8TH CANADIAN MASONRY SYMPOSIUM

Jasper, Alberta
May 31 - June 3, 1998

TESTS OF UNREINFORCED MASONRY WALLS STRENGTHENED WITH EXTERNAL FIBER REINFORCED PLASTIC

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ABSTRACT

A testing program conducted at the University of Alberta showed that externally applied fiber reinforced plastics (FRP) are effective in increasing the load carrying capacity of unreinforced masonry walls. Ten walls with a height of four meters were used to conduct thirteen tests in two series. Both undamaged and slightly damaged walls were tested. The following testing parameters were investigated: type, amount, and layout of reinforcement, axial load effects, and cyclic behaviour. This paper briefly reviews the existing rehabilitation methods available and explains why the use of FRP is a possible alternative. Results of material tests performed on the masonry and FRP materials are presented. The test setup, instrumentation of the specimens, and general test procedure are described. The general behaviour of the specimens is discussed with emphasis on the load – deflection characteristics and strain distributions. Finally, the modes of failure are identified and categorized. Overall results show that externally applied FRP greatly increases the strength and ductility of ungrouted unreinforced masonry walls.

INTRODUCTION

A large percentage of existing buildings in North America and around the world have been constructed with unreinforced masonry. The masonry elements in these buildings were designed to resist primarily gravity and wind loads with little or no consideration of the forces generated by a seismic event. Typical damage suffered by these buildings during an earthquake range from minor cracking to catastrophic collapse. Several conventional rehabilitation and strengthening methods have been reviewed and summarized by Hamid et al. (1994), Kingsley (1995), and Modena (1994). Of the methods considered, injection grouting, insertion of reinforcing steel, prestressing, jacketing, and various surface treatments are the most common. Each of these methods involves the use of skilled labour and disrupts the normal function of the building. Jacketing and surface treatments such as shotcrete, ferrocement, and reinforced plaster, can add anywhere between 30 to 100 mm of thickness to the existing wall. This translates into a significant increase in mass and may cause inertial forces induced by an earthquake to be greater than before the rehabilitation.

The use of Fiber Reinforced Plastics (FRP) as rehabilitation and strengthening material is a valid alternative. Appealing characteristics of FRP reinforcement are negligible weight, high strength to weight ratio, low strains at ultimate, and ease of application. Various tests have been performed on concrete and masonry elements reinforced with FRP with focus on the in-plane strength (Schwegler 1994, Weeks et al. 1994). Results show a marked improvement in the ductility and load carrying capacity of the elements tested. Out-of-plane tests were performed on small scale brick beams reinforced with glass fibers (Ehsani 1995). Results show that the strength of the fiber used directly affects the stiffness and governing mode of failure of the specimens.

Little information exists regarding the out-of-plane behaviour of unreinforced masonry walls reinforced with FRP. This paper presents the findings of a test program designed to provide information on this issue. An explanation of the test program and the variables considered is provided followed by a summary of the results.

EXPERIMENTAL PROGRAM

The experimental program consisted of ten masonry walls reinforced with externally applied fiber reinforced plastics (FRP). The walls were constructed in two series. Loading was out-of-plane with two line loads applied 1.2 m from the supports. The parameters investigated were the type, (carbon strap, carbon sheet, glass sheet), amount, and layout of reinforcement, axial load effects, and cyclic behaviour.

Material Properties

Two groups of materials were tested, those related to the masonry and those related to the reinforcement.

Ancillary masonry tests included 30 masonry units, 36 mortar cubes and ten masonry prisms five courses high. The results of the individual unit and mortar tests are summarized in Table 1. Table 2 summarizes the results of masonry prisms. Actual dimensions were used in the calculation of net compressive area for all masonry tests.

Tension coupons of the different types of fiber reinforcement were constructed and tested in accordance with ASTM Standard D3039/D3039 M-95a. Table 3 shows the results of all of the FRP tension tests. Both glass fiber coupons failed near the grips so the true ultimate stress and strain was not achieved. All other coupons failed near the center of the specimen. The stress – strain behaviour of all of the composite fibers is linear with no yield point typically associated with steel coupons.

