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STRUT-AND-TIE MODEL FOR PREDICTING THE SHEAR CAPACITY OF CONFINED MASONRY WALLS WITH AND WITHOUT OPENINGS

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Abstract

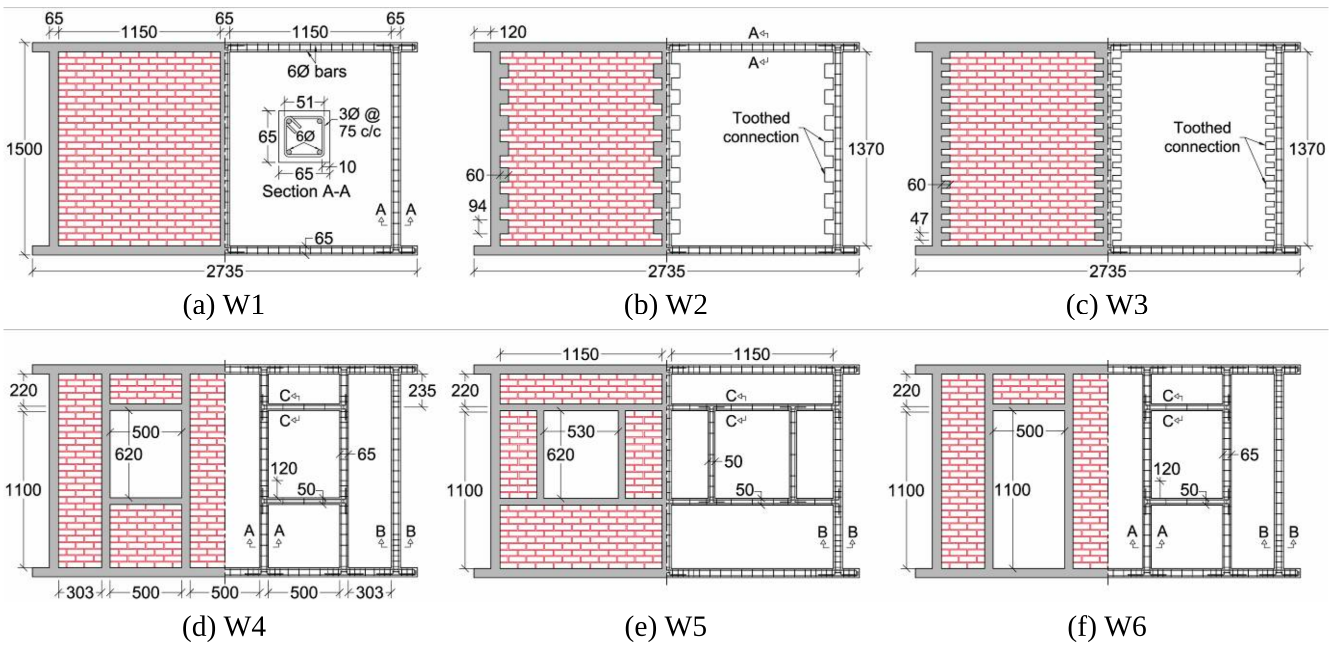
The confined masonry structure consists of load bearing walls strengthened with nominally reinforced concrete elements at the perimeter and other key locations. The confined masonry system has evolved based on its satisfactory performance in past earthquakes and can be considered as one of the most suitable alternatives to seismically vulnerable unreinforced masonry system due to its similar construction practice and economic feasibility. However, the existing analytical models for estimating the design shear capacity of confined masonry walls are primarily semi-empirical equations which are based on either friction theory or elementary theory of elasticity. These analytical equations are either highly influenced by the formulae originally developed for unreinforced and reinforced masonry walls or are based on the limited number of laboratory tests. The inconsistent predictions of in-plane shear strength by these existing models necessitate the development of a rational design method for confined masonry walls, especially with openings.

Strut-and-tie model has evolved as the most useful method for shear critical structures and disturbed regions (often called as D-region) in concrete structures. This method provides a rational and consistent design approach by idealizing the structural member with appropriate truss models. However, a prior knowledge of flow of stresses is required to develop strut-and-tie model for complex structural members, such as confined masonry walls with openings. Thus, initially nonlinear finite element analyses were performed on confined masonry walls which satisfactorily simulated their lateral load-deformation behavior and failure pattern. The principal stress vector diagrams obtained from the FE analyses were used to develop strut-and-tie models for confined masonry walls with and without openings. These strut-and-tie models provided an insight on how to construct the network of struts and ties for complex cases particularly in walls with openings surrounded with confining elements. With few simplifying assumptions, strut-and-tie models consistently provided good predictions for the in-plane strength of confined masonry walls within an error of 15% when compared to the experimental results. Due to its simplicity and reliability, the strut-and-tie model can be put into practice for the analysis and design of single to multi-storey confined masonry buildings.

