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SEISMIC PERFORMANCE OF PARTIALLY-GROUTED MASONRY SHEAR WALLS

A. E. Schultz

Department of Civil Engineering, University of Minnesota, 500 Pillsbury Drive, S.E., Minneapolis, MN 55455, U.S.A.

ABSTRACT

The seismic performance of partially-grouted masonry shear walls is studied. The advantages and benefits of this type of construction are highlighted. In-plane cyclic load tests of six partially-grouted masonry shear walls with bond beams are presented. Lateral force resistance mechanisms are discussed in conjunction with observed displacement response. Parameters relating to initial stiffness, shear strength, residual strength, energy dissipation and deformation capacity are highlighted, and predictions for the former two are compared with measurements.

KEYWORDS

Bond beams; cyclic tests; deformation capacity; energy dissipation; lateral loads; masonry walls; partial grouting; seismic design

INTRODUCTION

Earthquake-resistant design procedures in the U.S.A. for fully grouted and reinforced masonry has been validated by experiment and observed performance during recent earthquakes. However, cost-effectiveness has prevented reinforced masonry from gaining widespread popularity in the eastern and central U.S.A. where seismic hazards are lower than those in the west coast. In partially-grouted masonry, an alternative to fully-grouted construction, vertical reinforcement is concentrated in a few cells, and, only those cells with rebar are grouted. Horizontal bars are concentrated in bond beams, or they are replaced altogether by bed joint reinforcement. Large savings in construction costs are expected from partially-grouted masonry (Fattal, 1993a), yet, the seismic resistance of partially-grouted masonry must be verified, and current analysis and design methods must be validated as well.

The concept of partially-grouted masonry for lateral load resistance is not without precedent, as it shares similarities with confined masonry in Latin America (Casabbone, 1994). Confined masonry comprises unreinforced masonry walls which are erected between vertical gaps used for tie columns. A "confinement" frame of lightly-reinforced tie columns and tie beams is cast after the masonry mortar has hardened. Shrinkage of the concrete frame serves to confine the unreinforced masonry panel, as well as to provide structural redundancy and uniform distribution of lateral forces. This system has been successful throughout Latin American in mitigating seismic damage to low-rise masonry structures (Schultz, 1994).

A research program was initiated in the Building and Fire Research Laboratory (BFRL) of the National Institute of Standards and Technology (NIST) to study the seismic performance of partially-grouted masonry shear walls (Fattal, 1993a, Schultz, 1994). An experimental study was designed to determine the influence of horizontal reinforcement ratio, type of horizontal reinforcement and height-to-length aspect ratio on the shear strength and behavior of partially-grouted masonry walls. A secondary goal of the study is to generate physical data on partially-grouted masonry shear walls for the verification of finite element modelling techniques. This paper discusses the first of two series of partially-grouted masonry shear wall tests in which specimens utilize bond beams for the placement of horizontal reinforcement, and horizontal reinforcement ratio (ρ_h) and height-to-length aspect ratio (*H/L*) are the principal experimental variables.

EXPERIMENTAL PROGRAM

A total of six partially-grouted masonry shear wall test specimens with bond beams (Fig. 1) were constructed and tested. Only the outermost vertical cells of the walls were reinforced vertically and grouted, and a single bond beam containing all horizontal reinforcement was placed at mid-height of the walls. The remainder of the masonry in the walls was not grouted. All walls were built from seven courses of masonry for a total height of masonry equal to 1422 mm (56 in.). The first four masonry courses, including the middle course of bond beam units, were placed in a single day. Bond beams and the lower portion of exterior vertical cells were grouted on the second day. The top three courses of masonry were placed on the third day, and the remainder of the exterior vertical cells were grouted on the fourth day. Wall lengths equal to 1422 mm (56 in.), 2032 mm (80 in.), and 2845 mm (112 in.) were used to define three different height-to-length aspect ratios, as noted in Table 1.

