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EFFECT OF SPACING OF REINFORCEMENT ON THE BEHAVIOUR OF PARTIALLY GROUTED MASONRY SHEAR WALLS

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ABSTRACT

Partially Grouted Reinforced Masonry (PGRM) shear walls perform well in places where the cyclonic wind pressure dominates the design. Their out-of-plane flexural performance is better understood than their inplane shear behaviour; in particular, it is not clear whether the PGRM shear walls act as unreinforced masonry (URM) walls embedded with discrete reinforced grouted cores or as integral systems of reinforced masonry (RM) with wider spacing of reinforcement. With a view to understanding the inplane response of PGRM shear walls, ten full scale single leaf, clay block walls were constructed and tested under monotonic and cyclic inplane loading cases. It has been shown that where the spacing of the vertical reinforcement is less than 2000mm, the walls behave as an integral system of RM; for spacing greater than 2000mm, the walls behave similar to URM with no significant benefit from the reinforced cores based on the displacement ductility and stiffness degradation factors derived from the complete lateral load – lateral displacement curves.

Keywords: masonry shear walls; displacement ductility; wide spaced reinforced masonry; stiffness degradation.

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1. INTRODUCTION

Masonry walls are usually reinforced in cyclonic and / or seismically active zones, with the amount of reinforcement linked to the severity of the cyclone or seismicity. As Australian earthquakes are intraplate type characterised typically by low to moderate intensity, we have examined a lightly reinforced wall system, known as the partially grouted reinforced masonry (PGRM). Similar systems are also researched in other countries. For example, in the USA (Hamid, 1991; Shing et al, 1990a) and Canada (Khattab and Drysdale, 1993; Maleki et al., 2009; Oan and Shrive, 2009) walls containing bond beams at mid height and in New Zealand (Davidson and Brammer, D.R., 1996; Ingham et al., 2001) walls containing vertical and horizontal reinforcements have been examined. In Peru a masonry confined by mullions, known as 'confined masonry' is popularly adopted (Alcocer and Meli, 1995). In Australia, PGRM walls are constructed with vertical reinforcement at spacings ranging from 800 mm to 2000 mm; and horizontal bond beams spaced up to 3000mm (Haider and Dhanasekar, 2004).

Shear walls resist the inplane loading predominantly by their in-plane shear or flexural stiffness depending upon the wall geometry, the level of axial loading, properties of the constituent materials, and the amount and distribution of the vertical and the horizontal reinforcement. Shear walls that are fixed at the bottom and loaded vertically and horizontally as shown in figure 1 undergo deformation as a cantilever with tensile stresses predominating at the heel, especially when the vertical load is low in relation to the horizontal load, compressive stresses predominating at the toe and a biaxial tension – compression (or pure shear) state of the stress dominating the mid region of the wall. Tall walls usually fail due to flexural mechanism characterised by heel cracking followed by toe crushing; squat shear walls fail in a complex mechanisms characterised by diagonal cracking, bed joint sliding and toe crushing. Generally walls that fail in combined flexural and shear mode exhibit higher lateral load than those which fail purely under flexure.

To avoid premature failure of the masonry shear walls, it is necessary to provide reinforcement in the areas where significantly higher tensile and shear stresses occur. For example, reinforcement is provided in the heel region to resist flexural tension and with the anticipation of load reversal, flexural steel is also provided in the region shown as 'toe' in figure 1. From a flexural perspective, it is suggested that the lumped reinforcements placed only at toe and heel are better than the distributed reinforcement for enhancing the horizontal load capacity (Paulay and Priestley, 1992). Considering the masonry panel that would span between the lumped reinforced cores, this paper examines whether there should be any limit imposed on the spacing of the reinforcing bars. Diagonal reinforcement may be effective in resisting the diagonal tension due to 'pure' shear at the centre of the shear wall; such detailing is possible in reinforced concrete, but will be impractical for masonry due to the way the masonry blocks will have their hollow cores oriented, making either vertical or horizontal reinforcement only can be detailed. A question arises here on the effectiveness of the amount and the spacing of the vertical reinforcement to the enhancement of both the capacity and deformation characteristics (eg., ductility) of the shear wall.

Shing et al. (1990) have concluded from a series of experiments that properly detailed shear reinforcement higher than 0.2% helps avoid brittle shear failure and improves both the strength and the ductility compared to the unreinforced masonry (URM). Bernardini et al.(1997) have found that the 2200mm high, 1760mm long wall containing 0.12% vertical reinforcement was ductile. Davidson and Brammer (1996) have examined several reinforced masonry walls and concluded that the walls behave in a ductile manner sustaining cyclic loads even at the peak load level although the walls eventually fail in a diagonal shear, a mode normally considered brittle. Khattab and Drysdale (1993) have reported that increasing the amount of shear reinforcement can be beneficial to both the shear capacity and the ductility, although this cannot be achieved without proper detailing of reinforcement.

The effect of reinforcement on the reinforced masonry shear walls has so far been investigated for cases where the amount of reinforcement is relatively high and spacing of the grouted cores is limited to 800mm. PGRM shear walls allow for the spacing of reinforcement in the range of 800mm to 2000mm with no horizontal reinforcement except in the bond beam at the top of the walls. Interaction between the grouted cores and large unreinforced regions is quite complex and is not well understood. An experimental investigation on full-scale PGRM shear walls was carried out to examine their deformation characteristics; this paper reports the experimental tests, data and analysis.

For moderate earthquakes lightly reinforced masonry systems, such as the PGRM shear walls, are expected to be effective. However, due to lack of research, their behaviour is poorly understood and it is increasingly becoming clear that the design rules of these walls are highly nonconservative (Shrive et al (2009)). To provide an in-depth understanding and to contribute to the development of performance based design rules, a longer term, staged experimental work is being carried out; this paper reports a part of the work.

2. EXPERIMENTAL INVESTIGATION

Ten single leaf shear walls were constructed from hollow clay blocks and tested. The size of all walls was kept the same (2408mm high, 2870mm long and 150mm thick which corresponds to 9 blocks long and 28 blocks high). Blocks were of gross dimensions 310mm × 150mm × 76mm with two symmetrical voids of size 100mm × 80mm × 76mm to accommodate grouting and steel bars. Face shell width of the blocks was 35mm and the average ultimate compressive strength of blocks was 40.6MPa. A licensed mason of average workmanship built all the test walls. Each wall was reinforced with 4N12 vertical steel bars (normal ductility 12mm diameter steel bars of nominal yield strength 500MPa) providing a vertical reinforcement ratio of 0.1% (based on gross dimensions). In addition to this, a horizontal bond beam containing 4N16 bars was detailed at the top of the wall where the lateral concentrated load was applied horizontally. Where the horizontal load was reversed, a specifically designed cage system, due to pull from the actuator, pushed the bond beam from the opposite end. The horizontal steel was kept more than the vertical steel to specifically examine the effectiveness of the vertical steel to the inplane shear response of the walls. This design has also helped to maintain the area of vertical steel and the aspect ratio of wall constant for all walls with only the spacing between the bars a variable. As shear will be higher in the central region of the wall as shown in Figure 1, it was decided to measure the spacing with reference to the centre line of the walls as shown in Figure 2. Reinforcements were provided symmetric to the vertical centre line of the walls; this resulted in the definition of two spacing S_1 and S_2 as shown in Figure 2.

Only the cores containing reinforcement were grouted and all other hollow cores were left ungrouted. The ratio of the spacing (S_1/S_2) of the vertical reinforcement for PGRM walls (1, 1A), (2, 2A), (3, 3A), (4, 4A) and 5 was 0.05, 0.29, 0.42, 0.74 and 1.0 respectively. Walls 1A, 2A, 3A and 4A were tested under reversed cyclic loading and all other walls were tested under monotonic loading. Wall 6 was unreinforced masonry (URM) that contained no grouting (and hence reinforcement); spacing ratio does not apply to this wall. Design details of the tested walls are presented in Table 1.

