TONKIN CANE BAMBOO AS REINFORCEMENT IN MASONRY SHEAR WALLS

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ABSTRACT

In order to produce structurally sound housing for developing countries, different approaches need to be taken to produce economically viable buildings. One method to produce low cost housing is the use of indigenous materials in place of conventional materials such as steel. Bamboo is a rapid-renewable plant native in many places of the world, and has many beneficial properties that make it a promising, inexpensive alternative to steel reinforcement.

This paper presents an overview of experimental testing carried out using the bamboo species *arundinaria amabilis,* more commonly known as tonkin cane for use as internal reinforcement in masonry shear walls. The results of tensile strength tests on the bamboo, bond strength tests between a low-cement ratio grout and the bamboo, and experimental quasi-static cyclic testing on two shear walls are presented. One wall contained the required seismic reinforcement described by the Canadian Masonry Design Code S304.01-04, while the other contained tonkin cane bamboo as internal reinforcement. These results are analyzed and discussed.

KEYWORDS: Bamboo; Cyclic; Indigenous; Low Cost; Masonry; Shear Walls.

INTRODUCTION

In developing regions, affordable, safe housing is not readily available and approximately 50% of the population live in earthen buildings, such as common adobe masonry, which as a stand-alone building material is very vulnerable to seismic events [1]. As many developing regions are also in seismically active areas, affordable, safe housing is a necessity to prevent catastrophe. With masonry being one of the most common building materials used in developing countries, masonry structures reinforced with alternative, inexpensive reinforcement materials, such as bamboo present one possible solution for low cost housing. Bamboo is a widely available, rapid renewable resource that can obtain tensile strengths as high as steel [2].

With the exception of a few test programs where bamboo was tested mainly as an externally mounted reinforcement [3], there has been minimal research on bamboo reinforced masonry whereas there have been several studies on the use of bamboo in concrete structures. Studies of bamboo in cement matrices were completed as early as 1914 [4], however, the first major comprehensive study was carried out by Glenn at the Clemson Agricultural College [5]. This later study was commissioned by the United States War Production board due to the short supply of steel during WWII, in which *"...every available material was being investigated for its possible uses as a substitute for steel."* [5] This study, along with several future studies [6-17] investigated the tensile, shear and bending strength of bamboo, the bamboo-to-concrete bond strength and the performance of bamboo reinforced concrete members such as beams, slabs, and columns. While bamboo has its disadvantages, such as the need for waterproofing to prevent swelling and the limitation of crack control, it was generally found that the addition of bamboo reinforcement enhanced the performance of the structural element in question. Aziz & Ramaswamy [18] summarized their research stating that, *"bamboo technology for appropriate applications in various low cost construction works can definitely lead to the full utilization of bamboo resources of many developing countries."*

BAMBOO MATERIAL PROPERTIES

The bamboo species *arundinaria amabilis,* more commonly known as tonkin cane was investigated at the University of Calgary for use as internal reinforcement in masonry shear walls. The material properties of the tonkin cane were determined in experimental tests. Due to the hollow nature of the bamboo, it cannot not be tested for tension in the same manner as regular steel reinforcing, as grips of a standard testing machine would crush the ends of the bamboo samples. Therefore, the bamboo was split into sections, and epoxy was applied to the ends of the specimens to give a flat surface for steel grips to take hold of. These specimens were tested in a Tinius Olsen tensile testing apparatus, as shown in Figure 1.

Figure 1: Bamboo Tensile Testing Apparatus

A total of 10 specimens were tested: three had notches cut into the culm (T.C.1-3), three had notches cut in the nodal area (T.N.1-3), and one had no notches (T.N.A). Three notchless specimens (T.ME.1-3) had strain gauges attached to them, such that stress-strain diagrams and the modulus of elasticity could be determined. Table 1 shows the results from these tests.

Due to the brittle nature of the splitting failure, these results may actually represent a lower bound for the tensile strength of this type of bamboo. The values obtained appear to be in the lower range compared to results reported in literature, which are normally in the range of 48.0 – 170.6 MPa [11]. Conversely, strengths as high as 440 MPa have also been reported. [2] Since the notching of the specimens causes micro-fractures, it can be reasonably assumed that the values reported are lower than the actual failure stress of tonkin cane. However, the test results presented here are consistent with the literature in that the node tensile strength (84.7 MPa) is less than the culm tensile strength (117.6 MPa). Unfortunately, no other results for tonkin cane are reported in literature and therefore a direct comparison cannot be made.

