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SEISMIC BEHAVIOR OF CONFINED MASONRY WALLS WHEN SUBJECTED TO IN-PLANE AND OUT-OF-PLANE LOADING

Vaibhav Singhal[1] and Durgesh C Rai[2]

ABSTRACT

An experimental study was performed on half-scaled clay brick masonry wall panels to investigate the effect of type of interface present at the wall edge and the column face, such as toothing and the presence of openings on the in-plane and out-of-plane response of confined masonry walls. The wall specimens were subjected to a pre-defined sequence of slow cyclic in-plane drifts and shake table-generated out-of-plane ground motions. Four specimens were tested to assess the role of toothing at wall-to-tie-column interface on the bidirectional behavior of masonry walls. Another four wall specimens with openings for door and windows were tested till failure to evaluate the efficacy of three different confinement schemes used to enclose the openings. The toothing connections enhance the interaction between masonry walls and RC confining elements and were able to delay the failure by controlling the out-of-plane deflection. The appropriate confinement around the openings assisted in the uniform distribution of cracks which led to significantly enhanced strength and deformability and was able to compensate for deficiencies due to presence of openings. Under out-of-plane loads, confined masonry (CM) walls acted as a shear wall and due to the composite action between wall and the tie-column it could safely sustain large in-plane damage upto 1.75%. The strength parameters obtained from these tests were compared with those predicted using the existing models for solid and perforated masonry walls.

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An experimental study was performed on half-scaled clay brick masonry wall panels to investigate the effect of type of interface present at the wall edge and the column face, such as toothing and the presence of openings on the in-plane and out-of-plane response of confined masonry walls. The wall specimens were subjected to a pre-defined sequence of slow cyclic in-plane drifts and shake table-generated out-of-plane ground motions. Four specimens were tested to assess the role of toothing at wall-to-tie-column interface on the bidirectional behavior of masonry walls. Another four wall specimens with openings for door and windows were tested till failure to evaluate the efficacy of three different confinement schemes used to enclose the openings. The toothing connections enhance the interaction between masonry walls and RC confining elements and were able to delay the failure by controlling the out-of-plane deflection. The appropriate confinement around the openings assisted in the uniform distribution of cracks which led to significantly enhanced strength and deformability and was able to compensate for deficiencies due to presence of opening. Under out-of-plane loads, confined masonry (CM) walls acted as a shear wall and due to the composite action between wall and the tie-column it could safely sustain large in-plane damage upto 1.75%. The strength parameters obtained from these tests were compared with those predicted using the existing models for solid and perforated masonry walls.

Introduction

Confined masonry (CM) construction has evolved based on its satisfactory performance in past earthquakes [1]. It was observed that CM panels provide fair amount of in-plane shear capacity and ductility under seismic loads and its behavior can be significantly affected by wall-to-tie-column interface, detailing of confining members and presence of openings. Recent experimental studies have observed that the wall-frame connection details play a crucial role in the in-plane and out-of-plane behavior of masonry panels [2]. Moreover, according to the reconnaissance reports of several past earthquake and research evidences, openings have negative influence upon seismic resistance of CM walls [3]. Experimental studies on masonry infilled panels illustrated that an opening of size 20-30% of the panel area may reduce the in-plane stiffness and strength by about 70-80% and 50-60%, respectively [4]. Under out-of-plane loads the presence of opening may cause gross relocation of the yield lines, prevent developing of arching action and induces local effects at corners of opening, thus reduces the overall out-of-plane capacity of

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panel [5]. Therefore, to compensate for deficiencies due to presence of openings appropriate confinement should be provided on all sides of opening which facilitate the development of compression strut in masonry panel for the lateral load transfer. Several national standards and technical manuals provide guidelines for the suitable confinement around the door and window openings [6-9], however, the efficacy of these confining schemes is still not well known.

In addition, during an earthquake, the masonry panels are subjected to in-plane and out-of-plane loads simultaneously. The out-of-plane (OOP) load-carrying capacity of these masonry panels may be substantially weakened after being damaged, endangering their overall safety and stability. The present study is an extension of research on confined masonry walls and will evaluate the effect of wall-to-tie-column connection and presence of opening on the bi-directional behavior of CM walls. The accuracy of various existing models to predict the in-plane strength of CM walls with and without opening has also been evaluated.

Experimental Program

Specimen Details

The test matrix involved eight half-scaled wall specimens, having dimensions of 2.5 m long by 1.5 m high and 60 mm thick as shown in Figs. 1 and 2. These figures also summarize key observation and hysteresis behavior of all wall specimens. Two specimens were regular masonry infilled RC frame in which the masonry wall was built after the RC frame. In other six specimens, the confining (frame) elements were constructed after the masonry wall. Four solid wall specimens were prepared to study the influence of type of the interface present at the wall edge and column, such as toothing (Fig. 1). Another four specimens had openings for door and windows to evaluate the efficiency of various confining schemes for openings (Fig. 2). The walls are designated by alphanumeric symbol as SI, SC_{CT}, SC_{FT}, SC_{NT}, SI-O_{2WA}, SC-O_{2WB}, SC-O_{2WC} and SC-O_{DWB}, where alphabet I and C denote infilled and confined masonry panel, respectively and O signify the wall with openings. The subscript CT, FT and NT represents coarser, finer and no toothing present in solid walls, respectively. The subscript W and D represents type of opening, i.e., window and door opening, respectively and numeric symbol corresponds to number of window openings. The subscript A, B and C signify the type of confinement scheme used to enclose the opening as follows:

A. Only lintel beam provided over the opening.
B. Opening confined on all sides with tie-columns extending from bottom to top tie-beam and with no continuous horizontal band.
C. Opening confined on all sides with continuous horizontal band (sill and lintel bands).