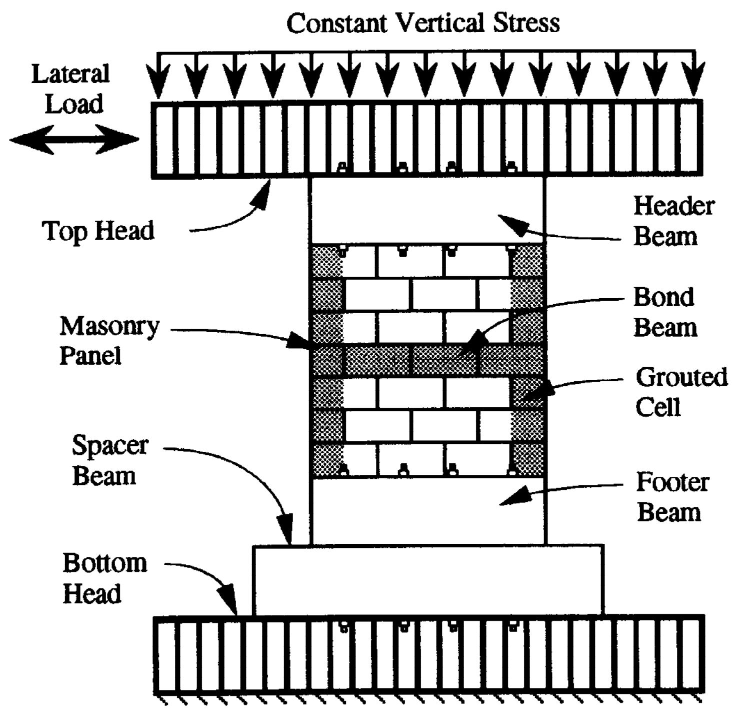

Fig. 1. Test setup for partially-grouted masonry walls

The six walls were reinforced with two #6 Grade 60 reinforcing bars in each exterior vertical cell, for a total area of vertical reinforcement equal to 1135 mm^2 (1.76 $in.^2$). Even though flexural yielding is a preferred mode of response for seismic design, the purpose of this study requires that the specimens respond and fail in shear. Two horizontal reinforcement schemes were used for the bond beams (Table 1) to define reinforcement ratios, based on gross dimensions, that are equal to 0.05% and 0.12%, respectively. The first of these is somewhat smaller than the minimum value cited in most building codes and standards of practice for masonry in the U.S.A. (*Masonry*, 1992, *International*, 1994, *NEHRP*, 1995).

The walls were constructed using two-cell concrete blocks with average length, thickness and height equal to 396 mm (15.6 in.), 195 mm (7.67 in.), and 194 mm (7.63 in.), respectively. The units were 51% solid with a minimum face-shell thickness equal to 33.7 mm (1.33 in.). All masonry was face-shell bedded, except for the ends of the walls in which exterior webs were also bedded. The mortar (ASTM Type S) and and coarse grout, respectively, had 28-day compression strengths equal to 21.7 MPa (3140 psi) and 29.6 MPa (4300 psi). The masonry had 28-day compression strengths equal to 17.1 MPa (2480 psi) and 17.6 MPa (2550 psi), respectively, for ungrouted and grouted prisms. The masonry panels were connected to precast concrete header and footer beams (Fig. 1) after the mortar hardened. Protruding ends of the vertical bars were inserted through sleeves into large pockets in the header and footer beams, and the pockets were filled with high-strength grout. Vertical bars, the ends of which had been previously threaded, were anchored by means of steel plates and high-strength steel nuts. Horizontal bars were anchored with 180° hooks that engaged the interior vertical bars in each exterior jamb.