Table 1: Bamboo Tension Test Results

Bamboo Specimen	Cross Sectional Area (mm^2)	Failure Force (kN)	Tensile Stress, f_b (MPa)	Mean Tensile Stress, f_b (MPa)	Standard Deviation (MPa)	Coefficient of Variation (%)	Comments
T.N.A	92.8	6.80	73.3	-	-	-	No notches
T.ME.1	51.2	7.10	138.7				Strain Gauged
T.ME.2	52.4	4.71	89.9	123.7	29.3	23.7	Strain Gauged
T.ME.3	44.0	6.27	142.5				Strain Gauged
T.N.1	75.7	5.88	77.7				Split failure
T.N.2	67.6	6.26	92.6	84.7	7.5	8.9	Split failure
T.N.3	37.7	3.16	83.8				Split failure
T.C.1	40.2	4.81	119.7				Failed at node
T.C.2	48.6	6.68	137.4	117.6	20.9	17.8	Failed at node
T.C.3	22.2	2.13	95.8				Failed at node

The stress-strain relationships for the specimens T.ME.1-3 are plotted in Figure 2. An almost perfect elastic linear behaviour is observed for all three specimens. Each specimen failed by splitting near the end of the steel grips, and then pulling apart at that location in a brittle manner.
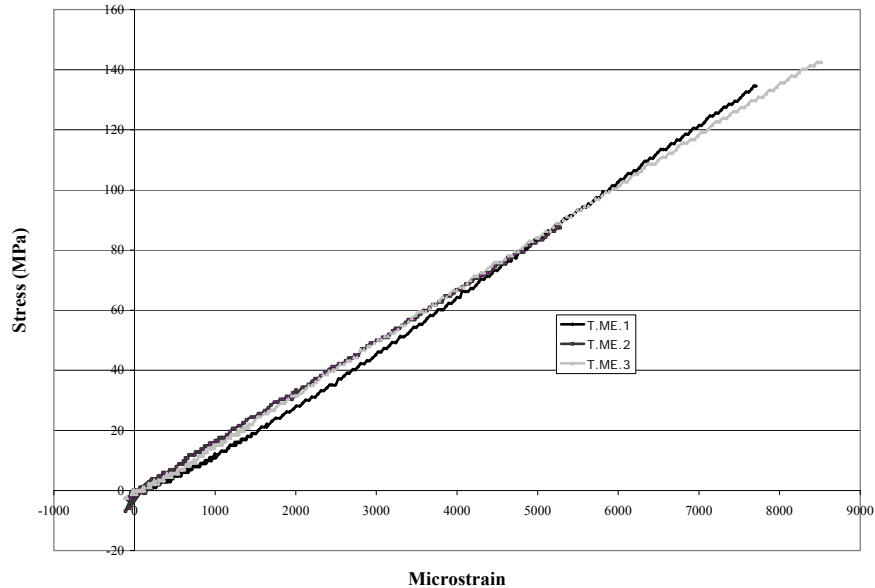

Figure 2: Stress Strain Diagram for Bamboo in Tension

Table 2 lists the modulus of elasticity for each of these specimens as calculated from the stress-strain diagrams. The stress-strain diagrams highlight the sudden failure that the bamboo experienced during these tests, since the behaviour is nearly perfectly linear, with no significant curvature or plastic behaviour at the end of the test. The results obtained here correspond well with values reported in literature, which range from 7-20 MPa [2,10], however, none of the species of bamboo tested in literature were tonkin cane bamboo.

Table 2: Bamboo Modulus of Elasticity

Bamboo Specimen	Modulus of Elasticity, E (GPa)	Mean Modulus of Elasticity, E (GPa)	Standard Deviation (GPa)	Coefficient of Variation (%)
T.ME.1	19.2			
T.ME.3	16.5	17.6	1.4	7.9
T.ME.3	17.2			

BOND STRENGTH
When bamboo is placed in fresh concrete, the bamboo in its wet environment will absorb water, and start to swell. The strength of the concrete over the first few days of curing is not strong enough to prevent the bamboo from swelling, hence, the expanding material will crack the concrete. Once the available water in the cement matrix is used up, the bamboo adjusts its moisture content to the drier environment, shrinking in volume during the process. This leaves a void in the cement matrix larger than the volume of bamboo that is present, leaving very little physical bonding of the bamboo to the concrete.

