# WIDER REINFORCED MASONRY SHEAR WALLS SUBJECTED TO CYCLIC LATERAL LOADING

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Abstract

**Partially grouted wider reinforced masonry wall, built predominantly with the use of face shell bedded hollow concrete blocks, is adopted extensively in the cyclonic areas due to its economy. Its out-of-plane response to lateral pressure loading is well defined; however, its in-plane shear behaviour is less well understood, in particular it is unclear how the grouted reinforced cores affect the load paths within the wall. For the rational design of the walls, clarification is sought as to whether the wall acts as a composite of unreinforced panels surrounded by the reinforced cores or simply as a continuum embedded with reinforcement at wider spacing. This paper reports four full scale walls tested under in-plane cyclic shear loading to provide some insight into the effect of the grout cores in altering the load paths within the wall. The global lateral load – lateral deflection hysteretic curves as well as the local responses of some critical zones of the shear walls are presented. It is shown that the aspect ratio of the unreinforced masonry panels surrounded by the reinforced grouted cores within the shear walls have profound effect in ascertaining the behavior of the shear walls.**

Streszczenie

**Zbrojone ściany murowane, wykonane z drążonych bloczków betonowych częściowo wypełnionych zaprawą, są powszechnie stosowane na obszarach cyklonicznych ze względu na ekonomiczność tego rozwiązania. Dobrze znane jest zachowanie się takich murów obciążonych bocznym naporem. Znacznie słabiej rozpoznane jest ich zachowanie w przypadku naprężeń ścinających w płaszczyźnie elementu, a zwłaszcza niejasne jest jak zbrojone rdzenie z zaprawy oddziałują na ścieżki obciążenia wewnątrz muru. Ze względu na racjonalność projektowania takich ścian, przyjmuje się, że ściana pracuje jako kompozyt niezbrojonych paneli otoczonych przez zbrojone rdzenie lub po prostu jako ośrodek ciągły z osadzonym zbrojeniem w szerszych odstępach. W artykule przedstawiono cztery pełnowymiarowe ściany badane pod cyklicznym obciążeniem ścinającym w płaszczyźnie elementu aby dostarczyć danych na temat wpływu rdzeni z zaprawy na zmieniające się ścieżki obciążenia wewnątrz muru. Przedstawiono również globalne obciążenie boczne – krzywe histerezy bocznego ugięcia, jak również lokalne odpowiedzi niektórych stref krytycznych ścian poddanych ścinaniu. Pokazano, że współczynnik kształtu niezbrojonych paneli murowych otoczonych zbrojonymi rdzeniami z zaprawy wewnątrz ścian poddanych ścinaniu ma znaczący wpływ na ustalenie zachowania się tych ścian.**

Keywords: **Concrete blocks; In-plane shear; Failure modes; Grout; Masonry shear walls.**

## 1. INTRODUCTION

Masonry is a one of the most popular and oldest materials used in building construction. However, due to limited ductility, modern unreinforced masonry walls are discouraged even in zones of moderate seismic risk; in such zones reinforced masonry is recommended. When reinforcement is closely spaced and fully grouted, concrete masonry walls behave very similar to that of the reinforced concrete shear walls; however, such fully reinforced systems are huge economic burden for moderate earthquake zones. In recent times masonry walls made from hollow concrete or clay blocks that are partially grouted and reinforced wider spaced cores surrounding unreinforced panels are being constructed with a view to conserving materials and making the walling system economical.

The structural behaviour of masonry is complex due to its anisotropy. Flexural failure of the reinforced masonry, characterised by the yielding of the tension reinforcement or compression failure of the masonry, can be predicted reasonably well using the simple-bending theory [1]. However, shear failure is more complicated involving diagonal tension, diagonal compression/ strut action, and sliding modes of failure over relatively larger zones, in which the mortar joints act as plane of weakness. This paper presents the results of the in-plane cyclic shear load performance investigation of four full scale walls, with each wall consisting of a series of unreinforced masonry panels surrounded by reinforced grouted cores, the geometry of which is similar to the confined masonry shear walls promoted as suitable building system for moderate seismic risk zones.

