A DESIGN METHODOLOGY FOR FRP SYSTEMS FOR MASONRY STRUCTURES

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ABSTRACT

As part of a combined effort at North Carolina A & T State University and The University of North Carolina at Charlotte, an investigation into the repair of unreinforced masonry structures is underway. The objective of this research is to develop a methodology for the design of fibre reinforced plastic (FRP) repair/strengthening systems for masonry shear walls. The prime focus of the investigation involves evaluating the use of small assembly tests to predict the behaviour of large scale structures. The small assembly tests were conducted on both brick and block masonry prisms and these prisms were subjected to either tension, compression, or shear loading to failure. The behaviour of the small tests were used to predict the performance of two large scale test specimens that were repaired/strengthened by FRP systems and the results were compared. Reasonable agreement was obtained during this comparison but further development is needed to improve the modeling procedures.

KEYWORDS: strengthening, FRP, design

INTRODUCTION

As part of a combined effort at North Carolina A & T State University and The University of North Carolina at Charlotte, an investigation into the repair of unreinforced masonry structures is underway. The objective of this research is to develop a methodology for the design of fibre reinforced plastic (FRP) repair/strengthening systems for masonry shear walls. The prime focus of the investigation involves evaluating the use of small assembly tests to predict the behaviour of large scale structures. The small assembly tests were conducted on both brick and block masonry prisms that were subjected to tension, compression or shear loading to failure. The behaviour of the small tests are then used to predict the performance of large scale test specimens that were repaired/strengthened by FRP systems The small scale test are used as the basis of both hand calculation methods and a simple Finite Element model.

SMALL SCALE TEST PROGRAM

A number of small masonry prisms were fabricated as companion specimens at UNC-Charlotte at same time that the large scale wall specimens were being constructed. Each of these prism sets was constructed as using the same units and mortar as the building specimens.

Six compression specimens (prisms) were fabricated using standard CMU, 200 mm x 200 mm (half blocks), and two units high. Three of these prisms were also reinforced with two layers of glass fibre mat (1.0 mm thick for each layer of fibre) set in an epoxy matrix on opposing sides of the prisms. Six double wythe brick specimens (prisms) were also constructed using standard clay brick five units (90 mm x 190 mm x 57 mm) high with one cross header course at mid-height. Three of these specimens were reinforced by cutting a 6 mm x 19 mm vertical groove into the centre of two opposing sides of the brick prism and setting a carbon fibre reinforced polymer (CFRP) bars, (Aslan 500 #2), into the grooves with epoxy. These prism specimens were allowed to cure at least 28 days in lab air environment.

After capping, each of the prisms was tested to failure using the procedures defined in ASTM C 1314 Standard Test Method for Compressive Strength of Masonry Prisms. Two displacement sensors were also affixed to the prisms, 150 mm apart about mid-height, as shown in Figure 1. The load and displacement sensor outputs were recorded.

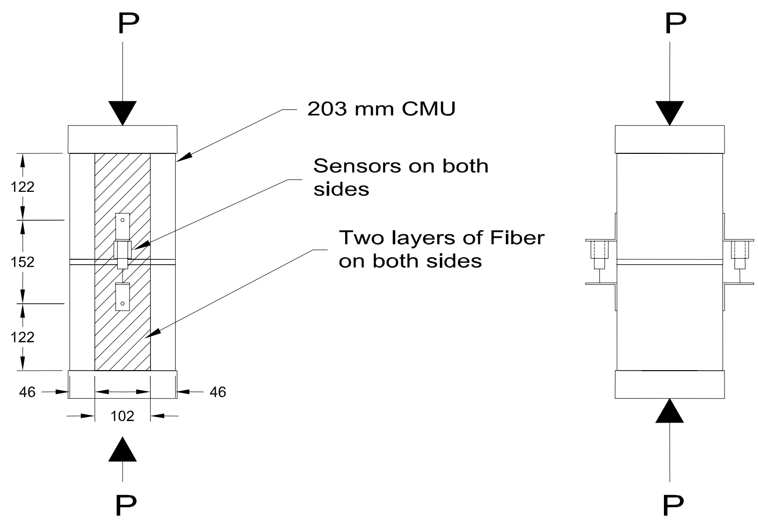

Figure 1 - Small Compression Test Specimens and Test Configuration

Three, two high-tension prism specimens were constructed using standard 400 mm x 200 mm concrete masonry units. Three double wythe brick tension specimens were also constructed using standard brick, eight units high. The brick and block prisms were reinforced as described for the compression specimens and allowed to cure at least 28 days in a lab environment. For the block specimens, two bolts were grouted into the top and bottom of the prisms. For the brick specimens, four bolts were grouted into the cavity between the brick wythes, on each end of the prism.