The use of seasoned culms as opposed to green culms, reduces the severity of cracking and shrinkage, while the use of pre-soaked culms negates any cracking, but there will be a loss of bond when the bamboo dries in the cement matrix [9]. In order to completely prevent cracking and shrinking, bamboo needs to be coated or saturated with a waterproof material. Materials that have been used in the past include asphalt emulsion [5], bitumen coatings [6], Negrolin and metallic wire [11,12], sulphur [16], anti-termite protective coating [15] and varnish [8] among others. Each material has produced varying results, with treatments such as Negrolin only allowing 4% absorption after 96 hours [11]. Cost of waterproofing materials can be a concern, with Ghavami [12] stating that while the treatment of bamboo with epoxy and fine sand was effective, it was an expensive treatment. He suggested that cheaper treatments such as asphalt paints, tar based paints, and bituminous materials would meet the requirements for making bamboo impermeable.

With the variation in different waterproofing agents as well as different node placements in the cement matrix, bond test results from previous research have had a high degree of variance. Kankam & Perry [6] tested 42 samples ranging from samples with only a roughened untreated surface, to specimens coated with bitumen and dusted with sand. Bond strength results ranged from 0.33 N/mm^2 for untreated samples to 2.6 N/mm^2 for samples treated with bitumen and dusted with sand. The low bond strength for untreated bamboo has been confirmed by others [9] with stresses ranging from 0.29 N/mm^2 to 1.18 N/mm^2. The pull-out stress from most types of

these tests give a non-uniform shear stress according to Ghavami [12], who proposed an improvement to the pull out test to give an evenly distributed shear stress as shown in Figure 3. With this method, bond strengths were found to range from 0.52 to 0.97 N/mm^2 [11].
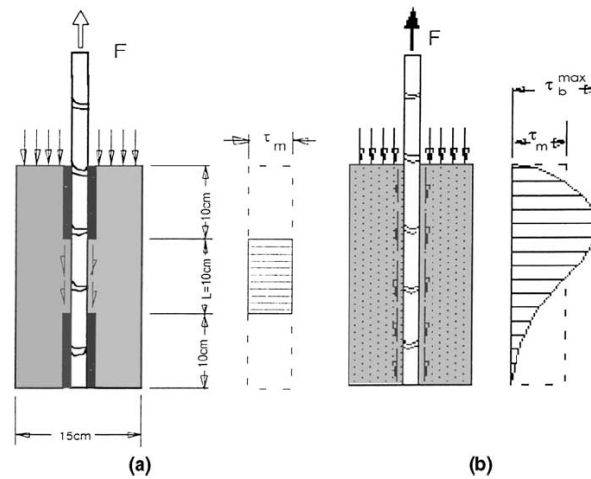

Figure 3: Bamboo Pull Out Tests: a) Improved; b) Conventional [12]

Bond strength tests were carried out on the tonkin cane based on the Ghavami [12] testing procedure. Bamboo splints were embedded a total of 165 mm in grouted cylinders, with 50 mm on each end covered with wax paper to ensure that no bond would occur between this surface and the grout. The 65 mm center portion of the embedded specimen was treated with spar varnish, and dusted with sand. Four specimens contained a node in the 65 mm treated center, while the other four were absent of a node.

These eight specimens were all tested in the same Tinius Olsen tension testing apparatus as the bamboo tensile tests. All results from this series of bond testing are given in Table 3. The bonding shear stress, τ, is calculated from the tensile force applied divided by the surface area of the 65 mm centre portion. Specimens with names starting with "N" and "C" denote specimens that were embedded with a node or culm respectively.