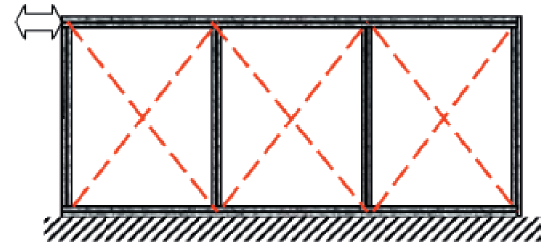

**Figure 1.**
**Composite action**

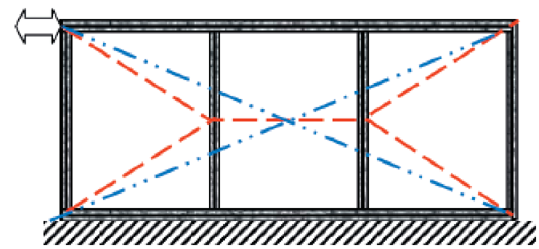

**Figure 2.**
**Continuum action**

## 2. WIDER REINFORCED MASONRY WALLS

A typical wider reinforced masonry wall is shown in Fig. 1. The wall consists of unreinforced masonry panels surrounded by reinforced grout cores. This wall system can be analogised to the confined masonry where the unreinforced masonry is first constructed with sufficient gaps for reinforced concrete tension column and beams fabricated later – thus confining the unreinforced masonry for improved inplane shear resistance with reduced shear distortions. The reinforced grout cores of the wider reinforced masonry also resist tension only – thus mimicking the tension columns of the confined masonry. Very little is known on the behaviour of the wider reinforced masonry walls to cyclic inplane shear; however, in line with the confined masonry, it is believed that it will be suitable for zones of moderate earthquake intensity.

Intra-plate type earthquakes of moderate intensity occur in many parts of the world; these earthquakes are characterised by very short duration shaking – as low as 30 seconds, thus may not demand very high energy dissipation as demanded by the plate boundary earthquakes. However, in the absence of lack of research on intraplate type of earthquake design spectra, we have kept the lateral load cycles in the experiments reports in this paper similar to those reported in the literature with high seismic research focus. We examine the lightly reinforced partially grouted masonry shear walls in this paper. A wall is deemed wider reinforced if it contains vertical reinforcement

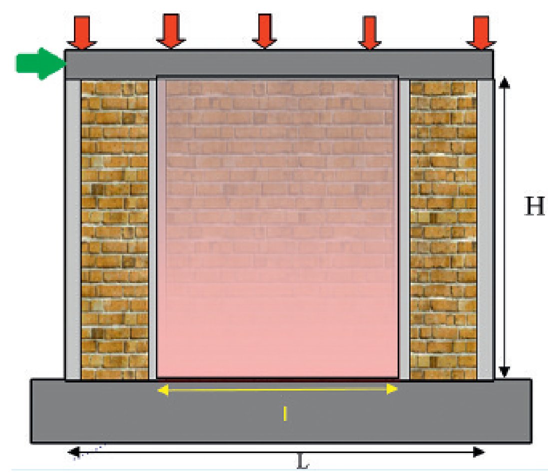


**Figure 3.**
**Central panel aspect ratio**

with the spacing ranging from 800 mm to 2000 mm; and horizontal reinforcement with the spacing of up to 3000 mm according to the definitions of the Australian masonry standard AS3700 [2]. The walls reported in this paper contain reinforcement with this range specified. This arrangement has effectively divided the wall into large unreinforced masonry panels surrounded by reinforced cores. Due to the large spacing between the reinforced cores in the horizontal and vertical directions, the wall as a whole could act as a *composite* of unreinforced masonry panels connected by reinforced grouted cores exhibiting strut mechanism of failure in each of the unreinforced panel as shown in Fig. 1 or it can also act as a unified *continuum* with embedded

