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# INFLUENCE OF OPENINGS ON QUASI-STATIC CYCLIC BEHAVIOUR OF PARTIALLY GROUTED MASONRY WALLS

**Fortes, Ernesto S.[1]; Parsekian, Guilherme A.[2]; Fonseca, Fernando S.[3] and Shrive, Nigel G.[4]**

## ABSTRACT
Openings in a wall alter its behaviour and add complexity to its analysis and design. The stiffness and load capacity of the wall can change significantly when the elements around an opening are taken into account. There has been some research conducted to determine the effects of openings on masonry shear walls, but most tests have been conducted on single-storey walls. The preliminary results from the testing of six half-scale, three-storey, concrete masonry walls with doors and windows are presented. The objective was to investigate the influence of openings in partially grouted and reinforced concrete masonry walls under cyclic loading. Tests were conducted on six walls—three with door openings and three with window openings. The walls experienced gradual strength degradation and failed in shear. The observed behaviour was attributed to the presence of a bond beam on the top of the walls, which apparently caused a frame-type action in the later stages of their testing. The typical bed joint and diagonal cracks indicated that the wall behaviour was controlled by shear.

**KEYWORDS:** *bracing walls, cyclic loading, masonry structural frames, concrete frames, coupling masonry beams, coupled walls*

## INTRODUCTION
Research has been conducted on masonry shear walls using single-storey specimens both with [1]-[2] and without openings [3] and also on multi-storey masonry walls with [5] and without openings [6]-[4]. The piers and coupling beams around an opening can affect the behaviour and capacity of a masonry shear wall with openings. To investigate the effect of the coupling beams

[1] PhD, Department of Civil Engineering, Federal University of Sao Carlos, Rodovia Washington Luis, km 235-SP-310 São Carlos - São Paulo – Brazil, ernestofortes@hotmail.com
[2] Associate Professor, Department of Civil Engineering, Federal University of Sao Carlos, Rodovia Washington Luis, km 235-SP-310 São Carlos - São Paulo – Brazil, parsekian@ufscar.br
[3] Associate Professor, Department of Civil and Environmental Engineering, Brigham Young University, Provo, UT, 84602, USA, fonseca@byu.edu
[4] Professor, Department of Civil Engineering, Schulich School of Engineering, University of Calgary, Calgary, Alberta, Canada T2N, ngshrive@ucalgary.ca

on the behaviour and capacity of multi-storey partially reinforced concrete masonry walls, six half-scale, three-storey concrete masonry walls with doors or windows were built and tested.

## EXPERIMENTAL PROGRAM

The experimental program focused on studying the behaviour of half-scale partially grouted and reinforced concrete masonry walls with openings under lateral in-plane loading. The mechanism of rupture, the maximum lateral load, and the degradation in post-cracking stiffness taking into account the effects of openings, cracking, shear distortions, and axial forces were assessed.

### *Wall Specimens*

Six specimens, three with door openings coupled by masonry beams and three with window openings also coupled with masonry beams, were constructed and tested. The configurations were selected to represent actual walls with openings. The overall dimensions of the specimens were 4.3 × 3.6 m, not including the top reinforced concrete beam and the footing. The specimens were thus equivalent to walls on each of three 2.87 m high storeys. The door size was 1 × 0.57 m and the window size 0.57 × 0.57 m. A schematic of a specimen with door openings is provided in Figure 1. The specimens with windows were the same except that they had windows starting after the 5$^{th}$ course of masonry from each floor. The masonry was constructed with units half the scale of a standard 190 × 190 × 390 mm hollow concrete block unit: similar half-scale units have been used by several other researchers (for example, [7]-[9]). The webs of the units were cut to form bond-beam units, which were used for the bond and masonry beams.

