CYCLIC SHEAR TESTS ON MASONRY PIERS

R. L. Mayes[I], Y. Omote[II], R. W. Clough[III]

ABSTRACT

The results of cyclic in plane shear tests on seventeen fixed-ended masonry piers are presented. The test set up is designed to simulate insofar as possible the boundary conditions the piers would experience in a perforated shear wall of a complete building. Each test specimen was a full-scale panel about 15 feet square consisting of two piers and a top and bottom spandrel. The panels were constructed from 6" wide x 8" high x 16" long hollow concrete block units. The variables included in the investigation were the quantity and distribution of reinforcement, the rate of load application, the vertical bearing stress and the effect of partial grouting. This paper discusses the effect of these parameters on the hysteresis envelopes and ductility of the piers.

I. Introduction

The test results presented herein demonstrate the cyclic behavior of concrete masonry piers subjected to the lateral loading. The variables included in the seventeen tests are the frequency of load application, the quantity and distribution of reinforcement, the vertical bearing stress and effect of partial grouting. The seventeen test specimen include eight sets of two identical panels. One of each pair is tested at an input displacement frequency of 0.02 cps (pseudo-static) and the other at 3 cps (dynamic). The other variables are listed in Table 1.

Only the characteristics of the load-deflection hysteresis envelopes are included in this paper, discussion of other parameters are included in references (1) and (2).

II. Test Specimen

The overall dimensions of the seventeen test specimen are the same and are shown in Figure 1. The piers with a height (5'-4") to width (2'-8") ratio of two were the elements of interest. The top and bottom spandrels were heavily reinforced in an attempt to prevent their failure, although this objective was not achieved in all cases.

The panels were constructed from standard two-core reinforcible hollow concrete blocks of nominal 6" wide x 8" high x 16" long dimensions. The core of each block was approximately 51.4 square inches with a ratio of net to gross area of 58%. Single units had an average gross compressive strength of 1714 psi (2944 psi net strength) with a range from 1340 psi to 2040 psi over five samples. The average tensile strength of the units was 267 psi with a range from 235 psi to 285 psi over five samples. The block test procedures followed the California Q-Block Quality Control Specification[(3)].

I Assistant Research Engineer, University of California, Berkeley
Principal, Computech, San Francisco
II Assistant Research Engineer, University of California, Berkeley
III Director, Earthquake Engineering Research Center, University of California, Berkeley

Tests 1, 2, 5, 6 and 9 to 12 had 2-#6 vertical re-bars in each jamb of the pier i.e. 0.92% reinforcement based on the gross cross sectional area. Tests 3 and 4 had 2-#4 vertical re-bars in each jamb of the piers - 0.41%. Tests 7 and 8 had 2-#6 vertical re-bars in each jamb and 3-#5 horizintal bars in each pier - 1.4% reinforcement. Tests 13 to 16 had a substantial amount of reinforcement, designed to ensure a flexural failure - 1.67%. In addition to the horizontal and vertical reinforcement, Tests 15 and 16 had steel plates inserted in the mortar joints at each of the three courses from the top and bottom of each pier. The plate used is shown in Figure 2. Test 17 was unreinforced.

III. Test Equipment and Procedure

The test equipment shown in Figure 3 permits lateral loads to be applied in the plane of the piers in a manner similar to that in which a floor diaphragm would load the piers during earthquake excitation. It consists of two, twenty-feet high, heavily-braced reaction frames to which a pair of hydraulic actuators are connected, a mechanism capable of applying vertical bearing loads similar to those experienced by the piers in an actual structure, and a concrete base on which the panel is constructed and bolted to the test floor.

The loading sequence of each panel consisted of 3 sinusoidal cycles of load applied at a specified actuator amplitude displacement and frequency. The actuator displacements generally followed the sequence 0.02", 0.04", 0.06", 0.08", 0.12", 0.16", 0.20", 0.25", 0.30", ---0.5", 0.6"----1.0"---1.5". After each set of 3 cycles of loading the walls were visually inspected and the crack pattern identified.

IV. Test Results and Discussion

A summary of the test results is listed in Table 1 and an example of a hysteresis loop and the mode of failure is shown in Figure 4 (for Test 3). Shear force indicators P_{u1}, P_{u2}, P_1, P_2 and P_3 and ductility indicators δ_1, δ_2, δ_3 and δ_4 listed in Table 1 are identified in Figure 4.