Specially made half-scaled burnt clay bricks (120.4 mm×61.8 mm×38.5 mm) and a lime-cement mortar mix of 1:1:6 proportion (cement: lime: sand) was used for the masonry panels. Micro-concrete was used in RC members of all specimens. The average reference properties of material; masonry compressive strength f_m, modulus of masonry E_m and compressive strength of concrete f_{ck} are listed in Table 1. The steel wires of 6 mm and 3 mm diameter were used as longitudinal and transverse reinforcement in RC members, respectively. The yield strength of 6 mm and 3 mm diameter bars were found to be 426 MPa and 623 MPa, respectively.

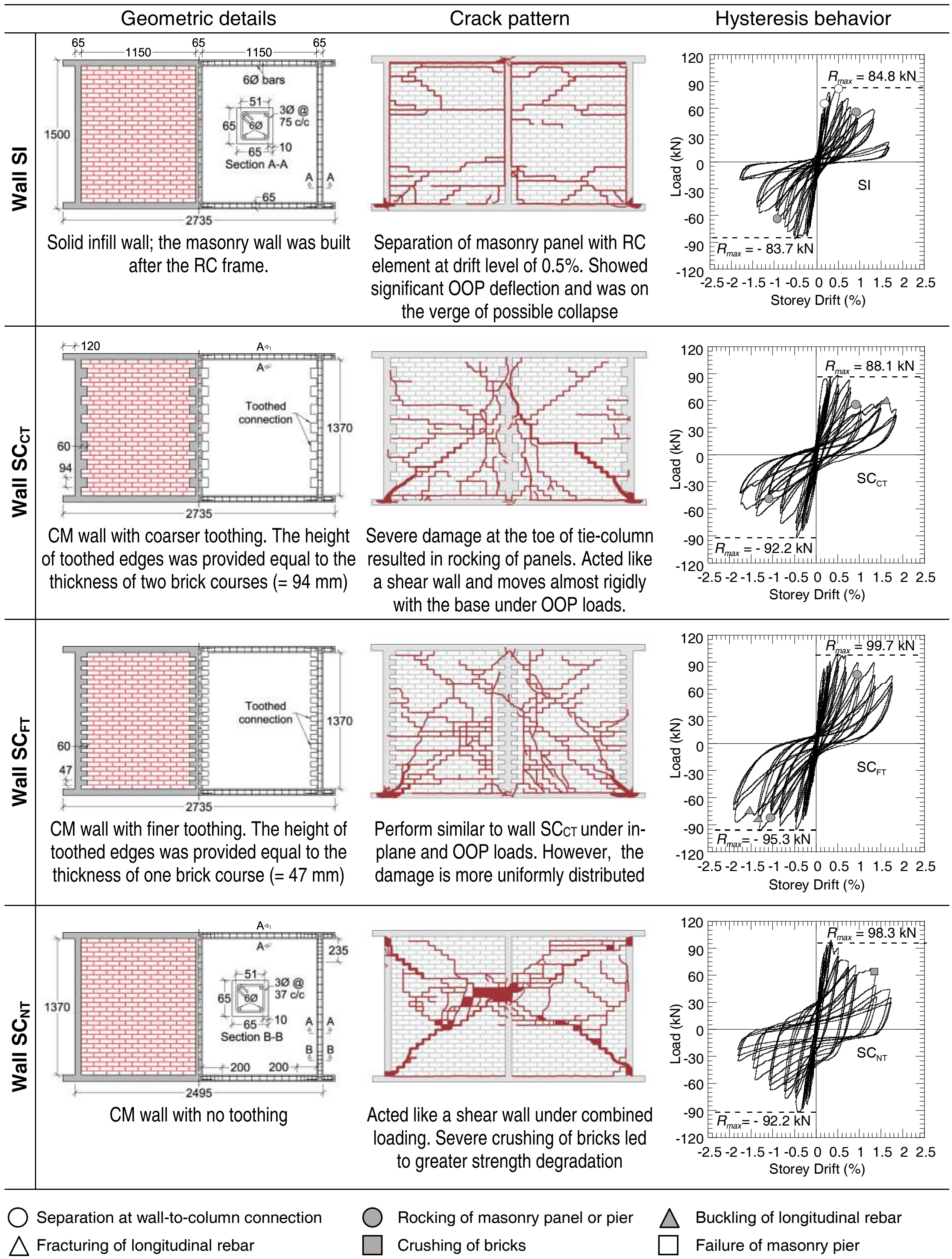


Figure 1. Details of solid walls along with summary of failure pattern and hysteresis behavior