The walls were reinforced with 3 #3 bars ($d_b$ = 9.5 mm) at each end and side of the openings. A two-course beam was constructed over each opening, reinforced with 2 #3 longitudinal bars at the bottom and 1 #3 longitudinal bar at the top. A continuous bond beam, reinforced with 1 #3 bar was constructed over each storey. This bar was hooked at the ends to the vertical wall reinforcement. A reinforced concrete slab, reinforced with 2 #3 longitudinal bars at the top and the bottom and #3 stirrups at 150 mm on centre, was constructed over the 1$^{st}$ and 2$^{nd}$ floors. The masonry beams over the openings had a #2 bar ($d_b$ = 6.3 mm) closed loop stirrup in every cell. To support the stirrups at the top, an additional #3 bar was placed in the reinforced concrete slab, resulting in the slab having 3 #3 bars at the top. A concrete beam, reinforced with 2 #5 bars ($d_b$ = 15.9 mm) on the top and bottom and stirrups 150 mm on centre, was constructed at the top of each specimen. The vertical bars in the specimens were extended into the beam and hooked around a 38 mm diameter, high strength, threaded bar that extended the entire length of the wall. This high strength bar extended past one end of the reinforced concrete beam to connect the specimen to the actuator. Four specimens (numbers 2 and 3 (door openings) and 5 and 6 (window openings)) had, at each level, a bond beam at the 5$^{th}$ course. The beam was reinforced with 1 #3 longitudinal bar. The bar was hooked at the ends to the vertical wall reinforcement. The specimens were constructed on a 305 × 1219 × 4064 mm concrete footing, which was reinforced with 5 #5 longitudinal bars top and bottom and #3 stirrups at 150 mm on centre.

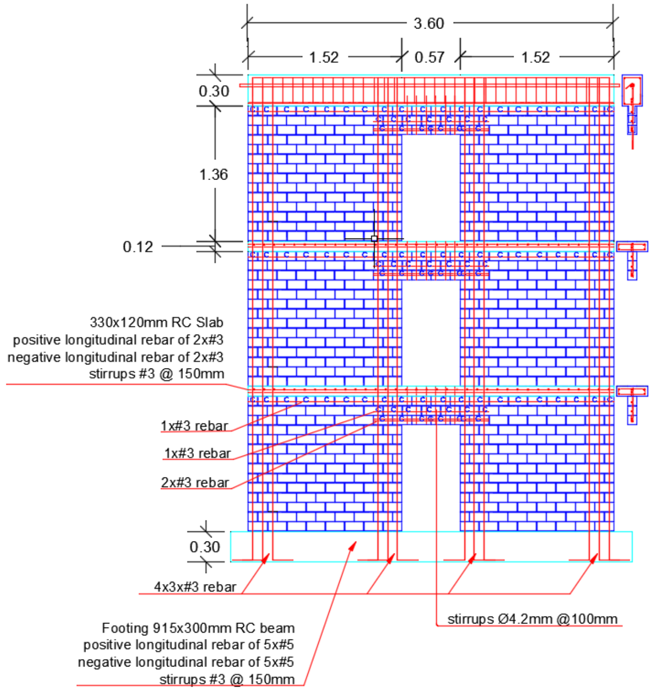



**Figure 1: Schematic of Specimen with walls with doors**

***Construction***

Construction of each specimen began with pouring the reinforced concrete footing, as shown in Figure 2a. The vertical reinforcement was tied to the reinforcement of the footing and protruded from the footing as shown in Figure 2b. The blocks were laid with 5 mm mortar joints. When the masonry was five courses high, the vertical cells containing reinforcement were grouted as shown in Figure 2c. For the walls with a bond beam at the 5$^{th}$ course, the horizontal reinforcement was hooked around the vertical bars as shown is Figure 2d. The top of each level, i.e., the 14$^{th}$ course, was also formed into a bond beam. Reinforced masonry beams were built under and over the openings. A small reinforced concrete slab was cast-in-place atop the 14$^{th}$ course of the 1$^{st}$ and 2$^{nd}$ storeys. The slab was used to connect the wall to the lateral bracing support of the testing frame. A reinforced concrete beam was cast-in-place atop the 14$^{th}$ course in the 3$^{rd}$ storey to support the vertical load and to distribute the applied lateral load along the length of the specimens. Two constructed specimens, one of each type, are shown in Figure 3.