Each tension specimen was attached to the load actuator using the embedded bolts as shown in Figure 2. Sensors were fixed 150 mm apart on two opposing face of all specimens. Tension load was then applied at even rate until failure.

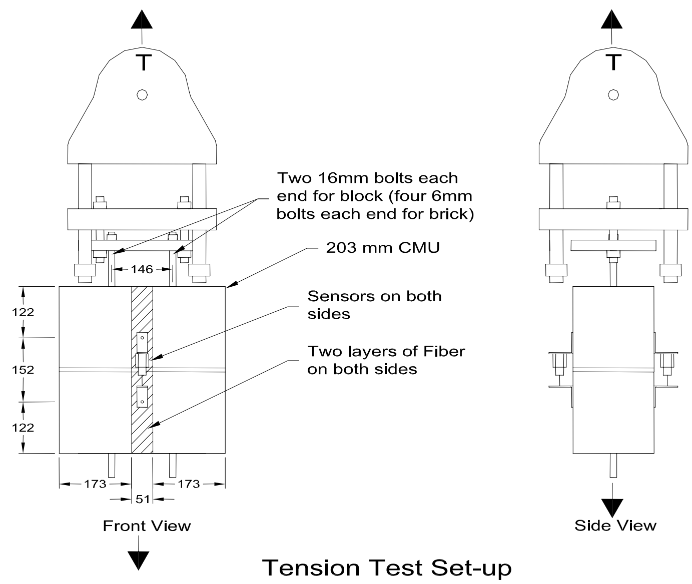

Figure 2 - Tension Test Specimens and Test Configuration

Three, three high shear prism specimens were constructed using standard 200 mm x 200 mm concrete masonry units. Three double wythe brick shear specimens were also constructed using standard brick, eight units high. The brick and block prisms were reinforced as described for the compression specimens and allowed to cure at least 28 days in a lab environment.

Each of the specimens was placed horizontally in the testing apparatus shown in Figure 3. A gypsum capping mixture was used to seat the specimens on the bearing plates. Displacement sensors were fixed onto each face of each specimen in the configuration shown in Figure 3. A shear load was applied to each specimen using an aluminium block 200 mm x 50 mm x 25 mm. The shear load was applied at an even rate until failure was observed. The load and displacements were measured throughout the test.

SMALL SCALE TEST RESULTS

The stress strain behaviour of the CMU compression prisms are shown in Figure 4. The reinforced block specimens typically failed by a bowing out of the fibre system followed by splitting of the units. The unreinforced specimens failed by vertical cracking and in some cases a spalling of the unit face.

The brick compression specimens behaved in a similar manner to the CMU prisms but failure typically occurred by the formation of vertical splitting cracks. Table 1 summarizes the maximum loads measured for all of the compression specimens.

The typical tension load-deflection behaviour of the reinforced block specimens is shown in Figure 5. For each specimen, there was an initial load region where the prism remained solid, followed by a softening of the prism after a crack formed in the mortar joint. Failure typically occurred by separation, either at top or bottom, of the mortar joint coupled with a peeling of the FRP and conical shaped section of the face of the masonry unit.