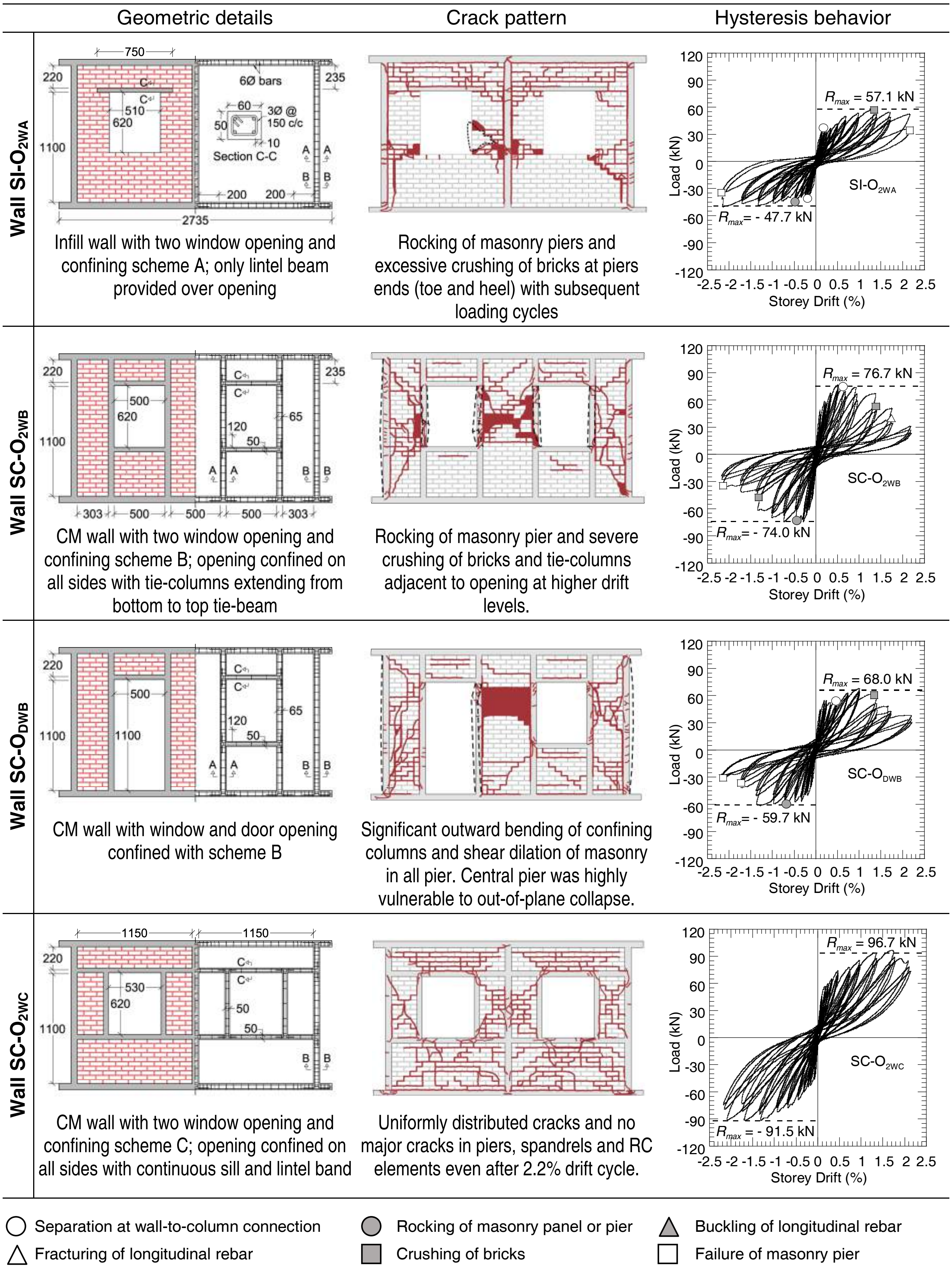


Figure 2. Details of perforated walls along with summary of failure pattern and hysteresis behavior

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

Wall	f_m	E_m	f_{ck}	Ultimate load, R_{max} (kN)	Displacement at peak load (mm)	Cumul. energy dissipated (kN m)	Strength degradation
SI	10.3	2960	30.1	84.2	7.2	8.1	0.24
SC_{CT}	8.2	3165	43.6	90.1	7.1	11.8	0.58
SC_{FT}	9.4	3353	40.7	97.5	7.2	12.8	0.80
SC_{NT}	8.8	3845	33.1	95.2	5.8	18.2	0.41
SI-O_{2WA}	7.5	2628	30.6	52.4	20.2	7.9	0.88
SC-O_{2WB}	7.8	2854	30.3	75.3	6.9	11.5	0.44
SC-O_{DWB}	8.5	3927	34.1	63.8	14.7	10.2	0.53
SC-O_{2WC}	7.9	3026	30.3	94.1	25.4	12.0	0.93

Test Procedure and Loading History

The testing method involved successive applications of out-of-plane and in-plane loading, so that there was no need to move the specimen for the repeated cycles of loading in the in-plane and out-of-plane directions [10]. The test setup for the out-of-plane and in-plane loading are shown in Fig. 3. The desired boundary conditions of diaphragm flexibility were achieved by providing a sufficient number of lateral supports. A vertical force of 0.10 MPa was applied over the wall specimen to simulate the gravity loads. For simulation of out-of-plane forces the artificial mass in form of lead blocks were attached to the walls in a regular grid pattern on both faces as shown in Fig. 3.