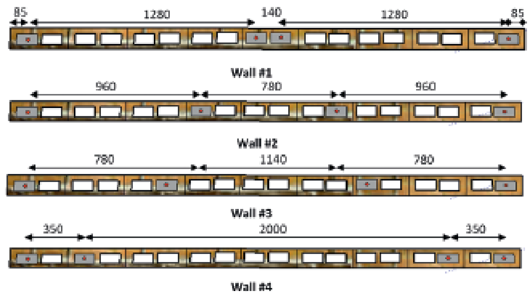


**Figure 4.**
**Reinforced core arrangement**

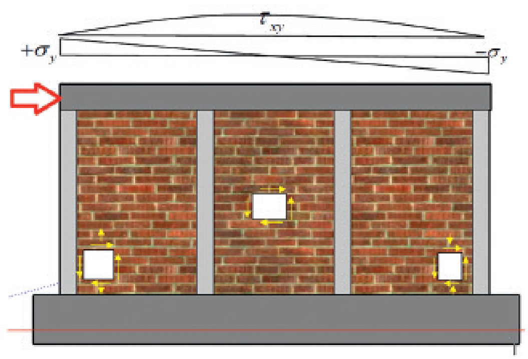


**Figure 5.**
**Critical zones of shear walls**

grouted reinforced cores, with single crack propagating diagonally with (shown in red) or without (shown in blue) bed joint sliding as shown in Fig. 2. Instead any other combination of failure path could occur. The horizontal spacing of the vertical reinforced grouted cores, the vertical spacing of the horizontal bond beams, the aspect ratio of the wall $\left(\lambda = \dfrac{H}{L}\right)$,where $H$ is the height of the wall and $L$ is the length of the wall, the imposed vertical load and the panel aspect ratio $\left(\mu = \dfrac{H}{l}\right)$, where $l$ is the length of the central URM panel of the wall as shown in Fig. 3, are the parameters that would affect the behaviour in a significant manner. The reason for considering the mid URM wall panel is that the central zone is the high shear that occurs at the central zone of the shear walls under inplane lateral loading.

When the wall aspect ratio $\lambda$ is lower than 1.0 (in some cases as low as 0.2), the walls are termed squat, the failure mode of which can be of any one of the combination, or of a combination of any few, shown in Figs. 1 and 2. There are dubious design formulae existing in many national standards [2-4], which are proved to be unconservative [5-8]. The effect of the panel aspect ratio $\mu$ is included as a novel means of understanding the in-plane shear response of wider reinforced masonry walls in this paper.

## 3. EXPERIMENTAL PROGRAM

The detail description of experimental work is explained in detail in Haider [9] and Dhanasekar and Haider [10]. A mason of average workmanship built all the four test walls. Each wall was reinforced with 4N12 (12 mm diameter deformed bars with area excluding deformed helix 110 mm$^2$) bars providing a vertical reinforcement ratio of 0.14%. All walls have gross dimension of 2408 mm high, 2870 mm long and 150 mm thick that represent 9 blocks long and 28 blocks high. All the walls were constructed from the commercially available hollow clay blocks in Australia. Hollow clay units of 310 mm (length) × 150 mm (width) × 76 mm (height) were used. These units contain two symmetrical cells with 100 mm × 80 mm in the centre where grouting and vertical reinforcement was accommodated. Each typical vertical grouting contains 1N12 bars at the centre. A bond beam size of 2870 mm × 150 mm × 172 mm consisting of two layers of masonry blocks reinforced with 4N16 bars was constructed at the top of each wall. The purpose of the bond beam was to enable uniform distribution of the applied vertical load and to minimize the chances for local failure of the loaded corner.

These walls were tested after 28 days of air curing within plastic wrapping. For each wall, three face-shell bedded prisms, three grout cylinders and three mortar cubes were also cast and tested on the day of testing of the walls. The mean strength of the test is provided in Table.1. The location of reinforced core is shown in Fig. 4. The typical test setup and push-pull arrangement for cyclic load is shown in [9] and [10] and hence not repeated. A 2000 kN compression capacity hydraulic cylinder was used to apply the vertical load. Horizontal load was applied under controlled displacement using 500 kN capacity tension-compression hydraulic cylinder. To capture strength and stiffness degradation, two cycles were conducted at each drift level of cyclic loading.