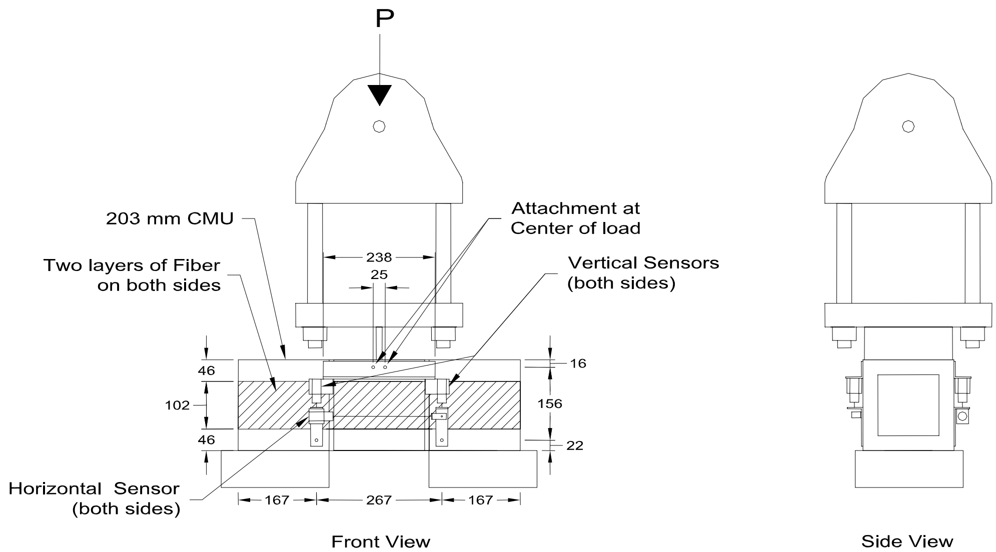

Figure 3 - Shear Test Specimens and Test Configuration

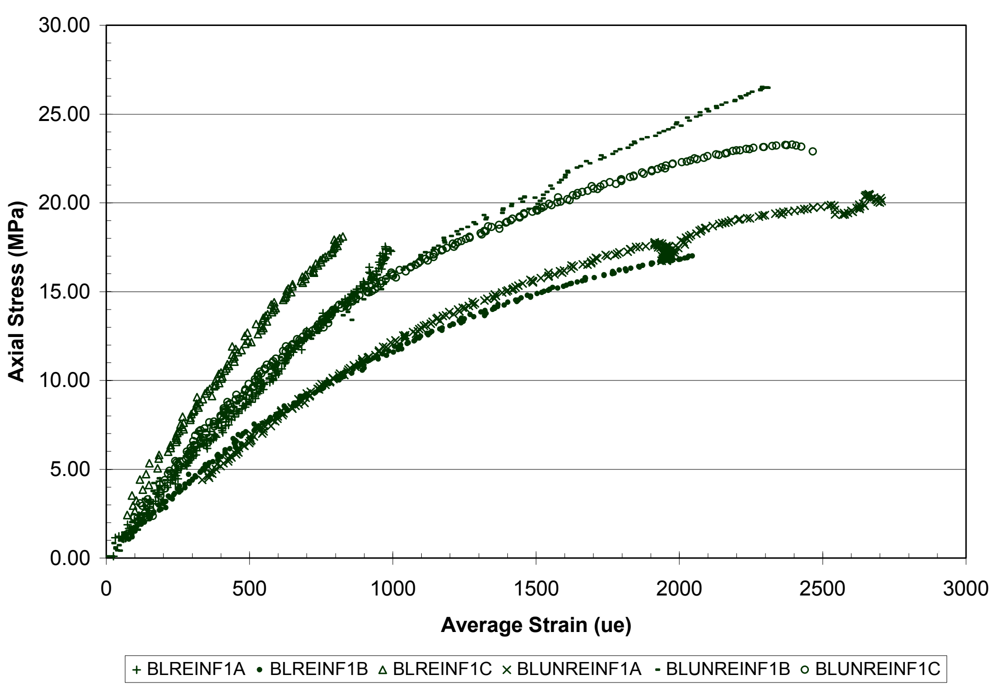

Figure 4 - Stress Strain Behaviour

Table 1 - Small Specimen Compression Test Results

Specimen	Max Stress (MPa)	Ave. Stress (MPa)	COV (%)
Block Unreinf 1A	20.53		
Block Unreinf 1 B	26.52		
Block Unreinf 1 C	23.29	23.45	12.8
Block Reinf 1 A	18.45		
Block Reinf 1 B	17.20		
Block Reinf 1 C	20.24	18.63	8.2
Brick Unreinf 1 A	19.06		
Brick Unreinf 1 B	19.35		
Brick Unreinf 1 C	18.70	19.03	1.73
Brick Reinf 1 A	15.57		
Brick Reinf 1 B	15.20		
Brick Reinf 1 C	18.48	16.42	11.0