(a) Footing (b) Dowels (c) Bond Beam (d) Hooks

**Figure 2: Specimen construction**

(a) With door openings

(b) With window openings

**Figure 3: Specimen configuration**

***Materials***

The properties of the constituent materials used to construct the walls were determined from standardized testing methods. Concrete strengths were determined by testing three standard 100 x 200 mm cylinders, as shown in Figure 4a at 28 days after casting. The average compressive strength of the foundation concrete was 49 MPa (C.O.V = 7%). The concrete used in the slabs representing the floor at each story was mixed in the laboratory from dry, pre-mixed bags of concrete. For the 120 mm thick concrete slabs, the maximum crushed limestone aggregate size was 10 mm. and three concrete cylinders were tested for each floor slab. The average compressive strength of the concrete used in all floor slabs was 37.7 MPa (C.O.V = 9.8%). The concrete used for the top concrete beam was ready-mix with a specified maximum aggregate size of 10 mm. The 28-day compressive strength of this concrete was 30 MPa (C.O.V = 5.2%). Compression tests were also conducted on mortar cubes as shown in Figure 4b. Several mortar batches were prepared during the construction of each specimen. Type S mortar was used and 24 randomly chosen 40 mm mortar cubes were tested in compression. The average compressive strength of the mortar was 15.7 MPa (C.O.V. = 15.1%). Pre-mixed bags of fine grout were used, with the grout being mixed in the laboratory. The grout was sampled by constructing three grout prisms for each level. The prisms were 89 x 89 x 190 mm as shown in Figure 4c. The average compressive strength of the grout was 35.7 MPa (C.O.V. = 12.8%). Compression tests were conducted on six of the half scale units, as shown in Figure 4d. The average compressive strength of the blocks, based on net area, was 16.9 MPa (C.O.V. = 9%). Three grouted and three hollow masonry prisms were built during the construction of each specimen and tested, as shown in Figure 4e, to determine their compressive strengths. The prisms were three blocks high by one block long. The grout was rodded during grouting of the prisms to ensure complete filling of all cells. The prisms were capped with gypsum prior to testing. The average compressive strengths, based on net area for grouted and ungrouted prisms, were 10.4 MPa and 12.0 MPa, respectively. The reinforcing bars were also tested. The vertical and horizontal bars had a yield strength of 638 MPa and the reinforcing bar used for stirrups had a yield strength of 312 MPa.

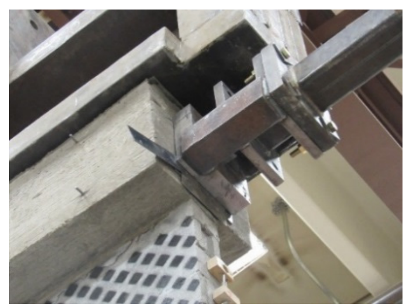
(a) Concrete b) Mortar (c) Grout (d) CMU (e) Prism

**Figure 4: Testing of the Materials**

**TEST SETUP**

The test arrangement is shown in Figure 5. Horizontal cyclic loading was applied to the top of the specimen via the concrete beam on top of the specimen. The concrete beam served to distribute the lateral and axial loads along the length of the specimen. The high strength bar embedded in the concrete beam was connected to the actuator as shown in Figure 5b. A set of steel shapes, weighting approximately 50 kN, was placed on top of the specimen to simulate a distributed axial dead load as shown in Figure 5c. The wall was stabilized from moving out-of-plane by a frame as shown in Figure 5d. The out-of-plane bracing system was used to represent the stabilizing influence of rigid diaphragm floors and consisted of three steel beams pinned to a steel frame at each floor level. These beams were connected to the reinforced concrete slabs and to the top concrete beam. The link members at each story were designed to resist out-of-plane displacements while not influencing the in-plane and vertical wall movements.

(a) Overall Setup



(b) Wall-to-Actuator Connection

(c) Wall-to-Actuator

(d) Out-of-Plane Bracing

**Figure 5: Specimen test setup**

The concrete footing was fixed to the concrete structural floor by ten post-tensioned Dywidag bars. Lateral in-plane cyclic load was applied using a hydraulic actuator with a capacity of 500 kN and a stroke of ± 250 mm.

## INSTRUMENTATION

String potentiometers and Linear Variable Differential Transducers were used to measure horizontal and vertical displacements, as shown in Figure 9. Displacements at the window and door levels, sliding of the wall relative to the concrete footing, and uplift at wall heels were monitored. Movement at the corners of the openings were also measured. String potentiometers attached to an independent column were used to monitor the movement, if any, of the test frame and the concrete footing. String potentiometers were also placed to obtain the shear and flexural components of the wall deformations. The overall in-plane lateral displacements of the walls, along the height of the walls, were monitored using seven horizontally positioned potentiometers. Two additional potentiometers were mounted at the base of the wall to monitor any horizontal slip and vertical uplift that might occur between the specimen and the concrete foundation.