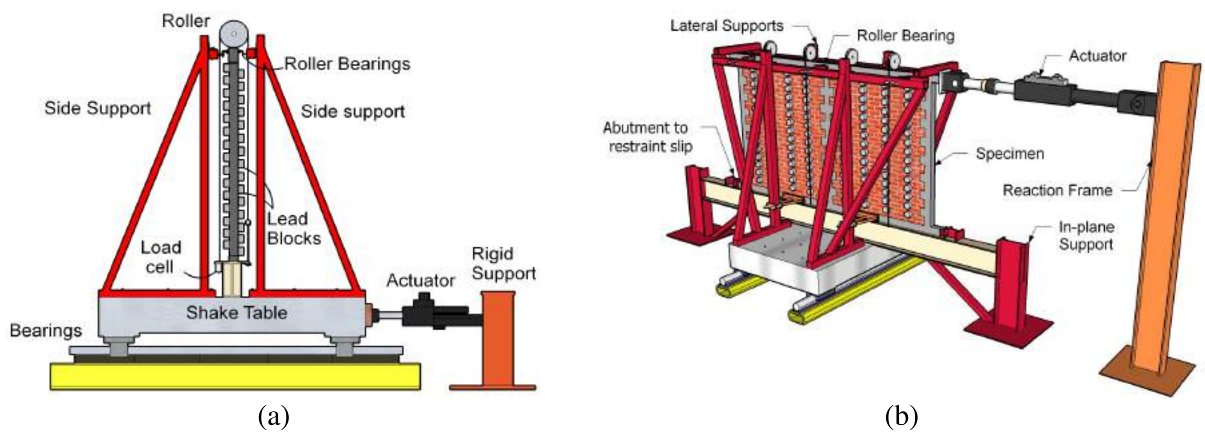
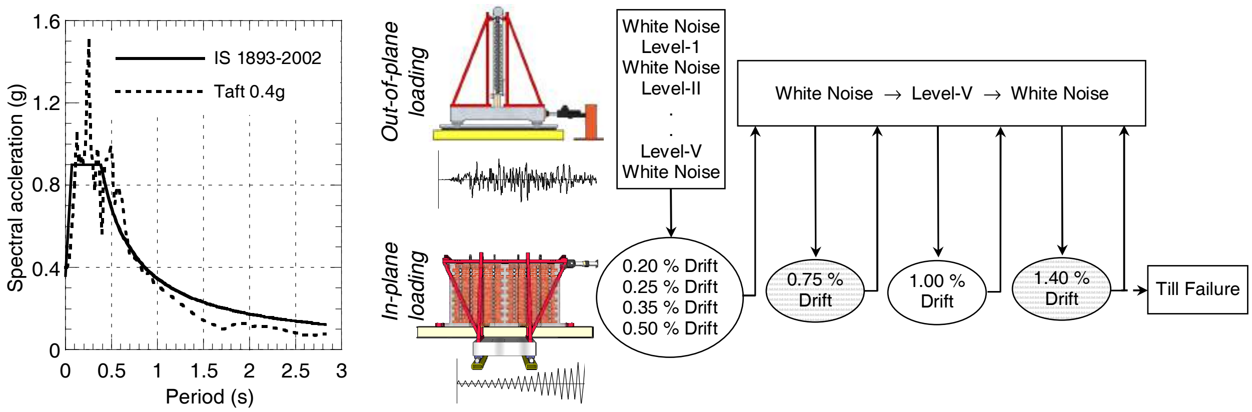


Figure 3: Schematic showing various components of the test setup for (a) out-of-plane and (b) in-plane loading

Taft earthquake was chosen for the out-of-plane target ground motion. The response spectra of this motion scaled to 0.4g corresponded well with the response spectrum specified in the Indian seismic code IS 1893: 2002 for a design earthquake of PGA = 0.36g as shown in Fig. 4a [11]. This Taft motion is referred as Level V and was scaled to 0.05g, 0.11g, 0.18g and 0.27g which are denoted as Level I, II, III and IV motions, respectively. The in-plane loading consists of displacement controlled slow cycle of gradually increased storey drifts from 0.10% to 2.20% as per ACI 374.1-05 [12]. The test procedure and loading sequence are summarized in

Fig. 4b. The load test started with the out-of-plane shake table motions consisting of a series of incremental Taft motions from Level I to Level V. After the completion of this out-of-plane loading schedule, the specimen was subjected to quasi-static in-plane cyclic loading and continued until cracks were visible (till 0.50% drift cycle). Subsequently, the second cycle of out-of-plane loading was applied which consisted of Level V Taft motion and an alternate process of in-plane and out-of-plane loading was continued until the specimen failed.