Keywords: Confined Masonry; Shear Strength; Strut-and-Tie Model; Finite Element Analysis

1. Introduction

Many in-plane strength and stiffness predictive equations are available in the literature for confined masonry (CM) walls but these equations are primarily semi-empirical which are based on the limited number of laboratory tests [1, 2]. The reliability of available equations has been judged by comparing the predicted shear capacity with the results of experimental study conducted on CM walls with and without openings [1, 3]. Key details of six tested CM wall specimens along with the material properties are given in Fig. 1 and Table 1. Two different types of confinement scheme were used to enclose the opening: Scheme A - opening confined on all sides with tie-columns extending from bottom to top tie-beam (i.e., full-height of the wall) and Scheme B - opening confined on all sides with continuous horizontal band (sill and lintel bands) and vertical confining members extend only to the height of opening.



(a) W1 (b) W2 (c) W3

(d) W4 (e) W5 (f) W6

Fig. 1 – (a) – (f) Details of confined masonry wall specimens

Table 1 – Average properties of materials and summary of various response parameters

Wall specimen	$f_m^{c\prime}$ (MPa)	E_m (MPa)	$f_c^{c\prime}$ (MPa)	Ultimate load, R_{max} (kN)	Initial Stiffness (kN/mm)
W1 (No toothing)	8.8	3845	33.1	95.2	40.8
W2 (Coarser toothing)	8.2	3165	43.6	90.1	39.5
W3 (Dense toothing)	9.4	3353	40.7	97.5	36.6
W4 (Two window opening confined with scheme A)	7.8	2854	30.3	75.3	33.8
W5 (Two window opening confined with scheme B)	7.9	3026	30.3	94.1	33.4
W6 (Window and door opening confined with scheme A)	8.5	3927	34.1	63.8	25.8

To simplify the design procedure most of the existing analytical models neglected the contribution of the tie-column reinforcement which resulted in a highly conservative estimates of the shear capacity of CM walls. Better shear strength predictions were obtained for those models which include the amount of longitudinal reinforcement in tie-columns [1, 2]. Table 2 compares the values of ratios $R_{\max(exp)}$ /$R_{\max(cal)}$ obtained from the existing models which include the contribution of amount of the longitudinal reinforcement in tie-columns. Majority of existing models either greatly overestimate [4, 5] or underestimate the shear strength [6, 7] of confined masonry walls (Table 2). Among all the existing models, the semi-empirical equation given in the Chinese code [8] provided the best prediction for the in-plane strength of solid CM walls. However, the existing equations were unable to consistently predict the shear strength for perforated CM walls.

The inconsistent predictions necessitate the development of a rational design method for CM walls, especially with openings. Strut-and-tie model could be the possible approach for estimating the shear strength of CM masonry walls. This method idealizes the complex structural member with an appropriate simplified truss model. However, a prior knowledge of flow of stresses is required to develop strut-and-tie model for complex structural members, such as walls with openings confined with RC members. Thus, a nonlinear finite element analyses were performed on CM walls which assisted in determining the stress flow and based on the obtained principal stress vector diagrams strut-and-tie models have been developed for walls with and without openings. This paper will first briefly discuss the finite element analyses and its adequacy in simulating the lateral load-deformation behavior and failure pattern of CM walls. Lastly the strut-and-tie model developed for predicting the in-plane strength of CM walls is presented.

Table 2 – Comparison between the experimental and predicted values of maximum strength

Model [Reference]	$\boldsymbol{R}_{\max(exp)} / \boldsymbol{R}_{\max(cal)}$						
	Solid CM walls			Perforated CM walls			Mean ± SD
	W1	W2	W3	W4	W5	W6	
Flores 1996 [6]	1.47	1.36	1.43	1.31	2.09	1.29	1.49 ± 0.30
Marinilli 2006 [7]	1.36	1.31	1.37	1.10	1.94	1.08	1.36 ± 0.31
Tomaževič 1997 [4]	0.86	0.78	0.86	0.74	1.20	0.74	0.86 ± 0.17
Bourzam 2008 [5]	0.78	0.72	0.79	0.58	1.10	0.58	0.76 ± 0.19
NSPRC 2001 [8]	1.06	1.04	1.07	1.14	1.52	1.11	1.16 ± 0.18

2. Finite Element Model

Finite element macro-models were developed for all tested CM walls using Abaqus [9] and the predicted behavior were compared with experimental results. The masonry was modeled at the macroscopic level; however, the interaction between the masonry and confining RC elements was done at the microscopic level. The discretization of masonry and confining elements was achieved by eight-node 3-D brick elements (C3D8R) of the Abaqus element library. 2-noded 3-D truss elements (T3D2) were used to model the longitudinal and transverse reinforcing bars, which were 'embedded' to the surrounding concrete elements. Tie-constraints were specified to represent the composite interaction at all wall-to-tie-column interfaces except at top surface of the horizontal RC member above which the masonry had been laid. To account for the possible slip phenomena at such interfaces of masonry and confining RC elements, appropriate description through surface interaction was assigned for tangential direction (friction factor, $\mu_f = 0.6$) and normal direction (hard contact). No cohesion was defined for the interaction of feeble contact along these surfaces. A schematic diagram of a typical wall representing the different finite elements is shown in Fig. 2.

The explicit finite element analysis technique was used in this study, which is more suitable to capture and represent the nonlinear behavior of the masonry wall. Concrete Damaged Plasticity (CDP) model available in the Abaqus material library was used to simulate the inelastic behavior of the masonry [9]. The model uses the concept of isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity to represent the inelastic behavior. The CDP model is based on the non-associated flow rule and provides necessary control to dilatancy in modeling friction and quasi-brittle materials [10, 11]. The stress-strain curves were developed for masonry and concrete under compression and tension for each confined masonry wall using properties specified in Table 1.