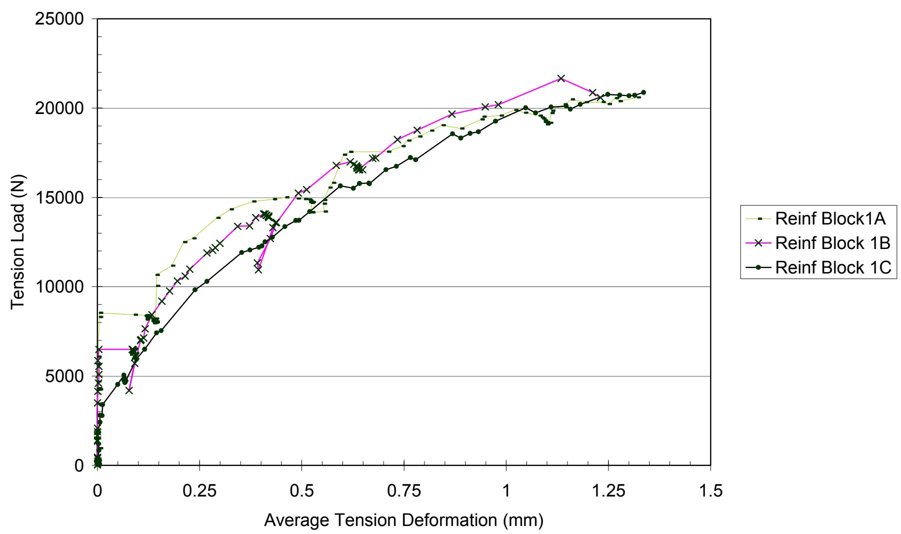

Figure 5 - Reinforced CMU Tension Load Deflection Behaviour

The reinforced brick tension specimens exhibited similar load-deflection behaviour with a softening after one, or more, mortar joints cracked. The failure of the brick prism was significantly different, however. At failure, brick units near the top or bottom of the prism rotated off the mortar joint due to the eccentric restraint provided by the carbon rods at the outside faces of the prism and ultimately failed by complete separation of the unit/rod from the prism. Table 2 summarizes the maximum tension forces measured for the block and brick specimens.

Figure 6 shows the typical shear load-deflection behaviour of the reinforced block specimens. These specimens typically failed by separation of mortar joints at the supports, a formation of a diagonal crack on the upper portion of the specimen that propagated to the central loading point. A conical section of the masonry was pushed up and out at failure.

Table 2 - Maximum Tension Load for Reinforced Brick and Block Specimens

Specimen	Max. Tension Load (kN)	Ave. Tension Load (kN)	COV (%)
Block Reinf 1 A	20.60		
Block Reinf 1 B	21.66		
Block Reinf 1 C	20.88	21.04	2.6
Brick Reinf 1 A	12.14		
Brick Reinf 1 B	24.19		
Brick Reinf 1 C	15.36	17.23	36.2

The reinforced brick specimens exhibited similar shear load-deflection behaviour and typically failed by conical cracking that extended through the reinforcement and centre of the specimen. Table 3 summarizes the maximum shear forces measured for the block and brick specimens.