In the hysteresis envelopes plotted in Figures 5 to 8, there clearly are two distinct types of behavior. First those typified by Test 1 in Figure 5 where the hysteresis envelope reaches a maximum load and as the lateral displacement increases the load gradually decreases. Second those typified by Tests 11 and 15 in Figures 6 and 8 where the hysteresis envelope reaches a maximum load and as the lateral displacment increases this load is maintained until a certain displacement at which point the load decreases. This is somewhat similar to an elasto-plastic force deflection relationship. The first type of behavior is characterized by low values of the parameter $(\delta_1 + \delta_2)/2$ (about 1.5) and larger values of the parameter $(\delta_3 + \delta_4)/2$ (in the range of 2 to 6). The corresponding values for the second type of behavior are 2-5 and 4-10 respectively.

The effect of bearing stress on (Tests 1, 2, 5, 6, 9 and 10) the ductility of the piers is somewhat inconclusive. Evaluating the hysteresis envelopes of Figure 5 and the ductility indicators of Table 1, there is a trend towards a more ductile behavior as the bearing stress increases, however, this is offset by the fact that the piers with a bearing stress of 500 psi can only withstand a maximum lateral displacement of 0.5" as opposed to 1.0" for the 0 and 250 psi cases. If the maximum displacement of 0.5" is not a limiting factor, then an increase in bearing stress could be considered to have a desirable effect on the ductility of the piers for the tests performed, however as the number of tests are limited and because this conflicts with the conclusion of other investigators[2] this indication obviously requires further investigation.

The effect of partial grouting on the ductility of the piers is also inconclusive. From the hysteresis envelopes of Figure 6, partial grouting produces a tendency towards an elasto-plastic type of force-deflection behavior and when compared to the fully grouted pseudo-static test (Test 1) the overall effect appears to be favorable. However, when compared to the fully grouted dynamic test the force-deflection curves are different and the fully grouted pier must be considered to have the more desirable ductile behavior[1]. In addition, both the partially grouted piers collapsed at a lateral displacement of 0.5" as opposed to 1.0" for the fully grouted walls. Because of the limited number of tests performed and the lack of any definite trend in the results, it is clear that further tests are required.

Horizontal reinforcement has a very desirable effect on the shear mode of failure (Tests 1, 2, 7 and 8). As seen from the hysteresis envelopes of Figure 7 and the ductility indicators of Table 1, horizontal reinforcement substantially increases the overall ductility of the piers, and the dynamic test shows a better performance than the pseudo-static test.

The performance of the flexural mode of failure was evaluated in Tests 3, 4 and 13-16 with the resulting hysteresis envelopes plotted in Figure 8. The basic difference between Tests 3, 4 and 13, 14 was the inclusion of horizontal reinforcement in Tests 13 and 14 to ensure that a pure flexural mode of failure was obtained. As expected, Tests 3 and 4 had characteristics of both the shear and flexural modes of failure, showing a more sudden drop in load carrying capacity at larger lateral displacements. Tests 13 and 14 show that the force deflection relationship of the flexural mode of failure tends towards elasto-plastic characteristics. The most significant result of this series of tests was the effect of joint reinforcement, (Figure 2), which was included in Tests 15 and 16. The addition of the horizontal plates substantially increased both sets of ductility indicators as well as the maximum displacement, that the piers could withstand, leading to an extremely desirable ductile behavior.

The main conclusion of this paper is that much more research is required on the shear strength of masonry piers. Trends of behavior were indicated but because the number of tests was small, definitive conclusions on many facets of the initial goals of the investigation could not be made. However, the following conclusions did emerge in the results obtained.

1) The inclusion of a sufficient amount of horizontal bar reinforcement significantly enhances the ductile behavior of piers failing primarily in the shear mode.

2) The inclusion of 1/8" plates in the toes of the piers produces extremely desirable ductile behavior for piers failing primarily in the flexural mode.

3) An increase in vertical bearing stress demonstrates a tendency towards a more ductile type of behavior for piers which fail mainly in

shear.

4) Partial grouting produces a tendency towards an elasto-plastic force-deflection relationship prior to failing in the shear mode. However, it is not clear whether or not this enhances the overall ductile behavior of the piers when compared to the behavior of fully grouted piers.

Acknowledgement

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References

(1) Mayes, R. L. Omote, Y., and Clough, R. W., "Cyclic Shear Tests on Masonry Piers, Part I - Test Results," EERC Report No. 76-8, Earthquake Engineering Research Center, University of California, Berkeley, May 1976.

(2) Mayes, R. L., Omote, Y. and Clough, R. W., "Cyclic Shear Tests on Masonry Piers, Part II - Analysis of Test Results," EERC Report No. 76-16, Earthquake Engineering Research Center, University of California, Berkeley, 1976.

(3) 1974 Masonry Codes and Specifications, Published by Masonry Industry Advancement Committee, California, 1974.