Strain gauges were installed on the reinforcement along the wall height for each floor. Strain gauges were also placed on the two outermost reinforcing bars at the end of each specimen to monitor strain penetration into the RC footing. These strain gauges were located at heights of +100 mm, -100 mm and -1200 mm, relative to the wall-footing interface.

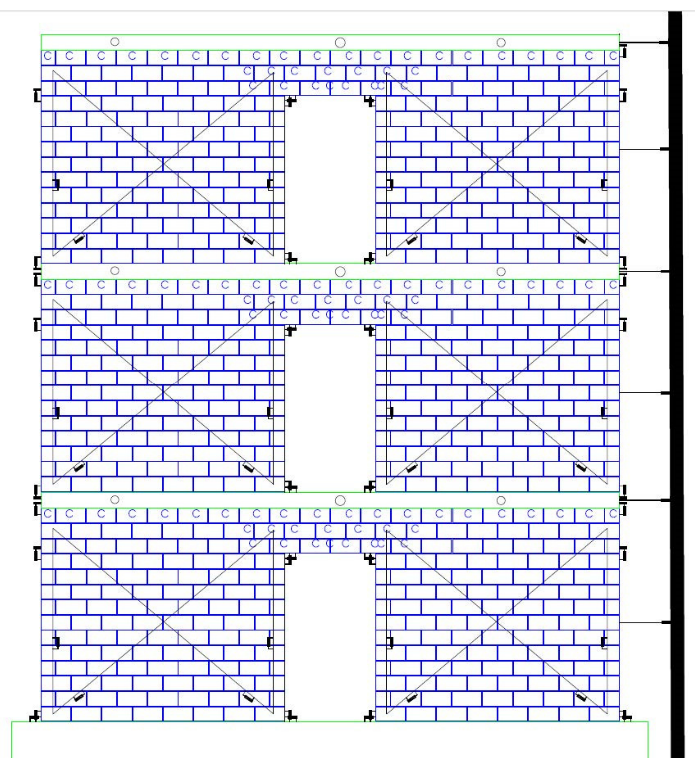


**Figure 9: Specimen Instrumentation**

## TEST PROTOCOL

The walls were tested under displacement control with a prescribed lateral displacement plan, as shown in Figure 10. The displacement-controlled procedure was designed to allow assessment of different targets such as yielding of the reinforcement, the first crack, the end of the linear region, peak load and ultimate load. At each new displacement level the test was paused briefly in each displacement direction (push and pull) to mark and take pictures of the cracks. Failure of a specimen was defined as the point where the lateral resistance dropped below approximately 60% of the measured peak lateral load.