Table 3: Bamboo Bond Strength Properties

Specimen	Surface Area (mm^2)	Failure Force (kN)	Bonding Shear Stress, τ (MPa)	Mean Bonding Shear Stress, τ (MPa)	Standard Deviation (MPa)	Coefficient of Variation (%)	Comments
C.7.1	2597.1	2.55	0.98	-	-	-	7 Day
C.28.1	2680.9	7.83	2.92				
C.28.2	2571.9	1.41	0.55	1.33	1.46	109.8	
C.28.3	2695.6	2.72	1.01				
N.7.1	2916.4	4.16	1.43	-	-	-	7 Day
N.28.1	2557.9	5.72	2.24				Bamboo Split
N.28.2	2745.1	5.23	1.91	2.16	0.23	10.6	Bamboo Split
N.28.3	2651.0	6.20	2.34				Bamboo Split

An extremely large coefficient of variation was obtained for the splints with no node embedded whereas when a node was embedded, the coefficient of variation was only 10.6%. The mechanical anchorage of the node appears to not only significantly increase the bond strength, but also reduces much of the variance of the bond strength of bamboo in grout. Given the high variability, a greater number of specimens should be tested in future research.

The results reported here are on the high end of the values reported in the literature for bamboo specimens embedded in concrete. The average tested value of 2.16 MPa for Tonkin cane with an embedded node is nearly on par with specimens tested with a bitumen coating and dusted with sand [6] which achieved bond strengths of up to 2.6 MPa. While these specimens are similar to the tonkin cane bamboo, there are a few major differences that make a direct comparison difficult. First, the effect of the reduced aggregate size in grout compared to concrete is unknown. The second difference is that the tests reported in literature [6] most likely produced non-uniform stress, as described in Figure 3. While the values are a good start for comparison, more tests would need to be completed with the same conditions to make a direct comparison.

QUASI-STATIC TEST PROGRAMME
As part of a larger test programme [19] two walls were constructed using standard 200 mm (190 x 390 x 190 mm nominal dimensions) block seven courses high, and four blocks in length and Type N mortar (1:1:6 cement:lime:sand by volume). Each wall was constructed by the same master mason, and knock-out blocks were used for all of the bond beams. One wall (QS.S.1), has the minimum seismic reinforcement as per the CSA S304.1-04 Design of Masonry Standard [20], including a total of 3 bond beams (in the 1st, 3rd & 5th courses from the top), and 4 evenly spaced vertical cores each reinforced with a single 10 M rebar. This rebar was tested for tensile strength producing an average yield strength of 462.3 MPa. The second wall (QS.B.2) has 4 evenly spaced vertical cores containing whole culm tonkin cane reinforcement, which was coated with varnish, and dusted with sand.

For grouted masonry in Canada, the CSA A179 [21] has a required sand to cement ratio of 3:1 by volume. Since the goal in low cost housing applications is to reduce the amount of expensive materials such as cement, a ratio of 4:1 was used. A high water to cement ratio of 1:1.62 was also used. Grouted masonry prisms were tested for compressive strength, with an average strength of 11.5 MPa.

A testing frame for the quasi-static testing was constructed as shown in Figure 4. The masonry wall specimen was placed between two upright W-section columns supporting a 500 kN actuator, which was used to apply a vertical load to the top of the wall. The wall was held stationary laterally by two braces bolted to the concrete floor, and a hand-pumped hydraulic jack was used to ensure that the wall would not slip along the concrete floor. Spherical seats were placed at the base of each side of the wall to ensure that there were no eccentric forces being transferred into the specimen.
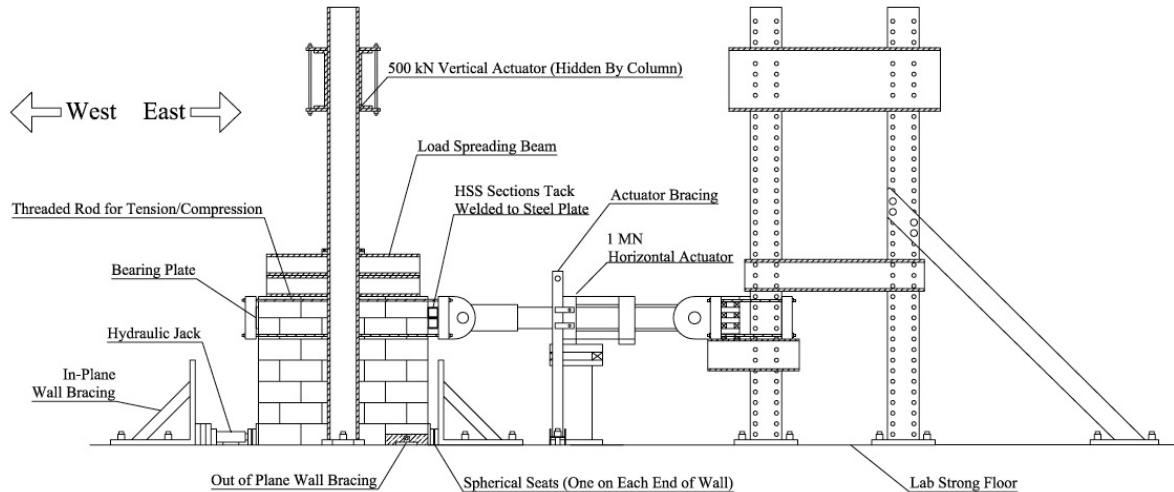
Figure 4: Quasi-Static Testing Frame Apparatus