(a) (b)

Figure 4. (a) Comparison of scaled response spectra of DBE (Design basis earthquake) and original Taft motion upscaled to 0.4g (b) Summary of test procedure and loading sequence

Experimental Results and Discussion

The failure patterns of all specimens at the end of the test along with key observations and hysteresis behavior are illustrated in Figs. 1 and 2. The infill masonry walls showed separation of masonry panel with tie-beams and tie-columns even at in-plane drift level of 0.5%. However, CM walls did not experience any such separation till the drift cycle of 1.75%. The confined masonry construction strengthened the wall to tie-column interaction and helped reduce the likelihood of out-of-plane instability. These confined masonry walls behaved more like a shear wall with boundary elements. No appreciable difference in overall behavior was noted in walls SC_{CT} and SC_{FT}, suggesting that both types of toothing were nearly equally effective. However, specimen SC_{FT} with higher density of toothing, demonstrated a more uniform distribution of cracks. The walls SC-O_{2WB} and SC-O_{DWB} with no continuous horizontal band suffered severe damage to masonry pier at higher drift levels, which implies inadequate confinement to piers in these walls. However, the wall with continuous horizontal band (SC-O_{2WC}) showed uniform distribution of cracks over the entire wall panel and no major cracks were observed in piers, spandrels and confining elements even after 2.2% in-plane drift cycle.

The observed in-plane response in terms of ultimate load, hysteretic energy dissipated and strength degradation (ratio of residual strength and peak load) are listed in Table 1. The envelope backbone curves for all wall specimens are compared in Fig. 5. The wall SC_{FT} which possessed a high density of toothing and wall SC-O_{2WC} with continuous sill and lintel band maintain the in-plane resistance with less than 20% strength degradation even after the 1.75% drift cycle. As observed from Table 1 and Fig. 5, both CM walls SC_{FT} and SC-O_{2WC} performed better than other schemes due to higher in-plane capacity and reduced rate of strength and stiffness degradation in the post-peak range. Confinement configurations in wall SC-O_{2WB} and SC-O_{2WC} considerably improves the strength and energy dissipation potential (> 40%) as compared to specimen with only lintel beam (SI-O_{2WA}). Both confinement configurations B and C were able to recapture the strength deficiency due to the presence of openings.

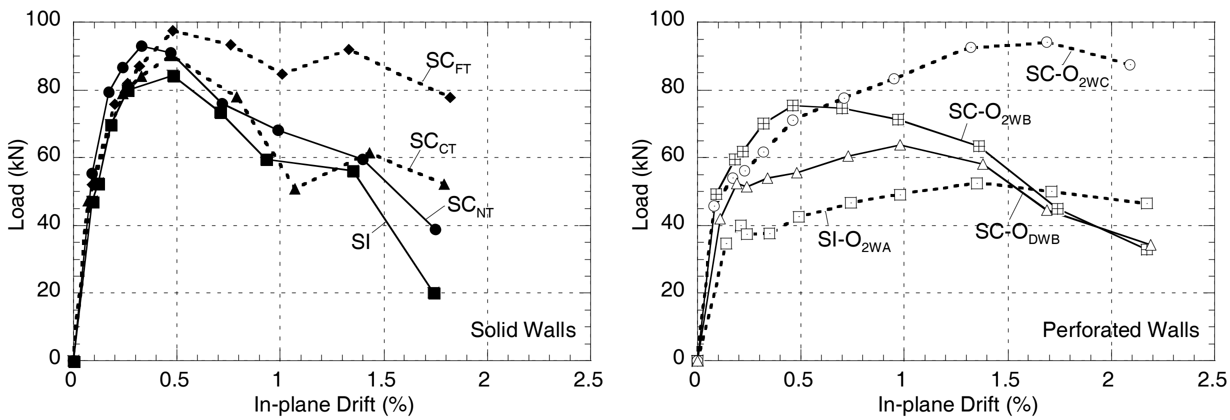


Figure 5. Comparison of observed envelope value of load versus story drift

The variation of equivalent out-of-plane (OOP) uniform pressure (calculated from observed inertia forces) with in-plane drift (damage) is shown in Fig. 6. The equivalent uniform pressure was calculated by multiplying average of peak acceleration at each location with the total mass of wall and then divided by the wall area. All specimens experienced relatively small variations in uniform pressure during the out-of-plane motion except for wall SI-O_{2WA}. The increase in uniform pressure in specimen SI-O_{2WA} after few in-plane drift cycles was may be due to higher local acceleration resulted from rocking of damaged masonry fragments.

