



BEHAVIOUR OF WIDE SPACED REINFORCED MASONRY WALLS UNDER INPLANE CYCLIC LOADING

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Abstract

Wide spaced reinforced masonry (WSRM) walls subjected to inplane loading exhibit complex structural behaviour that is not well understood in spite of a few years of investigation in several parts of the world. WSRM walls described in this paper contain only vertical reinforcement with spacing ranging from 800mm to 2000mm. With a view to examining the effect of the spacing ratio (defined as the ratio of spacing of vertical reinforcement at the central region to the end region of the wall) of vertical reinforcement, several full-scale, single leaf, clay block WSRM shear walls were constructed and tested under inplane horizontal and vertical loading at Central Queensland University. This paper presents the failure and deformation characteristics of the WSRM shear walls tested under monotonic and cyclic loading. It has been shown that WSRM shear walls possess a good level of ductility and do not exhibit rapid stiffness degradation until the ultimate stage.

Key Words

Wide Spaced Reinforced Masonry; Shear Walls; Lateral Loading; Ductility

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 Australia is subjected to intraplate type earthquakes characterised by moderate intensity, we have examined a lightly reinforced wall system, known as the wide spaced reinforced masonry (WSRM) in this paper. The WSRM walls are constructed with vertical reinforcement at spacing ranging from 800mm to 2000mm; and horizontal bond beam placed only at the top of the wall. Similar systems in the USA (Fattal, 1993 and Shing et al., 1993) contain bond beams at mid height and in New Zealand (Priestley, 1977 and Ingham et al., 2001) contain vertical and horizontal reinforcements at maximum spacing of 800mm. As the behaviour of this pattern of WSRM shear walls is not well understood, we have carried out an experimental investigation on ten full-

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scale walls at the Heavy Testing Laboratory, Central Queensland University. Eight of them satisfy the requirement of reinforcement spacing for the WSRM walls as specified in the Australian masonry code, AS3700(2001), the ninth one exceeded the maximum spacing requirement and the tenth wall was unreinforced masonry (URM).

2 Experimental program

2.1 The walls – Design & Construction

The reinforcements in all the walls (except the URM) were arranged symmetric to their respective vertical mid section. Two reinforcing steel bars of 12mm diameter (Y12) each were placed on either side of the vertical mid section of the wall that provides a total of 4Y12 bars or a total steel reinforcement of 440mm^2 (percent of steel $\approx 0.14\%$). The wall design details are contained in Table 1.

Table 1 Details of Shear Wall Design

Wall No.	Average Compressive Strength of Constituent Materials (MPa)			Spacing of Vertical Bars (mm)		
	Mortar Cubes (50mm)	Grout Cylinder (100mm x 200mm)	Masonry Prism (4 high stack bonded)	Middle	Ends	Spacing Ratio Middle / End
WSRM 1	10.3	22.0	12.5	140	1280	0.11
WSRM 2	10.7	32.1	11.9	140	1280	0.11
WSRM 3	9.7	29.5	13.7	780	960	0.81
WSRM 4	9.0	31.5	14.4	780	960	0.81
WSRM 5	5.3	34.8	16.9	1140	780	1.46
WSRM 6	6.9	39.7	17.6	1140	780	1.46
WSRM 7	5.0	36.5	18.4	2000	350	5.71
WSRM 8	6.4	34.7	18.1	2000	350	5.71
WSRM 9	10.0	39.7	20.1	2685	0	∞
URM	4.0	-	15.6	-	-	-

A mason of average workmanship built all the test walls. 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 mortar bed of 10mm thickness and was grouted after seven days of construction. A bond beam of size $2870\text{mm} \times 150\text{mm} \times 172\text{mm}$ reinforced with 4Y16 bars was constructed at the top of each wall to distribute the vertical and horizontal loads. The heavy reinforcement has ensured the integrity of the bond beam throughout the loading history. The walls were tested after 28 days of air curing.

2.2 Testing of Walls

All walls were tested under a portal-frame testing rig as shown in Fig. 1. The footing of the walls was bolted into the strong floor to provide maximum fixity. A purpose made roller was placed on top of the bond beam along its full length to allow the top surface of the wall to freely drift laterally. A spreader beam was then positioned on the top of the roller and a servo-controlled actuator of capacity 2000kN was used to apply the vertical load (0.5MPa) at the top of the wall. A 500kN servo-controlled tension-compression hydraulic actuator was used to apply the horizontal displacement on the vertical face of the bond beam. Under monotonic loading, the 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 (due to limitation of test set-up). The history of cyclic loading was characterised by cycling the amplitudes of displacement twice for each amplitude prior to increasing. Initially the increase in amplitude of displacement was kept small until the onset of yielding of the wall, and then the larger increments were made until the failure of the wall. The load, the displacement and the surface strain data were collected using the LABVIEW data acquisition program on a PC.