The cyclic horizontal force was applied to the wall specimen by means of a 250 kN actuator (later replaced by a 1 MN actuator) mounted to a braced W-section column, East of the wall specimen. This actuator had a total stroke of 250 mm, allowing it to push or pull the wall to a maximum deflection of 125 mm each way. At the top of each side of the wall (East and West), steel plates were suspended, and tightened to each other through the use of four steel threaded rods, clamping the top of the wall. One of the steel plates was directly attached to the actuator.

Fibreboard (tentest board) was placed on top of the wall to accommodate any irregularities in the top of the wall surface. A composite steel W-section was placed on the fibreboard and rollers perpendicular and parallel to the plane of the wall were placed on top to ensure the vertical load applied to the wall would be evenly distributed. The two plates on the East and West side of the wall were then attached to each other via threaded rods and tightened.

Metal targets were glued to each side of the test specimen to provide a flat surface for the Linear Strain Converters (LSCs) used to record the displacements of the wall at 8 locations: four on the West end and four on the East end. During loading, these displacements, the stroke and force of the horizontal and vertical actuators, were recorded in the data acquisition system.

An initial vertical load of 100 kN was applied. This load was used to pre-compress the fibreboard at the top of the wall so that the load would not fluctuate because of crushing of the board. The vertical load from the actuator was then reduced to a constant 60 kN. When this load was reached, the actuator was put into stroke control, keeping the position of the actuator locked. The racking (horizontal) load was then applied to the specimen, also in stroke control. This load was applied at a constant rate of 1 mm/s, with two cycles per deflection step. The step-displacement history that was used for the cyclic testing is shown in Figure 5.
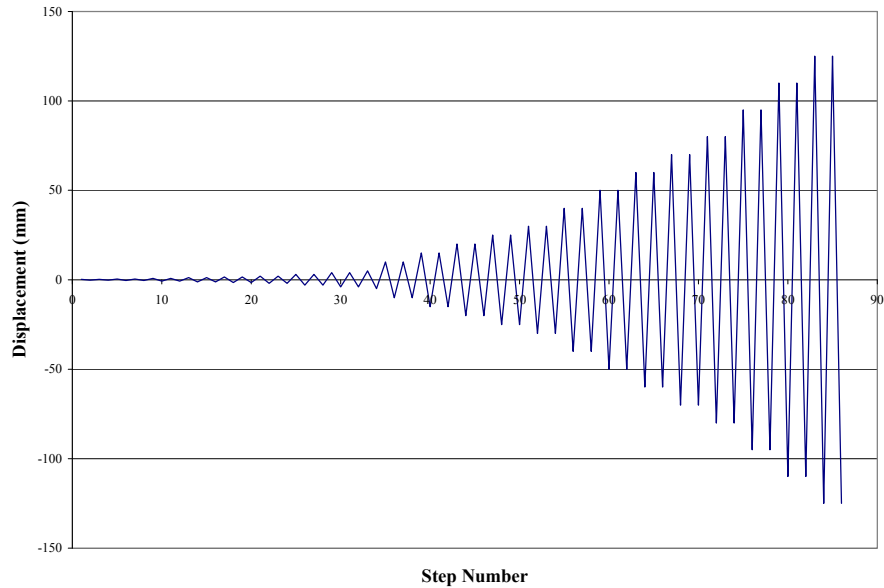
Figure 5: Step-Displacement History