A reinforced concrete base slab was specifically designed and constructed for each wall to allow fixing of the test walls to the strong floor. Masonry was laid in face shell bedding using a mortar bed of 10 mm thickness and was grouted seven days after construction. Mortar mix of 1 cement : 1 lime : 6 sand was used in the construction of all walls. A bond beam of size 2870mm × 150mm × 162mm reinforced with 4N16 bars was constructed at the top of each wall to distribute the vertical and horizontal loads. The heavy reinforcement ensured the integrity of the bond beam throughout the loading history. The walls were tested after 28 days of drip line moist curing. For each wall, three four blocks high stack bonded face shell bedded prisms, three grout cylinders and three mortar cubes were also cast and tested in compression on the day of testing of the walls. The mean compressive strengths of these constituents are provided in Table 1.

2.1 Test Setup and testing procedure

All walls were tested under a portal-frame testing rig as shown in Figure 3. The footing of each wall was bolted onto the strong floor to provide maximum fixity. A chain of rollers was placed on top of the bond beam along the full length of wall to allow its top surface to freely drift laterally under the load application. A spreader beam was then positioned on the top of the roller and a servo-controlled actuator of capacity 2000 kN was used to maintain the vertical load at constant level (0.5MPa) throughout the horizontal loading history at the top of the wall. A load cell was used to monitor the constancy of the vertical load throughout the testing. The spreader beam was simply resting on the top of the wall allowing it to rotate as a true cantilever. A 500 kN servo-controlled tension-compression hydraulic actuator was used to apply the horizontal displacement on the vertical face of the bond beam. The load, the displacement and the surface strain data were collected using the LABVIEW data acquisition program on a PC.

A total of 23 channels of loads, displacements and surface strains were measured using load cells, LVDTs and potentiometers as shown in Figure 4. Channels 0 and 1 were used for vertical and horizontal loads respectively. Channels 2 and 3 recorded the horizontal displacement of the bottom of the bond beam on either side of the thickness of the wall at the right end; channel 4 recorded the horizontal displacement of the centerline of the bond beam. Channel 5 recorded the bottom displacement bond beam at the left end of the wall (just below the horizontal actuator). Channels 6 and 7 monitored horizontal displacement of the toe and the heel of wall respectively. Channel 8 recorded the relative movement of the slab to the floor, channels 9, 10, 11 and 12 recorded the vertical displacement of the heel and toe of the wall on either side of the wall; channels 13-18 monitored surface strains at the center of the wall. The final four channels (19 – 22) measured the global diagonal deformation of the wall on both sides. Channel numbers shown in Figure 4 and separated by slash (/) refer to the instrument on the opposite sides of the wall.

Displacement data collected from channel 4 provided important information in developing the load – displacement response curve. Monotonic tests were performed first. From the load - displacement curves achieved under monotonic loading, the yield point, maximum load point and failure point for each wall were recognised and the displacement history (number of cycles and increments in displacement) was accordingly designed for the cyclic load tests. For both monotonic as well as cyclic load tests, the magnitude of displacement was increased until the cracks became wide open and the peak load dropped by at least 20% of the peak load or the walls exhibited deformation larger than 25mm (or ~>1% drift for the 2408mm high wall) due to the stroke limits of the actuator.

2.2 Loading History

The appropriateness of several load or displacement histories for testing walls and columns under cyclic loading have been examined by various researchers because post crack and post peak behaviour are load path dependent. Some loading histories found in the literature consisted of two phases (requiring a switching algorithm), namely load controlled history for pre-crack to pre-peak behaviour followed by displacement controlled history until failure of specimens. Other common histories include displacement cycles repeated two or more times at each amplitude; this method will require quite long time to complete the test if the displacement rate is not adjusted at the post-peak stage. The dispacement history adopted in the testing program is shown in Figure 5, where the horizontal displacement was cycled twice at each amplitude. The displacement rate was increased five fold during post peak stage to minimise testing time to a more manageable 4 hours and 30 minutes; the corresponding data sampling rate was also reduced five fold to minimise problems of data overloading that inhibit analysis at later stage.

3. RESULTS

3.1 Mode of Failure of PGRM shear walls

Ultimate crack patterns of the four PGRM shear walls (1A, 2A, 3A & 4A) tested under cyclic lateral loading are presented in Figures 6a – 6d respectively. For convenience, the location of the reinforcing bars is also shown. The effect of the spacing of the vertical reinforcement is clearly evident in these four walls. Walls 1A, 2A & 3A whose spacing of mid reinforcing bars was less than 2000mm ($S_1 < 2000$mm) exhibited a combination of “>” and “<” shaped diagonal cracks in the end panels with the central panel not showing significant damage. Wall 4A with $S_1 = 1000$mm exhibited “X” shaped cracking in the central panel typical of unreinforced walls and little to no observed damage in the edge panels typical of fully reinforced walls.

The ultimate cracking of four monotonically loaded PGRM walls (1, 2, 3 & 4) is shown in Figure 7a – 7d. Irrespective of the spacing of the reinforcement, central or edge, all walls cracked first at the centre and with the increase of the horizontal load, the cracks propagated towards diagonally opposite corners. Once the entire wall was split by the diagonal cracking, either the toe crushed (Figure 8) or the wall became laterally unstable forcing the testing to be stopped. The apparent insensitivity of the spacing of reinforcement to the mode of failure of PGRM walls under monotonic loading is in sharp contrast to the failure mode of the walls with the same spacing of reinforcing bars tested under cyclic loading. It appears that the progressive reverse loading damages the unreinforced panels more than the reinforced cores, where as under monotonic loading, the damage in masonry is more localised with sufficient energy to cause cracking through the grouted reinforced cores. To examine if a reverse-monotonic loading would make any difference to the failure mode, the monotonically loaded and damaged walls were ‘pulled’ under reverse monotonic loading. The crack patterns of the walls are shown in Figure 9. Walls 1, 2, and 3 exhibited cracking different to wall 4. In walls 1 – 3, whilst the forward monotonic loading (on undamaged walls) caused localised unidirectional diagonal cracking, the reversed monotonic loading (on pre-damaged walls) induced cracks distributed much wider resembling to “>” and “<” shapes. Although the cracks due to forward loading closed initially, after the formation of cracks due to reversed loading, the forward direction cracks re-opened showing dilatancy. The spacing of reinforced cores appears to have more influence on walls subjected to progressive cyclic damage or those walls pre-damaged due to any previous event than the monotonically loaded walls. Wall 4, however, exhibited localised damage with the final failure showing "X" cracking typical of URM or fully reinforced walls.

3.2 Mode of Failure of URM shear walls

One URM wall (Wall 6) and a wall with only the vertical edges reinforced (wall 5), with the spacing of reinforcement 2700mm or, S_1/S_2 = 1.0 were also constructed and tested. These two walls were only tested under monotonic loading (forward and reversed). The failure of these two walls is shown in Figure 10. The failure planes (under the forward and revered loading) are highly localised with the typical "X" pattern for URM. For all practical purposes these two walls were considered as unreinforced.

3.3 Ultimate Load

The maximum lateral load of each of the ten walls tested is listed in Table 1 along with the strength of the grouted masonry determined from the four high stack bonded, grouted prisms. Also given in the table are the strength of the mortar and grout. As can be seen, even though the blocks were of consistent quality (and of high strength, viz, 40MPa grade), the mortar and grout and hence the grouted masonry strengths varied significantly. The lateral strengths of the walls were therefore normalised using 'nominal' shear capacities determined as a multiple of the gross area of the horizontal bed joint ($A_h = 430,500mm^2$) and a 'nominal' shear strength, which was related using an empirical relationship (Shing et al., 1990b) to square root of the mean compressive strength of the masonry ($0.22 \times \sqrt{f_m^m}$ (in MPa)). Therefore, the normalised shear capacity ($\overline{V}$) for each wall was determined using Eqn. 1 and is presented in Table 1.

$$\bar{V} = \frac{V}{\left(0.22\sqrt{f_m^m}\right)A_h} \qquad (1)$$

In contrast to the observations of the failure mode in which there existed a distinct difference between the cyclic and monotonic loaded panel, the normalised capacities did not show major differences based on the type of loading. In the absence of major difference, it was decided to average the failure loads of the 'pairs' of the walls (eg., 1 and 1A). The average normalised load has shown a distinct difference between the five groups of 10 walls. Whilst the walls with spacing of reinforcement < 2000mm (types 1, 2 & 3) exhibited, normalised load of ≥ 0.50, the walls whose reinforcement is spaced ≥ 2000mm (types 4, 5 & 6) exhibited normalised load of ≤ 0.35. The mean of the normalised ultimate load for the first 'group' of walls (spacing < 2000mm) was 0.50 (COV of 4.7%) and for the second 'group' (spacing ≥ 2000mm, including URM) the mean was 0.34 (COV of 12.1%). Therefore, it is evident that it's not only the material constituents but more importantly the distance of the intermediate cores from the centre of the wall that affects the ultimate load capacity of walls.