Table 1 Summary of the Test Results

TEST NO.	Frequency (1) (cps)	Bearing Stress (2) (psi)	Vertical Reinforcement (3)	Horizontal Reinforcement (4)	$\frac{P_{u1}+P_{u2}}{2}$ (kips) (5)	$\frac{\tau_{u1}+\tau_{u2}}{2}$ (psi) (5)	$\frac{P_1+P_2}{2}$ (kips) (6)	$\frac{\tau_1+\tau_2}{2}$ (psi) (6)	P_3 (kips) (7)	τ_3 (psi) (7)	$\frac{\delta_1+\delta_2}{2}$ (8)	$\frac{d_1+d_6}{2}$ (in) (9)	$\frac{\delta_3+\delta_4}{2}$ (10)	$\frac{d_1+d_5}{2}$ (in) (11)	Max. Input Stroke of Actuator (in) (12)
1	0.02	250	2 - #6	--	26.0	135	24	125	20	104	1.55	.095	3.5	.065	1.0
2	3	250	2 - #6	--	33.2	173	31	161	28	146	1.55	.130	2.4	.095	1.0
3	0.02	125	2 - #4	--	27.3	142	26	135	21	109	1.5	.180	4.1	.105	1.0
4	3	125	2 - #4	--	26.0	135	22.8	119	18	94	1.8	.165	5.6	.085	1.0
5	0.02	0	2 - #6	--	20.5	107	18.5	96	15	78	1.55	.180	5.6	.075	1.0
6	3	0	2 - #6	--	25.5	133	21.7	113	19	99	1.85	.105	5.3	.070	1.0
7	0.02	250	2 - #6	1 - #5	40.7	212	39	203	33	172	1.5	.235	4.4	.123	0.7
8	3	250	2 - #6	1 - #5	48.4	252	44	229	33	172	1.45	.350	3.0	.180	1.0
9	0.02	500	2 - #6	--	29.5	154	28.7	149	24	125	2.1	.080	4.1	.060	0.45
10	3	500	2 - #6	--	34.1	178	32.7	170	28	146	2.8	.130	5.6	.055	0.5
11 (13)	0.02	250	2 - #6	--	20.0	104 (132)	18.9	98 (124)	17	89 (113)	3.8	.120	6.5	.055	0.5
12 (13)	3	250	2 - #6	--	21.8	114 (143)	20.6	107 (136)	18	94 (119)	5.1	.065	8.1	.048	0.55
13	0.02	125	2 - #4	3 - #7, 2 - #5	29.1	151	26.0	135	18	94	1.8	.215	5.2	.075	1.0
14	3	125	2 - #4	3 - #7, 2 - #5	28.8	150	24.0	125	19	99	3.1	.160	6.6	.080	0.9
15	0.02	125	2 - #4	3 - #7, 2 - #5L	35.2	189	33.6	175	23	120	2.5	.320	9.2	.090	1.5
16	3	125	2 - #4	3 - #7, 2 - #5L	36.2	189	32.4	169	22	115	3.4	.190	10.5	.090	1.5
17	3	250	--	--	23.7	123	20.7	108	17	89	1.6	--	--	--	0.7

Notes:
1. Frequency of the sinusoidally applied actuator displacement.
2. Bearing Stress based on the gross area (192 sq. in.).
3. Vertical reinforcement in each jamb of the piers.
4. Horizontal reinforcement.
5. P_{u1} and P_{u2} are the peak shear loads in either direction, and defined in Figure 4. τ_{u1} and τ_{u2} are the corresponding shear stresses based on the gross area.
6. P_1 and P_2 are the average ultimate shear strengths as defined in Figure 4. τ_1 and τ_2 are the corresponding shear stresses based on the gross area.
7. P_3 is a working ultimate shear strength defined in Figure 4. τ_3 is the corresponding shear strength based on the gross area.
8. δ_1 and δ_2 are approximate ductility ratios associated with P_1 and P_2 and defined in Figure 4.
9. Average value of deflection associated with P_1 and defined in Figure 4.
10. δ_3 and δ_4 are ductility indicators associated with P_3 and defined in Figure 4.
11. Average value of deflection associated with P_3 and defined in Figure 4.
12. Maximum input displacement of activator.
13. Grouted at Re-bars only. Values in parentheses are stresses based on net area (152 sq. in.).