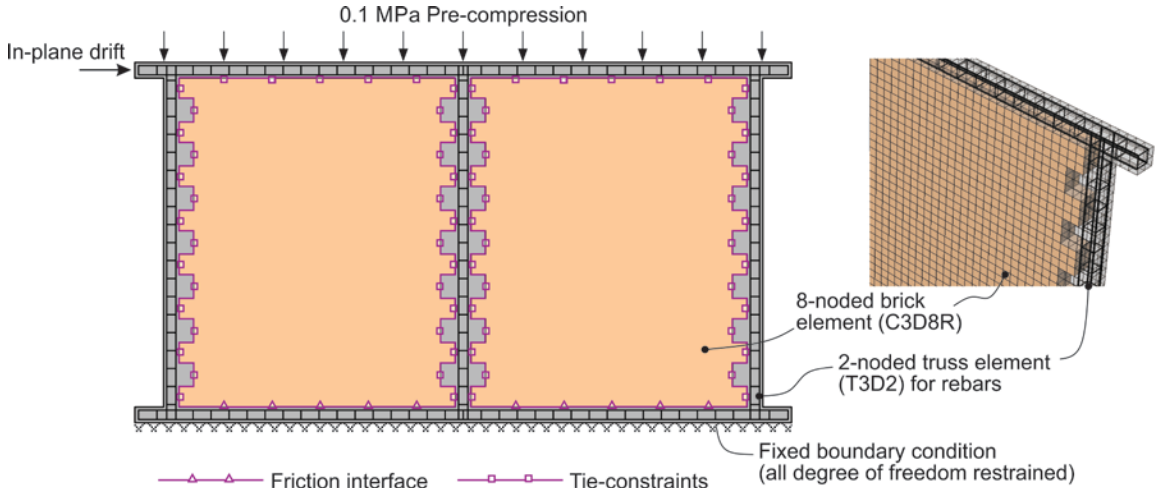


Fig. 2 – Schematic diagram of a typical wall illustrating various elements used in the finite element analyses

The stress-strain curve specified for masonry in compression was developed using the simplified tri-linear model as shown in Fig. 3a [12]. For tensile behavior, the ascending branch was obtained from the elastic behavior of masonry and the post-peak softening region was approximated by a linear function (see Fig. 3b). The tensile strength of masonry was taken as one-tenth of its compressive strength [13]. Equations to derive ascending and descending branch of compressive and tensile stress-strain curves are given in Figs. 3a and 3b.

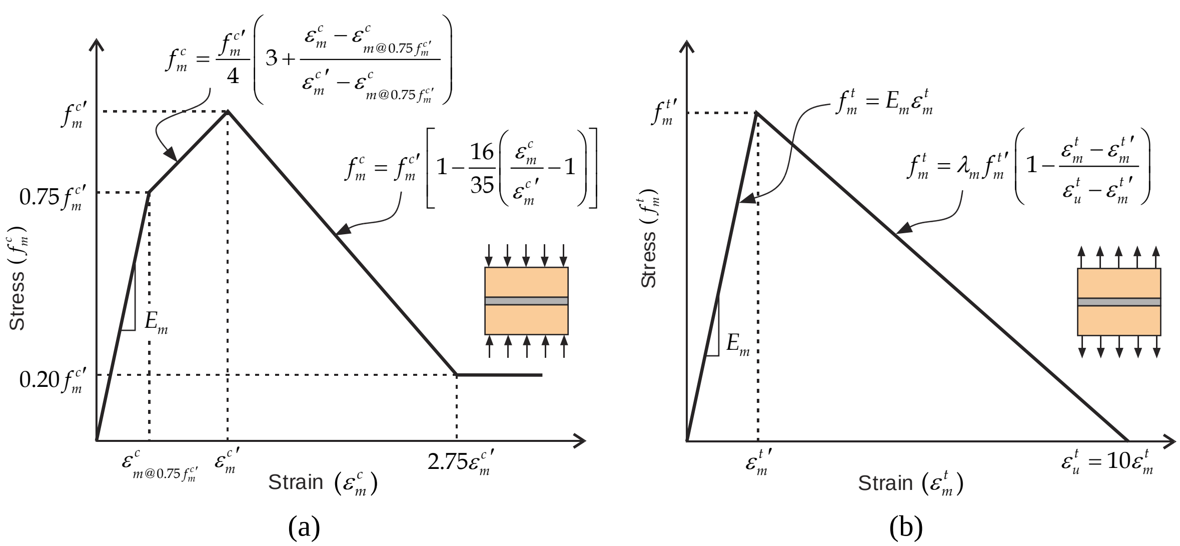


Fig. 3 – Material model of masonry specified in finite element analyses: a) compressive stress-strain curve, and b) tensile stress-strain curve

The model proposed by Kent and Park [14] to describe the compressive behavior of concrete was specified for the confining elements in all specimens (Fig. 4a). The tensile stress-strain behavior of concrete was defined based on the tension softening law given by Gopalaratnam and Shah [15] (Fig. 4b). Equations which described the ascending and descending branch of this curve are given in Figs. 4a and 4b. The strains corresponding to peak compressive and tensile stress were assumed as 0.002 and 0.00015, respectively. To define the tension softening behavior of concrete, the crack width, w_c and constants, k_c and λ_c are required to be specified. The value of these constant are, k_c = 0.063 and λ_c = 1.01, when the crack width is in micrometers [16]. Other required material properties were taken as default values in Abaqus [9].