4. DEFORMATION CHARACTERISTICS

4.1 Load – Displacement response

Relative movement of the base slab and the wall (mostly negligible), were subtracted from the displacement of the bond beam to provide true representation of the load – displacement behaviour of walls. All walls gradually deformed until the peak load was attained and then exhibited softening. Under monotonic loading, upon 20% loss in the peak shear capacity or excessive displacement (~ 25mm), the walls were pulled in the reverse direction until they either failed or exhibited excessive displacement (~ 25mm) to achieve a large push and pull cycle for comparison with both monotonic and cyclic envelope curves.

Typical normalised test data on horizontal load - displacement of the PGRM shear walls subjected to cyclic and monotonic loading are presented in Figures 11 and 12 respectively. The normalised cyclic load – displacement data (Figure 11) also contains an envelope curve. The normalised monotonic load – displacement data (Figure 12) also contains smoothed idealised curve.

The hysteretic load-displacement curves are generally fat indicating good cyclic load performance without severe localisation; this is consistent with the failure modes observed from these cyclically loaded walls.

Smoothed, idealised and envelope load – displacement curves for walls 1 & 1A, 2 & 2A, 3 & 3A and 4 & 4A under monotonic and cyclic loading respectively are shown in Figures 13a – 13d. The envelope curves of cyclically loaded hysteretic curves exhibit faster degradation rates when compared with the smoothed, idealised monotonic load-displacement curves of PGRM walls.

4.2 Displacement Ductility

Ductile walls safely resist loads with increasing displacement beyond peak loads. Displacement ductility (μ_d) is defined as the ratio of the ultimate displacement (D_u) to the yield displacement (D_y) as shown in Eqn. 2.

$$\mu_d = \frac{D_u}{D_y} \qquad (2)$$

As the identification of the yield and the ultimate displacements was not straightforward with the normalised, smoothed load – displacement response curves, a method used for the determination of the displacement ductility of reinforced concrete columns (Saatcioglu, 1991) was used to identify the yield and ultimate displacements. This method idealises the curve such that the yield point is set to divide the pre yield and post yield – pre peak energy equally as shown in Figure 14. The ultimate displacement was taken corresponding to 20% reduction in the peak load.

The displacement ductility of the cyclically tested walls (1A, 2A, 3A and 4A) was worked out as 5.7, 4.7, 5.3 and 4.4 respectively. The post peak response of these walls subjected to severe cyclic lateral loading appears quite good from a practical design perspective as it is generally believed (Watson and Park, 1994) that displacement ductility of 4.0 or more is sufficient even for severe earthquake regions. The walls remained in tact and performed well during handling from the test lab to the storage yard, surviving overhead crane lifting, truck transport and crane unloading without any concern to safety. However, no conclusive generalised statements can be made on the viability of the PGRM systems for seismic applications due to the limited number of the tests carried out. Furthermore, the walls have had mixed spacing of reinforced cores with some cores spaced less than 800mm. In practice designers tend to keep the spacing of reinforcement constant and generally 800mm or more (up to 2000mm) in high wind areas of Australia; although variation in spacing occurs near openings and wall ends. In spite of these factors, it appears PGRM is a viable earthquake resisting system, particularly in areas of moderate, intraplate type seismicity characterised by low magnitude, short period ground shaking with no significant after shakes.

The monotonically tested walls (1, 2, 3 and 4) exhibited more apparent ductile behaviour as seen in Figures 13a – 13d; their displacement ductility values were worked out to be 7.5, 8.3, 8.9 and 9.4 respectively. These values are significantly larger than the corresponding ductility values determined from the cyclically loaded walls. The observed characteristics of large plateaus of post peak response for these walls, corresponding to the sliding of the walls at discrete bed joints, are the major reason for this apparent ductility enhancement. Therefore, to characterise the ductile behaviour of PGRM walls, it appears that full cyclic load testing is required. The monotonic load tests carried out for the full post peak range can not characterise the seismic response of PGRM wall systems as evidenced from the failure modes and the ductility values determined. This is in contrast to the generalised conclusions in reinforced concrete columns (for example, Watson and Park, 1994) that the envelope curves of the cyclic test response curves can easily be obtained from monotonic tests. As reinforced concrete columns are generally closely reinforced (often with reinforcement ratio 1% ~ 4%), more uniform damage in concrete occurs irrespective of the load being cyclic or monotonic and hence this conclusion from the reinforced concrete research. Whilst fully reinforced masonry (for example, spacing of both the horizontal and vertical reinforcement limited to 800mm maximum) might exhibit similarity to the reinforced concrete, PGRM appears certainly not to conform to this finding and its behaviour under cyclic and monotonic loading is dissimilar and must be treated accordingly.

4.3 Stiffness Degradation

Stiffness degradation (C_K) of structural elements is defined by the rate of reduction in stiffness after occurrence of the yield stiffness. Mathematically it is expressed as the ratio of the secant modulus at any particular load stage (K) to the secant modulus at the yield load (K_o) of the load – displacement curve as shown in eqn 3.

$$C_k = \frac{K}{K_o} \tag{3}$$

'K_o' is the slope of a line passing through the origin to the yield point and 'K' is the slope of a line passing through origin to any other point on the load - displacement curve where the stiffness degradation value is determined. Normalised load – displacement curves were used for the determination of the stiffness degradation. Values of stiffness degradation were evaluated from envelope curves of the cyclically loaded walls at different values of lateral displacement and are plotted in Figure 15. Walls 1A and 2A exhibit high initial stiffnesses compared to that of wall 3A. Wall 4A exhibited only 50% of the initial stiffness of the wall 1A. In spite of the differences in the initial stiffness, the stiffness of all cyclically loaded walls degraded at a similar rate.

5. CONCLUSIONS

The response of partially grouted and reinforced masonry (PGRM) cantilever shear walls containing 0.1% of vertical reinforcement and large unreinforced wall panels in between the reinforced cores was determined from ten full scale experiments under monotonic and cyclic loading. The walls were constructed from hollow clay blocks; the dimension of the walls (2870mm long × 2408mm high × 150mm thick) and the amount of vertical (4N12) and horizontal (4N16) reinforcements were kept constant for nine of the ten walls. The spacing of the reinforcement was the only design variable examined in this investigation. Two of the 4N12 bars were placed one each at the vertical edges; the other 2N12 bars were located at varying distances to the edge reinforcements. Spacing of the bars was measured from the vertical symmetric axis; resulting in two spacing S_1 and S_2. Five spacing ratios were considered (0.05, 0.29, 0.42, 0.74 & 1.0) for the nine reinforced walls; the tenth wall was unreinforced. Spacing ratios 0.74 and 1.0 correspond to maximum spacing of $\geq$2000mm. The major finding of this research was that unlike reinforced concrete members, ductility can not be reliably measured from the monotonic load tests on PGRM walls. For true seismic response studies, cyclic loading tests of PGRM shear walls are absolutely essential.