Figure 1 Detail of test arrangement

3 Failure of shear walls

3.1 Mode of failure

Regardless of the spacing of the vertical reinforcement, in all WSRM walls first cracks appeared at the centre and with the increase of the horizontal load, the cracks propagated towards diagonally opposite corners until final failure occurred. A typical failure of shear wall subjected to cyclic loading is shown in Fig. 2. In general, all walls showed much wider (up to 25mm) diagonal cracks at the ultimate stage. No damage of any significance was observed in the remaining regions of the walls.



Figure 2 Failure of a cyclically loaded WSRM shear wall

Under monotonic loading, after the crack had formed along the full diagonal, a kinetic mechanism developed so that the block above the crack slid relative to the lower block, thus causing toe crushing resulting in large wall displacements. Cyclically loaded specimens exhibited gradual degradation of load capacity with no apparent kinetic mechanism. Placement of vertical bars did not significantly affect the size of the final cracks in both monotonic and cyclic loading cases.

The failure mechanism of the URM wall tested in this investigation was consistent with those reported in the literature and is different from the WSRM walls. Vertical reinforcing bars in the grouted cores of the WSRM walls transmitted the horizontal load towards the toe diagonally, whereas the URM wall transmitted the horizontal load to its toe through a rocking mechanism. The occurrence of relatively larger toe deformations of the URM wall indicated rocking and sliding mechanism (relative to the WSRM and the URM walls). It is, therefore, evident that the widely spaced vertical reinforcing bars provide dowel action in resisting the shear load when masonry walls are monotonically loaded (URM was not tested under cyclic loading).

3.2 Shear capacity

The maximum load attained by each wall is presented in Table 2. As the strength of the constituent materials differ significantly in each wall (Table 1), to get a true representation of the shear capacity of the walls were normalised as shown in Eq. (1). The coefficient 0.22 is used as the constant of conversion from compressive to shear strength of masonry.

$$\text{Normalised Load} = \frac{\text{Experimental Load}}{0.22 \times \sqrt{\text{Masonry Compressive Strength} \times \text{Area of wall}}} \quad (1)$$

Table 2 Shear Capacity of Shear Walls

Wall	#1	#2	#3	#4	#5	#6	#7	#8	#9	URM
Spacing Ratio	0.11	0.11	0.81	0.81	1.46	1.46	5.71	5.71	∞	-
Capacity (kN)	179.6	200.0	183.2	160.9	155.7	190.5	125.6	164.4	144.8	107.5
Normalised Capacity	0.54	0.61	0.52	0.45	0.40	0.48	0.31	0.41	0.34	0.29

The normalised shear capacity of the shear walls shown in Table 2 appears to be affected by the spacing ratio. For example, the normalised shear capacity for the monotonically loaded shear walls whose spacing ratio increased from 0.11 to 5.71 (ie, 0.11, 0.81, 1.46 & 5.71) has shown gradual reduction, namely 0.54, 0.52, 0.40 and 0.31. Although not quite as consistent, the four cyclically loaded walls have shown a gradual reduction in normalised shear capacity (0.61, 0.45, 0.48 and 0.34). It should be noted that although all these four walls contained the same amount of reinforcement ($\approx 0.14\%$), their normalised shear capacity varied by 74% in the case of monotonically loaded walls and 79% in the case of cyclically loaded walls. It therefore appears that the reinforcement ratio is not only a good measure to estimate the shear capacity of WSRM walls containing non-uniform spacing of reinforcement, but also is an important parameter that could be carefully used in maximising the design capacity of WSRM shear walls.

