THE EFFECT OF JOINT REINFORCEMENT ON VERTICAL LOAD CARRYING CAPACITY OF HOLLOW CONCRETE BLOCK MASONRY

M. Hatzinikolas
J. Longworth
J.Warwaruk

THE EFFECT OF JOINT REINFORCEMENT ON VERTICAL LOAD CARRYING CAPACITY OF HOLLOW CONCRETE BLOCK MASONRY

ABSTRACT:

The effectiveness of wire joint reinforcement in load bearing masonry is experimentally evaluated. Tests on prisms and full scale walls were conducted under axial and eccentric loads. #9 gauge truss type wire reinforcement was used as joint reinforcement. It was used in two forms: as supplied (normal) and flattened to 60% of the original diameter.

All reinforced specimens failed at lower loads than the plain specimens. Those reinforced with normal reinforcement exhibited, lower failure loads than those with flattened reinforcement. The reduction in capacity is attributed to stress concentrations produced by the joint reinforcement.

THE EFFECT OF JOINT REINFORCEMENT ON VERTICAL LOAD CARRYING CAPACITY OF HOLLOW CONCRETE BLOCK MASONRY

Most building codes specify a certain minimum amount of reinforcement to be placed in the horizontal joints of reinforced masonry walls. The Canadian Code(1) in article 4.6.8.1.1, specifies that reinforced masonry load-bearing and shear walls shall be reinforced horizontally and vertically with steel having a minimum area calculated in conformance with the following requirements:

$$A_v = 0.002\ A_g\ \alpha \quad [1]$$

$$A_h = 0.002\ A_g\ (1 - \alpha) \quad [2]$$

where
A_v = area of vertical steel per unit of length of wall
A_h = area of horizontal steel per unit length of wall
A_g = gross section area per unit length of wall reinforcement
α = distribution factory varying from 0.33 to 0.67 as determined by the designer.

The purpose of the horizontal reinforcement is to provide a certain amount of two way action for resisting lateral loads. Theoretically, there is no reason to expect that joint reinforcement will increase the load bearing capacity of concrete masonry walls, especially with the construction procedures commonly used in Canada. The actual effect on vertical load capacity is not well defined.

As a result of the substantial difference in the elastic properties of steel and mortar it can be assumed that the stress distribution in the mortar joint will be similar to the one for a plate with a rigid inclusion. Figure 1 shows the shape of the stress in a uni-formly loaded reinforced mortar joint. This stress distribution has a has a peak of at least 1.56 W, where W is the uniformly distributed load acting on the joint. This distribution is based on the assumption that the steel is infinitely stiffer than the mortar. This is a realistic assumption considering that the ratio of modulus of elasticity of steel to that of mortar is of the order of 40.

In reality the stress distribution is more complex because of the presence of confine-ment stresses and inelastic action. Exact analytical evaluation of the stress distribution in anisotropic plates is complex and beyond the scope of this paper. Reference (1) gives a complete detailed account of stress patterns created in anisotropic plates under various loading conditions.

The purpose of this investigation is to experimentally examine the effect of joint reinforcement and its shape on the load carrying capacity of hollow concrete block masonry.

SPECIMEN MANUFACTURE

Walls and prisms were constructued of 8x8x16 in. (nominal) concrete blocks. The blocks were manufactured locally using 4 parts of light weight aggregate mixed to 1 part sand. The mean compressive strength of the block was 2350 psi. Type S mortar, proportioned by volume, was used. The mortar was mixed in an electrically driven mixer and the workability adjusted to the blocklayers requirements. The average water cement ratio of the mortar (w/c) was 1.2. The mean strength of 50 - 2x2x2" mortar cubes tested was 2500 psi. The hori-zontal joint reinforcement was #9 gauge truss type wire as shown in Figure 2. This reinforcement was used either as supplied or in a flattened form. The wire was flattened to 60% of its original diameter by passing it through a set of rollers. The diameter of the wire was reduced by about 40% in this process. Walls and prisms were constructed by a skilled blocklayer and were cured in laboratory environment at 72° F temperature and 42% relative humidity.

A total of 30 two-block prisms, as shown in Figure 3, were built. Twenty prisms had no joint reinforcement. Ten of these unreinforced prisms were fully bedded in mortar. All other prisms were constructed with face shell mortar bedding. Five of the prisms had "normal" truss-type joint reinforcement and five had "flattened" reinforcement.