From the tests the following specific conclusions emerged:

1. Cyclic loading caused widespread damage in the PGRM shear walls containing reinforcement spacing < 2000mm and discrete, localised diagonal cracking for walls with maximum spacing of reinforcement of ≥ 2000mm;
2. Monotonic loading caused localised, discrete diagonal cracking in all PGRM walls irrespective of the spacing of reinforcement;
3. Reversed monotonic loading of pre-damaged walls caused wide spread damage similar to that of the cyclically loaded walls;
4. Ultimate inplane load of walls was affected by the strength of constituent masonry and grout;
5. The lateral load – displacement hysteresis of cyclically loaded walls was fat indicating good energy dissipation;
6. Displacement ductility of cyclically loaded walls was significantly lower than the corresponding monotonically loaded walls;
7. Irrespective of the difference in initial stiffness, the stiffness of all walls tested as part of the investigation reported in this paper degraded at a similar rate.

The above conclusions are limited to the parameters of the tests carried out and can not be generalised. The observed response of the PGRM shear walls, especially the failure mode and ductility, provide encouragement for further examination of this system of construction, especially for moderate seismic zones. Further research is warranted as to define whether the PGRM shear wall is a (i) unreinforced masonry system with some embedded reinforced grouted cores; or (ii) reinforced masonry system with wider spacing of reinforcements; or (iii) mixed or hybrid system made from reinforced and unreinforced masonry. Fundamental experimental investigations with well defined, non-contentious boundary conditions that would keep the PGRM system under uniform state of stress would be required to exactly define the nature of the PGRM system.

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NOTATIONS

C_K Stiffness degradation factor

D_u Displacement at ultimate lateral load

D_y Displacement at yield lateral load

f_m^m Mean compressive strength of masonry

PGRM Partially grouted reinforced masonry

URM Unreinforced masonry

V Measured shear capacity of wall

$\overline{V}$ Non dimensional shear capacity

μ_d Displacement ductility

TABLES AND FIGURES

List of Tables:

List of Figures:

Table 1–Design details and ultimate load of shear walls

Wall	Average compressive strength of Constituent Materials (MPa)			Spacing of vertical bars (mm)			Failure Load		
	Mortar Cubes[1]	Grout Cylinder[2]	Masonry Prism[3]	S_1 (mm)	S_2 (mm)	Spacing Ratio S_1/S_2	Actual (kN)	Normalised	Group Average
1	10.3	22.0	12.5	70	1350	0.05	179.1	0.51	0.50
1A	10.7	32.1	11.9	70	1350	0.05	191.0	0.49	
2	9.7	29.5	13.7	390	1350	0.29	180.3	0.48	0.50
2A	9.0	31.5	14.4	390	1350	0.29	160.5	0.52	
3	5.3	34.8	15.5	570	1350	0.42	167.4	0.53	0.50
3A	6.9	39.7	15.7	570	1350	0.42	195.0	0.47	
4	5.0	36.5	18.4	1000	1350	0.74	130.0	0.31	0.35
4A	6.4	34.7	18.1	1000	1350	0.74	159.8	0.39	
5	10.0	39.7	20.1	1350	1350	1.00	141.1	0.34	0.32
URM	4.0	-	15.7			-	112.6	0.30	

1. 50 mm × 50 mm × 50 mm
2. 100 mm dia. × 200 mm high
3. Four high stack bonded prisms with grout

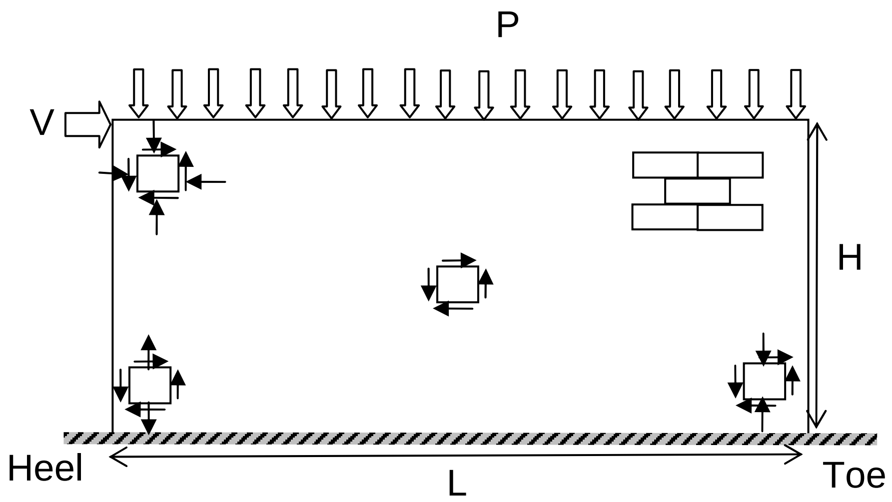


Figure 1 – Stress distribution in shear walls

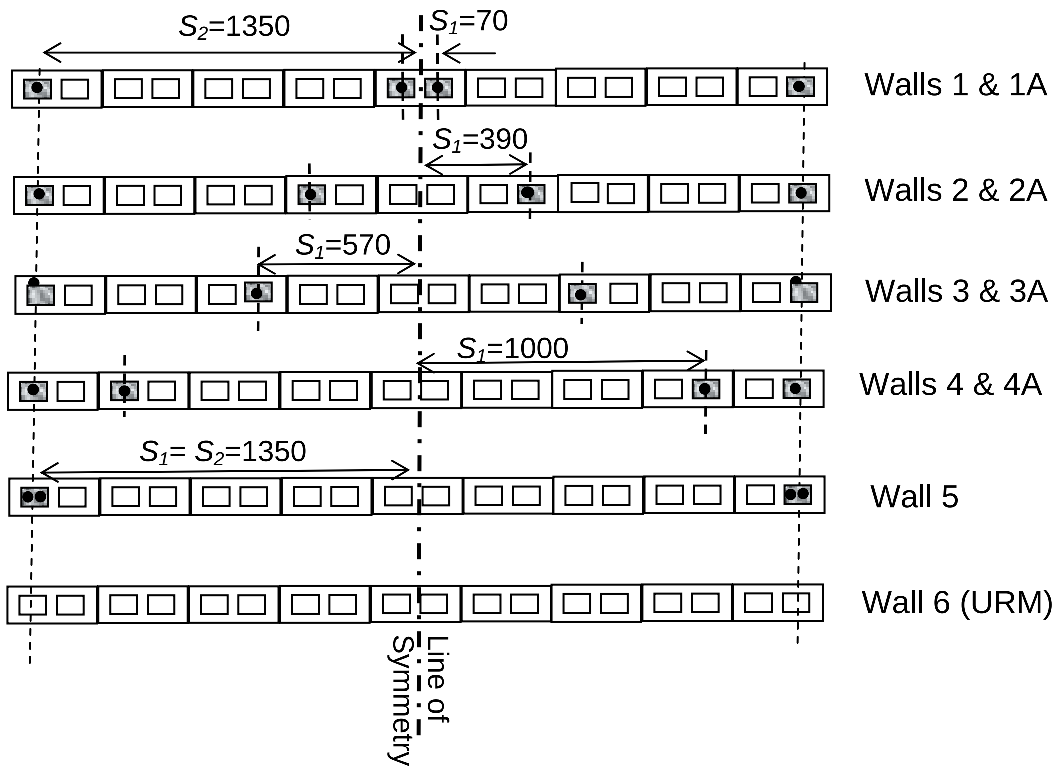


Figure 2 – Test Wall Details (Plan View)