Table 1. Schedule of shear wall specimens with bond beams

Wall No.	Length, mm[1]	Aspect Ratio, H/L	Horizontal Reinforcement	ρ_h, %	Vertical Load[2], kN[3]
1	2845	0.5	2-#3	0.05	267
3	2032	0.7	2-#3	0.05	191
5	1422	1	2-#3	0.05	133
7	2845	0.5	1-#4 & 1-#5	0.12	266
9	2032	0.7	1-#4 & 1-#5	0.12	177
11	1422	1	1-#4 & 1-#5	0.12	132

[1]1 mm = 0.03937 in. [2]Average of both loading directions [3]1kN = 0.2248 kips

The cyclic load tests were conducted in the NIST Tri-Directional Testing Facility (Woodward and Rankin, 1984), as shown in Fig. 1. In-plane cyclic drift histories were applied to the header beam while the footer beam remained fixed, and, since rotation of the header and footer beams was constrained, the panels were subjected to reversed bending. A summary of response forces and displacements from the cyclic load tests is given in Table 2. The cyclic drift histories were patterned after the TCCMAR phased-sequential displacement procedure (Porter and Tremel, 1987). In this procedure, groups of drift cycles are organized around peak amplitudes that are gradually increased to failure. Peak amplitudes after the initial elastic displacements are tied to the first-major event (FME). In the present study, the FME was associated with masonry cracking, and the loads and displacements at the FME were found to be very similar to those defining the limit of proportionality in the force-displacement relation.

Table 2. Summary of cyclic drift tests

Wall No.	Peak Lateral Response[1]		Deformation Capacity[1], mm[3]	Energy Dissipation Factor @ Δ_{054}	Horizontal Bar Peak Strain
	Load, kN[2]	Disp., mm[3]			
1	187	13.61	13.61[4]	103	0.00003
3	245	10.74	5.46	55	0.00226
5	133	10.41	5.00	81	0.00004
7	240	10.44	9.98	111	0.00011
9	192	7.29	4.70	66	0.00015
11	154	4.88	5.21	49	0.00007

[1]Average of both loading directions [2]1 kN = 0.2248 kips [3]1 mm = 0.03937 in.
[4]Deformation capacity was not reached, peak displacement is given

OBSERVED BEHAVIOR

At early stages, all specimens developed cracks in the ungrouted masonry above the bond beam, but there were two distinct cracking modes. For all specimens except Wall No. 3, the first cracks were vertical and they formed at the top of the walls near the interface between the grouted vertical cell and the ungrouted masonry (Fig. 1). With subsequent cycles, the cracks quickly propagated downward towards the joint between grouted vertical cell and bond beam. For Wall No. 3, the first cracks were inclined, they initiated at mid-length along the top edge of the wall, and they propagated towards the "bond beam"-"vertical cell" joints. Inclined cracks formed in the upper one-half of the masonry panels in Wall Nos. 1, 5, 7, 9 and 11, but this occurred after the vertical cracks along the jambs were well developed.

Initial vertical cracks in Wall Nos. 1, 5, 7, 9 and 11 occurred in response to the horizontal stress concentration between grouted and ungrouted masonry. Wall No. 3 did not exhibit this type of behavior, and it appears that different anchorage conditions for vertical bars in Wall No. 3, as compared with the other specimens, may have played a role. The cracking mode played a significant role in the post-cracking behavior and resistance of the walls. For Wall Nos. 1, 5, 7, 9 and 11, the vertical cracks dominated the behavior of the walls, as few other cracks formed during the tests. As the header beam of the walls were displaced, the width of the vertical cracks on the leading jamb grew very wide, i.e. up to 6 mm (1/4 in.). These cracks eventually propagated into the "bond beam"-"vertical cell" joint, disturbing the anchorage of the horizontal reinforcement.

Wall No. 3 responded in a manner similar to that of reinforced and fully-grouted masonry. Numerous inclined cracks formed following the initial cracks described earlier, and these well-distributed cracks did not open as widely as did the vertical cracks in the other specimens. Furthermore, some of the inclined cracks propagated into the "bond beam"-"vertical cell" joint, but this occurred late in the test and does not appear to have disturbed the anchorage of horizontal bars much. Horizontal reinforcement in Wall No. 3 achieved peak strains (Table 2) comparable to the nominal yield strain of the bar (0.002), whereas peak strains in the horizontal reinforcement of the other specimens were a small fraction of the nominal yield strain. Cracking mode of the walls seems to have affected the effectiveness of horizontal reinforcement.