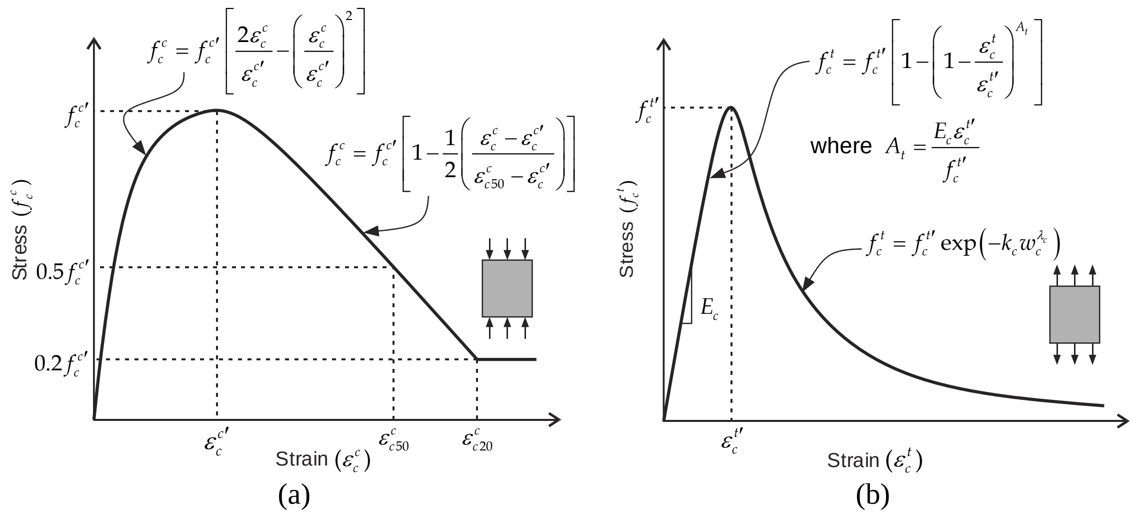


Fig. 4 – Material model of concrete specified in finite element analyses: a) compressive stress-strain curve, and b) tensile stress-strain curve

The appropriate boundary conditions were achieved by either deleting or constraining relevant nodal degrees of freedom of various elements. All the degrees of freedom of the nodes at the base were restrained in order to simulate a fixed boundary condition. The model was first subjected to constant vertical loading to represent loads from the upper floors, and then subjected to lateral displacements. FE models were subjected to a monotonically increasing load instead of cyclic lateral displacement at the top tie-beam. This simplified analysis scheme is capable of providing essential load-deformation behavior of confined masonry walls. A limitation of this FE modeling is that it is incapable of handling the strength and stiffness degradation associated with cyclic loading. The details about loading and boundary conditions are shown in Fig. 2.

The load-deformation results for the monotonically increasing loading are compared with the corresponding envelope values measured during the cyclic in-plane testing of confined masonry wall specimens as shown in Figs. 5a to 5d. Reasonable agreement was found between the analytical results and experimental data before any major cracking occurred; later load-deformation behavior was governed by the opening and closing of cracks. For solid wall the strength predictions from FE analysis R_{FE} were close to the experimental values, R_{exp} whereas the walls with openings showed some comparative discrepancy in strength and post-peak behavior (Fig. 5). Such discrepancies may occur due to the large concentration of stresses at the corners of wall openings which were not effectively mobilized during the simulation and, thus, resulted in higher strength degradation when compared to the experimental load-deformation curves.

The equivalent plastic strain contour plots obtained from the FE simulation are compared with the crack pattern observed during the test (Fig. 5). These contour plots reasonably agreed with observed cracking patterns, for example, solid CM wall W1 with no toothing developed a major diagonal crack extending to full-height and length of the wall. Similarly, FE analysis of the wall W4 with window openings indicated large plastic strains in central pier whereas for wall W5 the plastic strains in masonry piers appeared to be rather diffused; these observations correlate with the damage recorded during the experiments. This implies that the developed FE model was able to predict the pre-peak and post-peak behavior of confined masonry walls close to experimental results. The results of FE simulation assisted in determining the stress flow in perforated masonry walls with complex configurations of piers and confining members and eventually will help in developing the framework of strut and ties for confined masonry walls.