### 3.1. Failure mode

The critical zones of the shear walls for in-plane load are shown in Fig.5. The cracks initiate within the critical zones when stress level exceeds the material capacity. The two edge panels are dominated by axial stresses (tension/compression) whilst the central

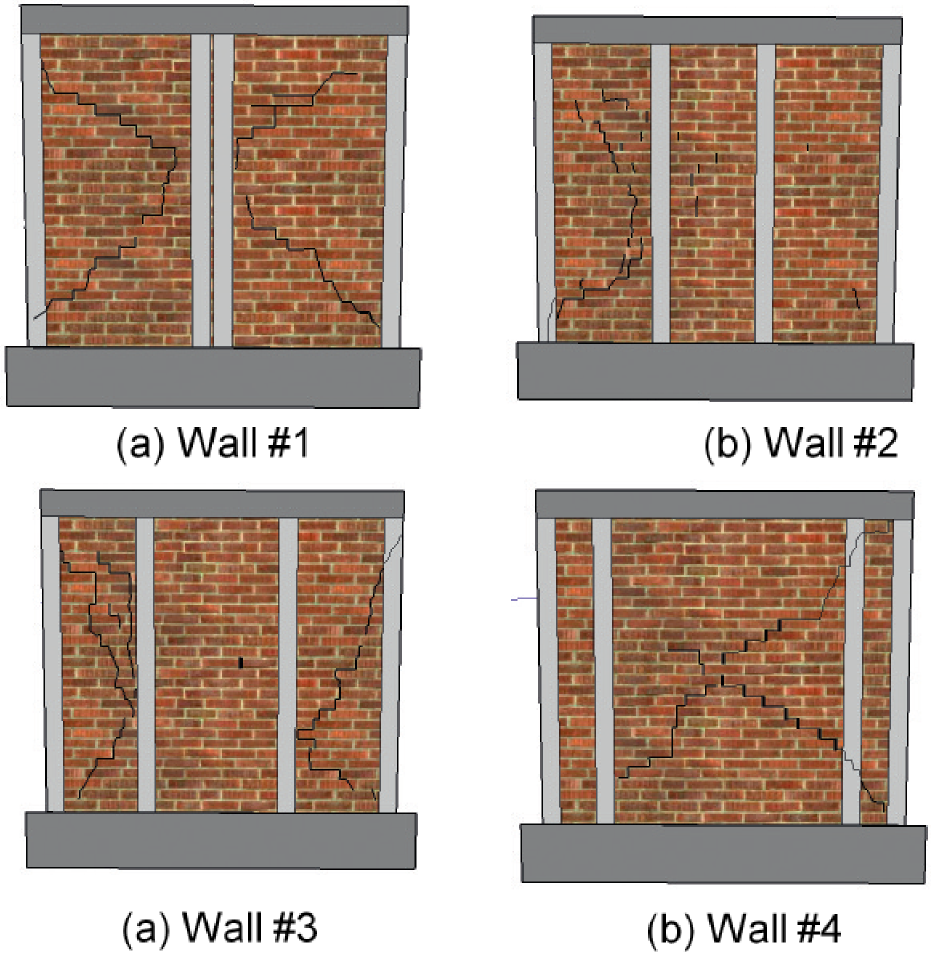


**Figure 6.**
**Crack pattern of the walls**

**Figure 7.**
**Integrity of typical reinforced grouted core**

panel is dominated by shear stress. Under cyclic loading, the tension and compression reverses with the change in loading from push to pull state.

The unreinforced masonry panels are assumed to be confined by the two edge reinforced cores – with the increase in the width of the panel (or, reduced aspect ratio of the panels), the panels are likely to experience reduced strength. Furthermore, the shear strength is normally greater than the tensile strength of the material. So it is more likely that the two edge panels that are under tensile stresses initiate damage prior to the central panel as long as the aspect ratio of the edge panels is of comparable magnitude as that of the central panel. This has been the case for Walls #2 and #3. Wall #1 did not have a central panel – so all damages have occurred in the edge panel. The edge panels of Wall #4 possess large aspect ratio (very small width – indicating high level of confinement potential); therefore, the tensile stresses in the edge panels could not exceed the higher capacities due to the confined nature of the edge panel prior to the onset of failure of the central panel that has failed due to shear.