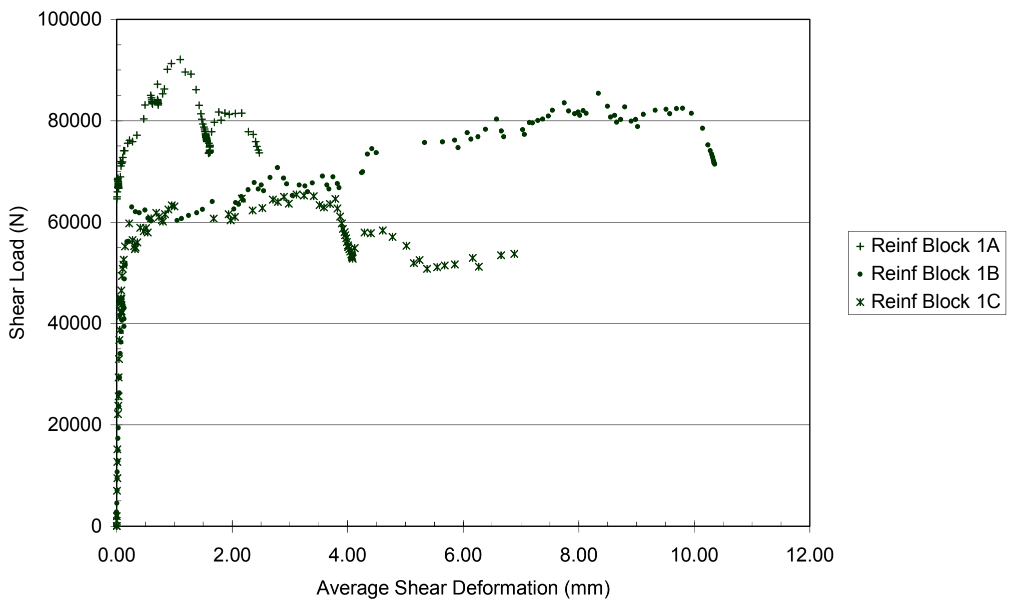

Figure 6 - Typical Shear Load Deflection Behaviour for Reinforced CMU Specimens

Table 3 - Maximum Shear Loads for Reinforced Brick and Block Specimens

Specimen	Maximum Shear Load (kN)	Ave. Load (kN)	COV (%)
Block Reinf 1 A	92.08		
Block Reinf 1 B	85.43		
Block Reinf 1 C	65.49	81.00	17.1
Brick Reinf 1 A	76.87		
Brick Reinf 1 B	66.96		
Brick Reinf 1 C	76.37	73.40	7.6

DISCUSSION

The results of the small scale tests were used to develop a model for prediction of the behaviour of masonry shear wall systems reinforced with fibre composites. This modelling took two forms, an approximate hand calculation method used to predict the ultimate strength of the composite masonry wall system and a conventional finite element analysis to attempt to predict the load deflection behaviour. This is being done to support development of a simple design methodology for use by professionals for fibre composite strengthening systems applied to unreinforced masonry walls. Due to paper length restrictions, the results of the finite element analysis will be reported at a later date.

Figure 7 shows the basic layout of the 4470 mm x 3250 mm x 2440 mm masonry building test specimen that was constructed in the Structural testing Lab at the University of North Carolina at Charlotte. This building was constructed using 200 mm concrete masonry units (CMU) meeting ASTM C 90 Standard Specification for Load Bearing Concrete Masonry Units and Type S masonry cement mortar.