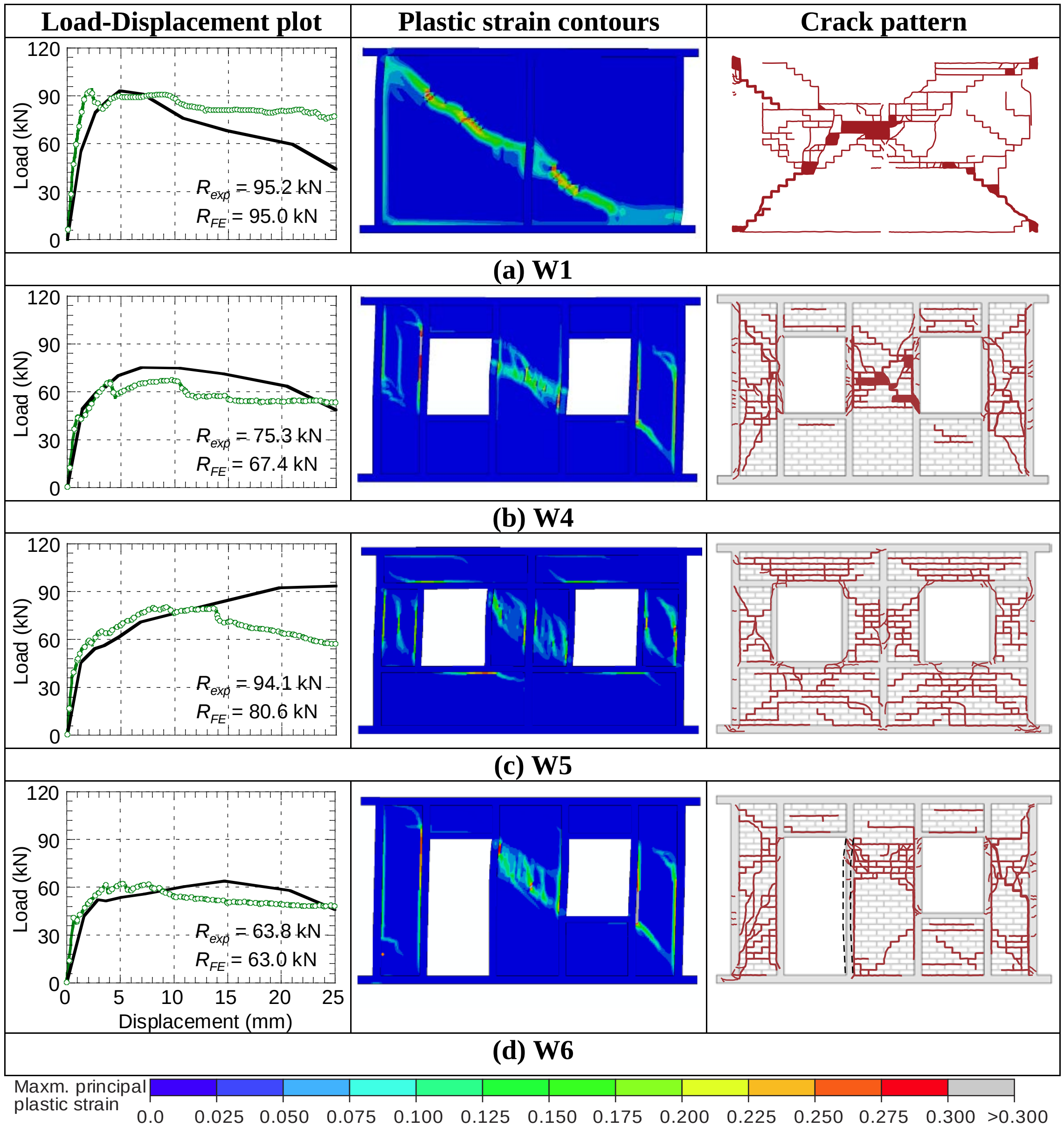


Fig. 5 – Comparison of load-displacement curve and crack patterns obtained from experiments and FE simulation for confined masonry walls: (a) W1, (b) W4, (c) W5, and (d) W6

3. Strut-and-Tie Model

Strut-and-tie model has evolved as a most useful method of analysis and design for shear critical structures and disturbed regions (often called as D-region) in concrete structures. This method provides a rational and consistent design approach by idealizing complex structural member with an appropriate simplified truss model. According to this procedure, based on the knowledge of direction of principal stresses obtained by a FE analysis of structural elements, load paths are drawn through the structure in form of a truss which is analyzed for the design loads.

The seismic design of RC structural walls with openings has been frequently under-taken with the use of strut-and-tie models [17, 18]. The wall-type structure is analogous to a very deep beam, where the disturbed region based upon St. Venant’s Principle would most likely include the entire wall. The strut-and-tie method has also been used for reinforced masonry walls [19, 20]. For masonry walls with irregular openings, New Zealand masonry code NZS 4230 recommends strut-and-tie model to establish the rational paths of internal forces and consequently the shear strength of the wall [21]. Voon and Ingham formulated the strut-and-tie model for the perforated reinforced masonry walls and showed the close prediction in the shear strength when compared to the experimental results. An example of resultant strut-and-tie analysis for perforated reinforced masonry wall is shown in Fig. 6, with struts indicated by a broader element thickness [19]. As illustrated in Fig. 6, the compressive force will be resisted appropriately by masonry struts whereas the longitudinal reinforcement will act as ties to resist the tensile forces.

