data show that the infilled frame gradually degrades and eventually at some point it will reach the ultimate capacity of the bare frame.

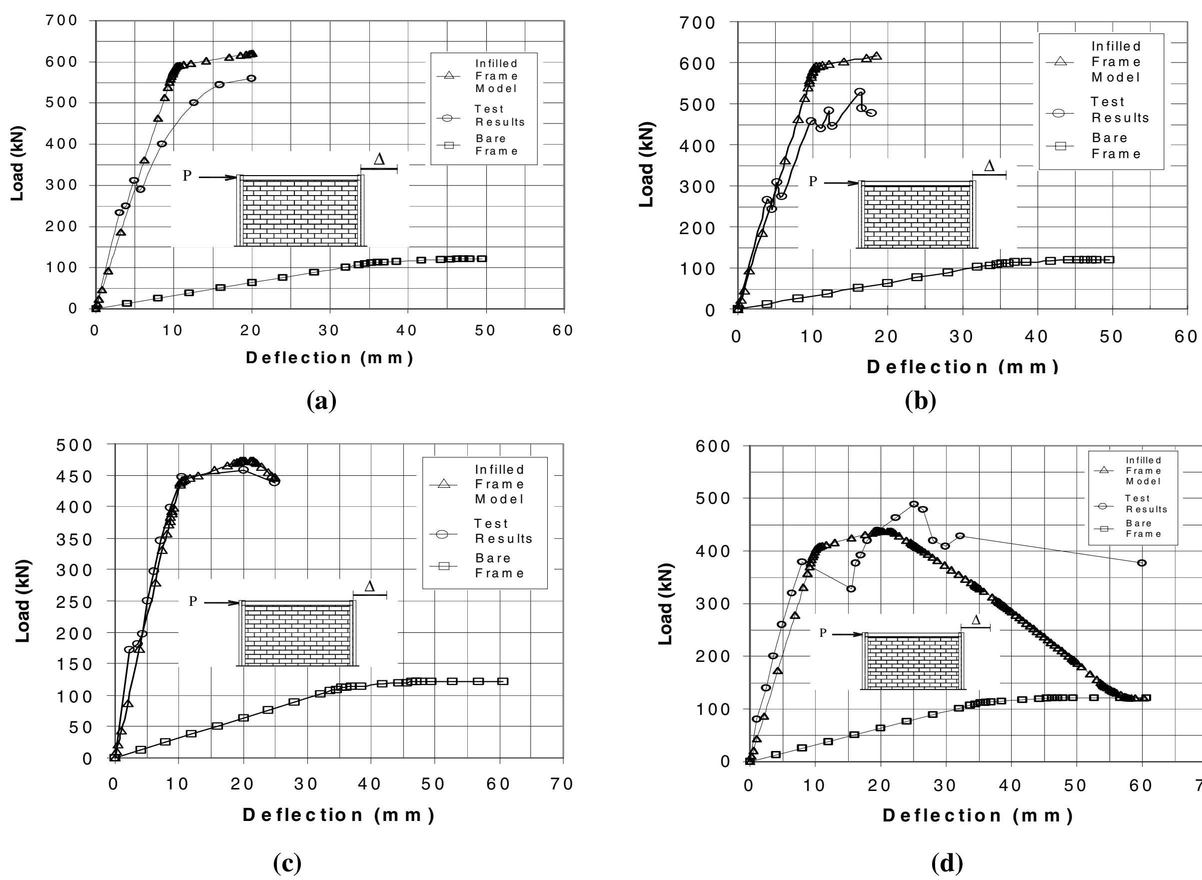

Fig. 5. Load-Deflection Relations for Specimens: a) WB2 (Yong [12]); b) WA3 (McBride [13]); c) Specimen WC7 (Amos [14]); and d) Specimen WD7 (Richardson [15])

Specimen Q21SSB, tested by Mosalam et al. [6] was modeled using the same technique; Fig. 6-a shows the load-deflection relation of the bare frame model and the infilled frame model along with the envelope of the cycling loading test. The model accurately represents the infilled frame up to a deflection of 6 mm, at which the model underestimated the specimen capacity by less than 2%. After this displacement, failure occurred in the specimen yet the model continued to carry more load, but with a very low stiffness, then it gradually loses its strength and fails. Fig. 6-b shows the bending moments in the model members at a load of 41.5 kN, before failure. These moments have the same trend as those obtained by Mosalam et al. [7] and suggested by Reflak and Fajfar [4], Saneinejad and Hobbs [5], and Buonopane and White [9].