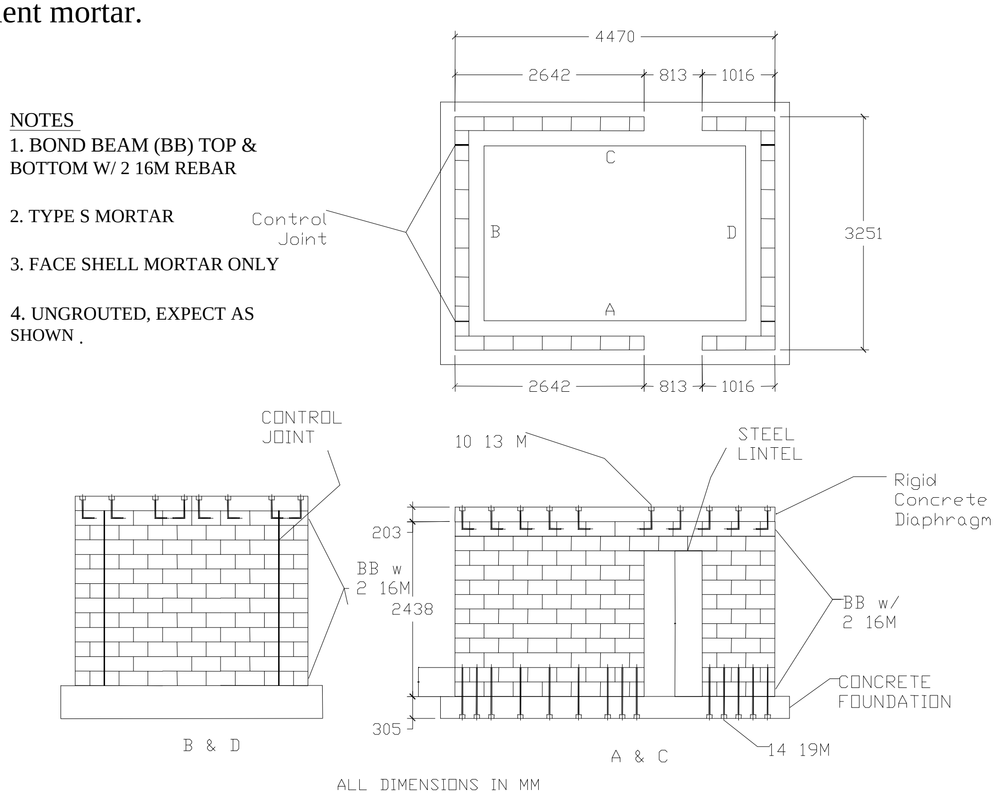

Figure 7 - Block Building Specimen

After allowing the specimen to cure in lab air for at least 28 days, a vertical load of 667 kN was applied to the top of the specimen to simulate the dead load of two floors above. A cyclic lateral load was then applied to the centre of the upper concrete diaphragm. This load was applied in push and pull cycles with three repetitions at each load level. The lateral load was increased in increments of 44.5 kN until there was a significant degradation in the specimen and failure was deemed imminent. Failure was deemed to be imminent at a lateral load of 400 kN where both the large and small piers exhibited significant diagonal tension cracks and there was a drop-off in the lateral load. A more complete description of these tests is contained in a paper by Foster et-al [1].

The block shear wall sections were then reinforced with the same glass fibre/epoxy system evaluated in the small tests, except that the width of the fibres was 300 mm as shown in Figure 8 and the mats were applied only to the exterior face of the masonry shear wall elements.

After the epoxy matrix had cured the vertical load of 667 kN was reapplied to the top of the specimen. A cyclic lateral load was again applied as described before. Failure occurred at a lateral load of 667 kN and was a result of face of the masonry units at the base of the diagonal fibre tension tie breaking away under the fibre (see Figure 9).

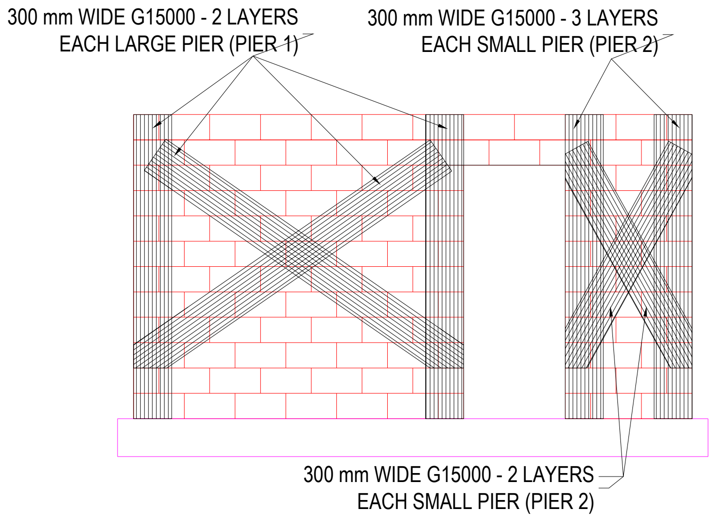

Figure 8 - Reinforcing Layout of Concrete Masonry Unit Specimens

Figure 9 - Lower Tension Tie Failure on Reinforced CMU Specimen

An almost identical building specimen was constructed using a double wythe clay brick system with a header at every third course (Type N Masonry Cement Mortar was used in this construction). The unreinforced building specimen was loaded with a 667 kN axial load and then with a cyclic lateral load. The brick specimen gave no warning of imminent failure and abruptly failed by diagonal shearing cracking at a lateral load of 489 kN.