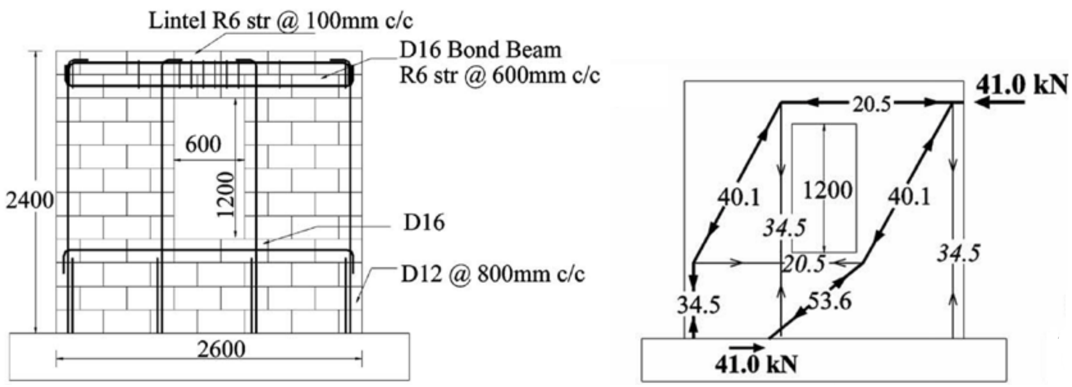


Fig. 6 – Strut-and-tie model for the reinforced masonry wall with an opening [19]

3.1 Application of Strut-and-Tie Model to Confined Masonry Walls

The Mexican code [22] suggests using the strut-and-tie method for the design of confined masonry walls with openings, however, no instructions/guidelines are provided on how to construct a strut-and-tie model for such perforated walls. Fig. 7a illustrates the confined masonry wall which resists the combined effect of moment M and axial load P similar to RC shear wall with boundary elements [23]. As shown in Fig. 7a, the tensile stresses are resisted by longitudinal reinforcement in tie-columns and compressive stresses are taken by concrete, masonry, and reinforcement in tie-columns and tie-beams. Therefore, analogous to the reinforced concrete and masonry walls, the strut-and-tie model for confined masonry wall will consist of pin-jointed structural truss which connects both tension and compression members. A possible strut-and-tie model for the confined masonry wall based on its load resisting mechanism is illustrated in Fig. 7b. The broken and solid lines in Fig. 7b represent the strut and tie, respectively.

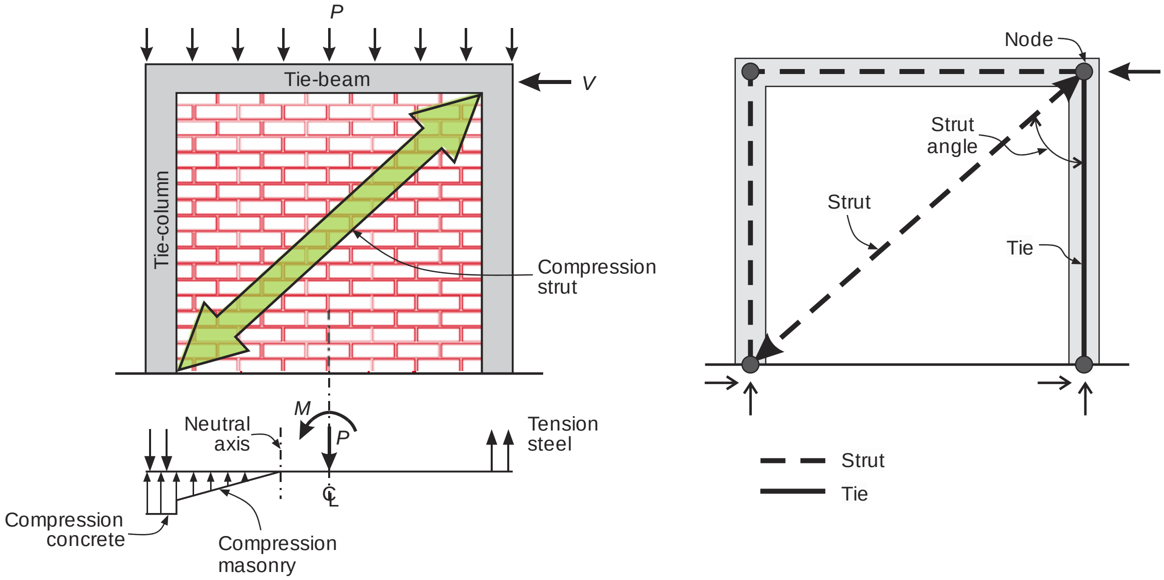
Based on the above strut-and-tie model, the basic design requirements for a confined masonry wall panel are as follows:

(i) sufficient shear resistance should be provided by masonry to carry the diagonal strut forces,

(ii) horizontal and vertical strut forces should be resisted by tie-beams and tie-columns, and

(iii) amount of longitudinal reinforcement in tie-beams and tie-columns should be determined from the tie forces.

Brzev and Gavilán have shown the application of strut-and-tie method through a design example of a four-storey wall panel [24]. However, they restricted the contribution of masonry in a wall panel to carry lateral loads by disregarding the strut action in panels with height-to-length (H_w/L_w) ratio greater than 1.5. The H_w/L_w ratio of 1.5 corresponds to a strut angle, θ_s = 34.0° between the axes of the strut and tie at any node, this limit on strut angle is more restrictive than the limit of 25° in ACI-318 [25]. This constraint on angle, θ_s imposed by Brzev and Gavilán [24] may result in significantly lower prediction in shear strength for confined masonry walls, especially with openings and slender masonry piers ($H_w/L_w \geq 1.5$). The contribution of slender masonry panels/piers can be reviewed based on the magnitude and direction of principal stresses obtained from the FE analysis.