Test Specimens

Details : A total of ten four meter high walls were constructed in two series by professional masons. A different crew was employed for each series. Specified dimensions for each specimen was 4 m high by 1.2 m wide by 0.2 m thick. Each specimen was 20 courses high with #9 gauge joint reinforcement every 3rd course. Standard 15 MPa masonry block and factory mix Type S mortar were used. None of the cores were grouted. The walls were built on 1200 mm x 200 mm x 50 mm steel base plates. Running bond was used and the joints were finished flush with the outside of the block. A communication error resulted in two different types of masonry block being used. Series One consisted of four walls built with standard 200 mm blocks. Series Two consisted of six walls built with standard 8" (203.2 mm) blocks. This changed the actual dimensions of the specimens to 4.05 m high by 1.205 m wide by 0.193 m thick when the imperial block is converted to metric dimensions. All specimens were allowed to cure for at least 28 days before fiber reinforcement was applied.

Reinforcement Strategy : Series One involved seven tests on the four walls and focused on varying the type of reinforcement. One wall was first tested without reinforcement, then tested again as a partially cracked wall, and finally as a fully cracked wall. One was reinforced on one side and tested until fully cracked, then additional reinforcement was placed on the opposite side and the wall was tested again in a cyclic manner. Series Two involved six tests on the six walls and focused on varying the layout and amount of carbon fiber sheet. The reinforcement was primarily orientated in the vertical direction to optimize the strength of the fibers; however, one specimen was tested with the reinforcement strips oriented diagonally. The purpose of this test was to determine the out-of-plane resistance of a wall reinforced primarily for in-plane loads. Effects of axial load were also investigated in this series. Figure 1 shows the different layout patterns tested. Table 4 summarizes the variables investigated for each test. Because metric blocks were used in the construction of the specimens in Series One the designation (M) is used to identify the tests. Similarly (I), for imperial, is used to identify the specimens from Series Two. Each test is designated by the series, (M) or (I), followed by the type of reinforcement used; (CS) for carbon strap, (CST) for carbon sheet, and (GST) for glass sheet, followed by the test number. An additional number preceded by a hyphen indicates the specimen is being used again for the current test. For example, MCST 7-4, indicates Series One (metric walls), carbon sheet, test 7, and it is using the same specimen from test 4.

Test Setup

All specimens were loaded in the test frame shown in Fig. 2. The walls were tested as a simply supported beam standing on end. A hydraulic jack supplied the loads that were transferred to the wall using a distribution frame constructed for the test. The jack load was centered on the distribution frame which then separated the concentrated load into two line loads located at a height of 1.3 m and 2.7 m from the base of the wall. The line loads rested along the full width of the wall. Knife edges and rollers were used for the loading and lower gravity supports. The top and bottom reaction supports consisted of a built-up hollow structural section which spanned the width of the wall. A series of loose hinges tied back with steel rods to the loading frame allowed for rotation and translation of the ends while providing stability by maintaining a tensile load.

For the tests involving axial load, a modification to the test frame was made. A combination knife edge and roller boundary condition was placed on the top of the wall to allow the axial load to remain vertical at all times. Load rods were run from the ends of the knife edge, down the sides of the wall, and through the strong floor in the laboratory. The load rods were attached to springs underneath the strong floor. When compressed by a hydraulic jack, the springs maintained a constant axial load for the duration of the test.

Instrumentation

The following instrumentation was used: load cells to measure the jack load and reaction loads, Linear Variable Displacement Transducers (LVDT's) to measure deflections, and Demec and electronic strain gauges to measure masonry and reinforcement strains.

A 100 kN capacity load cell was used to measure the force from the jack. The reaction load was measured at the four corners of the specimen using T-bone load cells. The deflection of the wall was measured using a series of 13 LVDT's placed at 400 mm intervals along the height of the wall and 200 mm intervals around loading points. The deflection measurements were taken on the compression side to minimize fluctuations in the readings due to separation of the mortar joints on the tension face.

Masonry strains were measured using a 50 mm Demec gauge. Gauges were placed primarily in the horizontal direction and measured the strain distribution from the edge of the wall to the centerline. Reinforcement strains were measured using 5 mm electronic strain gauges. The location of the reinforcement gauges varied from test to test but concentrated on the mortar joint strains along the height of the wall.

Test Procedure

The specimens were lifted into the test frame by an overhead crane. After the specimen was properly aligned, lateral load was applied at a rate of 0.87 mm per minute. The test was controlled using an existing computer program and all electronic readings were recorded using this program. Electronic readings were taken at approximately one quarter kN intervals. Demecs were recorded at regular intervals up to 20 kN of load. General observations such as crack patterns and crack widths were made throughout the test. For the tests involving axial load, the axial load was applied immediately after positioning of the wall and before any lateral load was applied.