The clay unit building specimen was then repaired with five vertical near surface mounted carbon FRP strips on the large wall section and three vertical carbon FRP strips on the smaller shear wall section. The carbon strips were placed by cutting a 6 mm x 19 mm vertical groove into the face of the wall and setting a carbon fibre reinforced polymer (CFRP) bars, (Aslan 500 #2), into the groove with epoxy. (See Figure 10).

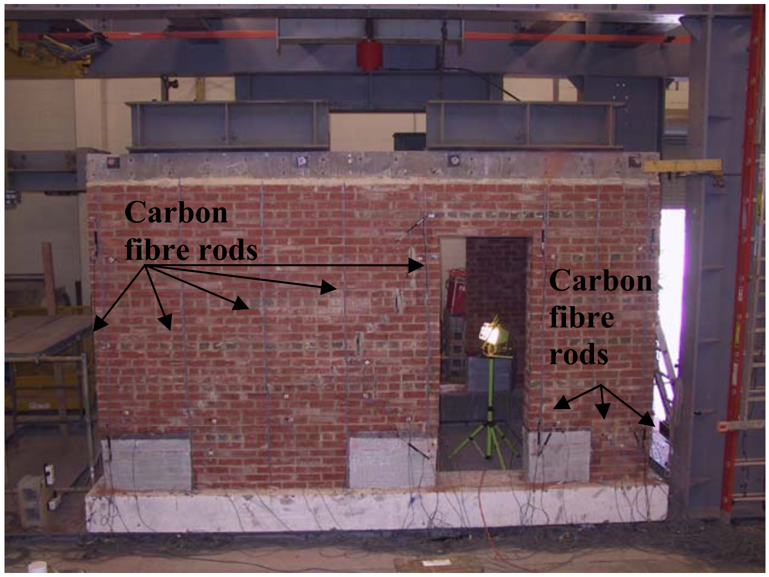

Figure - 10 Reinforced Brick Building Specimen Failure

After the epoxy matrix had cured, the building specimen was retested with a vertical load of 667 kN and a cyclic lateral load. Failure occurred at a lateral load of 489 kN and was a result of the existing diagonal tension crack opening up and causing failure of the carbon strips (See Figure 10).

Analyses of the building specimens show that they have a large shear wall element with an aspect ratio near 1.0, and are therefore shear dominated. In addition, these large shear wall elements govern the behaviour of the specimens due to their relatively large stiffness. If it is assumed that the lateral load is distributed with respect relative stiffness [2] and elastic solid sections are effective, then 83% to 87% of the lateral force is carried by the larger elements, depending on whether the sections are behaving as if there are fixed supports on the top and bottom, or are cantilevered off the base. This suggests that the maximum lateral load that can be applied to the building specimen would be 2/(0.83 to 0.87,- say 0.85) the capacity of the large shear wall elements, since there are two large shear wall elements, one on each side.

If it is assumed that the shear wall elements will fail in shear by diagonal tension cracking under any significant axial stress [3][4][5], then it is likely that a diagonal crack will form in the large pier of the block building specimen. The resultant free body diagram (FBD) of the cracked wall section at the ultimate lateral load can be developed and is shown in Figure 11. Note that this FBD looks at just the large shear section since it is assumed that the distribution factor will account for the effects of the small pier. The applied axial force, P, is assumed to be (667 kN/(2 x 4.47 m)) x 2.64 m + 2.4 kN (dead weight of slab and steel over just the shear wall element) = 199.4 kN. The tension force, T can be obtained from the average of the small tests and is 21.04 kN for two double fibre layers, 51 mm wide. This suggest that the 300 mm wide double layered strips on the larger specimen would have a capacity of T = 300 /100 x 21.04 = 62.9 kN. T2 is the tension force at the right side (T2 includes the fibre tension force and the restraint provided by the bond beam) required to maintain rotational equilibrium and is determined by summing moment about the lower crack location. The shear resistance (ignoring any masonry strength along the crack or weight) can be modelled as a combination of a fibre/masonry shear strength, V1, and a friction component, V2. If V1 was assumed to be the average value obtained from the small tests, V1 = 81.0 x 300/(100 x 4) = 60.5 kN for each of the two vertical two layer strips. The small scale tests had a double shear configuration with fibres on two faces so these test results were divided by four to represent the single layer and single shear configuration of the large specimens.