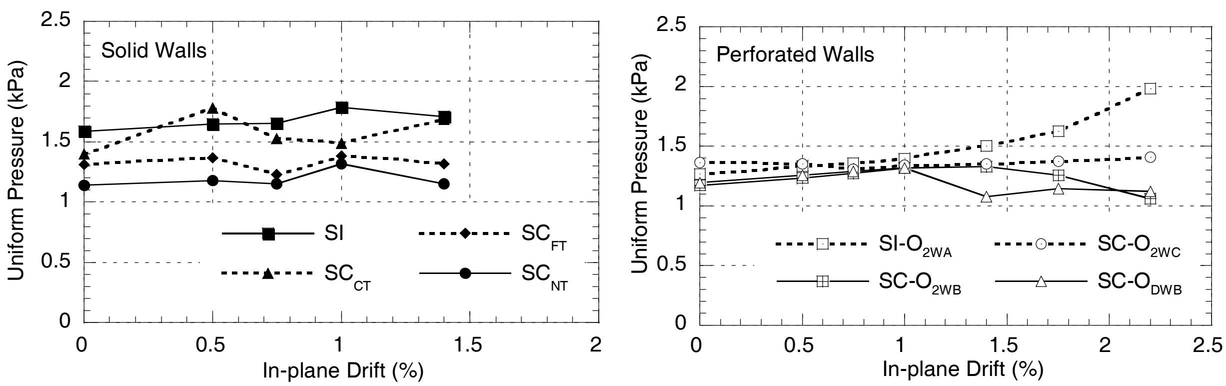


Figure 6. Variation of peak uniform acceleration with in-plane drift/damage

The peak out-of-plane deformations of masonry panels in solid walls and masonry piers A, B and C in perforated walls after each in-plane drift (damage) level are compared in Fig. 7. For walls SI-O_{2WA} and SC-O_{2WC}, the mean of peak out-of-plane displacement recorded in piers B_1 and B_2 were compared with pier B of other walls as shown in Figs. 7b-7d. The wall SI with infilled masonry showed continuous increase in out-of-plane deflection with in-plane damage and was likely to collapse after 1.75% drift cycle. Conversely, the maximum out-of-plane displacement in solid CM walls remains fairly invariable with the in-plane damage (Fig. 7a). Due to the severe damage of masonry piers in walls SI-O_{2WA}, SC-O_{2WB} and SC-O_{DWB} significant out-of-plane displacement was observed in these specimens when compared to the wall SC-O_{2WC} (Figs. 7b-7d). As depicted from Fig. 7d, the central pier B in wall SC-O_{DWB} experienced large out-of-plane deformation and was on verge of the collapse. In contrast, the peak out-of-plane displacement in the wall SC-O_{2WC} with continuous lintel and sill band remains fairly invariable with the in-plane damage (Figs. 7b-7d). This indicates that the observed out-of-plane instability was primarily due to excessive deflections and not governed by accelerations (inertia forces).

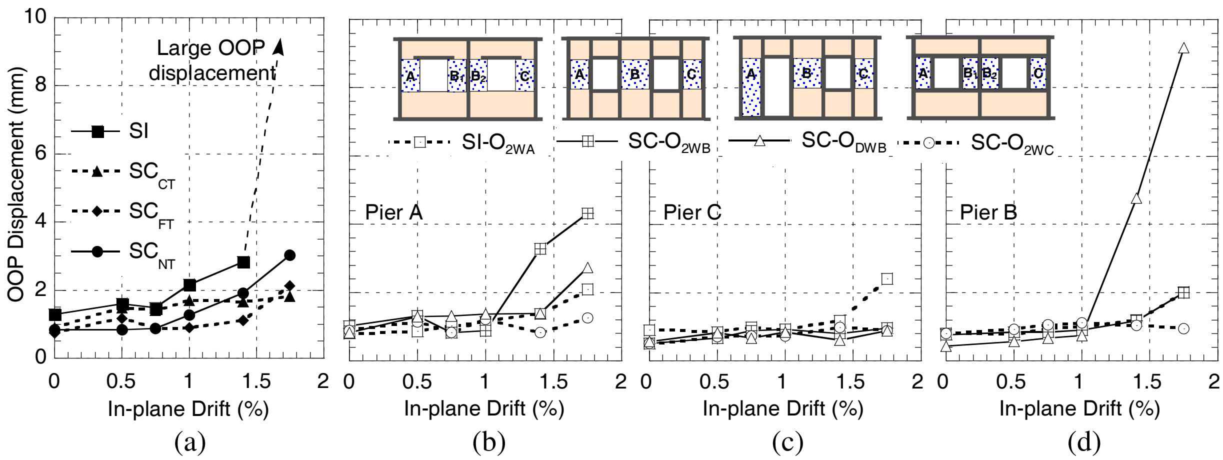


Figure 7. Variation in out-of-plane displacement with in-plane drift/damage (a) solid masonry walls (b) – (d) walls with opening in pier A, C and B

Analytical Prediction of the In-plane Strength of Confined Masonry Walls

Two different theories has been developed to physically model the shear failure mechanism of masonry. In the first hypothesis (HT1), the lateral resistance, $R_{w,s}$ of confined masonry is derived on the basis of the friction theory as given by Eq. 1.

$$R_{w,s} = \left(X v_{mo} + Y \sigma_o \right) A_w \tag{1}$$

where, v_{mo} is the shear strength under zero compression stress, σ_o is the design compressive stress and A_w is the cross-sectional area of the wall. The X and Y are constants defining the contribution of v_{mo} and σ_o, respectively. The value of constant X and Y in Eq. 1 has been proposed by many researchers based on the regression analysis of experimental results. For CM walls, the values of constant X and Y typically varies from 0.21 – 0.60 and 0.29 – 0.37, respectively. A significantly larger range of these constant can be attributed to the great disparity in materials used and construction techniques in the past experimental studies. To provide a practical and straightforward method for the design of CM walls, many international standards and earlier research studies neglect the contribution of the tie-column reinforcement.