The procedure for test MCST 7-4 involved cyclic loading. Because of the way the test was arranged, it was not possible to load the specimen in the reverse direction. As a result, the specimen was never taken past zero during the cycles of loading and unloading. The specimen was loaded using the jack load as a guide for the beginning of the cycles. Three cycles were performed at 5 kN and 10 kN as the upper limit. After this point, the wall was loaded to twice the deflection obtained at the 10 kN level and three cycles were performed. Then the wall was loaded to three times the deflection and so on.

TEST RESULTS

The major areas of interest are the load – deflection characteristics of the specimens, the tensile and compressive strain results, and the modes of failure. The overall general behaviour of a specimen during a typical test is also presented.

Load – Deflection Behaviour

Figures 3 and 4 show the combined load – deflection results for Series One and Series Two respectively. Comparing the load vs. mid-span deflections for all twelve reinforced tests, the overall shape of the curves can be divided into two distinct sections. The first section of the curve is a gradual arc which continues until around 10 to 20 mm of mid-span deflection. Most likely this initial portion of the curve is a result of the mortar joints debonding. Only occasionally did a crack form within the mortar itself. The second portion of the curve is approximately a straight line representing the contribution of the reinforcement stiffness to the behaviour of the specimen. At this stage, all horizontal joints within the constant moment region have debonded.

It is beyond the scope of this paper to explain in detail the significance of all the variables; however, some general observations can be made. It appears that the imperial block in Series Two causes a slight increase in initial wall stiffness. The type of reinforcement used does not appear to significantly affect the overall behaviour. The stiffness of a specimen increases with an increase in the amount of fiber reinforcement used. The axial load increases the stiffness of the initial portion of the response while decreasing the stiffness during the second portion.

Typical Strain Behaviour

The measured strains can be categorized into two main areas of interest: masonry strains and reinforcement strains. The next two sub-sections focus on general strain behaviour for a typical test, ICST11.

Masonry Strains : The strains along the length of the wall can be separated further into block and joint strain behaviour. Figure 5 shows the typical block and joint strain behaviour for specimen ICST11, which was reinforced with two 250 mm wide carbon fiber sheets. Symmetry was assumed and readings were only taken up to the vertical center line of the wall.

The block results shown are centered on the 6th course at a height of 1.112 m from the base of the wall. This course is outside of the constant moment region, just below the lower load point. The figure shows that the strains are much higher on the reinforcement than in the block itself. The weak bond between the mortar and the block does not allow it to carry much tension. At a load of 5 kN the reinforcement is already picking up most of the load; however, some strain is transferred to the block directly next to the strip of reinforcement. Within the reinforcement itself the strains become higher as they approach the center of the strip. While not shown, compression strains measured in the same location on the opposite side of the wall follow a similar pattern. The strains are slightly concentrated behind the reinforcement and gradually reduce to a constant value away from the reinforcement.

The joint results shown are centered over the mortar joint directly below the aforementioned block strains at a height of 1.010 m. The tensile joint strain behaviour is essentially the opposite of the block strain behaviour. The strain is high across the masonry joint and gradually reduces towards the reinforcement. The reinforcement experiences the lowest strains and as a result restrains the joint from opening freely. In this case, the compression strains directly behind the reinforced area are constant, if not a little bit lower, than the surrounding strain in the masonry joint. Again, the compression strains are not shown.

A typical load – strain response for joint strains is shown in Fig. 6. The curve is similar to a load – deflection plot. The curve becomes linear after about 15 kN of load has been applied which corresponds with the change in slope of the load – deflection curve for this specimen.

Fiber Reinforcement Strains : Figure 7 shows a typical load – strain plot for the fiber reinforcement at both joint and block locations within the constant moment region. Again there is a distinct difference between the joint and block strains. The block strains start out as linear up to a load of approximately 14 to 15 kN. Then the strains increase rapidly with very little increase in load until they stabilize and again take on a linear form. This initial change in slope of the response again corresponds with the change in slope of the load – deflection curve for specimen ICST11. The sudden increase in strain can be attributed to the reinforcement requiring a longer development length as the horizontal joints become fully cracked. The joint strains follow approximately the same initial slope as the block strains but then suddenly change slope around 5 kN. This may indicate the point at which the joint in that particular location has begun to debond. The remainder of the curve has a relatively linear behaviour.

Failure Modes and Overall Behaviour

Out of the thirteen specimens tested three general modes of failure were observed: *mortar debonding* or *mortar slip*, *flexure–shear*, and *flexure*.