FORCE-DISPLACEMENT RESPONSE

The response of all shear wall specimens to the cyclic drift histories was reasonably stable and well-behaved (Fig. 2). Initial response to load was linear and was characterized by large stiffnesses. As the peak amplitude of the drift cycles increased, the force-displacement hysteresis curves widened and peak resistance deteriorated. In addition, "pinching" behavior became evident, i.e. reduction in unloading stiffness at low lateral load levels. It was also noted that deterioration of post-peak strength and pinching of the hysteresis loops increased with increasing height-to-length aspect ratio. However, none of the specimens displayed sudden failure, and, peak resistance gradually deteriorated with load cycles. All specimens except Wall No. 1 were tested until the peak resistance for a given group of cycles was less than 75% of the largest horizontal force registered by that wall.

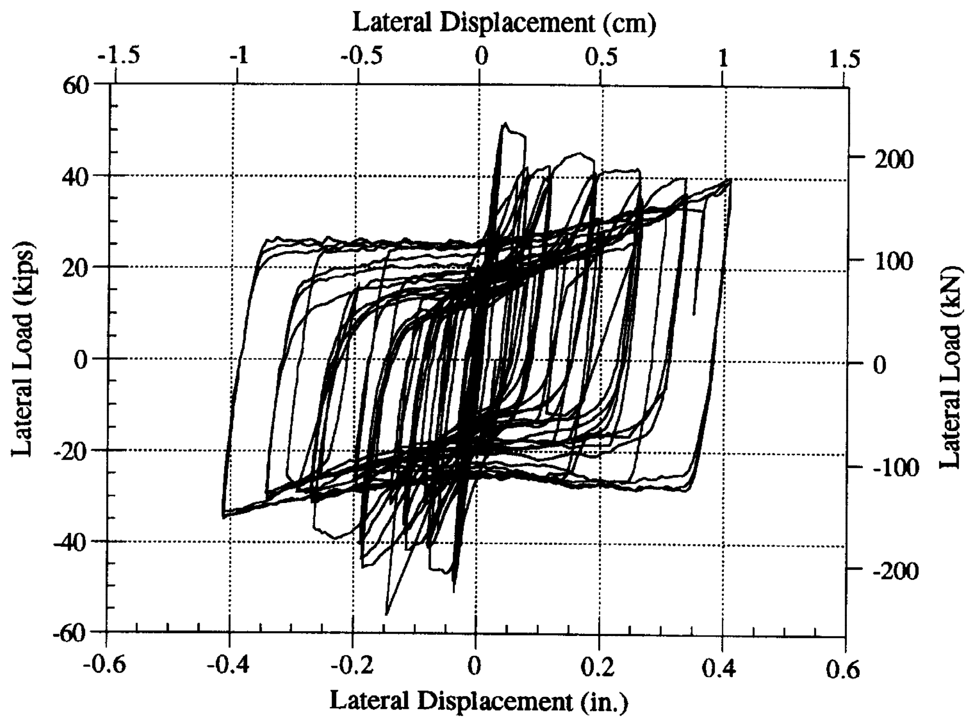

Fig. 2. Force-displacement response for wall 7

Envelopes of force-displacement response were constructed by sequentially scanning each history, collecting all force-displacement coordinate pairs which exceed previous peak displacement, and "smoothing" the trends using an 11-point moving average. These smoothed envelopes (Fig. 3) demonstrate that the response of the shear wall specimens is essentially bilinear. The envelopes have a marked limit of proportionality that strongly resembles a yielding system, even though there was no widespread yielding of reinforcement in these walls. These envelopes also illustrate the manner in which total lateral load resistance of the walls generally decreased with increasing aspect ratio (*H/L*).