The mode of failure of the walls are presented in [10] and hence not repeated here. Schematic sketches of

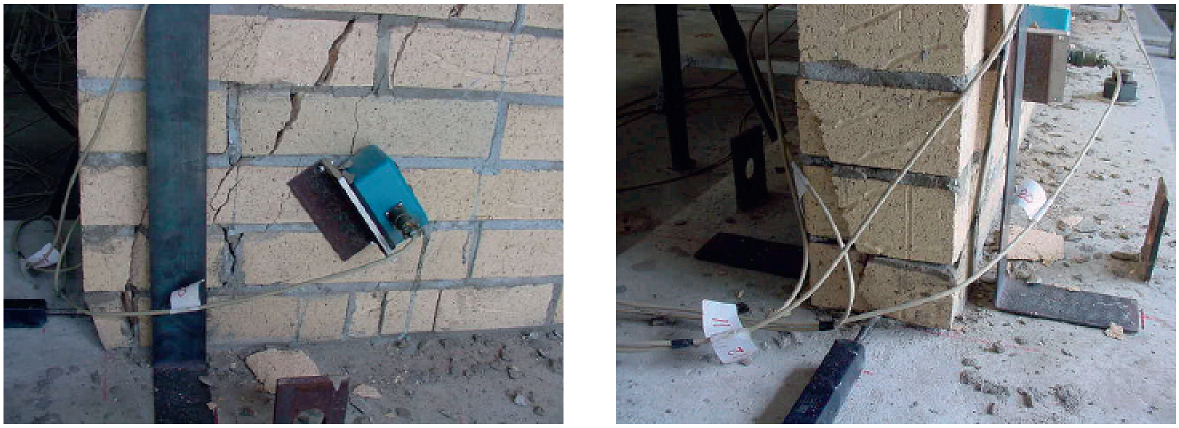

**Figure 8.**
**Side and end views of toe crushing in reinforced grouted**

the failure modes of the walls are presented in Fig. 6. It can be seen that the failure occurs in unreinforced masonry panels. Where the aspect ratios of all three panels in the wall are comparable to each other, damage is widespread across all panels. Where the difference in aspect ratio of the panels is significant, only the panels with the smallest aspect ratio suffered damage (i.e., in Wall #4 only the mid panel was damaged). The confinement of the surrounding reinforced grout core offered to the unreinforced masonry panel is thus evident; however, to quantify the exact nature of the confinement, further research is essential.

To verify if the damage passed through the reinforced grout core, the shells of the masonry units were hammered out carefully to expose the grouted core (Fig. 7). There was no evidence of damage along the height of the grout core, although local toe crushing was evident as shown in Fig. 8.