Both *mortar debonding* and *mortar slip* involve separation of the mortar from the adjacent masonry block. This accounts for the failure of two specimens, MU1, unreinforced, and MCS3-2, two, 50 mm wide, carbon straps. The unreinforced wall lost its ability to carry load through debonding failure at the 13th joint from the base of the wall. For specimen MCS3-2 the 1st joint from the base of the wall slipped in the horizontal direction. This mode of failure, as shown in Figure 8, occurred because the reinforcement did not have sufficient bonded area over the joint to restrain the shear forces. This was determined to be an undesirable mode of failure and for future tests carbon fiber patches were placed over the lower and upper reaction joints to provide enough shear resistance.

The second, and most common, mode of failure was *flexure–shear*. Because the shear span to specimen depth ratio is 6, shear failure was not expected to be an issue. However, the reinforced specimens experienced enough combined load and deflection to induce a flexural crack on the edge blocks, usually between the 5th and 7th courses. Once this flexure crack had progressed vertically about 15 mm in length, a shear crack would begin to propagate towards the compression face of the specimen. The resulting failure is shown in Fig. 9. Six specimens failed in this manner. An additional specimen was unloaded before the shear cracks induced failure.

The last mode of failure was *flexure*. Specimens ICST10, which was reinforced with two 125 mm carbon sheets, and ICST12, reinforced with ten 125 mm carbon sheets orientated at an angle of 37° from vertical, failed in this manner. In both cases the reinforcement ruptured along one or two of the horizontal joints in the constant moment region. Flexure–shear cracks did not develop. Figure 10 shows the failed joint for specimens ICST10. The figure clearly shows that the masonry is firmly bonded to the failed strip of reinforcement.

Table 4 summarizes the failure modes for each specimen tested along with the corresponding failure load and mid-span deflection. In all of the reinforced specimens failure occurred without significant warning.

Each reinforced specimen followed a series of steps before failure occurred. The first event that happens is the progressive separation of the horizontal mortar joints in the constant moment region. Once every joint has fully separated diagonal cracks begin to appear in random locations. The cracks begin at the edge of the reinforcement strips and angle up or down to the nearest horizontal joint. The difference in strain between the reinforcement and the adjacent joint, as shown in the section on strain behaviour, explains the formation of these diagonal cracks. In the tests that experienced mid–span deflections over 70 mm, the reinforcement would begin to pull the face of the masonry block away from the wall. This occurred when diagonal cracks had fully formed around a joint location. Next, horizontal flexure cracks would form in random locations. The cracks would typically start from the center of a block at the edge of the reinforcement strip and progress outwards towards the edge of the wall or towards the center of the wall. In some cases the flexure cracks spanned completely between the strips of reinforcement. Not all joints and blocks experienced these diagonal cracks and flexure cracks. Finally, flexure–shear cracks would form on the edge blocks of the wall. These cracks formed between the 5th and 7th courses from the base of the wall, just below the lower load point, as well as between the 14th and 16th courses, just above the upper load point. One of these cracks would then progress until failure. Figure 11 illustrates the typical crack pattern observed in the constant moment region.

SUMMARY AND CONCLUSIONS

Results show that the out-of-plane strength can be increased by 20 to 45 times that of an unreinforced wall tested under the same conditions. The ductility is also greatly increased with mid-span deflections ranging from 30 mm to 90 mm at failure compared to less than 1 mm for the unreinforced wall. The reinforced walls were able to resist between 4 kPa and 9 kPa of equivalent out-of-plane pressure. Failure modes ranged from shearing of the wall near the load points to flexure failure of the reinforcement. In both cases failure is brittle. Application of the reinforcement is simple and unobtrusive compared to traditional methods of rehabilitation. The behaviour of the wall is similar for all types of fiber tested. The amount of reinforcement used and relative strengths seem to govern the second portion of the response. Further research is required to investigate the behaviour of the walls under true cyclic loading. Overall, the tests showed that externally bonded FRP is an effective alternative to rehabilitating unreinforced masonry walls.

ACKNOWLEDGMENTS

The authors would like to thank ISIS Canada for providing funding and the Canadian Masonry Research Institute for providing masonry materials and labour for this research.

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Table 1 Masonry Unit and Mortar Compressive Strengths

	Series One		Series Two	
Statistics	Blocks (MPa)	Mortar (MPa)	Blocks (MPa)	Mortar[1] (MPa)
mean	19.9	12.3	15.9	14.7
stand. dev.	2.49	1.33	2.85	1.54
c.o.v.	0.12	0.11	0.18	0.10

[1]Eighteen specimens tested – all others, fifteen tested.