A total of 30 short walls, as shown schematically in Figure 4 were constructed in running bond (blocks overlapping by 50%). Ten wall were plain and seven were horizontally reinforced at every second course, five with normal and two with flattened reinforcement.

In addition to the prisms and short walls, twelve full scale walls, 16 blocks high and 2 1/2 blocks wide, were built in running bond. Six were plain and six had normal #9 gauge wire joint reinforcement.

All specimens were tested at an age of at least 28 days.

TEST METHODS

All two-block prisms were tested in axial compression in a 1.6 million lb. hydraulic testing machine, with flat-end conditions. 1/4-inch plates were placed at the ends, and even bearing was achieved by capping the specimen with high strength plaster of Paris. The walls were tested with pin-ended conditions using a roller and channel arrangement shown in Plate 1. To avoid local failure in walls tested with eccentric loads, the top an bottom courses were fully grouted. The full scale walls were tested in double curvature using the same arrangement as shown in Plate 1.

TEST RESULTS

a) Prisms

Failure loads and resulting stresses for the axially loaded two block prisms are given in Table 1. Average stresses for each group of similar specimens are also listed.

The average failure stresses were 2090 psi for the fully bedded prisms, 2009 psi for the face shell bedded, 1895 psi for the prisms with flattened joint reinforcement and 1642 psi for those with normal #9 gauge wire joint reinforcement.

b) Short Walls

Table 2 summarizes the test results for axially loaded short walls and Table 3 summarizes results for eccentrically loaded walls.

The average failure stress for the axially loaded specimens was 2323 psi for the plain, 2129 psi for those with flattened joint reinforcement and 1856 psi for those with normal joint reinforcment.

c) Full Scale Walls

The results of tests on wall specimens subjected to double curvature are shown in Table 4. The stress at failure is calcu-lated using linear stress distribution and the mortar bedded area. $P-\Delta$ effects are neglected in the stress computations.The average stress at failure for plain walls was 3662 psi and for walls containing joint reinforcement was 3215 psi.

DISCUSSION OF TEST RESULTS

a) Prisms

The average failure stresses for prisms with normal joint reinforcement was 18% lower than for plain prisms. For prisms with fiattened joint reinforcement the reduction was 8%. The results for fully bedded prisms indicate that the load capacity is influenced directly by the area of block covered by mortar. Failure for fully bedded specimens occurred at an average stress of 2090 psi as com-pared to 2009 psi for prisms with face shell mortar. The average failure load was in the order of 35.5% higher for the fully bedded prisms.

Failures were caused by splitting of the block at the cross webs for plain specimens and splitting at the flanges for the reinforced ones. These types of failures are illustrated in Plate 2.

b) Short Walls

Axially loaded short walls failed in a similar manner to prisms. Failure modes are illustrated in Plate 3. Short walls with normal joint reinforcement failed at average stresses 20% less than plain ones, an4 specimens with flattened joint reinforcement at 8% less than plain specimens. Fully bedded specimens carried only 10% additional load than specimens with face shell mortar. Eccentrically loaded short wall specimens with normal joint reinforcement failed at an average stress 22% less than the plain specimens.

c) Full Scale Walls

The average stress at failure for full scale walls with normal joint reinforcing tested under axial and eccentric loads was 12% less than for the plain walls. However, for eccentricities larger than 3.0 inches there was no effect due to the presence of the joint rein-forcement. If the results from tests with a 3.5" eccentricity are excluded, the average failure stress for walls with joint reinforcement was 16% less than plain walls.

The results indicate that the presence of joint reinforcement reduces the load carrying capacity of hollow concrete block masonry. It was observed that the mortar joint at failure, for prisms with joint reinforcement was completely crushed, whereas in the case of plain specimens, a ring of hard mortar remained on both blocks. Plate 4 shows this ring of hard mortar at the middle of the flanges and webs of a block after failure. This observation further strengthens the assumption of premature mortar failure at the location of the joint reinforcement.

CONCLUSIONS

On the basis of experimental evidence it is concluded that:

1. Joint reinforcement reduced the ultimate load bearing capacity of masonry walls as a result of a stress concentration created by the presence of the reinforcement.