WALL QS.B.2 TEST RESULTS

The crack pattern and hysteresis loops produced from the testing of wall QS.B.2 are shown below in Figure 6. A strange dip in the hysteresis loops can be seen, right after the wall achieves its ultimate load. For this test, only a 250 kN horizontal actuator was used. This actuator actually reached its maximum capacity, but this maximum load was continually applied until the target deflection was reached. The test program was paused at this point. This sustained load eventually caused the failure of the wall which caused this sudden decrease in load.
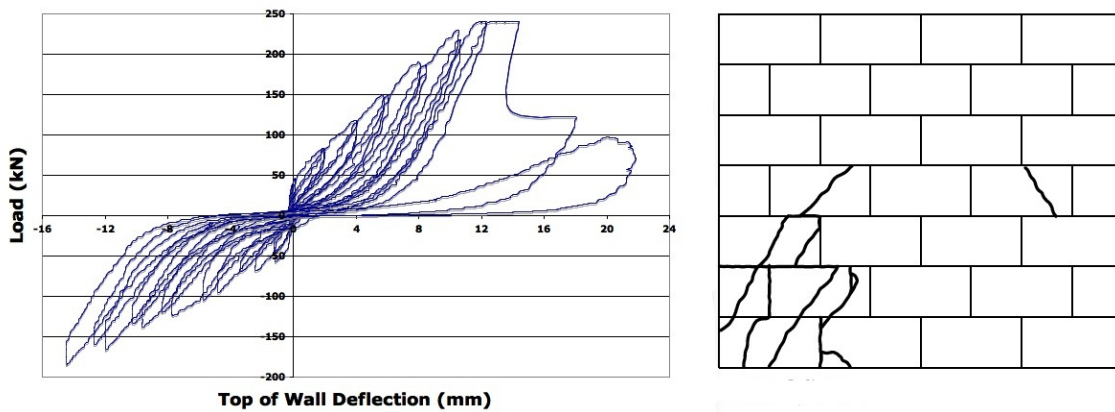

Figure 6: Hysteresis Loops and Crack Pattern for Wall QS.B.2

The crack pattern in this wall seems to indicate that the wall failed in diagonal tension, however, since the loading was not equal for the push and pull cycles, this is most evident on one side of the wall. This wall also exhibits what appears to be a compression strut in the crack pattern, which also indicates that this wall may have the influence of a partial flexural failure. Without a

base for the wall to be doweled into, flexural failure can only occur by crushing of the toe, with a diagonal compression strut, as opposed to yielding of dowels on the tension side.

It was noted that the loading and deflection curves for the push (positive deflection) and pull (negative deflection) cycles, were not equal each other. This disparity between the push and pull cycles comes from the fact that the stroke of the actuator was not necessarily the same as the top of wall deflections, with greater discrepancy on the pull cycle. This discrepancy comes from several possible sources, including movement of the frame itself, the steel rods holding the top of wall bearing plates being in tension and experiencing small elongation only on the pull cycle, and the difference between the steel bracing and the hydraulic jack used to provide in-plane support on the pull and push cycles respectively. When the test setup was changed to include the 1 MN actuator, the displacement measured on the frame and at the base of the wall on the west side were fed back to the computer control system to correct the stroke and more accurately relate the stroke to the top of wall deflection. As can be seen in the results from QS.S.1 (Figure 7) using the 1 MN actuator, the discrepancy between the push and pull cycles is significantly reduced.

It was also discovered that wall QS.B.2, due to mis-calibration of the control system, had an initial vertical load of only 20 kN, as opposed to the 60 kN that was placed on all other test walls. This was not noticed until the results from the data were analysed. With lower axial loads in-plane shear failure is known to tend more towards flexural failure than diagonal tension [22].