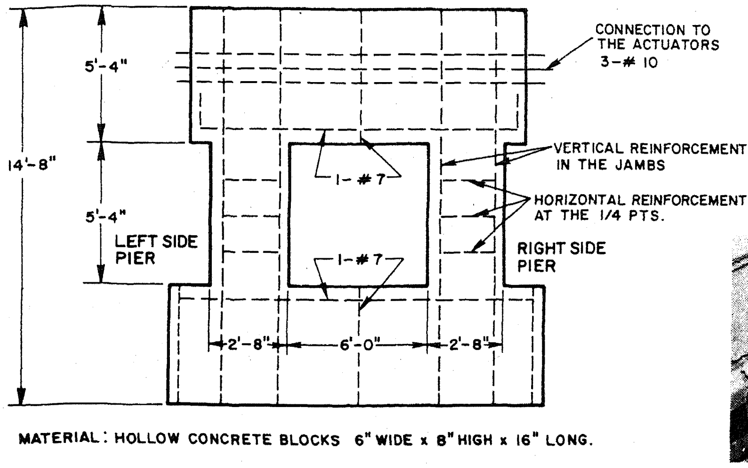

Fig.1 Typical Double Pier Test Specimen

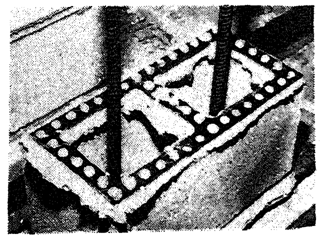

Fig.2 1/8-in. Plate

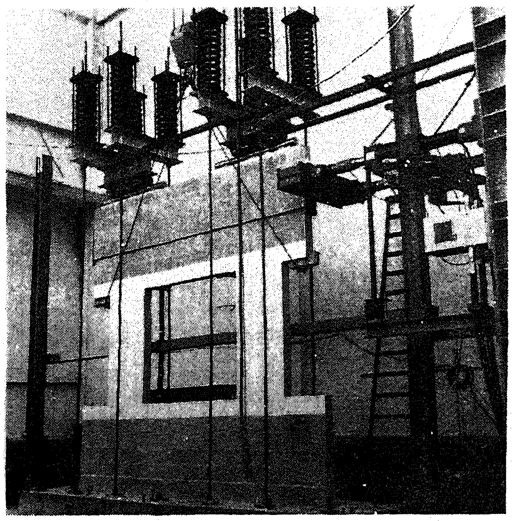

Fig.3 Double Pier Test Set-up

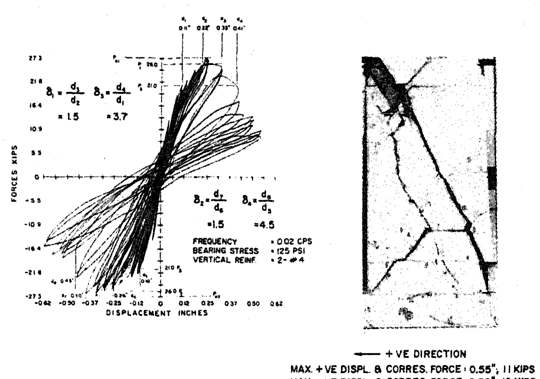

Fig.4 Definition of Shear Strength and Ductility Indicators

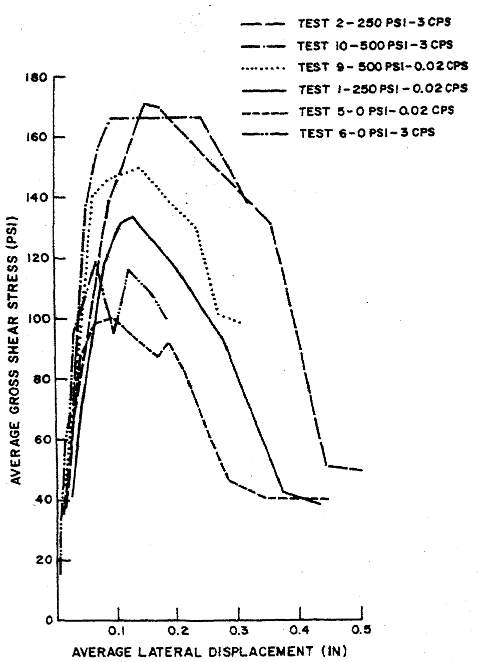

Fig.5 Effect of Bearing Stress

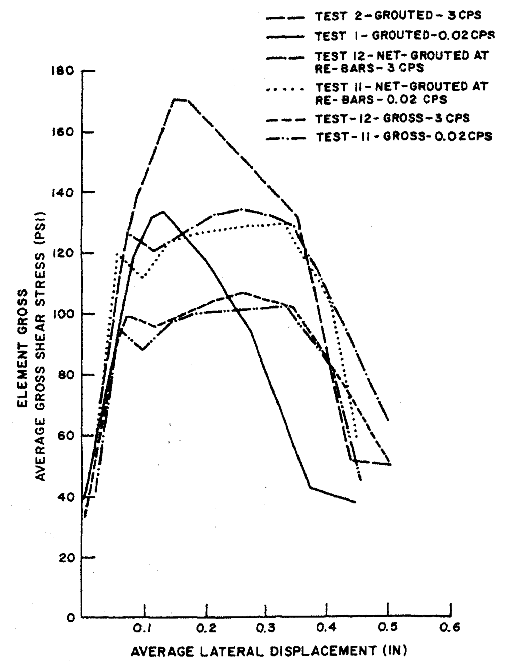

Fig.6 Effect of Partial Grouting

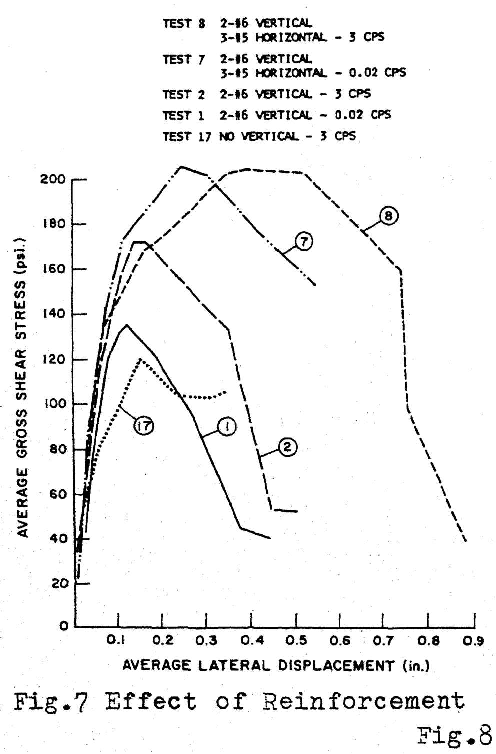

Fig.7 Effect of Reinforcement

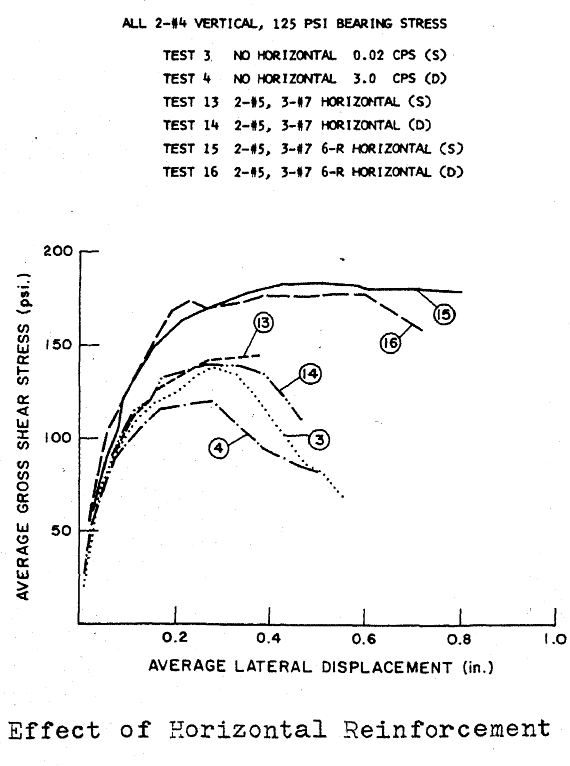

Fig.8 Effect of Horizontal Reinforcement

DISCUSSION

W.O. Keightley (India)

Would the authors please explain the mechanism by which the plates within the mortar joints (Fig. 2, p. 205) resulted in stronger and more ductile piers ?

Author's Closure

With regard to the question of Mr. Keightley, we wish to state that for piers failing in the flexural mode, the actual mechanism of failure is one of secondary compression at the toes of the piers. As the steel yields due to the movement developed at the top and bottom of the piers, the increase in tensile strain causes a corresponding decrease in the compression area of the cracked section, assuming that plane sections remain plane. At a critical strain level, compression failure occurs. This compression failure is characterized by tensile splitting of the masonry units along vertical planes. When this occurs the face shells of the masonry units spall allowing the vertical reinforcing steel in this zone to buckle.

The effect of the plates within the mortar joints is as follows: First the plates confine the tensile splitting of the masonry units to individual units thereby preventing spalling of the face shells. Second, by confining the tensile splitting of the masonry units the compressive strength is increased by 15 to 30%. This increase in compressive strength correspondingly increases the flexural strength of the walls. In addition, by preventing the spalling of the masonry units the ductility of the piers is increased.