Table 2 Masonry Prism Test Results

	Series One		Series Two	
Statistics	f'm (MPa)	E[1] (MPa)	f'm (MPa)	E (MPa)
mean	7.3	9155	13.4	10249
stand. dev.	1.74	1472	1.58	616
c.o.v.	0.24	0.16	0.12	0.06

[1]Data unavailable for one specimen – five specimens tested in each series.

Table 3 Fiber Reinforcement Coupon Test Results

Fiber Type	σ_u (MPa)	ε_u ($\times 10^{-6}$)	E (MPa)
Glass Sheet (two specimens tested)	106	6264	17770
Carbon Strap (four specimens tested)	2749	14842	185180 (std. dev. = 2256 c.o.v. = 0.012)
Carbon Sheet (six specimens tested)	545	12393	43701 (std. dev. = 9971 c.o.v. = 0.228)

Table 4 Reinforcement Strategy and Summary of Results

Specimen	Number of Strip: Tension Face	Number of Strip: Compression Face	Area of FRP in Tension (mm^2)	Layout	Axial Load (kN)	Maximum Load (kN)	Mid-Span Defl. (mm)	Mode of Failure	Location of Failure
MU1	N/A	N/A	N/A	N/A	-	1	0.7	mortar	13th joint
MCS2-1	2	-	127	A	-	12	14	not failed	-
MCS3-2	2	-	127	A	-	21.8	31	mortar	1st joint
MCST4	2	-	55	B	-	28.9	58	not failed	-
MGST5	2	-	500	B	-	36	70	flex.-shear	5th course
MCS6	4	-	254	C	-	46.4	42	flex.-shear	6th course
MCST7-4	2	2	55	B	-	32.7	78	flex.-shear	6th course
ICST8	2	-	110	B	-	50.2	63	flex.-shear	7th course
ICST9	2	-	55	B	10	33	71	not failed	-
ICST10	2	-	27.5	D	-	20.9	82	flexure	11th-12th joint
ICST11	2	-	55	B	-	41.7	91	flex.-shear	7th course
ICST12	10	-	40	E	-	22.7	62	flexure	12th joint
ICST13	2	2	55	B	30	37.7	88	flex.-shear	6th course