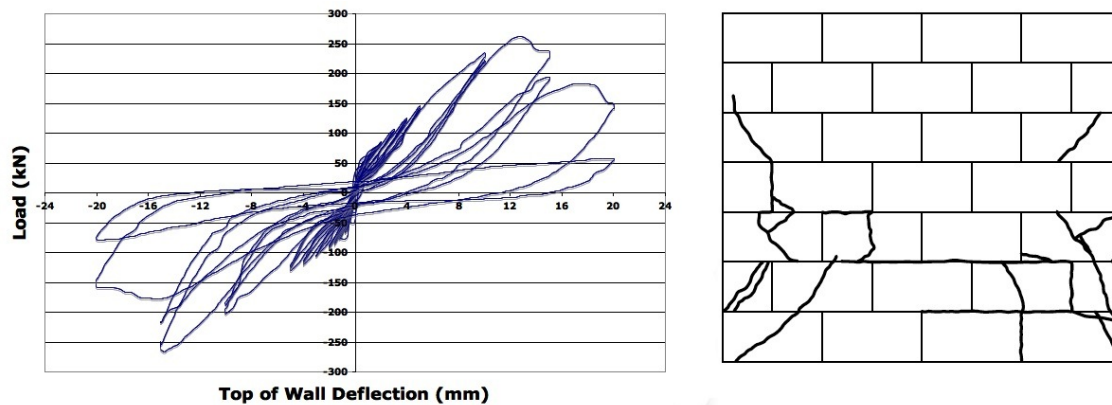**WALL QS.S.1 TEST RESULTS**

Figure 7: Hysteresis Loops and Crack Pattern for Wall QS.S.1

The hysteresis loops and crack patterns for the steel reinforced wall, QS.S.1 are shown in Figure 7. The wall exhibited a diagonal tension shear failure, with a slight rocking, and compression strut failure at each of the toes. This compression strut is likely due to an influence of a flexural failure, and despite not developing a full "X" pattern associated with diagonal shear failure, this wall does appear to exhibit a diagonal shear failure.
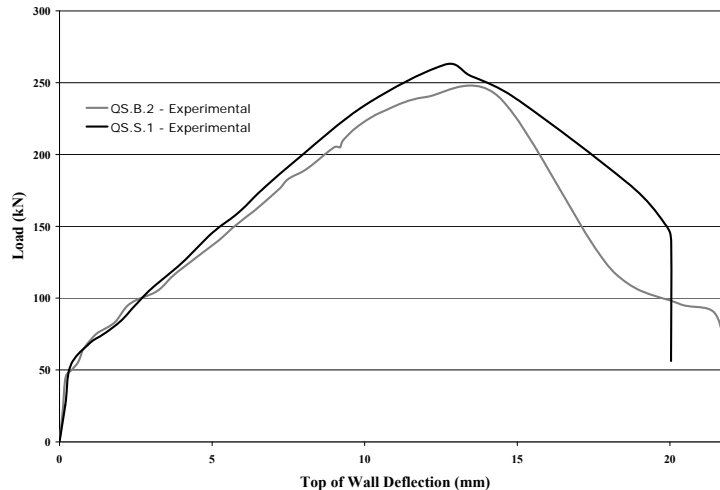
Figure 8: Experimental Hysteresis Envelopes

It can be observed that while the hysteresis loops on wall QS.S.1 encompass larger areas than those for wall QS.B.2, the envelopes of all the loops produced from both walls are somewhat similar. The comparison of the experimental hysteresis envelopes for both walls is shown in Figure 8. While the crack pattern on the bamboo reinforced wall is one sided due to the test setup, this crack pattern is again similar to the pattern produced on the steel reinforced control wall. The ultimate capacities of both of the walls were also similar, with the steel wall obtaining a maximum load of 262.6 kN, compared to a maximum of 240.6 kN in the bamboo reinforced wall. Finally, the maximum deflection of both the tested walls were nearly identical, with the steel reinforced wall reaching 20.0 mm and the bamboo reinforced wall reaching a maximum deflection of 21.8 mm.

CONCLUSIONS

The tensile strength of bamboo from species to species is variable. For use in any sort of construction, tensile testing of the specific bamboo species should be undertaken to obtain an accurate assessment of strength. Experimental testing showed the variability of bond strength of bamboo to grout using marine varnish as a waterproofing agent. Previous research has shown that there are many different waterproofing materials that have proven to meet the requirements to make bamboo reinforcement impermeable. For developing regions, only the most cost effective waterproofing treatment would be used, however, for any such treatment, proper testing needs to be performed to ensure no swelling occurs.

From the quasi-static testing, the bamboo reinforced shear wall showed remarkably similar behaviour to that reinforced with steel. The slightly lower ultimate resistance of the bamboo-reinforced wall could be attributed to a lower axial load applied to it, along with the lack of bond beams. This leads to the questions of whether bond beams are necessary in low cost housing applications. With the similarities in load and strength, the experimental testing shows that bamboo shows promise as reinforcing for low cost structures.

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