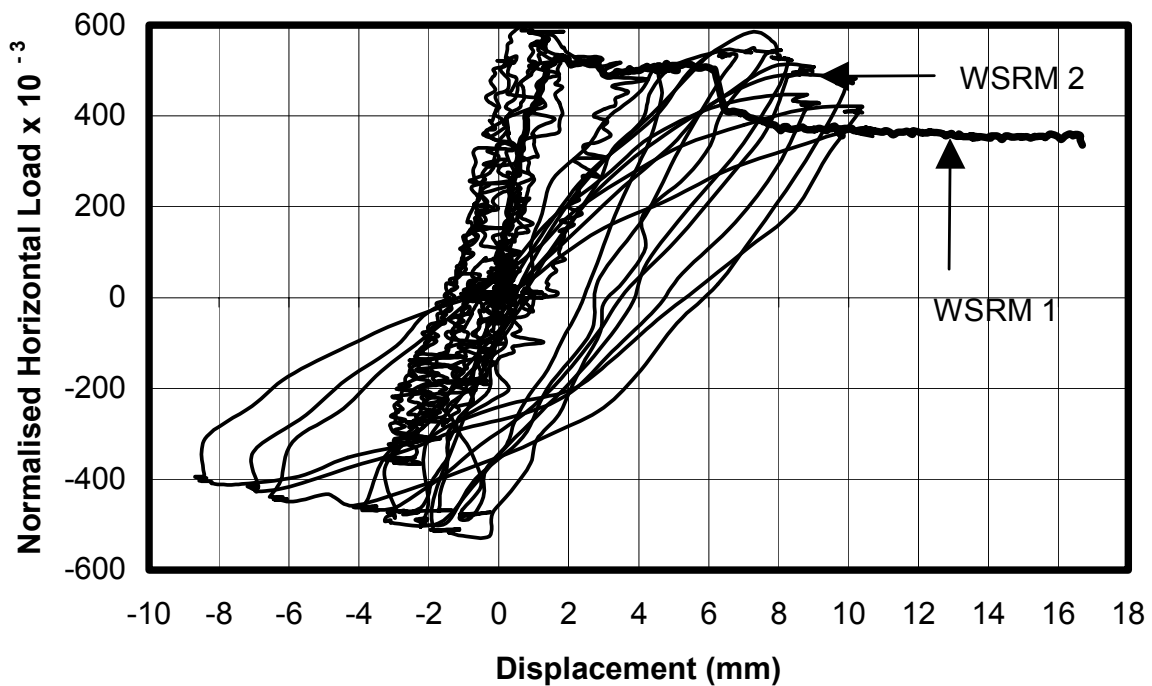
Pair-wise comparison of the normalised shear-capacities reveals that the cyclic loading generally increases the shear capacity of the WSRM walls (relative to the monotonically loaded walls). All pairs except the pair whose spacing ratio was 0.81 (walls 3 and 4) have exhibited this phenomenon.

When the spacing of the reinforcement is very high (ie, when the WSRM wall tends to be URM), the normalised shear capacity registers lower values. URM walls exhibited the lowest normalised shear capacity (0.29).