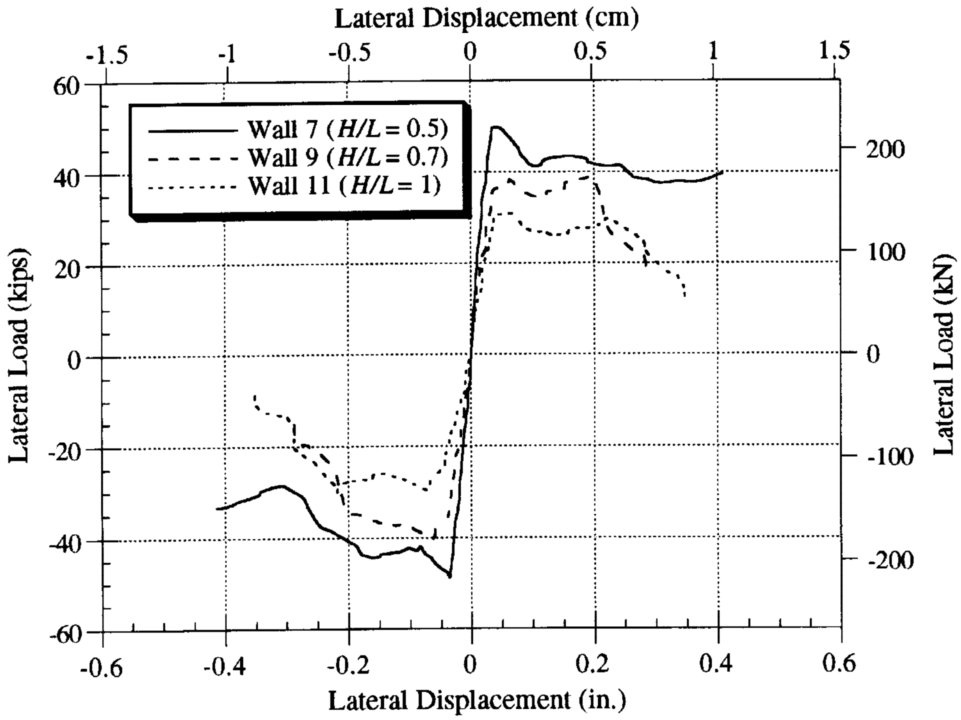

Fig. 3. Force-displacement envelopes for walls with $\rho_h = 0.12\%$

Using a linear least-squares fit, initial stiffnesses were calculated from the envelopes using data pairs which do not exceed 40% of the force limit for proportionality. These inferred stiffnesses are listed in Table 3 along with estimates based on elastic behavior. The calculated stiffnesses were obtained by combining flexural and shear contributions for an elastic panel, the thickness of which was taken as twice the face-shell thickness of the block. The modulus of elasticity E_m was estimated as $600f'_m$, where f'_m is masonry compression strength (Atkinson and Yan 1990), and the shear modulus G was approximated as $0.4E_m$. In the direction of first loading, the inferred stiffnesses are approximately 3/4 of the calculated stiffnesses. In the negative loading direction, actual stiffnesses are roughly 3/5 of the calculated stiffnesses.

Table 3. Initial stiffnesses and strengths

Wall No.	Initial Stiffness, kN/mm[1]			Calculated Lateral Load Capacity, kN	Mean Lateral Load-Vertical Force Ratio
	Calculated	Inferred			
		Positive	Negative		
1	431	330	235	0.611	0.567
3	287	175	149	1.066	0.917
5	175	166	160	0.783	0.642
7	431	348	241	0.708	0.676
9	287	202	168	0.755	0.832
11	175	129	109	0.825	0.806

[1] 1 kN/mm = 5.710 kips/in.