Figure 3 - Test Rig

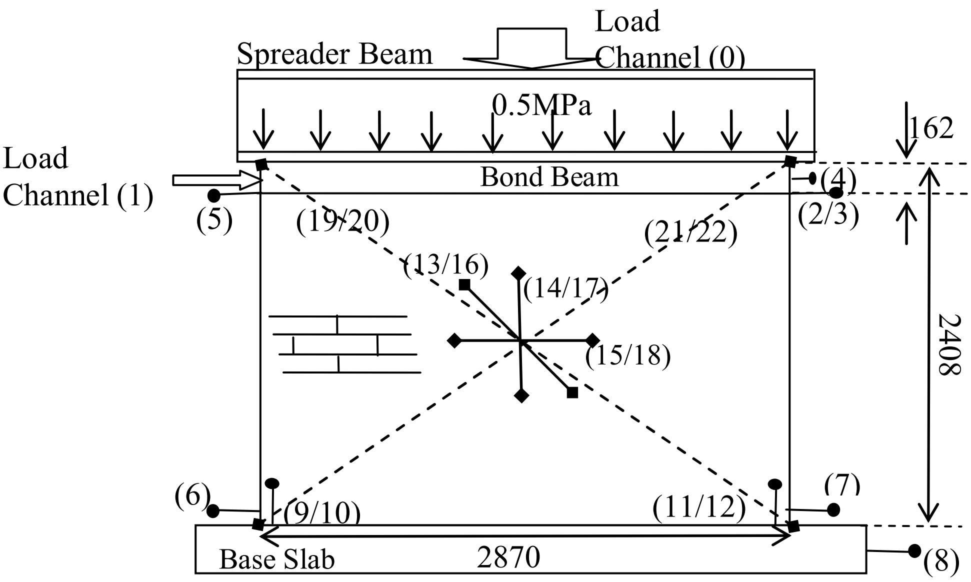


Figure 4 - Instrumentation and test setup

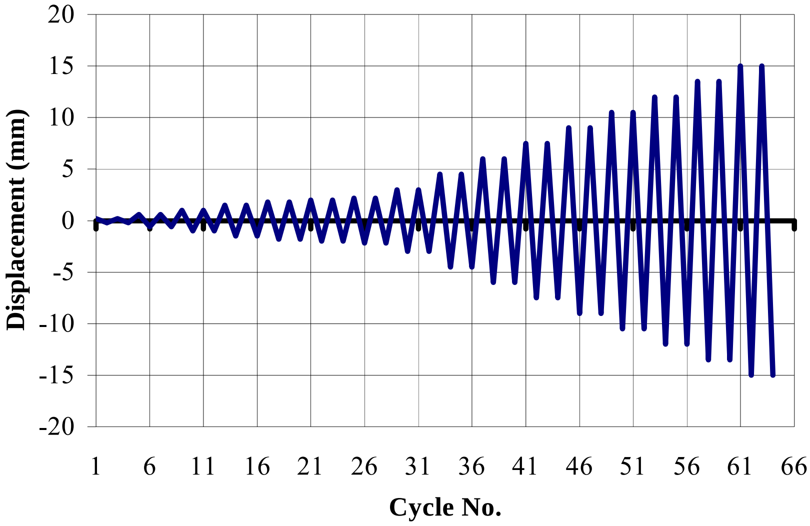
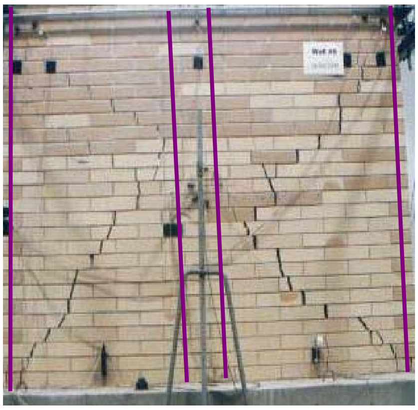
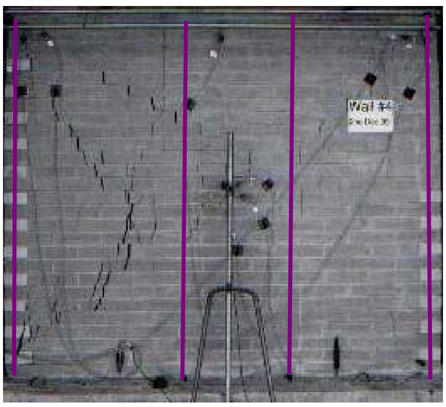