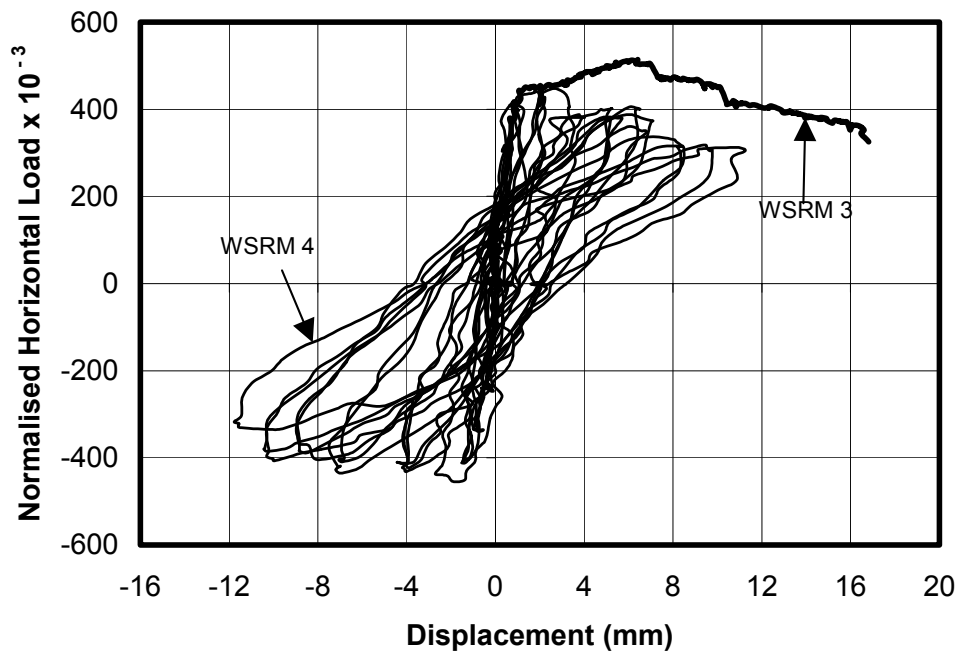
4 Deformation of shear walls

4.1 Load – deflection response

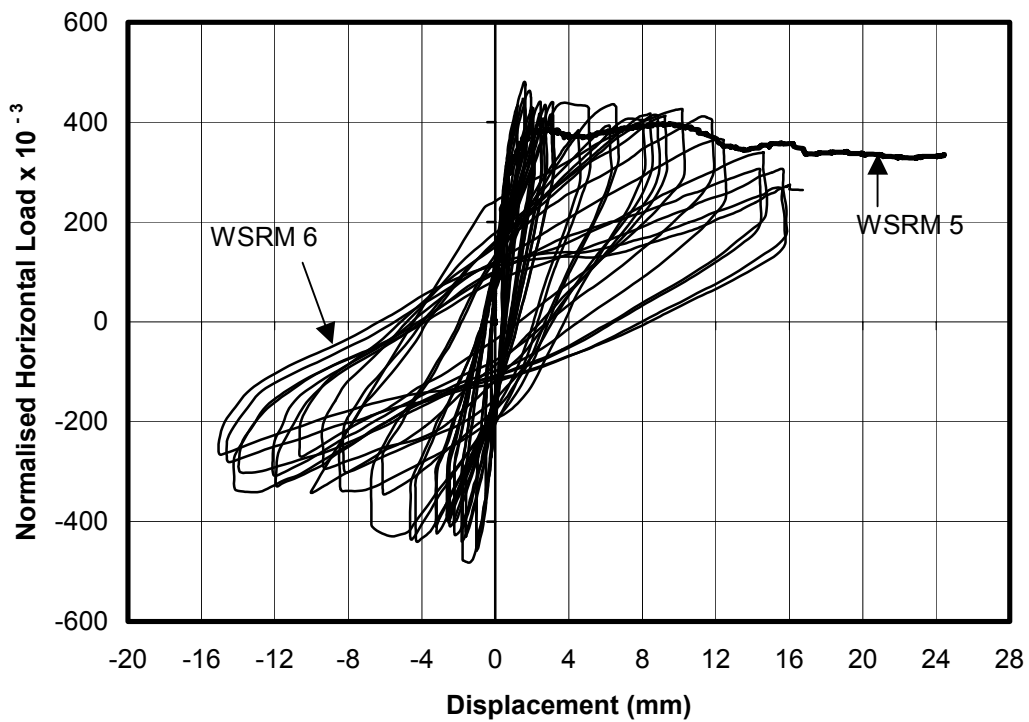
The normalised lateral load – lateral deflection curves for the four pairs (walls having the same spacing ratio) of the WSRM walls are shown in Figs. 3(a) – 3(d) respectively. Fig. 3(e) shows the same graph for WSRM 9 and URM as both these walls are currently considered as URM in AS3700(2001).



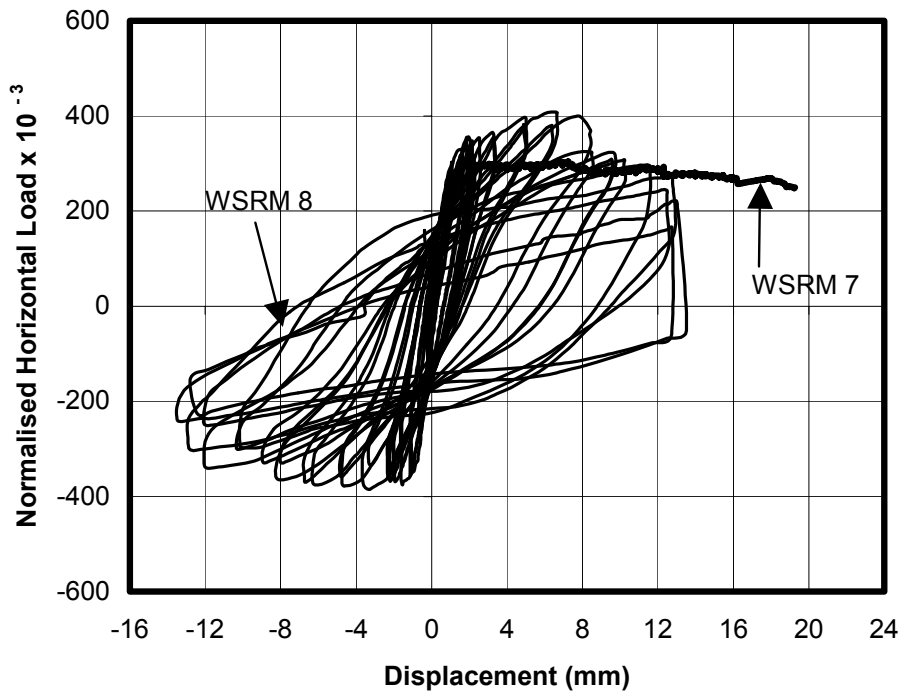
(a) WSRM Shear walls of spacing ratio = 0.11