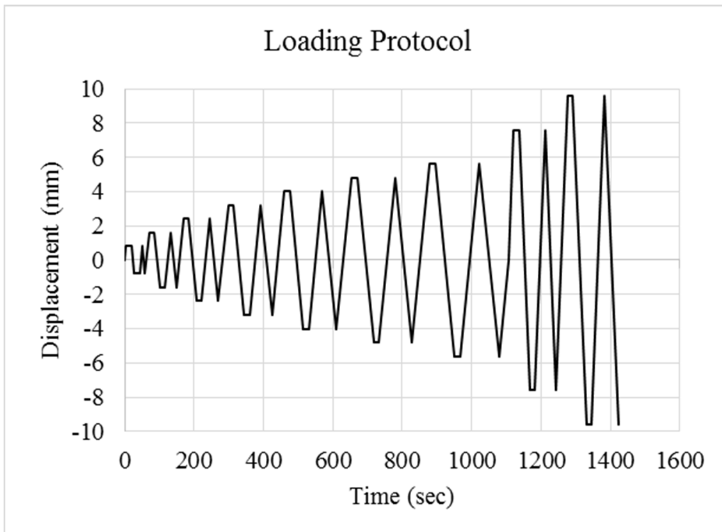


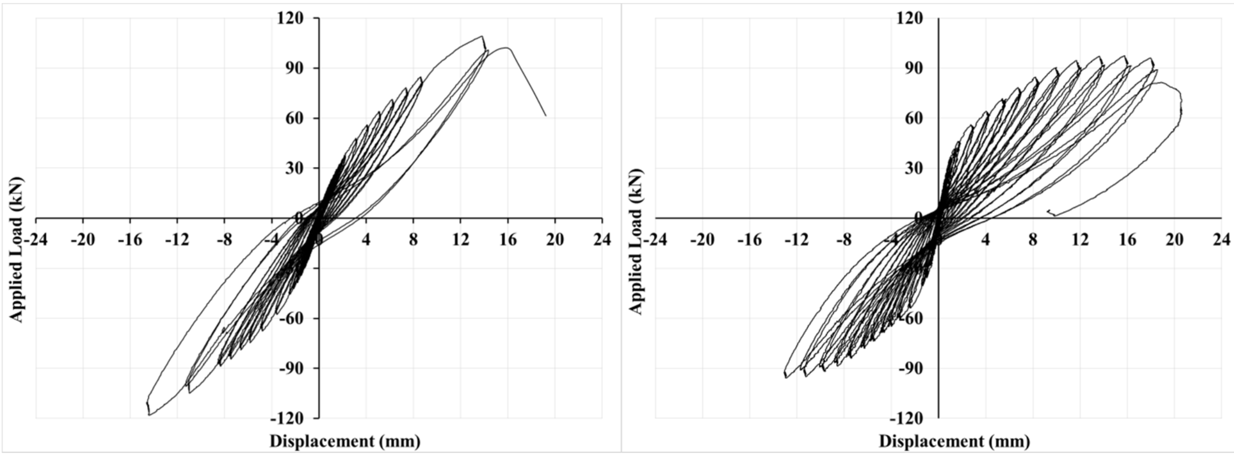

**Figure 10: Specimen Test Protocol**

## PRELIMINARY RESULTS

The hysteresis for specimens 1 and 2 with door openings and 4 and 5 with window openings are shown in Figures 11 and 12, respectively. The response of specimen 3 was similar to that of specimen 2 and the response of specimen 6 similar to that of specimen 5.

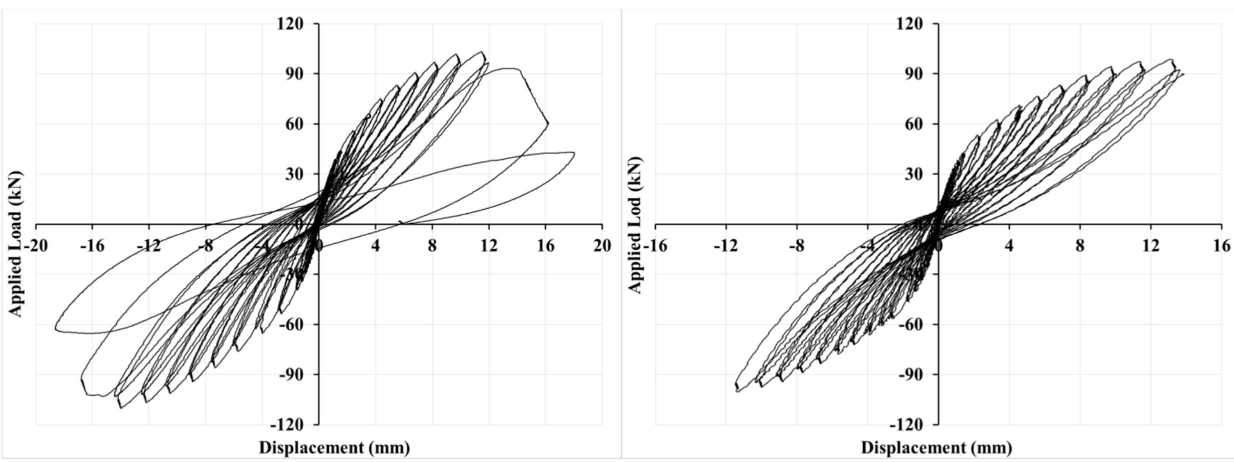
The hysteresis curves are symmetrical in general, indicating similar behaviour in both directions of loading until major cracks developed in one or other loading direction. After that, damage concentrated in that loading direction and the response started to become asymmetric. For all specimens a small decrease in strength and stiffness is observed during the second excursion to the same displacement level. At higher displacement levels, bigger hysteresis loops are evident, indicating higher amounts of energy dissipation and increased plastic deformations.

The envelope response of all specimens in the push direction is shown in Figure 13. The walls with door openings presented similar stiffness even after the reinforcement yielded. The responses are similar and apparently the bond beam at the 5$^{th}$ course of each storey (specimens 2 and 3) changed neither the behaviour nor the capacity. The capacity of specimen 2, however, is slightly less than that of its counterpart, specimen 3. The test results are being examined in depth to determine if there is a specific reason for the smaller capacity of specimen 2 or if it is simply due to the normal scatter of results expected when testing masonry.