WALL TOUGHNESS AND STRENGTH

Cumulative energy dissipation, a measure of toughness, was obtained by integrating each force-displacement loop and accumulating these areas. Because the specimens have different strengths and stiffnesses, cumulative dissipated energy was normalized by one-half of the product of force and displacement at the limit of proportionality. Energy dissipation factor is given in Table 3 at the instant when wall displacements correspond to a drift equal to 0.54% of panel height (Δ_{054}). The amount of horizontal reinforcement does not seem to affect the ability of the walls to dissipate energy, but increasing height-to-length aspect ratio has a detrimental effect on wall toughness. Deformation capacity, defined as the displacement at which lateral load resistance decreases to 75% of the peak value, is another measure of toughness. These capacities, which were obtained from the smoothed envelopes, are given in Table 4 and they range from 0.33% to 1% of the height of masonry. These values appear to be low for regions of high seismic risk, but may be acceptable for regions of low seismicity.

In-plane shear strength of the walls were calculated using a formula adapted by Fattal (1993b) from work by Matusmura (1987). These strengths are listed in Table 3, along with the ratios of measured peak load to calculated strength. On the average, measured peak load is 3/4 of the shear strength calculated using Fattal's formula. It is also interesting to note that Wall No. 3, which displayed a different cracking mode and vastly larger peak horizontal bar strains, also displays a peak load which is essentially equal to the estimated shear strength. All other specimens were weaker than their estimated shear strengths.

Certain characteristics of the force-displacement relation for stocky walls (Fig. 2), such as steep unloading slopes, flat post-peak loading branches, and wide, stable loops, strongly suggest that the stocky walls relied on friction to resist horizontal forces. The mean values of the ratio of lateral force to vertical force in the post-peak branch are given in Table 3, and this ratio is equal to the coefficient of friction for an ideal sliding system. These ratios are smallest for the stocky walls (H/L = 0.5), and a magnitude of 0.6 is not unreasonable for sliding friction between masonry, mortar and concrete. For walls with larger aspect ratios, larger values of this ratio suggest that other mechanisms contributed to lateral load resistance.

