Development and testing of timber/concrete shear connectors

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SUMMARY

With the advent of improved technology regarding trucks carrying heavier, longer loads and increasing the number of trips under the same routes, old timber bridges need to be monitored in order to ensure users of its structural integrity. In most cases the existing substructure in bridges are in excellent conditions but have defective decks. Some of these bridges are in remote areas such as timber logging tracks, secondary highways, inland routes, etc… In order to have these bridges constantly open and in good service, maintenance and/or re-surface must be performed promptly to avoid interruption to traffic flow. This requires new, innovative techniques that can offer all the requirements anticipated.

This paper presents the results of tests and an analytical evaluation of three types of shear connectors for the design of new timber bridges and the rehabilitation of old bridges with timber girders and concrete decks connected by composite action. Furthermore, the development criteria were to provide a suitable and economic shear connector that did not required costly or specialised equipment.

The *static* and *dynamic* response of shear connectors between concrete/timber interfaces has been determined from laboratory testing. For these types of composite materials, timber provides the tensile strength whilst the concrete provides the compressive strength. The degree of composite action depends on the type of connector, timber condition, initial slip and the effective flexural stiffness of the connector and/or system. Full-scale specimens were fabricated to obtain accurate results comparable to realistic existing conditions. Horizontal and vertical displacements were measured between the two materials up to and beyond failure in all cases. The research at UTS-Faculty of Engineering is concentrated on achieving full composite action between timber and concrete with the minimum amount of effort and maximum performance.

TYPES OF CONNECTORS

Three different types of connectors were pre-selected from a range of possible nine (9) types. These were fabricated from steel grades varying from 250 up to 350 MPa [1]. In order to have a benchmark the 20mm plain mild steel *"dowel"* connector (the most typical and common shear connector) was chosen to allow comparison with the UC (universal column) section and CHS (circular hollow section)[1]. The reasons for using these connectors include the availability of materials, its minimal fabrication and easy assembly on site. The use of steel connectors between timber and concrete is not new. This composite system was developed in the USA around 1930 at the University of Oregon [1]. In Australia the first major composite system was built on a highway bridge around mid 1950's on the Pacific Hwy in NSW over the Maria River. Worldwide others have also conducted research on this composite system, but mainly for domestic or light industrial construction, [2], [3].

TIMBER

Old timber railway sleepers from the Sydney Harbour Bridge were utilized for the timber component. The conditions of these timbers were ideal since they had been exposed to the elements for an extensive period showing signs of wear, weather exposure and natural defects. These timbers were also chosen to simulate the real conditions of an old timber bridge girder as they had been

TYPICAL BRIDGE CROSS SECTION
-Round logs with 50mm top sawn-off
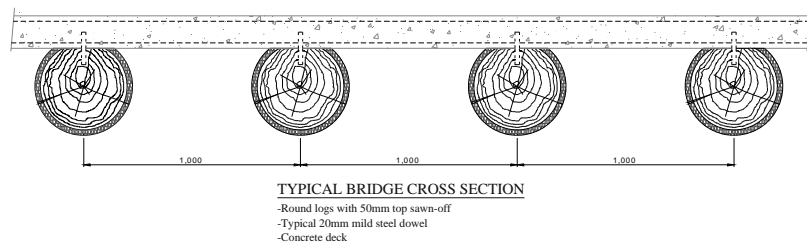-Typical 20mm mild steel dowel
-Concrete deck

Figure 1

exposed to heavy loading from freight and passenger train traffic. The most common type of girders in timber bridges is round logs (see Figure 1). These logs are machined on one side to provide a flat surface to support the deck. When the concrete or timber decks get damaged or are in need of repair the decks are either partially or fully removed. However, these girders are still fully functional, except where occasionally some sections that cover the first 25-50mm in the flat portion, (see Figure 1) become damaged due to entrapment of moisture. Fifty-four pieces were cut at approximately 500mm in length but only forty-six were suitable for the project. Selection of the timber was based on visual grading system. The species used for these tests were Red Iron Bark *(Eucalyptus sideroylon)* and Grey Box *(Eucalyptus microcarpa Maiden)*[2].

Before the timber was segmented some of the pieces showed very poor conditions on the outside. Closer inspection after cutting them showed that the majority of the natural defects were only on the first 20mm from the surface. The timber was conditioned for 3

weeks in the structures laboratory and one end machined to allow the specimen to sit perpendicular with respect to the applied load during testing. The surface moisture content was measured only on a few selected pieces having an average value of approximately 14%.