The second approach to evaluate the shear resistance of masonry is based on the assumption of elementary theory of elasticity. According to this hypothesis (HT2), the diagonal shear failure of wall are caused by the principal tensile stresses which develop in the plane of the wall. Thus, by considering masonry as an elastic, homogenous and isotropic structural element, the lateral resistance $R_{w,s}$ of plain masonry is given by following equation [13]:

$$R_{w,s} = A_w \frac{f_t}{\beta} \sqrt{\frac{\sigma_o}{f_t} + 1} \quad \text{with} \quad \beta = \min\left(\frac{H_w}{L_w}, 1.5\right) \geq 1.0 \tag{2}$$

where, f_t is the tensile strength of the masonry and β is the shear stress distribution factor which primarily depends on the geometry of the wall. In case of confined masonry walls, the part of shear capacity of the wall can also be attributed to the tie-column reinforcement. Therefore, the maximum lateral resistance R_{max} of the confined wall can be obtained by adding the shear resistance provided by the brick panel $R_{w,s}$ and vertical rebars $R_{rv,d}$. Besides the above two hypothesis the model proposed in Chinese code (2001) define a new variable v_{em} which signify the shear strength along the stair-stepped damage of masonry, this strength criteria is categorized as third hypothesis (HT3) for future reference [14]. The reference to various existing models are categorized in Table 2, each model is designated by alphabets (XX) which signify the initials of authors involved in developing a particular model.

Table 2. Existing models for maximum shear strength of confined masonry walls

Friction theory (HT1)		**Theory of elasticity (HT2)**	**Other methods (HT3)**
Neglect reinforcement	**Consider reinforcement**		
Inpres Cirsoc, 1983 (IC)[15]	Flores and Alcocer, 1996 (FA)[18]	Tomaževič and Klemenc 1997 (TK)[13]	Chinese code, 2001 (CH)[14]
San Bartolome et al., 2004 (SB)[16]	Marinilli and Castilla, 2006 (MC)[19]	Bourzam et al., 2008 (BO)[21]	
Marques and Lourenço, 2013 (ML)[17]	Riahi et al., 2009 (RI)[20]		

The lateral load capacity of solid CM walls obtained from the tests are compared with values predicted using the existing models. The ratio R_{exp}/R_{cal} obtained from all 9 existing models are plotted in Fig. 8, for a reasonable prediction of strength this ratio should be nearly equal to 1.0. Fig. 8 illustrates that the existing models which are based on the first hypothesis and neglects the contribution of the reinforcement highly under-estimated the shear capacity of CM wall, also indicated by the significantly higher values of ratio R_{exp}/R_{cal} (> 1.75). However, by considering the contribution of vertical rebars the models derived from the first method provide relatively better estimate of in-plane capacity of CM walls. The existing models based on second hypothesis slightly over-estimate the observed values and provide the prediction within a error of approximately 25%. As shown in Fig. 8, the most accurate prediction of shear capacity (with a maximum error of 7%) was made using the strength criteria proposed in the Chinese code [14].

Strength Reduction Factor for CM Walls with Openings

The effects of openings on the strength of masonry panels were usually taken into consideration in many previous research studies through reduction factors D_F. The reduction factor are generally defined as the ratio of strength of a perforated masonry to that of an identical solid masonry. Due to the lack of research on confined masonry walls with openings these reduction factors are primarily proposed only for masonry infilled frames. For CM walls, Riahi et al.

(2009) was first to propose a reduction factor for the cracking shear strength [20]. Moreover, the proposed relation for D_F are based on the regression analysis of limited number of test results and thus may be applicable to specific type of wall panels. In lieu of scarcity of detailed study on CM walls with openings the suitability and accuracy of various reduction factors primarily developed for masonry infilled frame was investigated. Summary of various reduction factors available in the literature was given by Mohammadi and Nikfar [22]. The existing equations of reduction factor for strength D_F of perforated masonry walls are summarized in Table 3.

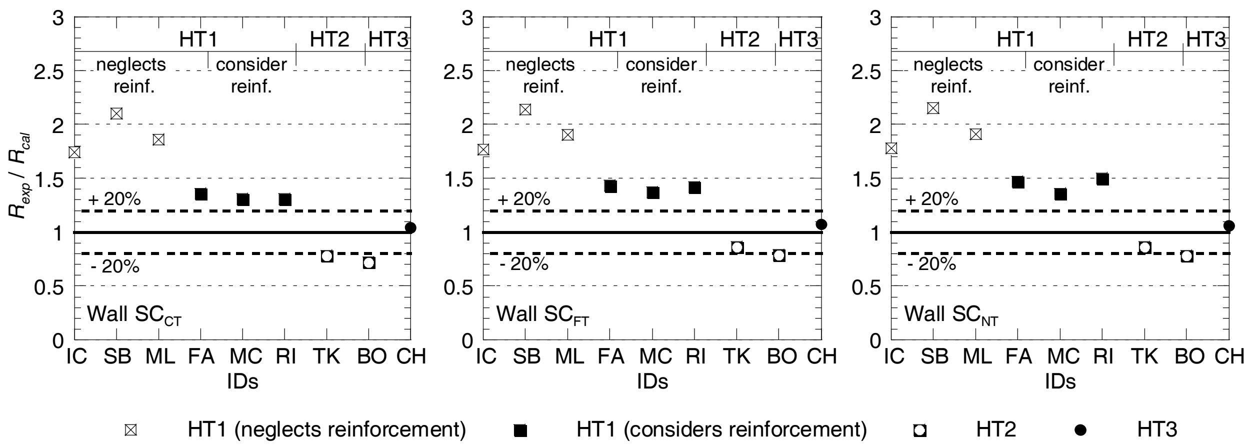


Figure 8. Comparing accuracy of existing model to predict the shear resistance of solid CM walls (HT1, HT2 and HT3 indicate the first, second and third hypothesis, respectively)