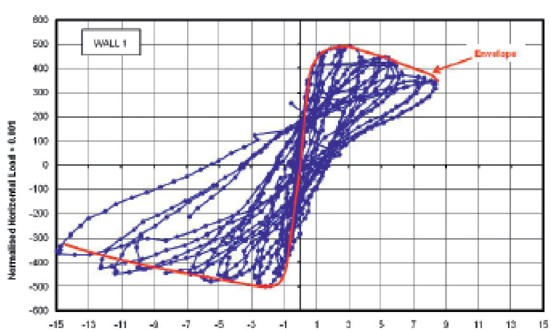


**Figure 9.**
**Hysteresis and envelope curves for WALL #1**

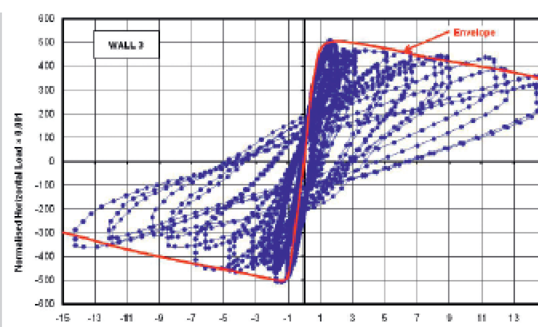


**Figure 11.**
**Hysteresis and envelope curves for WALL #3**

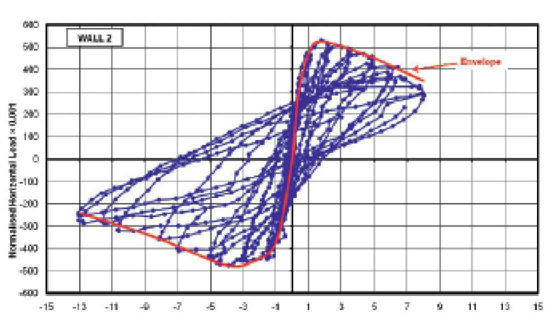


**Figure 10.**
**Hysteresis and envelope curves for WALL #2**

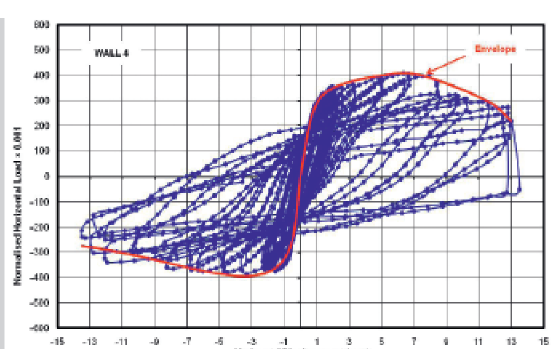


**Figure 12.**
**Hysteresis and envelope curves for WALL #4**

## 4. DEFORMATION RESPONSE OF SHEAR WALLS

Twenty two channels of LVDT and potentiometer data of displacements and surface strains were measured. Horizontal displacement measured in line with the position of the horizontal actuator was used in the development of the load-deflection hysteretic curves presented in this section. To get the true horizontal displacement at bond beam, the value measured by LVDT located at bond beam was subtracted by the relative movement of the base slab.

The properties shown in Table.1 exhibited significant variability. Therefore each wall's in-plane horizontal load then normalised using following equation (Eq-1).

$$V = \frac{P_u}{0.22\sqrt{f_m}A_g} \times 10^3 \qquad (1)$$

Where $P_u$ is the maximum ultimate in-plane shear loading (N), $V$ – normalised in-plane load, $A_g$ – gross sectional area (mm$^2$) and $f_m$ – mean compressive strength of masonry (MPa).

The normalised horizontal load-deflection curves of the shear walls are shown in Figs. 9-12. All curves are plotted in same scale to ease comparisons. The envelope curve is also shown in each plot and reported separately in Fig. 13. The Pinched hysteresis curve generated by unreinforced masonry [11] is not visible in the above four shear walls. Some selected loops from post softening region of the response curve for all walls are plotted in Fig. 14. All loops are fat indicating good energy dissipation. From the envelope curves shown in Figs. 9-12, the ductility of the walls in the forward and reverse direction were determined as per the procedure provided in [12, 13]. The ductility ($\xi$) was calculated using Eq-2.

$$\zeta = \frac{\delta_u}{\delta_y} \qquad (2)$$

$\delta_y$ is determined from the equivalent bilinear envelope curve and is determined as the displacement corresponding to 80% loss of the maximum load the wall sustained. The ductility and the drift index ($\Delta$) of the all four walls are reported in Table 2. It can be noticed that the drift index of the wall was ranging from 0.21% for Wall #2 to 0.46% for Wall #3 and Wall #4. The ductility of the wall was above 6 for all walls except Wall #4 and also Wall #4 exhibited low normalised load. These walls exhibited integrity after the test and also able to transport them safely from lab to storage yard. Secant stiffness at each loop of the hysteresis compared to the initial tangent stiffness of the corresponding wall. At ultimate stage all walls exhibited a stiffness degradation of 88% consistently. For combined-confined masonry similar level of stiffness degradation is reported [14].