TEST SPECIMENS

"The effective flexural stiffness of composite members is a function of the stiffness of connecting medium, that transfer the shear and the elements that are being connected to form a composite member."[Walter Burdizk]

in its cross-section and 500mm in length with concrete positioned between them (see Figure 5). The connectors were secured to the timber parallel to the grain. These were positioned in the widest face of the timber. Before the connectors were fastened a heavy-duty plastic membrane was attached to the timber, this would allow free bonding between the two surfaces permitting the applied force to act directly onto the connectors.

Having so many surface imperfections (splits, checks, gum veins, holes from bolts, etc.) in the timber, it would not be a true comparison if the two surfaces were allowed to react against each other, as friction would be introduced. In real situations, this would be an advantage, since the concrete would fill the voids in the timber increasing the shear capacity. In order to obtain a pure shear response and proper comparison between connectors, flexural steel reinforcement was not used. The concrete strength was 25MPa at 28 days.

The connectors were placed in the timber in a way that would maximise the degree of composite action. The plain mild steel dowels (see Figure 2) were arranged at 60° to the horizontal in order to carry the shear forces as well as the tensile forces trying to separate the two materials. The CHS (see Figure 3) was fit in the timber by drilling a groove of the same size diameter. This was possible by manufacturing a hole-saw from the same material with tungsten tip cutters attached, hence, a perfect fitting. In addition a 150 x M16 coach screw was placed in the centre of the ring to prevent the separation of the two materials, this would allow for the tensile forces (see Figure 3).

Due to the UC's (see Figure 4) shape and geometry the shear and tensile actions combine by relying on the flanges to account for the pullout forces and the web to add some of the shear without adding any extra elements. This is the smallest UC section available in Australia. These elements were fastened using four 75 x M16 coach screws to the timber that in turn provide the shear strength (see Figure 4). All bolts were mild steel grade 4.6. The ends of the UC's were flame cut at 45°° to allow easy fastening to the timber. The author manufactured all specimens.
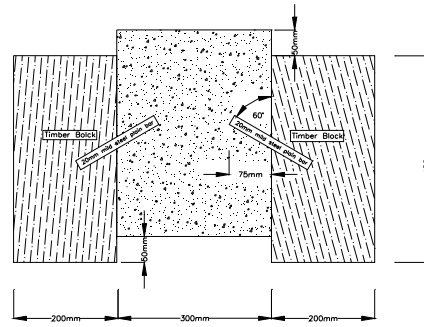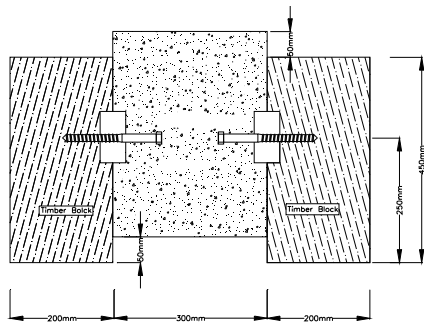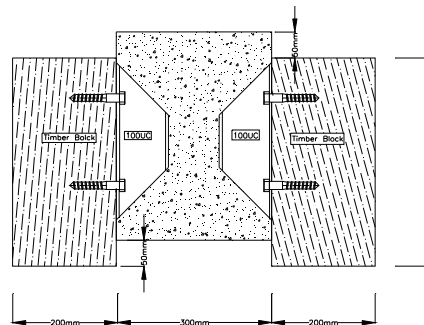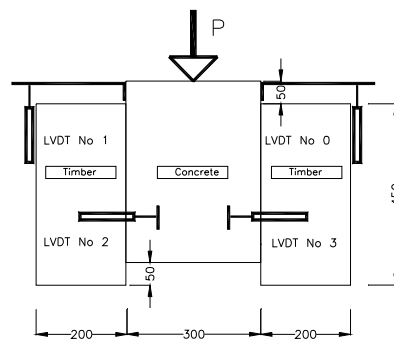
Figure 2

Figure 3

Figure 4

Figure 5

TESTING

Twenty-three specimens were tested under the same conditions. The first nine were used to adopt the best two out of three connectors. The specimens were instrumented with four LVDT's (Linear Voltage Differential Transducer) with an accuracy of 0.01mm and the load was read directly from the universal testing machine. Two LVDT's measured the relative vertical differential displacement between the timber and the concrete and the other two the separation or pullout (see Figure 5). The data was transferred electronically using *Spectra* data acquisition system.