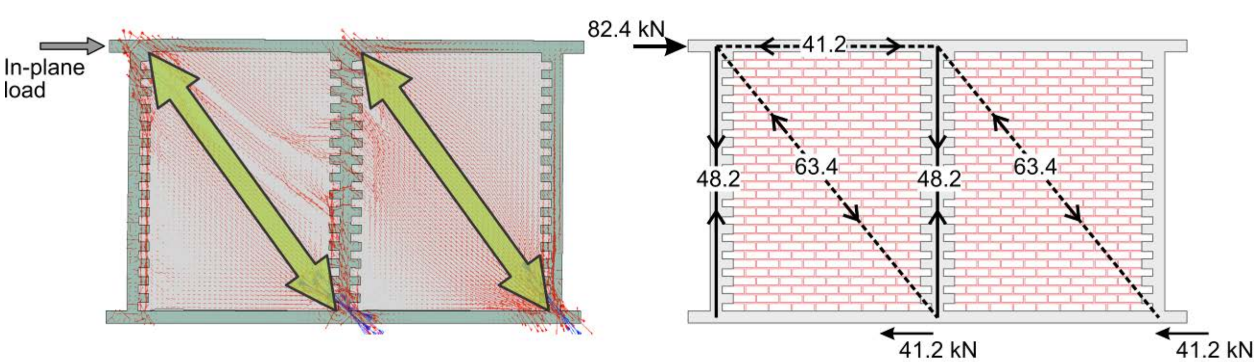


(a) (b)

Fig. 7 – (a) Internal force distribution in a confined masonry wall panel subjected to combined axial load and bending (redrawn from Meli et al. [23]), and (b) a possible strut-and-tie model for confined masonry wall

3.2 Predictions of Strut-and-Tie Models

The principal compressive stress vector plots for solid CM wall along with its strut-and-tie model is shown Fig. 8. The strut-and-tie model will be similar for all solid CM walls because it is incapable of considering the role of toothed connection between the tie-column and masonry. The solid CM wall W3 was chosen for the analysis which develops compression struts in each panel under the unidirectional lateral load as shown in Fig. 8a. The resultant strut-and-tie analysis for this wall is diagrammatically shown in Fig. 8b, where broken lines indicate the strut components. The strut-and-tie analysis was performed by first estimating the load carrying capacity of ties (48.2 kN as indicated in Fig. 8b), which was obtained from the yield strength of the longitudinal reinforcement. Subsequently, the skeleton of struts and ties was solved by maintaining the equilibrium at each node, i.e., a system of internal forces must be in equilibrium with the externally applied loads and support conditions. In order to ease the analysis procedure, the effect of pre-compression and wall self-weight was not considered. The obtained lateral load strength of 82.4 kN closely matches with the experimental values for all solid CM walls as compared in Table 3.



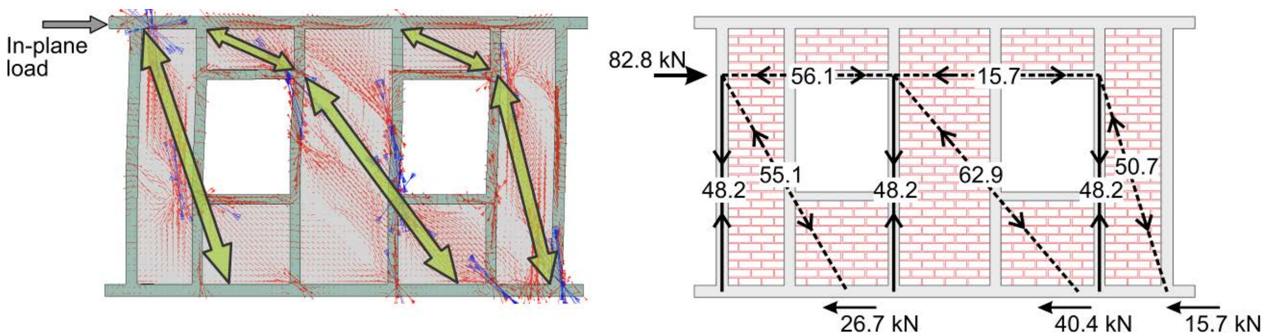
(a) (b)

Fig. 8 – Wall SCFT: (a) principal stress vector plots, and (b) strut-and-tie model

Table 3 – Comparison of lateral load carrying capacity obtained from experimental and strut-and-tie analysis