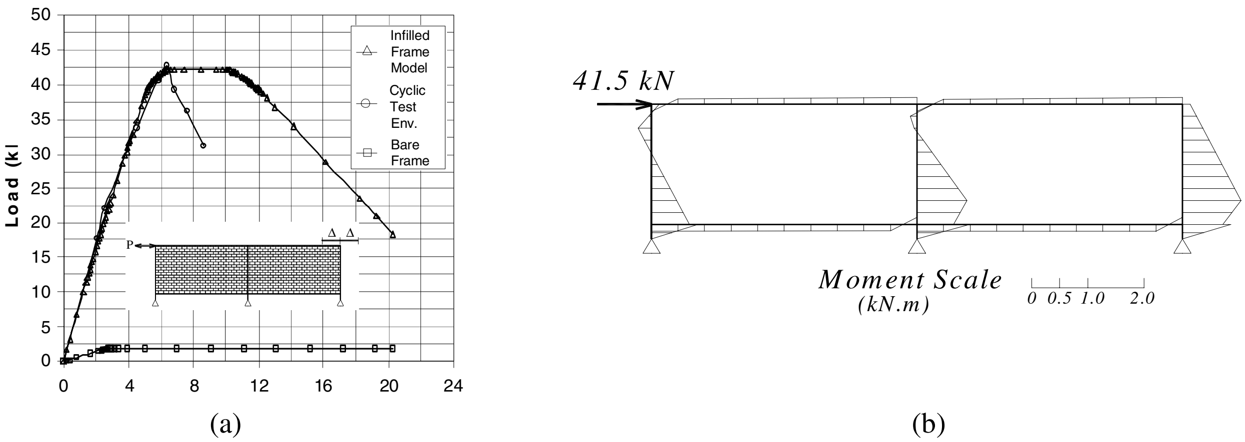

Fig. 6. Model Prediction of Specimen Q21SSB (Mosalam et al. [6]) Behaviour: a) Load-Deflection Relation; and b) Bending Moment Diagram (drawn on the tension side)

CONCLUSIONS

This paper presents an analytical method of predicting the stiffness and the ultimate load capacity of concrete masonry infilled steel frames (CMISF) failing in corner crushing mode. Based on the present study, the following conclusions can be inferred:

1. The proposed analytical technique predicts the lateral stiffness up to failure, and the ultimate load capacity of CMISF to an acceptable degree of accuracy. The technique accounts for the nonlinear behaviour that occurs in both the steel frame (due to formation of plastic hinges) and in the masonry panel (due to crushing). The technique considers the diagonal tension cracking in the masonry joints merely as a serviceability limit state.
2. The use of three struts instead of a single one is justified based on the observed bending moments in the frame members, which cannot be generated using a single strut. Furthermore, the three struts do not fail simultaneously, which is the case in actual infill panels, since the crushing starts at the corners and keeps propagating in the corner region leading to failure of the panel. The use of the three struts will also facilitate modeling the interaction among the different panels in multi-story buildings.

3. Instead of using the actual nonlinear stress-strain relation, an option, which might not be available in structural analysis software, a simplified tri-linear stress-strain relation is employed for the masonry. A similar relation is also used in modeling the steel frame load-deformation relation. It is worth mentioning that this simplification results in a less solution time, specially, in multi-story 3-D structures with large number of DOF. For linear elastic analysis purposes, a simpler way is to utilize the first part only of the tri-linear relation in order to obtain the stiffness and the ultimate load.

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