2. The reduction in strength was less in the case of flattened reinforcement.

ACKNOWLEDGEMENTS

This study was performed in the Department of Civil Engineering at the University of Alberta. Financial assistance from the Canadian Masonry Institute, the Alberta Masonry Institute and the National Research Council of Canada is acknowledged.

REFERENCES

1. CSA Standard S304-M, Masonry Design and Construction, Canadian Standards Association, Rexdale, Ontario, Canada.

2. Lekhnitskii, S.G., "Anisotropic Plates", translated for the second edition by Tsai, S.W. and Cheron, T. Gordon and Breach Science Publishers, New York, London, Paris.

3. Mayes, R.L. and Clough, R.W., A Literature Survey, Compressive, Tensile, Bond and Shear Strength of Masonry, U.S. Department of Commerce, National Technical Information Service, P.B.-246-296.

4. Supplement No. 4 to the National Building Code of Canada, 1975. Issued by the Associate Committee on the National Building Code, National Research Council of Canada, Ottawa.

TABLE 1 - Results From Axially Loaded Two Block Prisms

Prism	Mortar bedded Area in.^{2}	Joint Reinforc.	Load at Failure kips	Stress at Failure Based on Mortar Bedded Area	Stress based on Gross Area of psi in.^{2}
* 1	58.3	Plain	132.4	2271	1111
2	"		117.5	2015	986
3	"		112.9	1936	947
4	"		150.1	2574	1259
5	"		106.6	1828	894
6	"		127.9	2193	1073
7	"		129.8	2226	1089
8	"		136.0	2332	1141
9	"		90.0	1543	755
10	"		115.7	1984	971
		Average	121.9	2090	1023
11	39.1	Plain	75.7	1936	635
12	"		100.0	2557	839
13	"		68.9	1762	578
14	"		78.8	2015	661
15	"		94.3	2411	791
16	"		90.0	2301	755
17	"		60.0	1534	503
18	"		65.5	1675	549
19	"		87.5	2237	734
20	"		65.0	1662	545
		Average	78.57	2009	659
21	"	Flattened #9 Gauge Wire	90.0	2301	841
22	"		98.5	2519	826
23	"		60.4	1544	506
24	"		60.6	1549	508
25	"		50.8	1299	426
		Average	72.06	1842	621
26	"	#9 Gauge Wire	60.5	1547	507
27	"		45.8	1171	384
28	"		55.2	1411	463
29	"		60.1	1537	504
30	"		70.0	2790	841
		Average	58.32	1646	540

* Specimens 1 to 10 were fully bedded.

** For specimens 11 to 30 mortar was placed at the face shells only.

TABLE 2 - Results of Tests on Short Walls Axially Loaded

Specimen	Mortar Bedded Area in.^{2}	Joint Reinforc.	Load at Failure kips	Stress at Failure Based on Mortar Bedded Area psi	Stress Based on Gross Area psi
* 1	152.5	plain	257.4	1687	553
2	152.5	plain	260.0	1704	558
Avg.			258.7	1696	555
** 3	100	plain	215.5	2155	706
4	100	plain	249.1	2491	816
Avg.			232.3	2323	761
5	100	Flattened	234.8	2348	769
6	100	#9 Gauge Wire	191.1	1911	626
Avg.			212.9	2129	698
7	100	#9 Gauge	200.0	2000	655
8	100	Wire	171.2	1712	561
Avg.			185.6	1856	608

* Specimens 1 and 2 were fully bedded in mortar.

** For Specimens 3 to 8 mortar was placed only at the face shell.

TABLE 3 - Results From Eccentrically Loaded Short Wall Specimens

Specimen	Mortar Bedded Area $in.^2$	Joint Reinforc.	Eccen-tricity in.	Load at Failure kips	Moment at Failure k-in	Stress at Failure Based on Mortared Area (psi)
1	100	plain	t/6=1.27"	196.9	250.0	3537
2	"	"	t/6=1.27"	150.1	190.6	2696
3	"	"	t/3=2.54"	119.3	303.0	3094
4	"	"	t/3=2.54"	158.7	403.0	4115
Avg.						3360
5	"	#9 Gauge	t/6=1.27"	160.0	203.2	2875
6	"	Wire	t/6=1.27"	149.1	189.35	2679
7	"	"	t/3=2.54"	92.75	235.5	2405
8	"	"	t/3=2.54"	105.5	264.9	2717
9	"	"	t/3=2.54"	92.75	235.5	2405
Avg.						2616