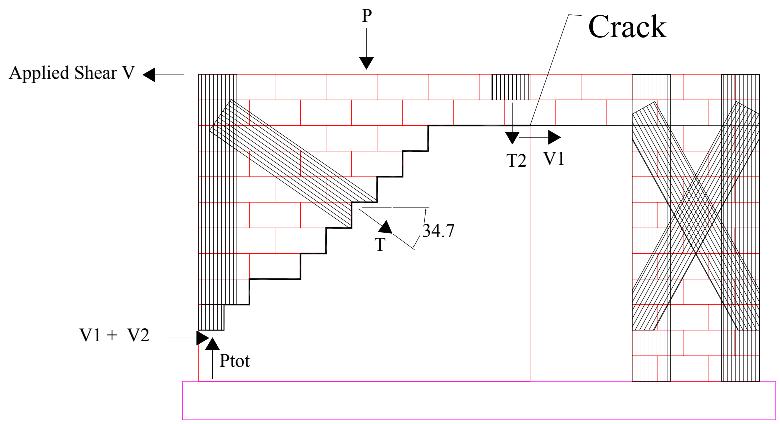

Figure 11 - Free Body Diagram of Cracked Pier

V2 is a shear friction component and varies depending on the roughness of the surfaces in contact and the axial load. Work by others [3,4] suggests that the friction coefficient, μ, varies from about 0.7 to above 1.0 and the MSJC Code [2] suggest that this value should be 0.45. If it is assumed that the friction is concentrated at the lower section of the cracked model then V2 can be calculated as = P_{total} x μ = (199.4 + 62.9 x sin34.7 + T2) x μ = 294 x μ. If the fiction coefficient is 0.45, then V2 = 137.5 kN. If μ is assumed to be 0.9 then V2 = 238.2 kN. Using these values, a maximum applied shear load for the large shear wall element can be determined through a lateral force summation as = 121+ 62.9 cos 34.7 + (132.3 or 238 kN) = 305 to 410 kN. This analysis would suggest that the maximum applied value for the building specimen of approximately 2/0.85 x 307 = 717 to 964 kN. The measured shear strength was 667 kN. The range of measured to predicted ratios is 0.93 to 0.69. This suggest that the friction in the pre-tested specimens may be lower than undisturbed walls and the combined tension and shear loads on the fibre systems are lowering the shear strength of the system. Also the building specimen tension tie failure appears to have been premature since it forced a splitting failure of the block that did not occur in the small scale tests. It should be noted that the T2 force at a μ = 0.9 (489 kN), is likely to exceed the tension fibre and bond beam shear capacity, and therefore limit the shear force developed. Finally, it is interesting to note that if the 300 mm width of vertical fibres at the top of the cracked section, which are under significant tension stress, are removed from the shear resistance calculation, then 575 to 824 kN capacities result (1.16 to 0.81 predicted to measured failure load ratios). Further investigation into these effects is needed, although a reasonable agreement was achieved with the lower code mandated friction coefficient.

A similar analysis was conducted on the brick specimens and a 402 kN to 631 kN building capacity were calculated assuming only four of the five carbon strips are effective in shear (the end carbon strips were near maximum tension loading). The measured maximum lateral load value of 489 kN results in a measured to predicted ratios of 1.21 to 0.78.

The investigation is continuing and will attempt to apply this methodology to a variety of reinforcing and specimen configurations. In addition, the deformation characteristics are being incorporated a simple finite element model and hand calculation methods to evaluate whether a simple method can be developed to predict the load deflection-behaviour of masonry wall systems reinforced with FRP systems.

CONCLUSIONS

The objective of this research was to develop a methodology for the design of FRP repair/strengthening systems for masonry shear walls using small assembly tests to predict the behaviour of large scale structures.

Based on the results of this investigation it can be concluded that reasonable agreement can be obtained between predicted failure loads and measured values using the capacities determined by small scale tests and simple static models. However, further evaluation of the effects of combined loadings on the reinforcing systems and extension into a greater variety of building and reinforcing configurations must be completed before this method can be generally used.

ACKNOWLEDGEMENTS

This research was conducted through funding by the National Science Foundation Award ID: 0324548. Their support is gratefully acknowledged.

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