The testing programme was divided in 6 stages:

1. Construction of 9 specimens, (3 for each type of connector)
2. Static testing of the 9 specimens
3. Construction of extra 14 specimens
4. Static and dynamic testing of the 14 specimens, 6 static and 8 dynamic
5. Analysis of results
6. Conclusions and recommendations

A pilot static test was set up and three specimens for each type of connectors were tested initially to obtain the following:
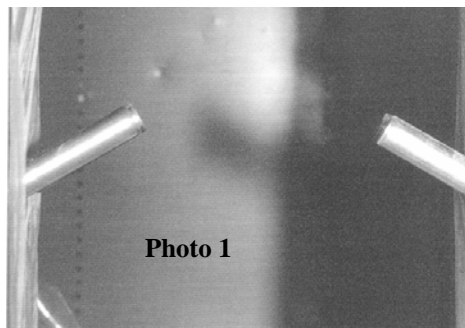
- Material behaviour
- Connector's behaviour
- Failure mode of connectors
- Slip between the two materials in both directions (vertical and horizontal)
- Ultimate strength of connector
- Failure mechanism of the system

In order to distribute the load uniformly over the concrete face a 4mm soft sheet of masonite was placed with a 30mm hardened steel plate on top it that in turn transferred the applied load from the universal testing machine.

RESULTS

1. 20mm Plain mild steel dowel

The failure of the steel dowel type connector (see Photo 1) was due to the propagation of concrete cracks at the timber/concrete interface. As the concrete was pushed downwards, due to their inclined angle, the bars created a point of stress concentration accelerating the failure mode. The cracking of the concrete started to develop at the connector's location in the interface between the concrete and the timber, 250mm from the bottom. After the tests were completed the concrete was removed and the connectors exposed for further analysis. The *"dowels"* showed no signs

of stress or bearing problems in the timber. It was quite noticeable that there was no bonding between the concrete and the *"dowels"*. This type of failure was identical for all three specimens.

2. Circular hollow section

These specimens (see Photo 2) failed at much higher loads with specimen CHS_3 being the strongest with a maximum load of 424.5kN (see Table1 & 2). These specimens showed a ductile balance failure (see Figure 10). Due to the internal forces the CHS pulled-out from the groove in a skewed angle but did not become disengaged from the timber (see Photo 4). In these specimens it was noted that after failure the roundness and the integrity of the pipe was kept, but the timber showed signs of crushing with the pipe being lifted at the upper end (see Photo 4).
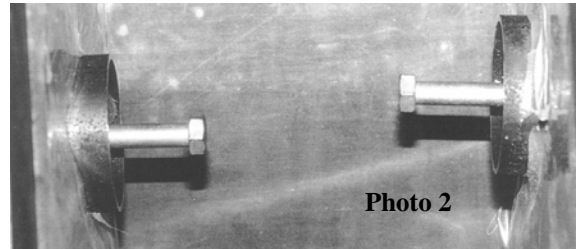
Photo 2

3. Universal Column Section

Deformations of the coach screws (see Photo 5) were the cause for the failure mode. Bearing failure in the timber together with tensile forces pulling the coach screws created enough stress to pull the connector out. In addition these forces developed internal concrete failure with a sudden collapse of the specimen. This substantiates the initial theory of horizontal force (tension) pushing the timber outwards.

Photo 3
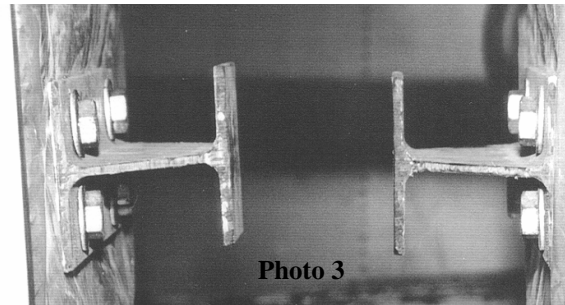
Static Tests

Only the results and analysis of two connectors are presented in this paper. The values given in Table 1 represent the base line for the selection of the best connectors. From these results the *dowels* were discarded due to the poor performance. Not only the load capacity was very low but also the ductile behaviour was very inferior to the other connectors.

The tested specimens were pre-loaded to 100kN in-order to allow for settlement of any imperfections either in the timber bases or in the concrete surface. Internally it allowed the different components to be re-arranged to a working condition. In addition from these tests it was found that this load was less than or equal to 0.5mm in the vertical slip (see Figure 6). This has a significant importance from the design point of view to comply with the Australian Bridge Code. The results given in Table 1 and 2 represent the total applied load from the pre-selected connectors. Hence, each connector would take half of these values.