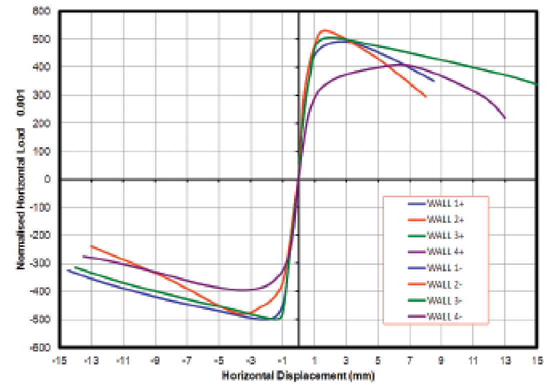


**Figure 13.**
**Envelope curves for all four walls**

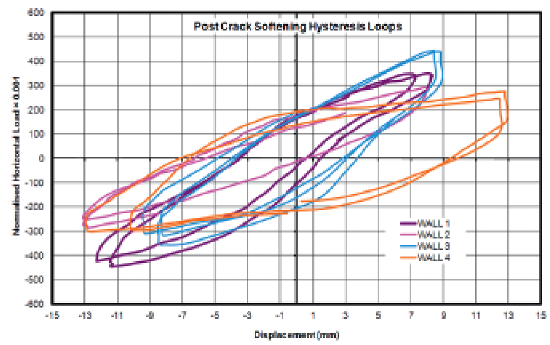


**Figure 14.**
**Hysteresis loop at post peak loading for all four walls**

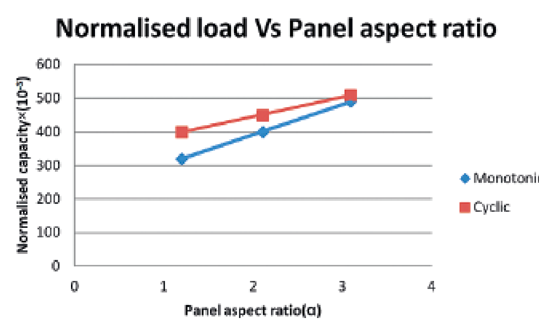


**Figure 15.**
**Effect of the central panel aspect ratio to the capacity of the shear walls**

## 5. EFFECT OF PANEL ASPECT RATIO

Each wall specimen has two edge panels and one central panel. Both edge panels have same aspect ratio as reinforced core located symmetrically to the centre of the wall. The effect of sandwiched centre panel aspect ratio was considered as shown in Fig. 3. The panel aspect ratio is defined as the ratio in between height of the panel (***h***) and the length of the panel (*l*). The panel aspect ratio of Wall #2, Wall #3 and Wall #4 are 3.1, 2.1 and 1.2, respectively (Wall #1 does not have central panel as its inner reinforced core located close enough to each other).

The normalised in-plane shear capacity versus the central panel aspect ratio is reported for cyclic and monotonic load in Fig. 15. Cyclic normalised in-plane load produced linear correlation with central panel aspect ratio. In the same figure monotonic load curve also reported – the data of which is extracted from [10]. It can be noticed that cyclic load curve exhibits higher strength than the monotonic load curve indicating that the slow damaging of the unreinforced masonry panels and the undamaged reinforced masonry have offered higher resistance to the lateral cyclic deformations. The monotonically loaded walls [10] exhibited failure of the grouted cores, the damage of which reduce the ability of the wall to offer higher resistance as it has happened with the cyclically loaded walls. The gradient of these curves is also affected by the type of loading where the monotonic load walls show higher sensitivity to the aspect ratio of the panel compared to the cyclically loaded walls. From the finding it appears that where the aspect ratio of the central panel is much higher, it might be more likely the monotonic load would produce higher in-plane load. Indeed this has been so, with the monotonically loaded wall of configuration similar to that of Wall #1 [refer to 9 and 10], it has exhibited higher lateral load capacity. The effect of the axial load on the gradients of these curves could not be determined; this is because only one single level (0.5 MPa) of pre compression was used in the experimental study.

## 6. CONCLUSIONS

In the absence of clear understanding of the behaviour of partially grouted wider reinforced masonry shear walls, they are designed with questionable assumptions. With a view to understanding the effect of the vertical grouted reinforced cores to the behaviour and the global deformation response of such walls, four full scale walls were fabricated and tested under constant vertical pre-compression (0.50 MPa). The following conclusions are drawn from the tests specific to the walls tested:

1) The behaviour of wider reinforced masonry shear walls is influenced by the aspect ratio of the unreinforced masonry wall panels surrounded by the reinforced grout cores. The relation between the wall aspect ratio and the panel aspect ratio remains inconclusive as only limited number of walls has been tested.
2) The walls exhibited good energy dissipating fat hysteretic loops in the post crack hardening and post crack softening stages, illustrating the benefit of the partially grouted wider reinforced masonry systems to seismic zones.
3) The central panel aspect ratio produced linear correlation with the in-plane shear capacity of the wall.

a. Cyclically loaded walls exhibit higher shear load resistance than the monotonically loaded walls of similar design – primarily due to the undamaged grouted cores.

b. Monotonically loaded walls exhibit higher sensitivity to the central aspect ratio of the wall panel than the cyclically loaded walls. Walls with very large central panel aspect ratio are likely to exhibit higher lateral load under monotonic load compared to the cyclic load.

## ACKNOWLEDGEMENTS

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