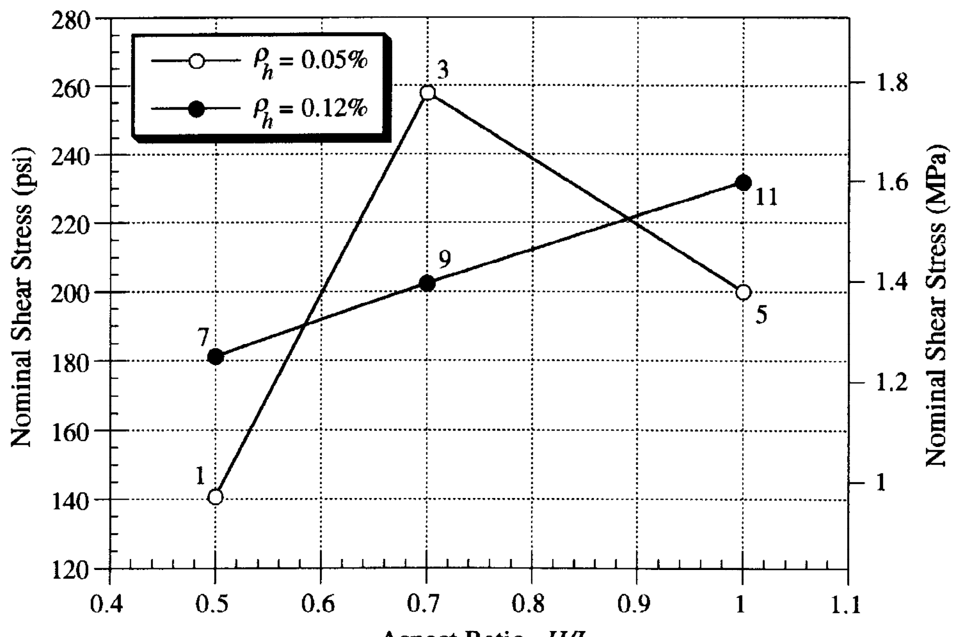

Fig. 4. Influence of aspect ratio on ultimate shear stress

INFLUENCE OF EXPERIMENTAL PARAMETERS

The peak load resisted by the wall specimens, after averaging the values for both loading directions, were divided by net horizontal area to define ultimate shear stresses. The net horizontal area was taken as the product of twice the minimum face-shell thickness of the block and the total length of the wall. These ultimate shear stresses, except that for Wall No. 3, show a marked dependence on aspect ratio (Fig. 4). As aspect ratio doubles from the stocky walls to the slender walls, ultimate shear stress increases by 42% for walls with ρ_h = 0.05% and by 28% for walls with ρ_h = 0.12%. Horizontal reinforcement ratio can be seen to have a modest beneficial influence on ultimate shear stress (Fig. 5), with the stocky walls (H/L = 0.5) and slender walls (H/L = 0.5) seeing 29% and 16% increases, respectively, as ρ_h increases from 0.05% to 0.12%. However, strength data for Wall No. 3 does not fit these trends. In fact, the ultimate shear stress for Wall No. 3 exceeds that of all other specimens. This and other observations suggest that the response of Wall No. 3 is out of character for this series of shear wall tests.

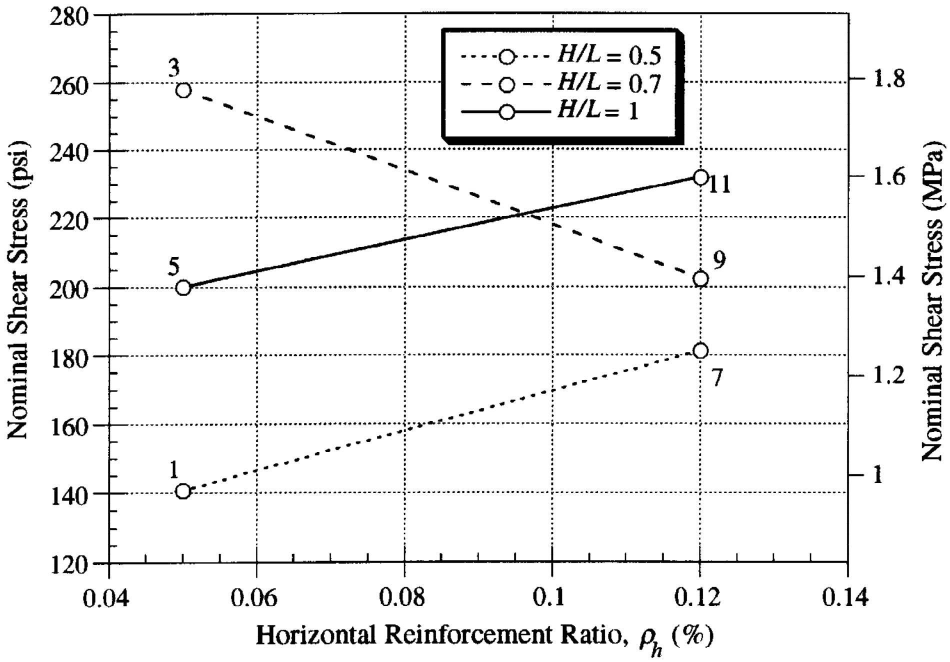

Fig. 5. Influence of horizontal reinforcement ratio on ultimate shear stress

CONCLUSIONS

The experimental observations presented in this paper suggest that partially-grouted masonry is a viable lateral-load resisting system for regions of moderate and low seismic risk. Resistance to the drift histories is stable and features high initial stiffness and ample energy dissipation. The lateral load resisting mechanism is vastly different from that for reinforced masonry walls. Vertical cracks arising from stress concentrations between ungrouted and grouted masonry appear to dominate wall behavior, and sliding friction between masonry panels and concrete surfaces contributes to the resistance mechanisms. Height-to-length aspect ratio has a beneficial effect on ultimate shear stress, but a detrimental effect on toughness (strength deterioration, deformation capacity and energy dissipation capacity). Horizontal reinforcement ratio has a modest beneficial effect on ultimate shear stress, but it does not appear to affect toughness.

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