Table 1   Static tests results

Spec. Nº	Force (kN)	Spec. Nº	Force (kN)	Spec. Nº	Force (kN)
CHS_1	320.5	UC_1	337.5	DWL_1	113.5
CHS_2	314.5	UC_2	370.0	DWL_2	119.0
CHS_3	424.5	UC_3	325.0	DWL_3	130.0
Average	**353.2**		**344.2**		**120.8**

Table 2 Average values for shear connectors tests

Spec. No	Force (kN)	Spec. No	Force (kN)	
100mm Pipe + Coach screw		**UC-Section**		
CHS_1	320.5	UC_1	337.5	Pre-Dynamic test
CHS_2	314.5	UC_2	370.0	
CHS_3	424.5	UC_3	325.0	
CHS_4	278.9	UC_4	385.5	
CHS_5	294.3	UC_5	285.20	
CHS_6	273.9	UC_6	386.3	
CHS_7	335.9	UC_7	371.1	
CHS_8	306.1	UC_8	343.8	Post-Dynamic test
CHS_9	N/A	UC_9	357.7	
CHS_10	322.0	UC_10	298.1	
Mean	**319.0**		**346.0**	
Std Deviation	**44.5**		**35.0**	

The values in Table 2 can be divided in two categories the first six (6) values represent static tests results only with the rest also the static results but after the end on the dynamic tests. The CHS's demonstrate a drop of strength after the dynamic test whilst the UC's no including UC_5 an increase in strength.
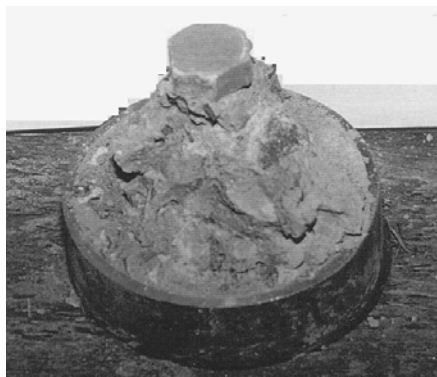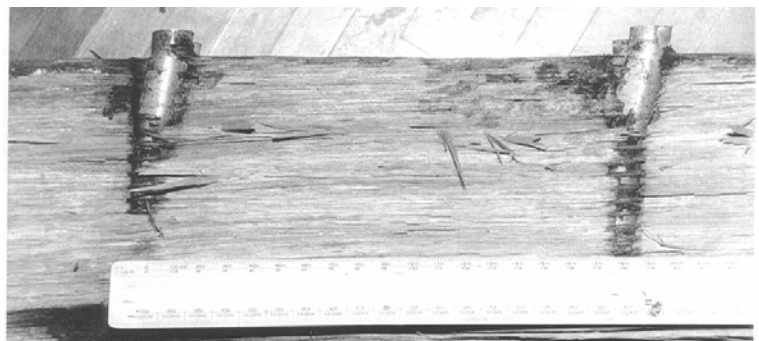

Photo 4

Photo 5

The stiffness of the connectors were calculated as follows:
1. UC sections $\varpi = 409$kN/mm $\sigma = 123$kN/mm
2. CHS sections $\varpi = 231$kN/mm $\sigma = 24$kN/mm

ϖ = mean
σ = standard deviation

The values presented above represent the linear behaviour of the initial vertical differential deflection between the timber/concrete interfaces. Due to the scatter in the data all the calculations values (i.e.: force) were subjected to be equal or less than 1mm displacement. No lateral restraints were applied to allow the development of internal tensile forces in the connectors and their effect in the materials.

The static test data is too large to be included in this presentation, however a representative sample of graphs is given in Figure 6.

Dynamic Tests
These tests were divided between the Materials Testing Laboratory and the Structures Laboratory due to time constraints and loading capacity of the testing machine at the Materials Testing Laboratory. This testing machine is capable of a maximum of 12 tonnes dynamic load and 16 tonnes static load. This presented a major problem since all of the specimens showed a linear behaviour in the vicinity of 250kN or 125kN per connector.