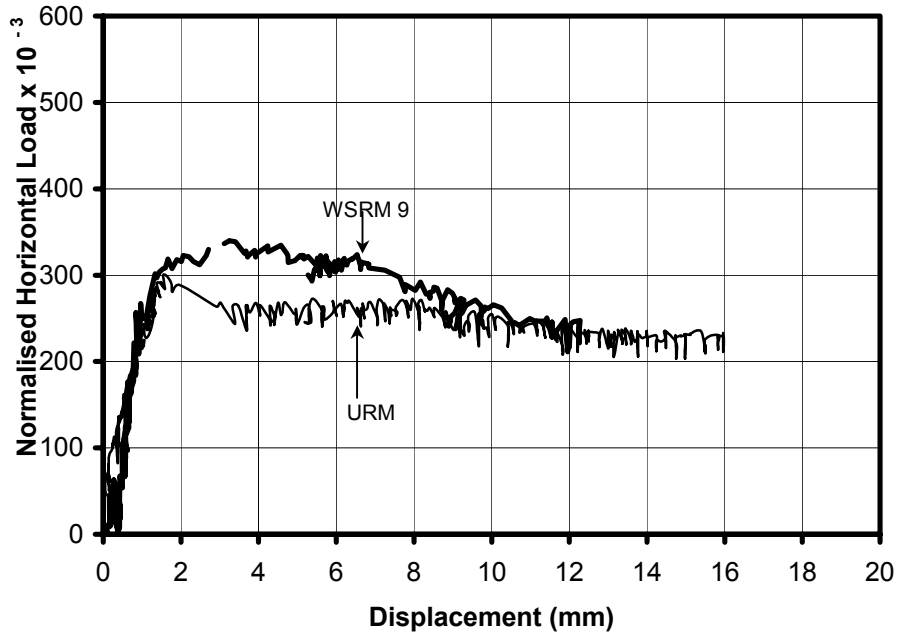
(b) WSRM Shear walls of spacing ratio = 0.81



(c) WSRM Shear walls of spacing ratio = 1.46



(d) WSRM Shear walls of spacing ratio = 5.71



(e) WSRM Shear wall 9 (spacing ratio = ∞) and URM shear wall

Figure 3 Normalised Lateral Load – Lateral Deflection Curves

The normalised load axis (vertical) is graduated ± 0.600 in all graphs presented in Figs 3(a) – 3(e) with a view to enabling quick inference on the influence of reinforcing steel spacing ratio on the deformation characteristics of the walls. It is apparent that with the increase in spacing ratio, the “height” (representing the normalised capacity) of each curve reduced with the URM wall exhibiting the lowest normalised capacity. Furthermore, the “width” of these graphs indicates that the walls shown in Fig. 3(c) exhibit the highest deformation capacity. Perhaps it could be concluded that a spacing ratio approximately equal to 1.5 would be good if improved deformation capacity is desired (as in earthquake design). Further testing is required to confirm this finding.

4.2 Displacement ductility

Displacement ductility is defined as the ratio of ultimate displacement to the yield displacement. To determine the yield point, a concept of equivalent elastic - perfectly plastic system proposed by Muguruma et al. (1991) was used. According to this model a horizontal line was drawn at the maximum (peak) load level of the load deflection curve and an inclined line was then drawn from the origin such that it provided equal pre and post yield energy. A point where the inclined line intersected the actual load – deflection curve was considered as the yield point of the wall, and its corresponding load and displacement were defined as the yield load and the yield displacement respectively. Similar method of determining the yield point is contained in Tomazevic and Lutman (1996).

According to the models provided by Muguruma et al. (1991) and Saatcioglu (1991), RC columns generally reach their ultimate displacement when reduction in their shear capacity exceeds 20% of the peak load. In contrast, some of the walls tested during this research did not lose 20% of the peak load until they reach very large displacement (where experiment were to be stopped). Therefore the curves were smoothened to extrapolate the values of ultimate load and the corresponding displacement. From the smoothened curves, yield, peak and ultimate displacements were calculated and are presented in Table 3.