TABLE 4 - Loading Conditions and Test Results
From Full-Scale Wall Segments

Wall	Joint Reinforc.	h/t	Eccen-tricity of top in.	Eccen-tricity at bottom in.	Failure Load kips	Maximum Stress at Failure Based on Mortared Area psi
D1	plain	19.97	0.00	0.00	218.3	2183
N1	"	"	+1.27	-1.27	191.3	3511
N2	"	"	+2.54	-2.54	158.6	4236
N3	"	"	+3.00	-3.00	154.9	4606
N4	"	"	+3.50	-3.50	123.3	4072
N5	"	"	+1.27	0.00	183.5	3368
Avg.						3662
F1	#9 Gauge	"	0.00	0.00	160.0	1600
F2	Wire	"	+1.27	-1.27	160.0	2936
F3	"	"	+2.54	-2.54	144.6	3862
F4	"	"	+3.00	-3.00	124.6	3705
F5	"	"	+3.50	-3.50	128.8	4253
G1	"	"	+1.27	0.00	160.0	2936
Avg.						3215

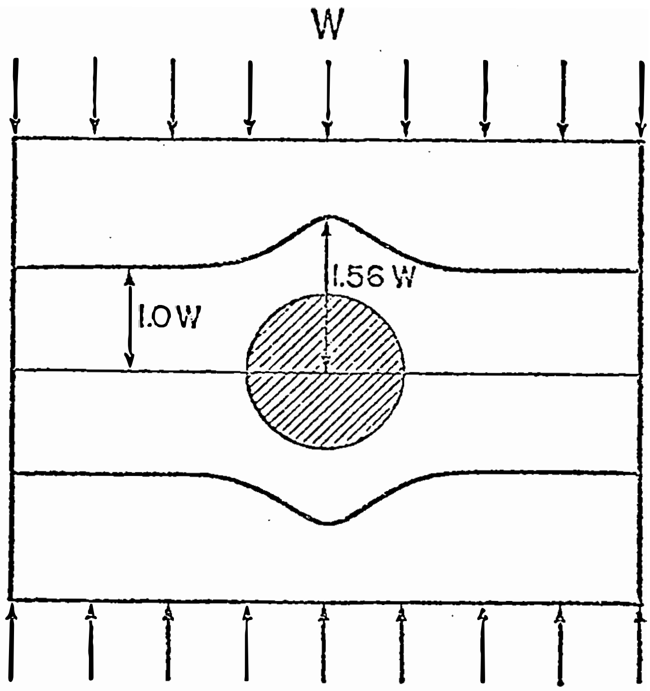

FIGURE 1 -Stress Distribution in a Plate
With Rigid Inclusion

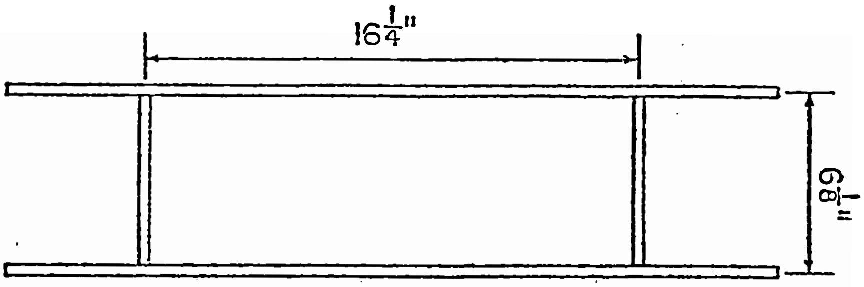

FIGURE 2 -#9 Gauge Wire Joint Reinforcement

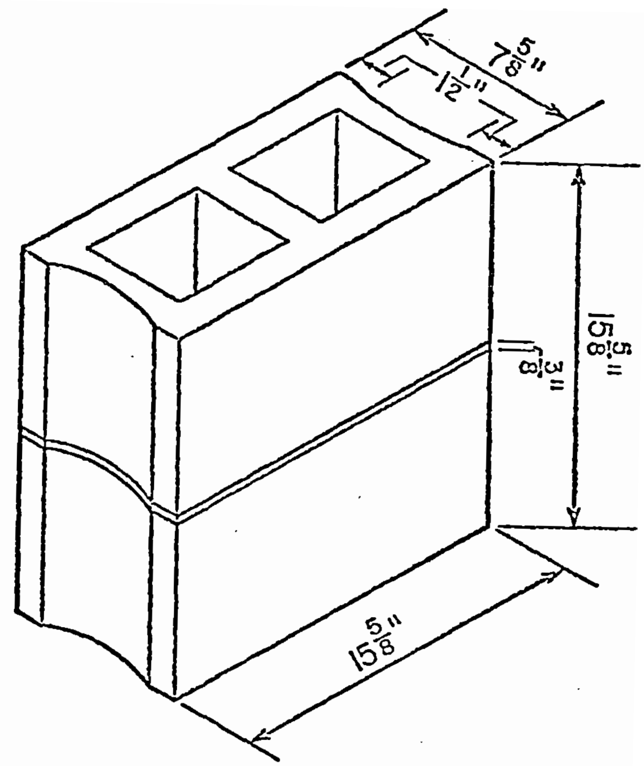

FIGURE 3 - Two Block Prism

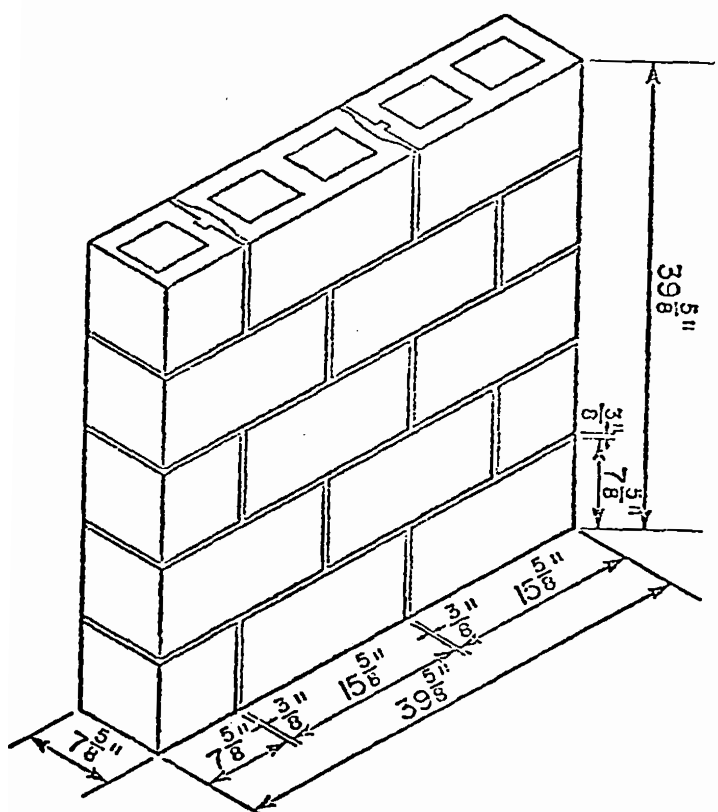

FIGURE 4 - Short Wall Specimen

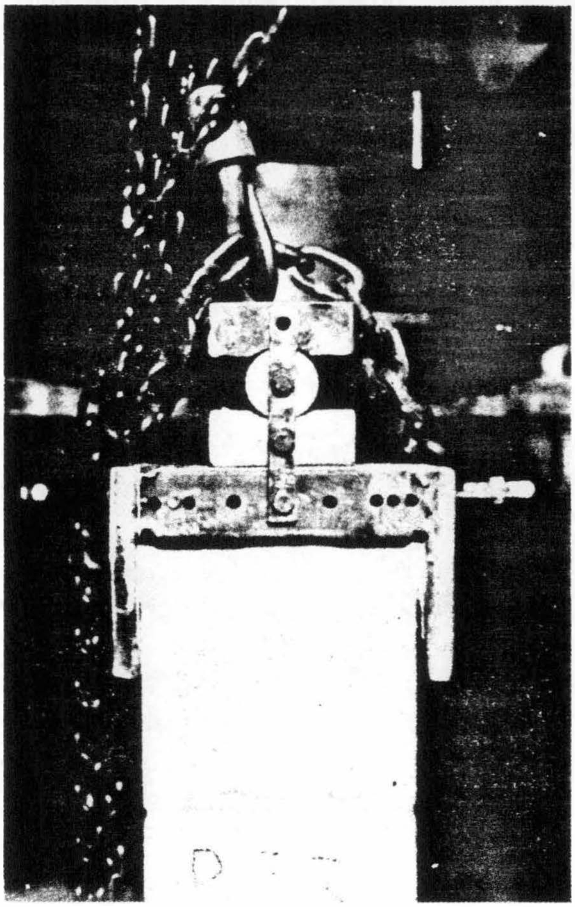

PLATE 1 - Loading Arrangement for Prism and Walls Tested With Pin-Ended Conditions