To observe the behaviour under a "working load" the specimens were subjected to 100kN for 100,000 cycles. As the testing machine in the Structures Laboratory could not be classified as a true dynamic machine, the numbers of cycles were reduced to 10,000, but the load was increased to 150kN. Close examination on static test graphs gave values that would not displace more than 0.5 to 1.0 mm (see Figure 9). The frequencies were set to 1.0Hz and 0.1Hz respectively.
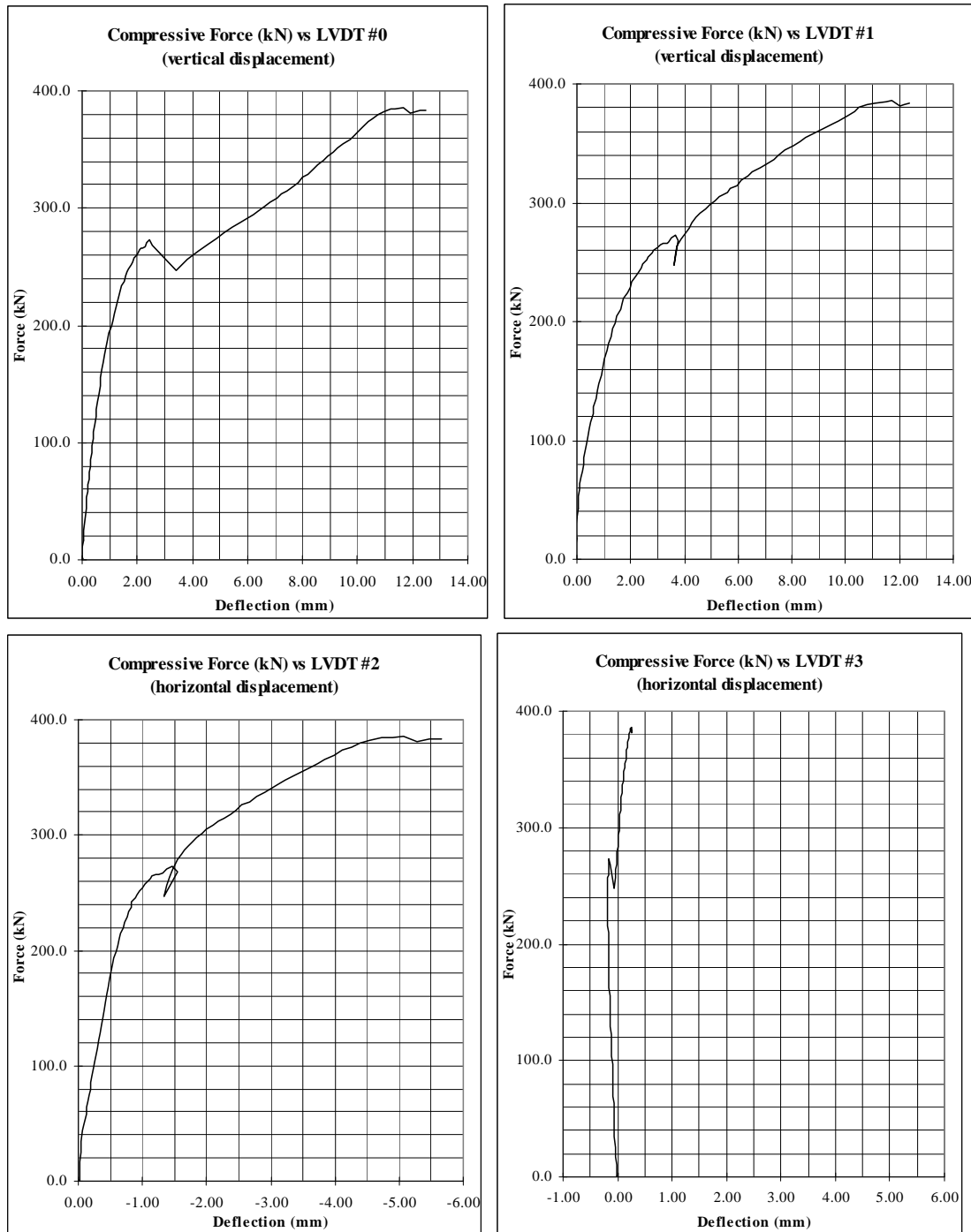
Figure 6 typical graphical responses for vertical and horizontal displacements.