Table 3. Strength reduction factors to account for the presence of opening

IDs	Equations	Remarks	Reference
$D_{F\text{-AL}}$	$0.6(A_r)^2 - 1.6(A_r) + 1$	A_r = ratio of area of the opening and wall panel = A_o/A_p	Al-Chaar et al., 2003 [23]
$D_{F\text{-RI}}$	$-2.2(A_r) + 1$	—	Riahi et al., 2009 [20]
$D_{F\text{-TM}}$	$1.49(A_r)^2 - 2.238(A_r) + 1$	For $A_o/A_p > 0.4$, $DR_{F\text{-TM}} = 0$	Tasnimi and Mohebkhah, 2011 [24]
$D_{F\text{-}MF}$	$-1.085(A_r) + 1$	Valid for $A_o/A_p \leq 0.4$	Mohammadi and Nikfar, 2013 [22]

The accuracy and reliability of the existing models of strength reduction factors was judged by the first comparing the predicted values with the ratio of strength of a perforated infill wall SI-O$_{2\text{WA}}$ to that of the similar solid wall SI. Among the above four proposed D_F, the reduction factors $D_{F\text{-AL}}$, $D_{F\text{-RI}}$ and $D_{F\text{-TM}}$ estimate the reduction due to the presence of opening within an error of 20% and will be further used for predicting the strength of perforated CM walls. For in-plane strength estimation the equation proposed in Chinese code was chosen which provided the best prediction of strength as shown in Fig. 8. Table 4 provides the comparison of measured (R_{exp}) and calculated strength (R_{cal}) for CM walls SC-O$_{2\text{WB}}$, SC-O$_{\text{DWB}}$ and SC-O$_{2\text{WC}}$ with openings. As presented in Table 4, the reduction factor D_F proposed by Al Chaar et al. (2003) gives the best prediction of strength for wall SC-O$_{2\text{WB}}$ and SC-O$_{\text{DWB}}$. In wall specimens SC-O$_{2\text{WB}}$ and SC-O$_{\text{DWB}}$ the contribution of all confining columns (two exterior columns + four interior columns) were included while estimating the in-plane strength using the equations listed in Table 3. However, for wall SC-O$_{2\text{WC}}$ larger deviation was observed in the predicted strength because the contribution of only one interior tie-column was included due to the uncertainty about the role of other vertical members in shear resistance which were extending only to the opening height.

Table 4. Comparison of ratio R_{exp}/R_{cal} for perforated CM walls

Walls / D_F	**SC-O$_{2WB}$** (R_{exp} = 75.3 kN)			**SC-O$_{DWB}$** (R_{exp} = 63.8 kN)			**SC-O$_{2WC}$** (R_{exp} = 94.1 kN)		
	AL	**RI**	**TM**	**AL**	**RI**	**TM**	**AL**	**RI**	**TM**
R_{cal}	66.1	53.8	57.9	57.5	38.7	47.6	61.9	49.0	53.8
R_{exp}/R_{cal}	1.14	1.40	1.30	1.11	1.65	1.34	1.52	1.92	1.75

Conclusions

Eight half-scale clay brick wall panels were tested using a unique loading protocol to study the effect of toothing at wall-to-tie-column interface and presence of openings on the in-plane and out-of-plane response of confined masonry walls. The test results show that the toothing is essential and it helped delay the failure by controlling the out-of-plane deflections. The wall with toothing at each alternate brick course performed superior when compared to other wall panels. The CM walls maintained structural integrity even when severely damaged and performed much better than masonry infill RC frames. Due to the composite action developed between wall and the tie-column, the CM walls acted as a shear wall with tie-column as boundary elements. The lateral strength and energy dissipation capacity of masonry walls with window openings confined on all four sides (walls SC-O$_{2WB}$ and SC-O$_{2WC}$) was more than 40% higher as compared to regular infill wall SI-O$_{2WA}$ with openings. More prevalent confining scheme of tie-columns with no continuous horizontal bands failed to provide ample confinement to masonry piers, which resulted in severe damage to wall piers. Confining openings on all sides with vertical elements, and horizontal sill and lintel bands clearly improved the bidirectional response of masonry walls and compensated for strength deficiencies caused by openings. Various existing models for estimating the shear capacity of CM walls are primarily empirical equations which are based on the specific experimental studies. The existing models which consider the contribution of reinforcement reasonably predicts the in-plane strength, however, their reliability should be verified for CM walls with different geometric details and material properties.

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