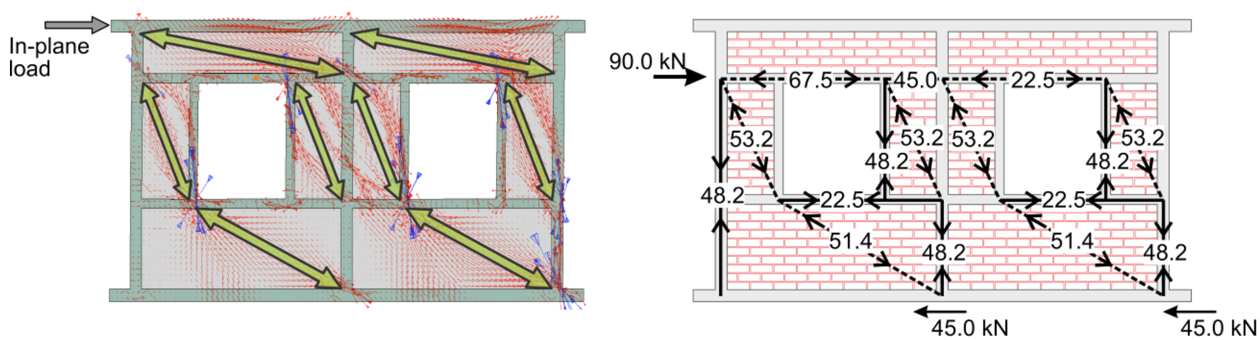
	Solid wall			**Wall with openings**			
	W1	W2	W3	W4	W5	W6	
R_{exp} (kN)	95.2	90.1	97.5	75.3	94.1	+68.0	-59.7
R_{STM} (kN)	82.4			82.8	90.0	+71.8	-65.9
R_{exp}/R_{STM}	1.16	1.09	1.18	0.91	1.04	0.95	0.91

Similar to the solid wall, the strut-and-tie models for CM walls W4, W5, and W6 with openings were developed based on the principal stress vectors as illustrated in Figs. 9 to 11. Considering an intricate flow of compressive stresses in walls with openings as observed from the principal stress vector plots in Figs. 9a and 10a, the strut-and-tie analysis was simplified by assuming the location of external force at the level of sill beam/band. This assumption was appropriate because the inclination of struts in sill masonry were almost leading to horizontal strut (angle of strut, θ_s > 65°) especially in the wall with continuous sill and lintel bands (Fig. 10a). The principal stress vector plots for the wall W4 with no continuous horizontal bands illustrate that the compressive stresses developed almost over the full-height of masonry panels (except in central panel), which conform to the damage pattern observed during the experiments. This observation is in contrast with the standard procedures where the effective height of masonry pier/panel in perforated walls is defined based upon the vertical dimension of smallest adjacent openings [26]. The current standard definition for masonry piers may result in non-conservative estimates of lateral resistance as taller masonry panels have lesser capacity than shorter masonry panels and, thus, need to be revised accordingly. As shown in Table 3, the strut-and-tie analyses closely predict the in-plane resistance for walls W4 and W5 with two window openings.



(a) (b)

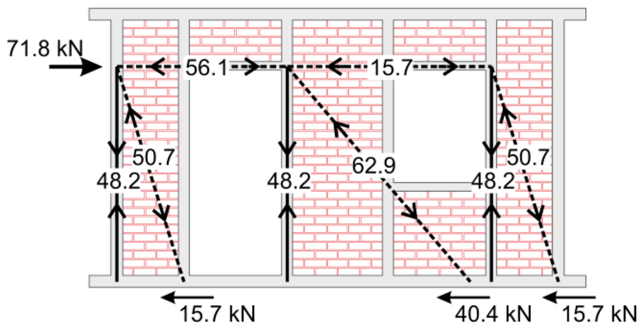
Fig. 9 – Wall W4: (a) principal stress vector plots, and (b) strut-and-tie model



(a) (b)

Fig. 10 – Wall W5: (a) principal stress vector plots, and (b) strut-and-tie model

The confined masonry wall W6 with asymmetric openings, i.e., one window and one door opening exhibited different arrangements of struts and ties with the change in loading directions. The resultant strut-and-tie analysis for wall W6 in push (positive) and pull (negative) direction are shown in Figs. 11a and 11b, respectively. A slightly higher lateral resistance was obtained when the load was applied in push direction as compared to pull direction. This observation is consistent with the asymmetric hysteresis behavior of the wall specimen W6 during the in-plane cyclic load test. Moreover, the predicted lateral load capacity in both directions compare well with the experimental values (Table 3). These results indicate that the strut-and-tie analyses can provide good predictions on in-plane capacity of CM walls with and without openings.



(a) (b)

Fig. 11 – Strut-and-tie model for wall W6: (a) push direction, and (b) pull direction

4. Conclusions

The inconsistent predictions of in-plane shear strength by existing equations necessitate the development of a rational design method for confined masonry walls, especially with openings. To develop rational and consistent design approach, the nonlinear finite element simulations were conducted on CM walls. These FE models closely resemble the load-deformation behavior and failure pattern obtained from the laboratory test on CM walls with and without openings. Based on the knowledge of the magnitude and directions of principal stresses, obtained by a nonlinear finite element analyses the strut-and-tie models were developed for CM walls. These strut-and-tie models provided an insight on how to construct the network of struts and ties for complex cases particularly in CM walls with openings surrounded with confining elements. With few simplifying assumptions, strut-and-tie models consistently provided good predictions (within an error of 15%) for the in-plane shear strength of CM walls with and without openings, and, thus, can be confidently used as an analysis and design tool for confined masonry structures. The analysis using strut-and-tie model was found to be simple, yet powerful approach for CM walls and can be easily implemented in computer-aided analysis and design tools.

5. Acknowledgements

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