The trend obtained (see Figure 8) shows that at a constant load the initial slip starts to be non-linear at around 50kN per connector. Even after 100,000 cycles the difference between the initial and the final slip does not alter greatly and maintains the full composite action.
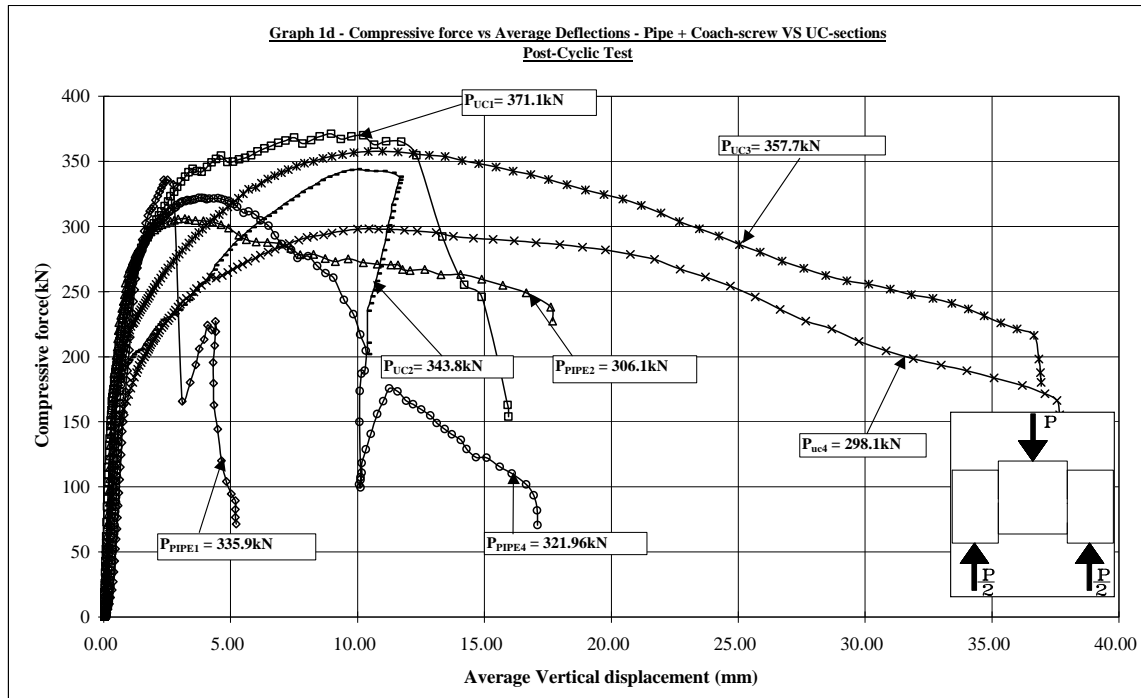
Graph 1d - Compressive force vs Average Deflections - Pipe + Coach-screw VS UC-sections
Post-Cyclic Test

Figure 7 typical comparative graphical responses

Average Displacements vs No of Cycles for Spec. PIPE 1

$y = 0.0299Ln(x) + 0.7702$
$R^2 = 0.8648$

$y = 0.0391Ln(x) + 0.51$
$R^2 = 0.9574$

Ave values for 35kN (mm)
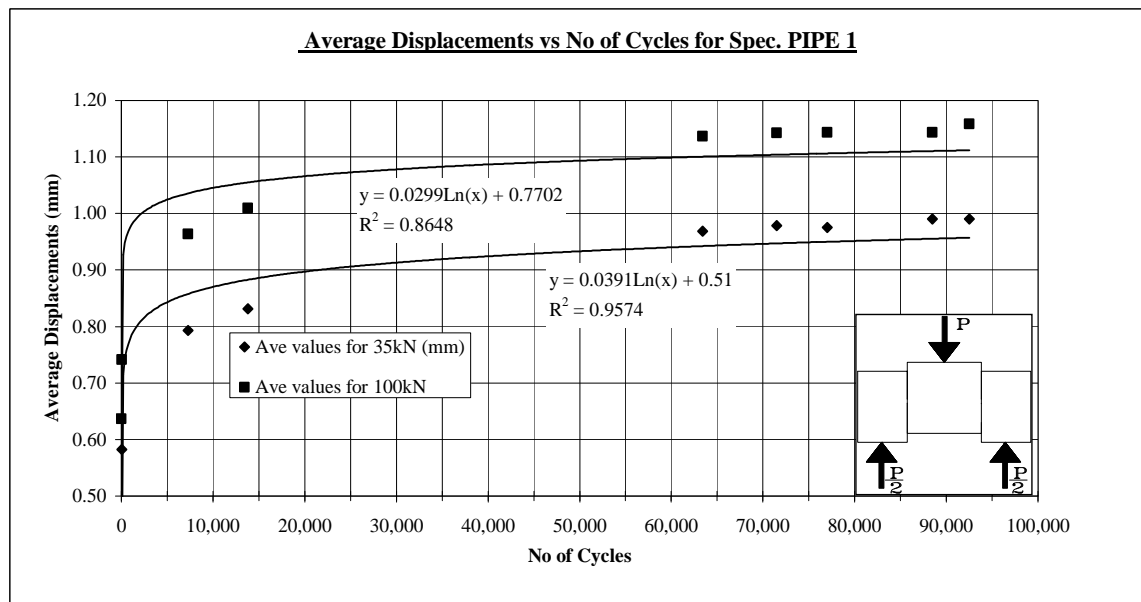Ave values for 100kN

Figure 8 typical dynamic response between the UC's and CHS's.