Figure 5 – Lateral loading (displacement control) history for cyclic loading

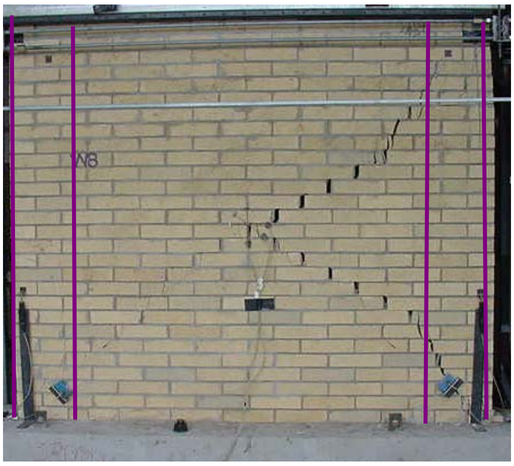


(a) Wall 1A



(b) Wall 2A

(c) – Wall 3A



(d) – Wall 4A

Figure 6 – Final crack patterns of PGRM shear walls tested under cyclic loading

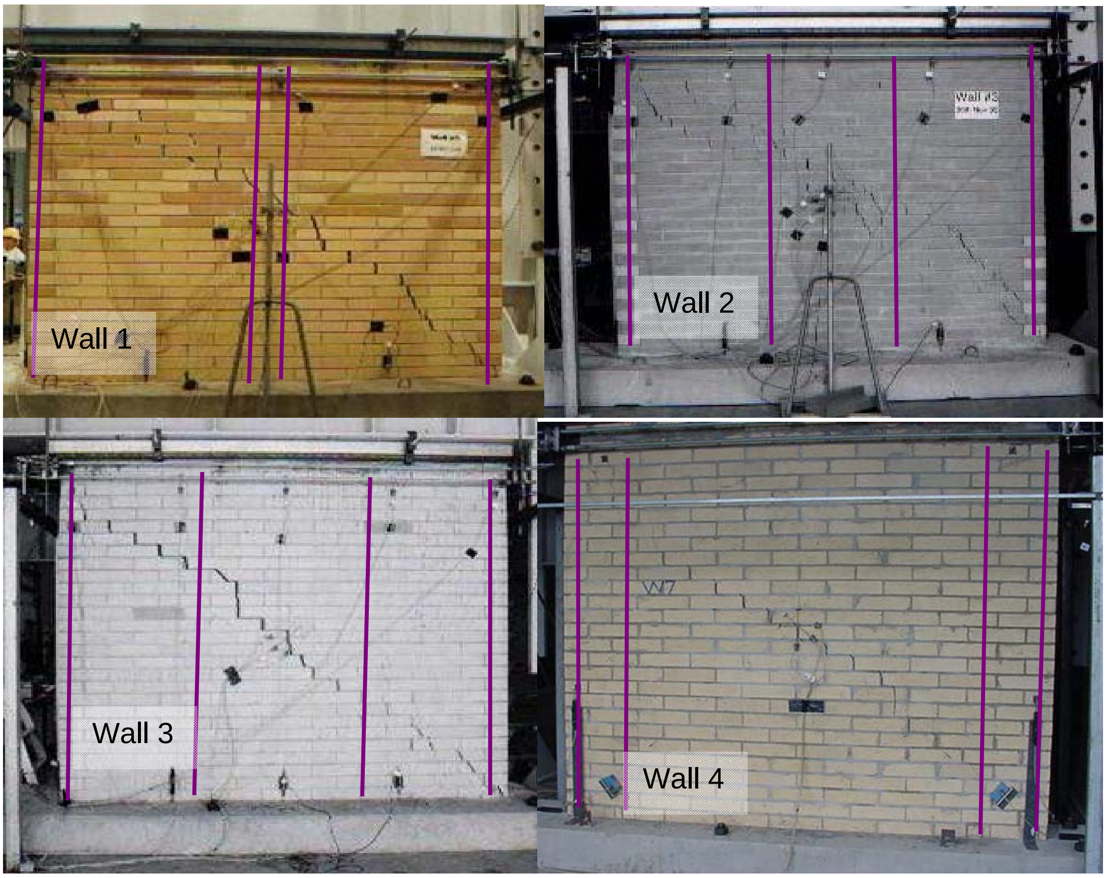


Figure 7 – Final crack patterns of PGRM walls tested under Monotonic loading

Figure 8 – Compression Toe Crushing under Monotonic Loading

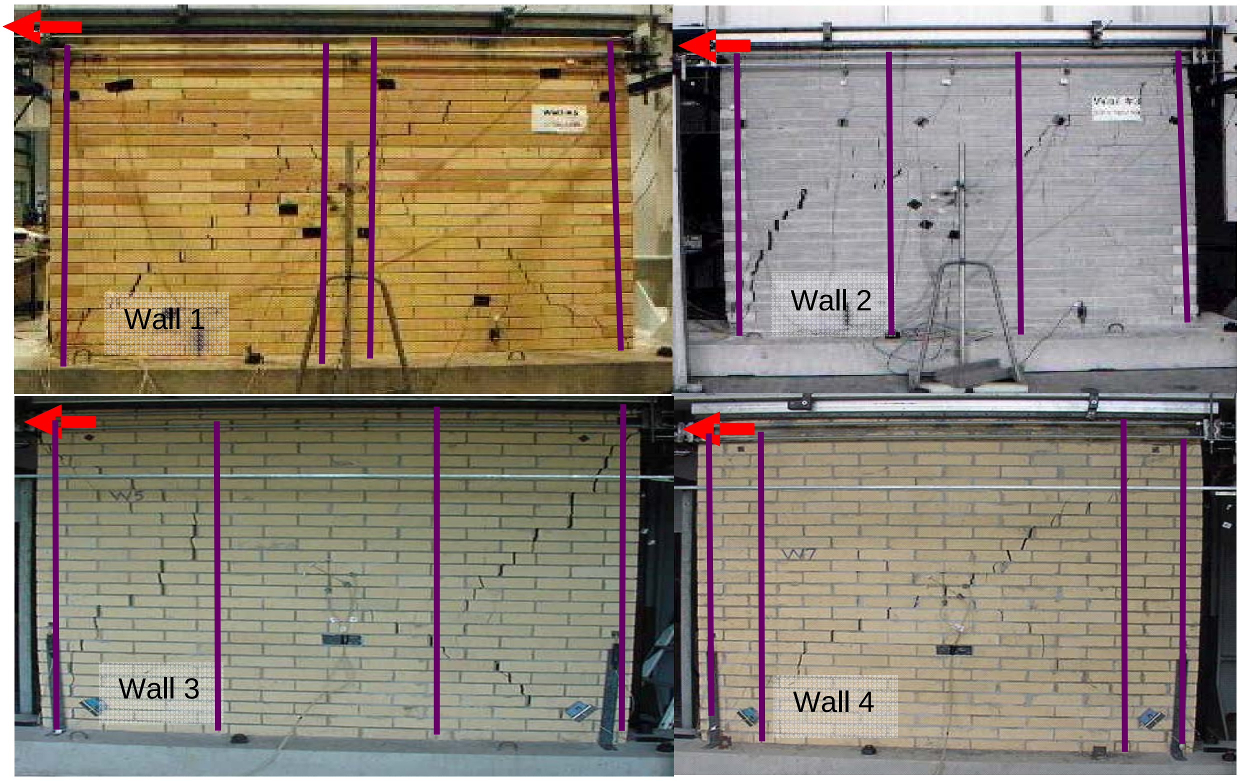
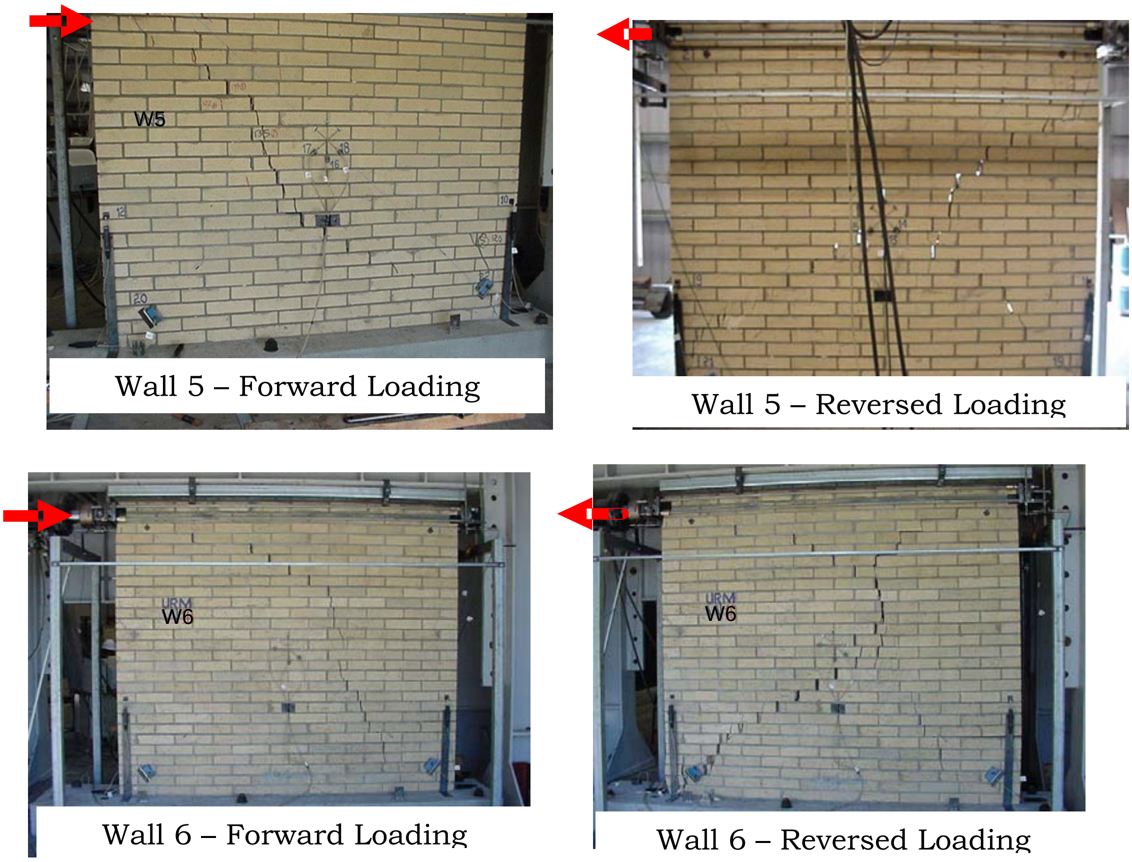


Figure 9 – Final crack patterns of PGRM walls tested under reversed monotonic loading



Wall 5 – Forward Loading

Wall 5 – Reversed Loading

Wall 6 – Forward Loading

Wall 6 – Reversed Loading

Figure 10 – Final crack patterns of wall 5 and URM wall tested under reversed monotonic loading

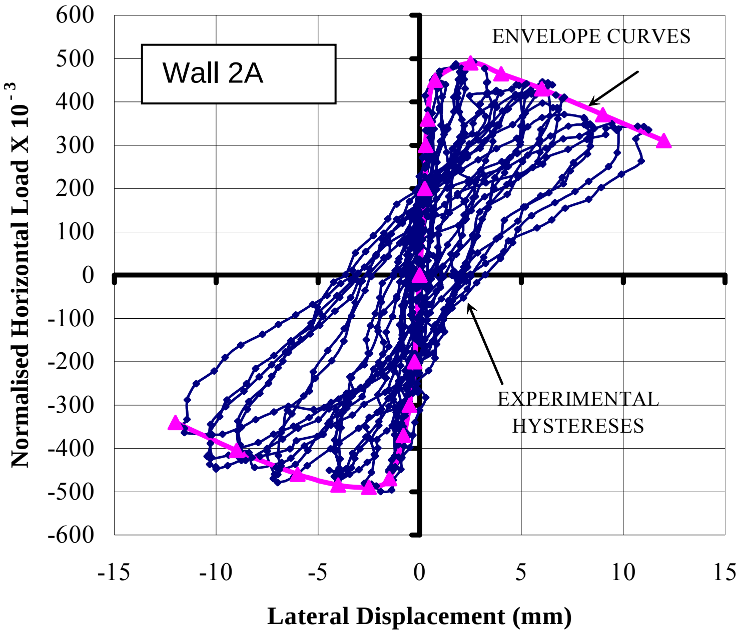


Figure 11 - Typical normalised load-displacement curves of PGRM walls under cyclic loading