(a) Wall 1 (b) Wall 2

**Figure 11: Response of specimens with door openings**



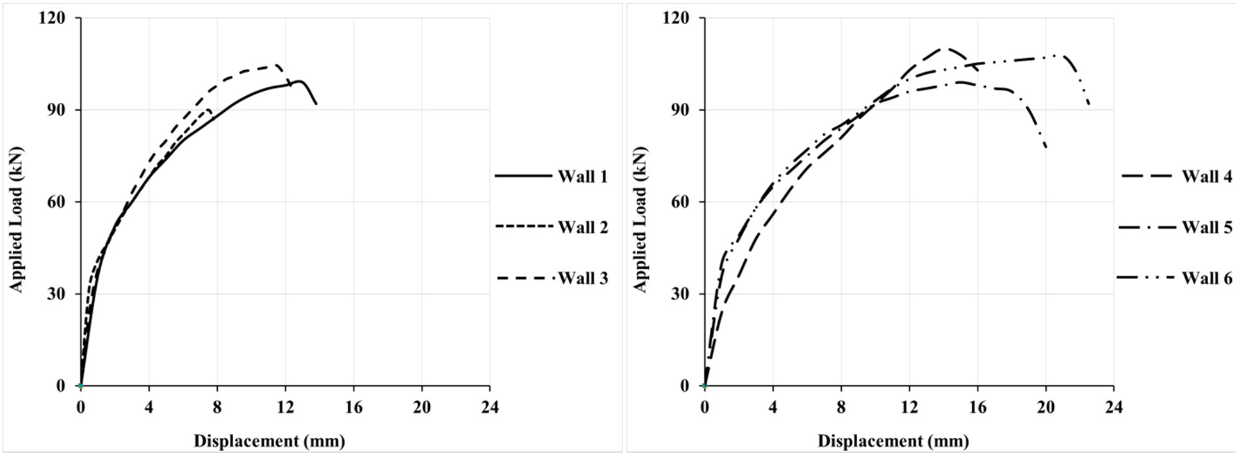

(a) Wall 4 (b) Wall 5

**Figure 11: Response of specimens with window openings**

For the specimens with window openings, the response of the specimen without a bond beam at the 5$^{th}$ course of each storey is slightly different from that of the specimens with such bond beams. That specimen is slightly more flexible and slightly less ductile. The capacity of that specimen, however, is slightly more than that of the other specimens.

The capacities of all specimens are similar, except that of specimen 2. The specimens with window openings appear to be slightly more ductile that those with door openings.

The numerical results are summarized in Tables 1 and 2. The results reported here include only the peak lateral load, Z, in both pull and push directions and the displacement, D, at the top of each specimen, corresponding to the peak lateral loads.




(a) Specimens with door openings (b) Specimens with window openings

**Figure 13: Envelopment response**

**Table 1: Specimen test results – peak lateral load**

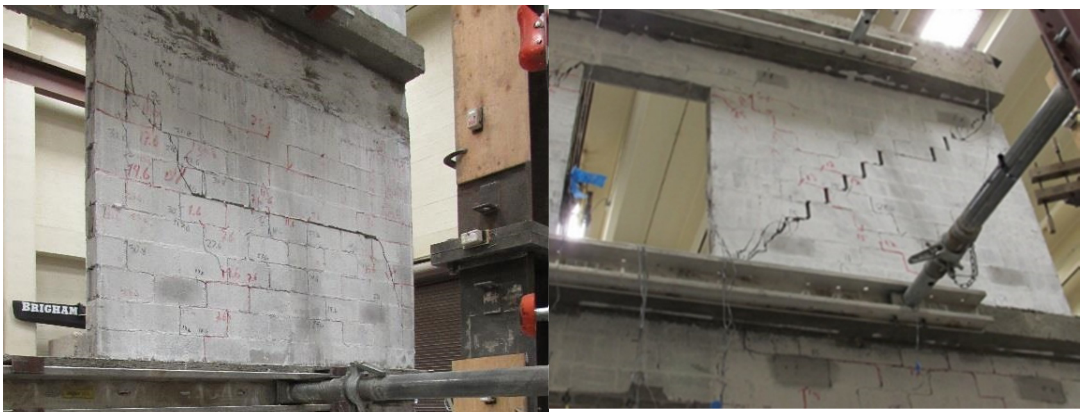
| Specimen | Z (kN) | | Average Z (kN) | C.O.V (%) | Group Average Z (kN) | C.O.V |
|---|---|---|---|---|---|---|
| | (+) ve | (-) ve | | | | |
| Wall 1 | 99.0 | 101.3 | 100.2 | 1.6 | 100.1 | 6.3 |
| Wall 2 | 89.6 | 98.7 | 94.2 | 6.8 | | |
| Wall 3 | 103.5 | 110.2 | 106.8 | 4.5 | | |
| Wall 4 | 109.2 | 118.1 | 113.6 | 5.5 | 103.6 | 8.2 |
| Wall 5 | 97.5 | 96.1 | 96.8 | 1.1 | | |
| Wall 6 | 101.0 | 106.2 | 103.6 | 3.6 | | |