CONCLUSIONS

Imperfections in the timber produced mixed results as the displacements from the same connector type varied; it was assumed that the concrete was of uniform mix. During testing as the loading rate increased the specimen started to tilt towards the weakest side, hence, incrementing the load on one side. In addition this caused stress concentration on the concrete producing an earlier failure. The steel dowel exhibited the poorest performance against the CHS and UC's. Not only the CHS and the UC's showed close values in strength and approximately three times stronger than the steel dowel but also better flexural behaviour. The range of capacity varies quite considerably in order to try to create a trend. Nonetheless the specimens are able to sustain large forces, even after 100,000 cycles.

Test results have indicated conclusively that the shear connectors investigated and tested have followed well-defined behaviours. In both of the specimens tested full composite action has been achieved with the Universal Column section reaching far better ductility than the Pipe + coach-screw. The test have proven that the connectors are easy to fabricate and install with minimal effort
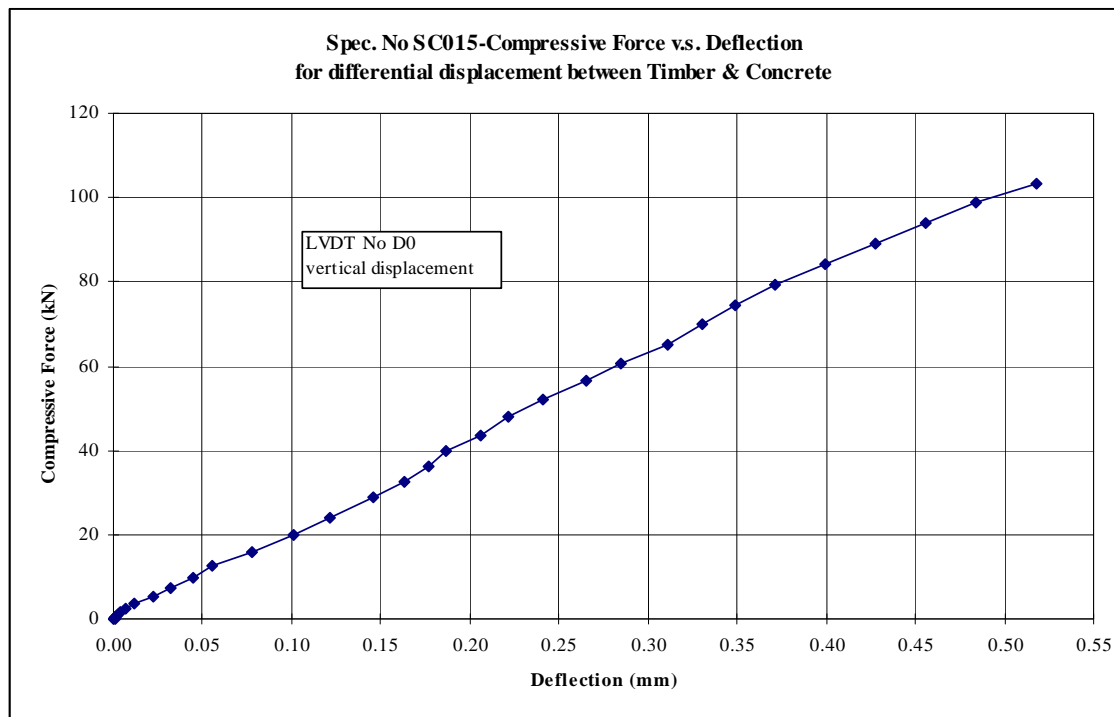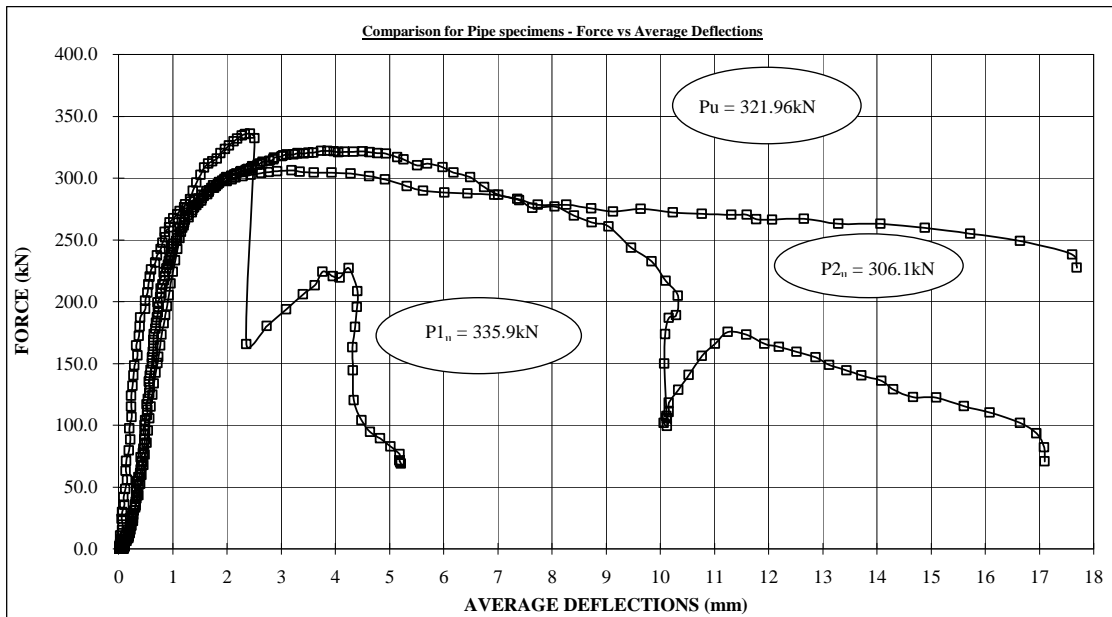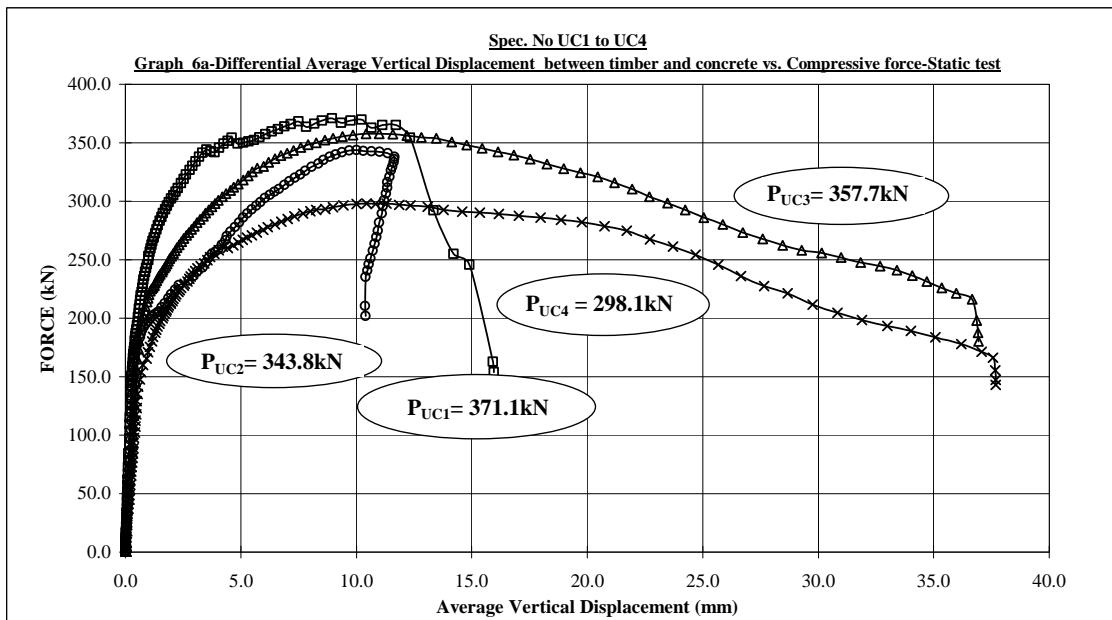

Figure 9

Comparison for Pipe specimens - Force vs Average Deflections

$Pu = 321.96kN$

$P2_n = 306.1kN$

$P1_n = 335.9kN$

Figure 10

Spec. No UC1 to UC4
Graph 6a-Differential Average Vertical Displacement between timber and concrete vs. Compressive force-Static test

$P_{UC3} = 357.7kN$

$P_{UC4} = 298.1kN$

$P_{UC2} = 343.8kN$

$P_{UC1} = 371.1kN$

Figure 11

ACKNOWLEDGEMENTS

RECOMMENDATIONS

As suggested by the outcome of this investigation, more work needs to be done in developing these connectors for the purpose of applying it to concrete decks over timber beams. Further investigations should concentrate on long term repetitive loading and full-scale deck testing in order to see the overall effect under flexure conditions

 The investigation should continue on the two types of connectors selected if a comparison between pure shear and shear due to bending is to be made.

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