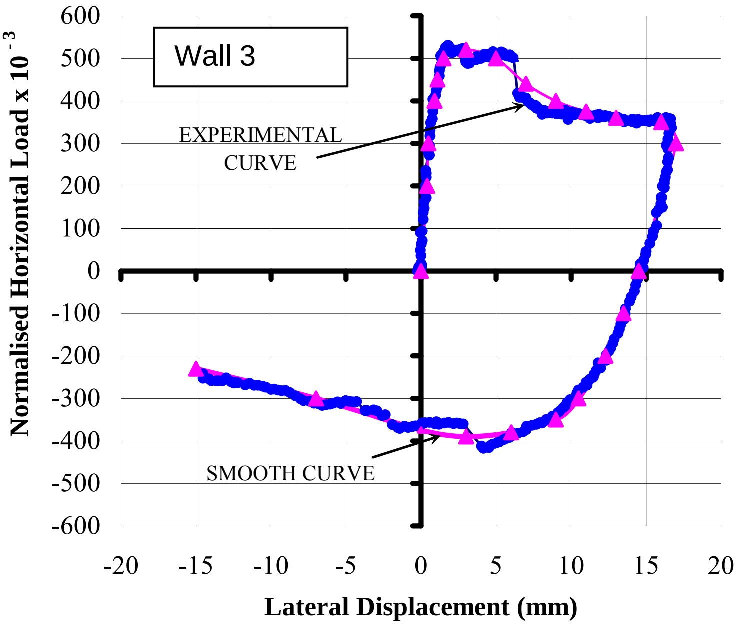
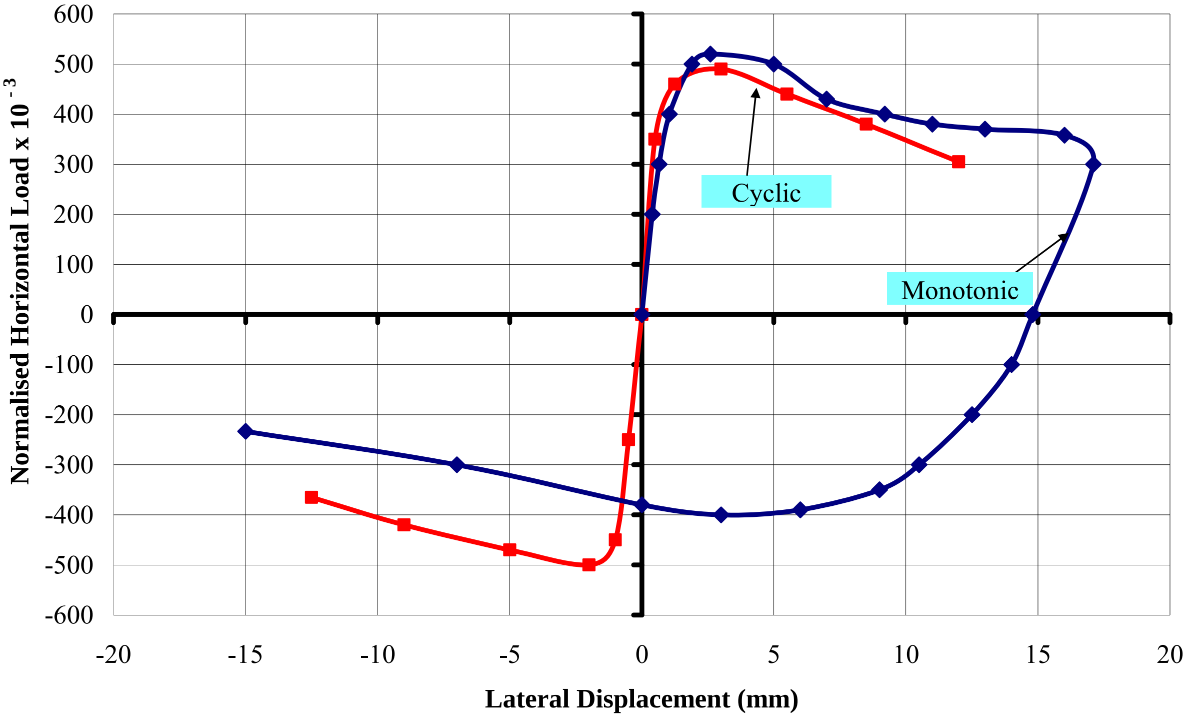
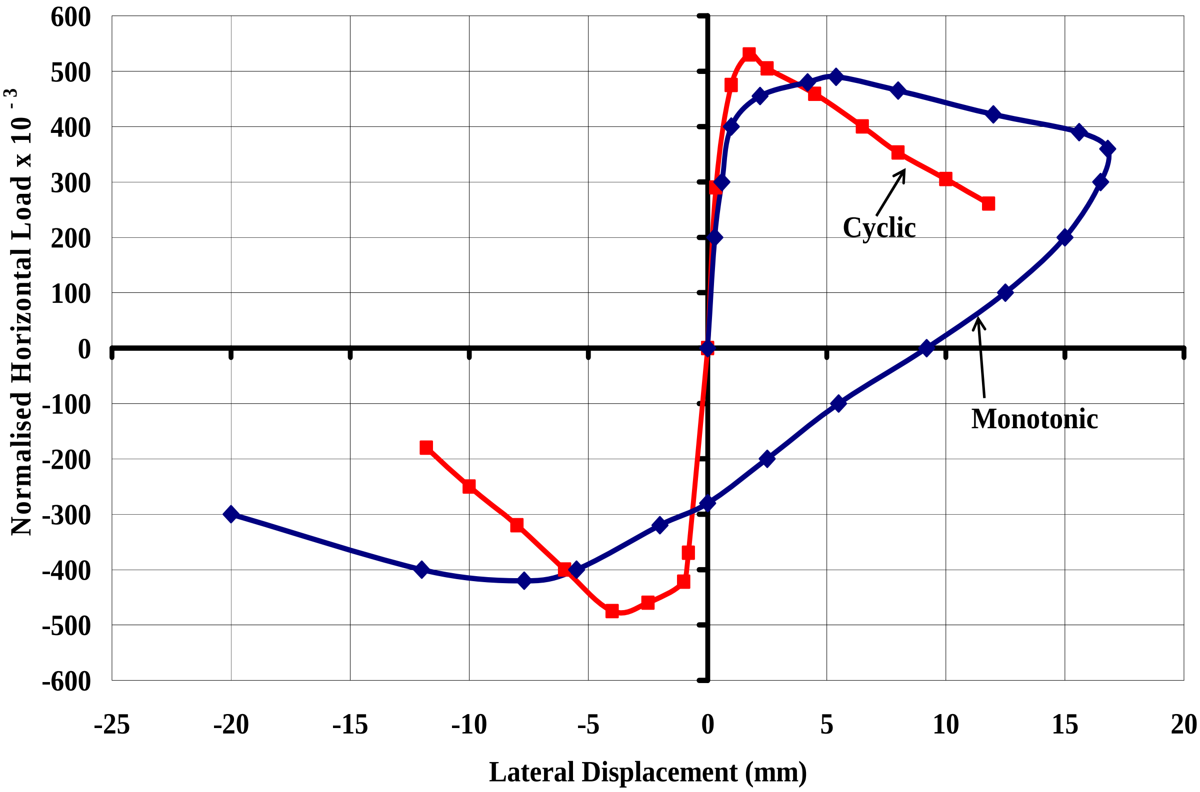