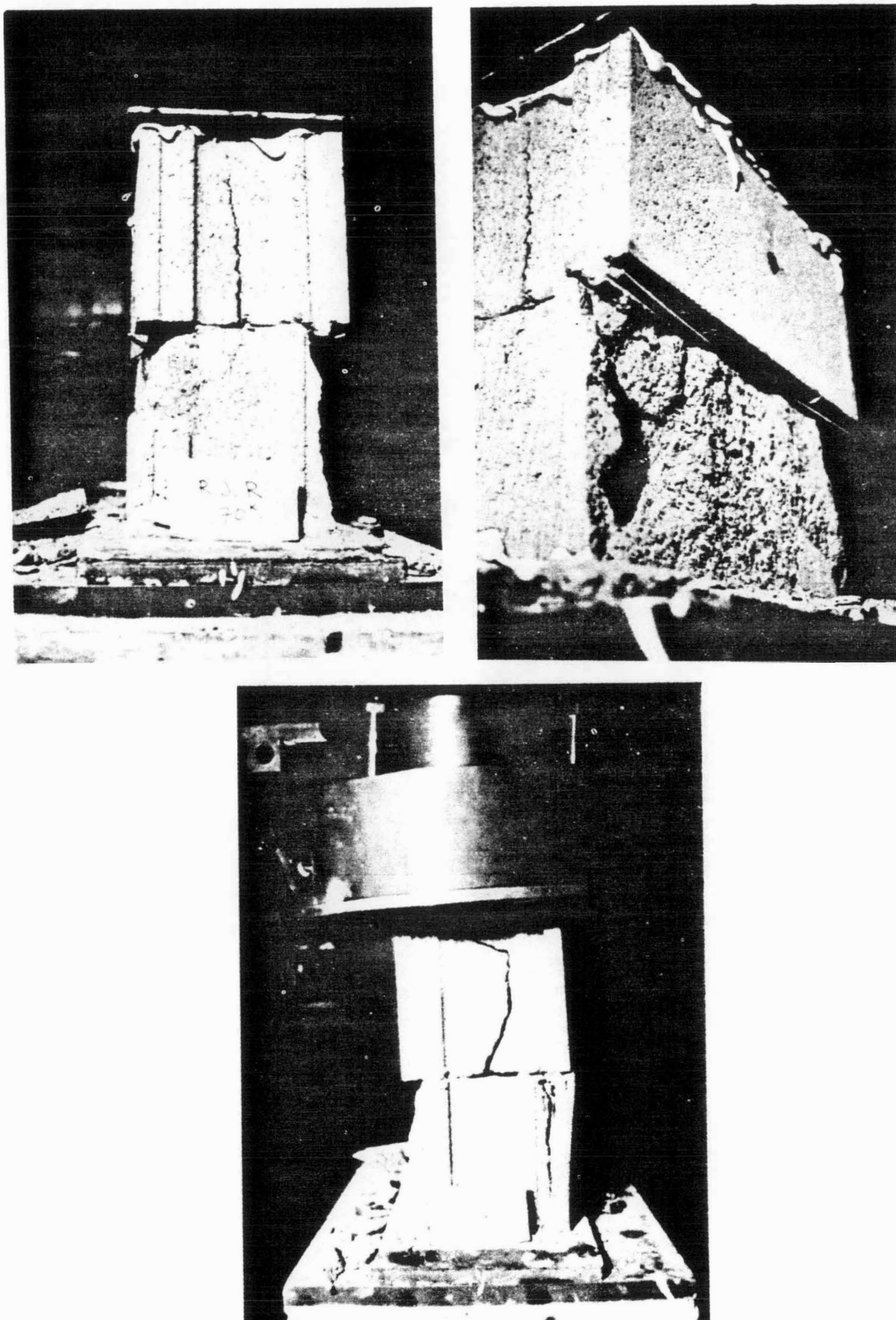

PLATE 2 - Typical Failures of Prisms With No. 9 Gauge Wire Joint Reinforcement

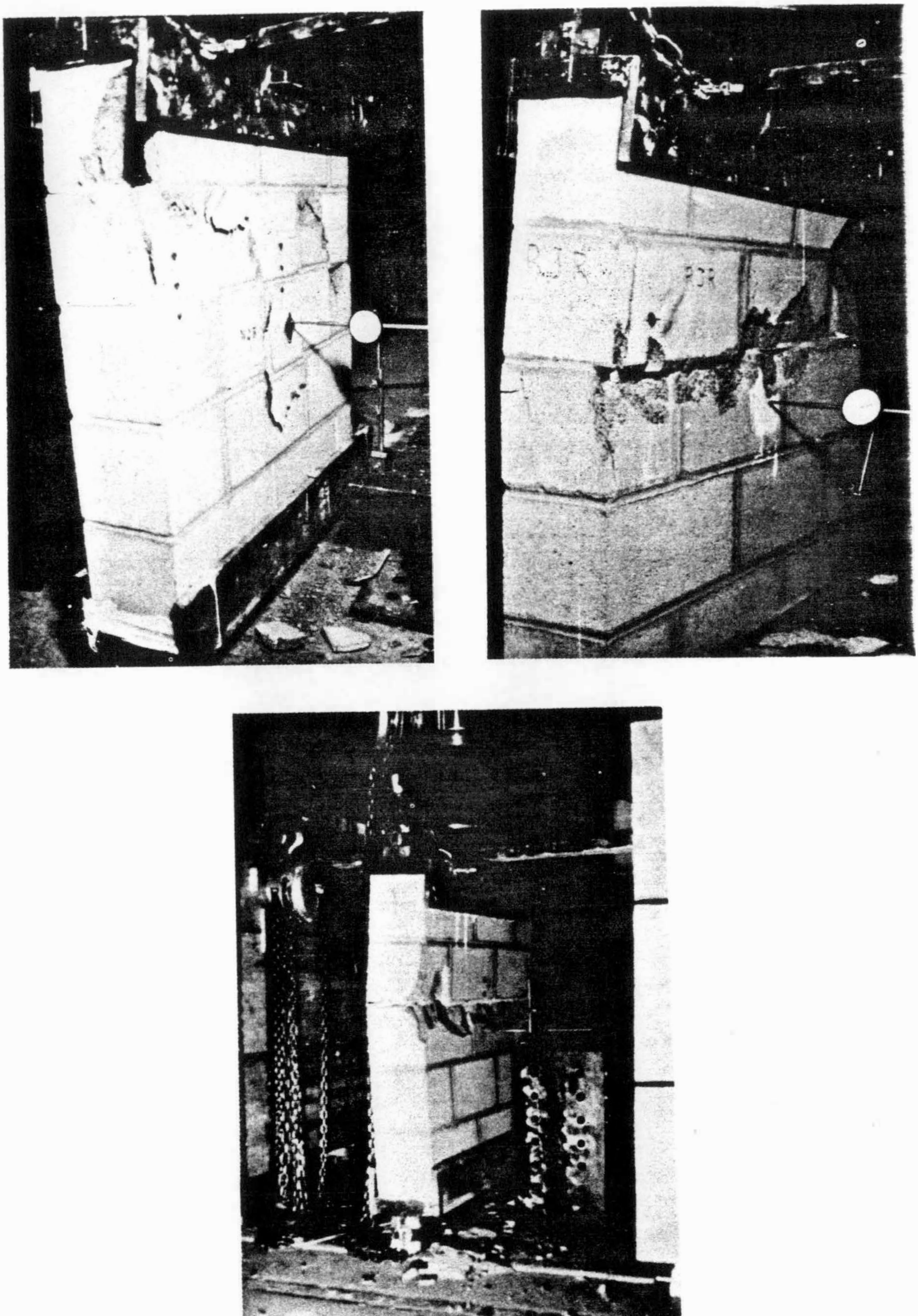

PLATE 3 - Short Wall Specimens With and Without Joint Reinforcement After Failure

PLATE 4 - Ring of Hard Mortar on Fully Bedded Prism After Failure