**Table 2: Specimen test results – Specimen Top displacement, D**

| Specimen | D (mm) | | Average D (mm) | C.O.V (%) | Group Average D (mm) | C.O.V |
|---|---|---|---|---|---|---|
| | (+) ve | (-) ve | | | | |
| Wall 1 | 13.2 | 11.5 | 12.4 | 6.9 | 12.4 | 9.9 |
| Wall 2 | 7.5 | 12.1 | 9.8 | 23.5 | | |
| Wall 3 | 11.5 | 14.0 | 12.8 | 9.9 | | |
| Wall 4 | 13.9 | 14.7 | 14.3 | 2.9 | 14.4 | 12.5 |
| Wall 5 | 15.8 | 13.0 | 14.4 | 9.7 | | |
| Wall 6 | 15.4 | 21.4 | 18.4 | 16.3 | | |

It appears that the capacities of the specimens are not affected significantly by the type of opening. The average capacity considering all specimens is approximately 102 kN, with a coefficient of variation of 9.2%. The displacements corresponding to the maximum loads, however, vary more than the maximum loads themselves. The average displacement at peak load in the push (+) direction is 13.2 mm and the average displacement at peak load in the pull (-) direction is 13.0 mm, with coefficients of variation of 22.3% and 27.3%, respectively.

Some of the cracks on walls with door openings are shown in Figure 14 and some of the cracks on walls with window openings in Figure 15. Typically, horizontal bed joint cracks were first observed at an applied displacement of 2.4 mm, which corresponded to approximately 40 to 60 kN lateral load. The cracks formed in the mortar joints between the third and tenth courses in the first, second and third storeys and extended, on average, approximately 800 mm along the length of the wall. As the applied displacement was increased, the cracks widened and propagated. Cracks would then develop at head joints. A step crack pattern would develop, indicating a shear dominant effect. Ultimately, walls failed due to shear as shown in Figure 14.




(a) 2nd storey cracks (b) 3rd storey failure

**Figure 14: Specimen with door openings**


(a) 1st storey cracks (b) 3rd storey cracks

**Figure 15: Specimen with window opening**

**PRELIMINARY OBSERVATIONS**

Specimens 2, 4 and 5 failed in the first storey and specimens 1, 3 and 6 failed in the third storey. Walls experienced slipping along bed joints, a step crack pattern and, at the end of the test, cracking along joints and through units, as shown in Figures 14 and 15. The cracks suggest a diagonal strut infilled within the reinforced masonry regions and a dominant diagonal shear behaviour. It appears that the concrete beams on the top of the walls and the masonry beams above the openings affected the behaviour of the walls. These beams appear to have induced coupling between the vertical segments of the walls.

The lateral load capacity of all specimens was similar, except for one wall, which may simply be due to the normal variability in masonry construction. The similar load capacity may be an indication that the capacity of the walls is independent of the type of the opening, as long the widths of the openings are reasonably similar. However, the top lateral displacement of the specimens at peak load was distinctly more variable. This may be an indication that the type of opening and the coupling beams may have an effect on the ductility of the wall.

The specimens exhibited gradual strength degradation and did not experience sudden failure. Such desirable behaviour was probably due to the continuous bond beam at the top of the walls at each level and the coupling beams on top of each opening. It appears that these structural elements cause a wall with openings behave like a frame.

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