Table 3 Displacement ductility of WSRM Shear Walls

Spacing Ratio	Wall	Test Method	Yield Displacement	Ultimate Displacement	Ductility
0.11	WSRM 1	Monotonic	1.3	7.2	5.5
	WSRM 2	Cyclic	1.0	8.3	8.3
0.81	WSRM 3	Monotonic	1.3	15.0	11.5
	WSRM 4	Cyclic	0.7	8.8	12.6
1.46	WSRM 5	Monotonic	1.3	25.0	19.2
	WSRM 6	Cyclic	0.7	10.7	15.3
5.71	WSRM 7	Monotonic	1.3	16.0	5.3
	WSRM 8	Cyclic	1.3	12.0	9.2
∞	WSRM9	Monotonic	1.6	9.6	6.0
N/A	URM	Monotonic	1.7	16.6	9.8

WSRM walls with approximately uniform spacing ratio (walls 3, 4, 5 & 6) of the vertical reinforcement exhibited higher ductility factor than the other walls under both monotonic and cyclic loading. Ductility of walls with uniform spacing of the vertical reinforcement was higher than that of walls with non-uniform spacing of the vertical reinforcement. This observation is made consistently for both monotonic and cyclic loaded walls. It is therefore apparent even a very small amount of reinforcement

($\approx 0.14\%$) has the potential to provide good levels of ductility sufficient to resist moderate intensity earthquake spectra.

The ductility of the monotonically loaded walls exhibited higher range of values (5.3 – 19.2). Cyclically loaded shear walls exhibited much reduced range (hence more reliable estimates) of ductility values (8.3 – 15.3). As these values translate into “structural response factor (R_f)” in AS3700 (2001) (similar to the “load reduction factors” in text books (Pauley and Priestley (1992))), it could be recommended that the ductility factors derived from the cyclically loaded walls be used in its evaluation. The R_f value is calculated from the displacement ductility factor (μ_d) as shown in Eq. 2.

$$R_f = \sqrt{2\mu_d - 1} \quad (2)$$

The R_f values calculated from the displacement ductility factors obtained from the cyclically loaded WSRM walls range from 4.0 to 5.4. The current R_f value reported in AS3700 (2001) is 2.5 for WSRM walls.

5 Conclusions

Behaviour of masonry walls containing small amount of vertical reinforcement (approximately 0.14%) at larger horizontal spacing (800mm - 2000mm) under a vertical load of 0.5MPa, derived from full scale walls tested under monotonic and cyclic loading is reported. Spacing ratio (defined as the ratio of spacing of vertical reinforcement at the central region to the end region of the wall) of vertical reinforcement is used as the key parameter in the investigation and was varied from 0.11 to infinity. With a view to appreciating the effect of the light reinforcement to shear capacity and ductility, an unreinforced masonry wall was also tested.

The following general conclusions were made from the analysis of the test results:

1. WSRM shear walls with approximately uniform spacing of the vertical reinforcement exhibited good level of shear capacity and ductility and degraded gradually when compared with the walls of non-uniform spacing of the vertical reinforcement under monotonic as well as cyclic loading.
2. WSRM shear walls under cyclic loading degrade at faster rate than under monotonic loading. This exhibits higher rate of accumulation of damage during the cyclic loading.
3. The normalised shear capacity of the shear walls exhibited gradual reduction with the increase in the spacing ratio of vertical reinforcement. The normalised shear capacity of WSRM shear walls has generally increased under cyclic load relative to the monotonic loading.

From the ten walls tested, some specific conclusions were also made as listed below:

1. The lowest normalised shear capacity was observed in the URM wall (monotonic loading) and the higher normalised shear capacity was observed in the WSRM wall of spacing ratio 0.11 subjected to cyclic loading (WSRM 2).

2. The ductility of the monotonically loaded walls exhibited higher range of values (5.3 – 19.2). Cyclically loaded shear walls exhibited much reduced range (hence more reliable estimates) of ductility values (8.3 – 15.3).
3. The R_f values calculated from the displacement ductility factors obtained from the cyclically loaded WSRM walls range from 4.0 to 5.4 whilst the current R_f value reported in AS3700 (2001) is 2.5 for WSRM walls. It appears that the R_f value could be revised upwardly making the WSRM system competitive with reinforced concrete shear wall systems.

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