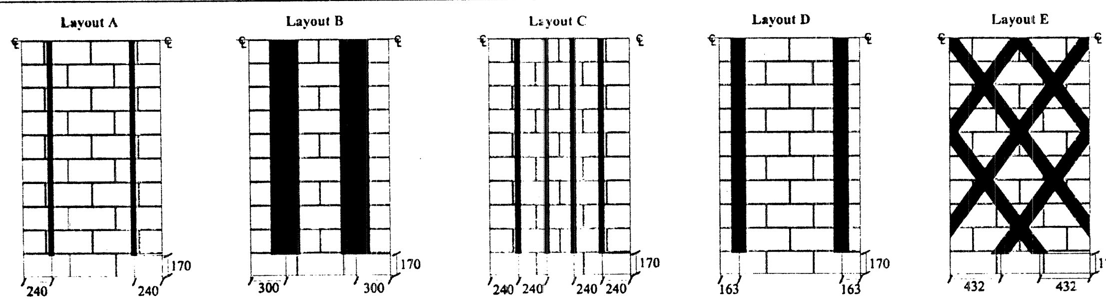

Figure 1 Reinforcement Layout Patterns

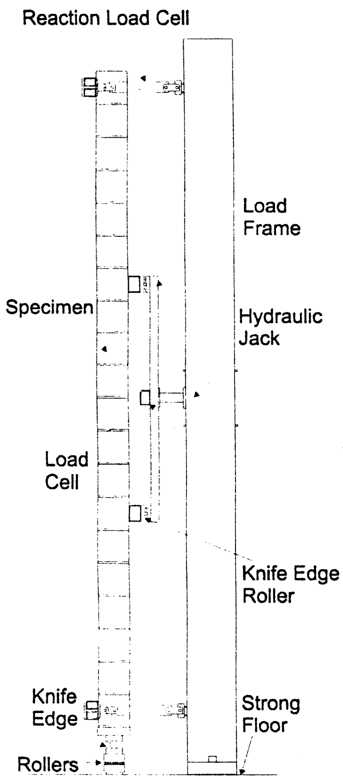

Figure 2 Testing Frame Details

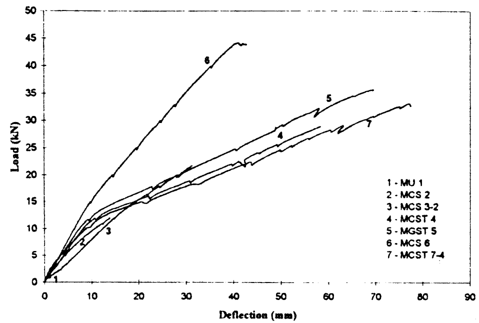

Figure 3 Series One Load – Deflection Behaviour

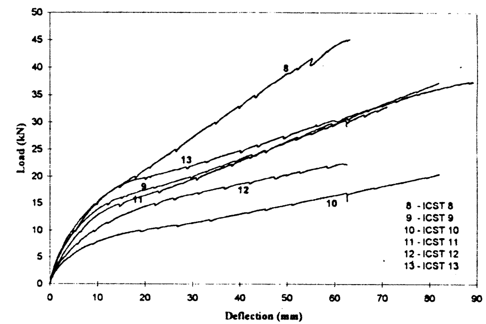

Figure 4 Series Two Load – Deflection Behaviour

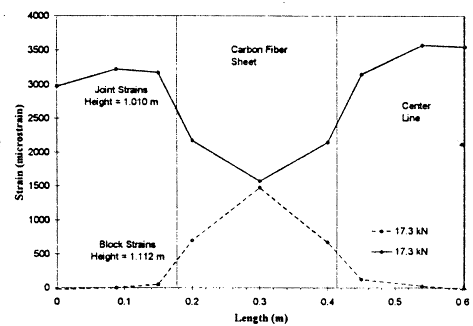

Figure 5 Typical Strains Along the Width of the Wall

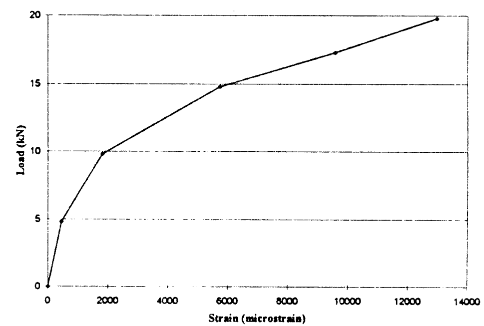

Figure 6 Masonry Joint Strain Behaviour

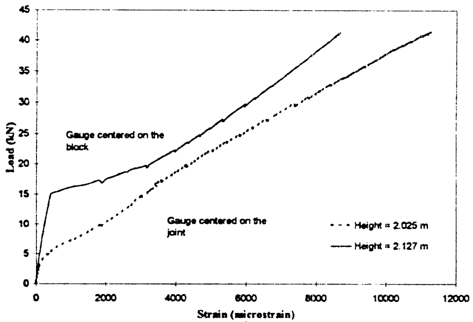

Figure 7 Typical Load – Reinforcement Strain Behaviour

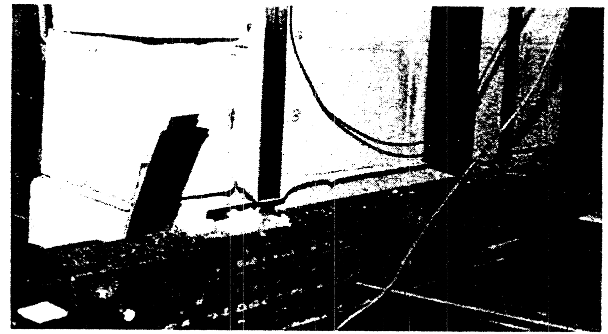

Figure 8 Mortar Slip Failure

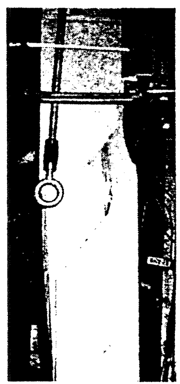

Figure 9 Flexure – Shear Failure

Figure 10 Flexure Failure

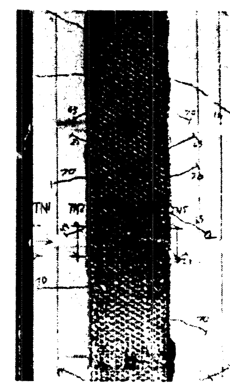

Figure 11 Typical Crack Patterns