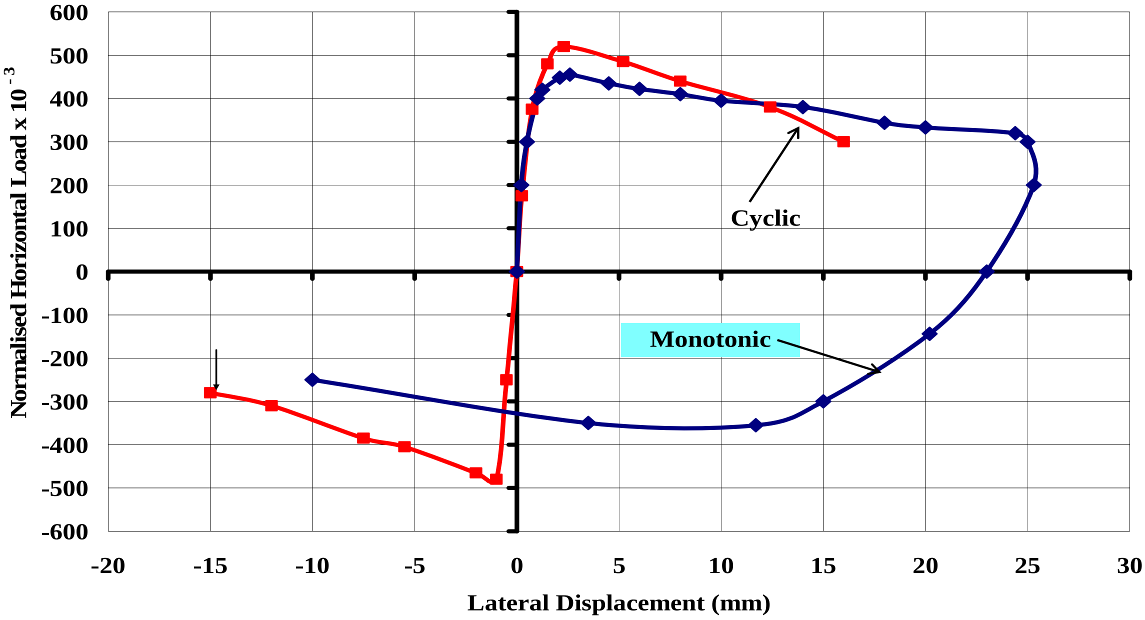
Fig. 12 - Smoothed load –displacement curve of a PGRM wall under monotonic loading



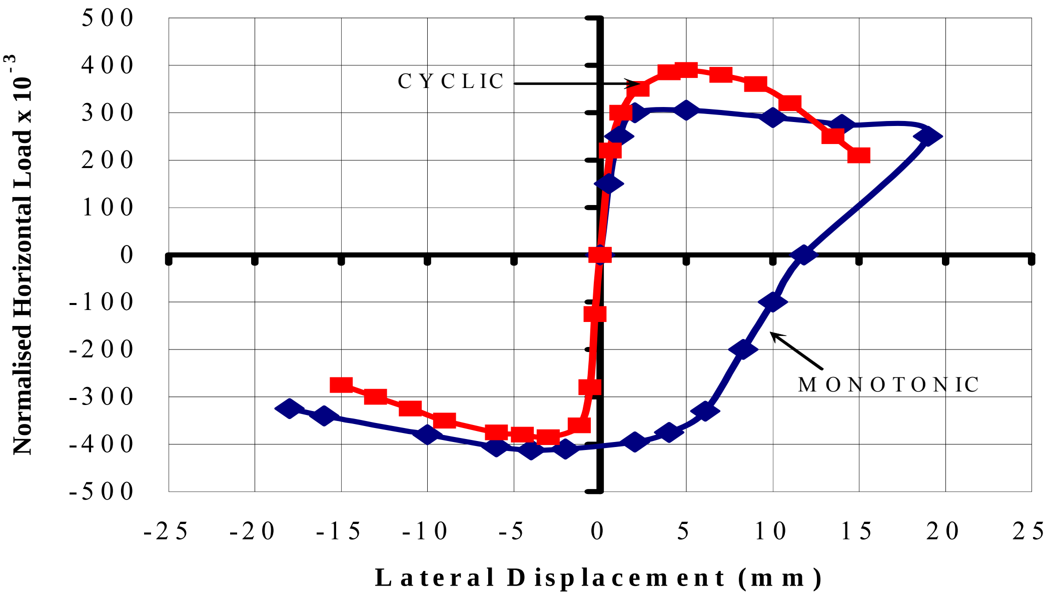
(a) Walls 1 & 1A



(b) Walls 2 & 2A



(c) Walls 3 & 3A



(d) Walls 4 & 4A

Figure 13 – Monotonic and cyclic envelope curves

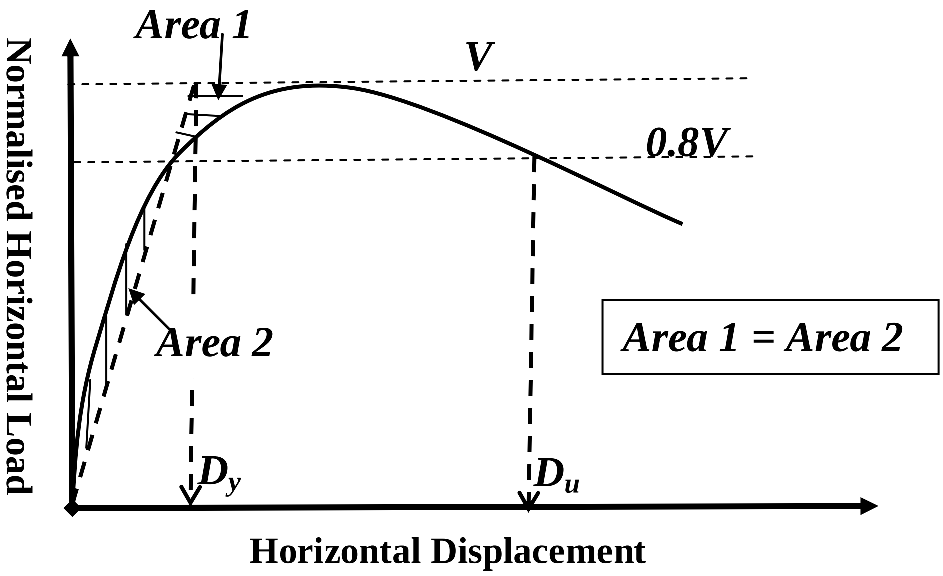


Figure 14 –Definition of yield and ultimate dispacements

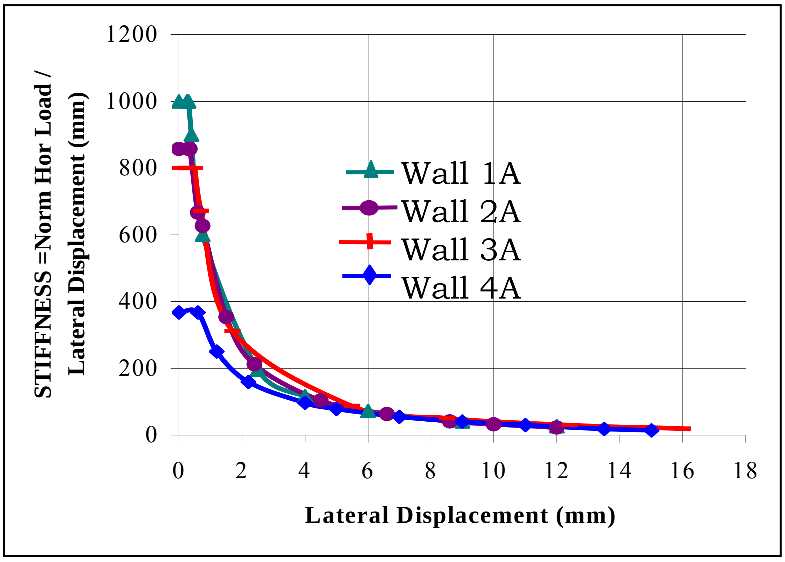


Fig. 15 – Stiffness Degradation of PGRM walls tested under cyclic loading