Preliminary research towards a semi-prefabricated LVL-concrete composite floor system for the Australasian market *

D Yeoh †
Faculty of Civil and Environmental Engineering, Universiti Tun Hussein Onn Malaysia, Malaysia

M Fragiacomo
Department of Architecture and Planning, University of Sassari, Italy

A Buchanan
Department of Civil and Natural Resources Engineering,
University of Canterbury, Christchurch, New Zealand

C Gerber
Faculty of Engineering, University of Technology Sydney, NSW, Australia

SUMMARY: *The choice of the best floor solution has always been a key issue in the design and construction of multi-storey timber buildings. Strict performance requirements such as effective acoustic separation of inter-tenancy floors, thermal mass, fire resistance, limitation of deflection, resistance to vibrations and effective diaphragm action are very hard to comply with if only timber is used. The main purpose of this paper is to present the preliminary and some ongoing research in the short- and long-term carried out mainly at the University of Canterbury, New Zealand, for the realisation of a semi-prefabricated laminated veneer lumber (LVL)-concrete composite floor system in both the local and Australasian market. The paper discusses a novel semi-prefabricated LVL-concrete composite system where panels made from LVL joists and plywood flooring are prefabricated off-site. Once the panels are lifted onto the supports and connected side-by-side, a concrete topping is cast-in-situ so as to form a continuous slab connecting all the panels. Composite action between the concrete topping and the panels is achieved using different types of connectors, such as various forms of notches cut from the LVL joists and reinforced with coach screws or toothed metal plates pressed in the LVL joists. After pointing out the advantages of the proposed system over traditional only-timber and only-concrete floor solutions, the paper describes push-out tests in the short-term on connections used in the LVL-concrete composite. Tests to failure of small LVL-concrete composite blocks (push-out tests) with different types and shapes of connection systems were performed at the University of Canterbury, New Zealand, and at the University of Technology Sydney, Australia. The results are parametrically evaluated and discussed in detail. The failure mechanism of the notched connection is highlighted together with the strength and stiffness values for each tested connection system. Subsequently, the four best connection systems were identified and used in beam specimens of 8-10 m in span. The experimental program on the beams is presented briefly in order to provide information of the different phases of the project.*

1 INTRODUCTION

The timber-concrete composite (TCC) beam represents a construction technique widely used overseas for new and existing construction (Ceccotti, 2002). This technique consists of connecting an existing or new timber beam or joist with a concrete slab cast above a timber flooring using a connection system (see figure 1). A steel mesh is placed into the concrete flange in order to resist possible tensile stresses due to slab bending and to reduce the crack width. A plastic membrane is generally laid on the timber flooring in order to prevent concrete leaking

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† Corresponding author David Yeoh can be contacted at dey12@student.canterbury.ac.nz.

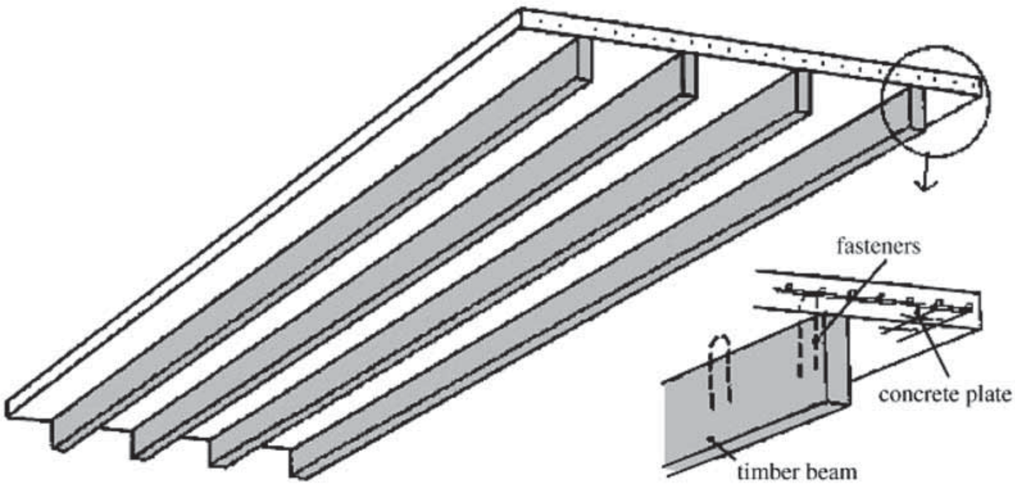

Figure 1: Schematic of a typical timber-concrete composite floor system (Ceccotti, 2002).

during the concrete placement. By interconnecting the lower timber beam with the upper concrete flange, a degree of composite action can be achieved.

A general definition of complete, partial and no composite action is provided in figure 2. A high degree of composite action is highly desirable in TCC structures as it increases both stiffness and load-carrying capacity, with improved structural performance. In a simple member subjected to bending, the bottom outermost fibres are stressed in tension, whereas the top outermost fibres are stressed in compression. The TCC beam is an attempt to combine the high compressive behaviour of concrete with the tensile and flexural resisting behaviour of timber to provide an improved composite beam. When complete composite action is achieved, the layered beam acts as a one-layer beam with mixed material properties. In this case, the beam is stressed such that all or most of the concrete is in compression and all or most of the timber is in tension, depending on the depth of each material. Also there is complete transfer of stresses between the two layers on the layer interface, and no interlayer slip (relative horizontal movement) occurs (see figure 2(a)). Complete composite action is the most efficient combination of the two materials in a layered beam configuration.

Conversely, when the beam has no composite action, the behaviour of the TCC beam is that of an individual concrete beam deflecting on top of an individual timber beam. In this case, the concrete beam and the timber beam are both stressed in pure bending. Furthermore in beams with no composite action, there is no transfer of stresses between the two layers, and large relative movement of the concrete layer with respect to the wood layer, ie. significant interlayer slip, occurs (see figure 2(c)). As a consequence of that, the beam will deflect more, and the material will be stressed more. When connectors are placed between the concrete layer and the timber layer, partial composite action is generally developed (see figure 2(b)). Although the different layers are stressed both in tension and compression, the situation is significantly better than that for the case where there is no composite action. Most of the concrete is stressed in compression and most of the wood is stressed in tension. Interlayer slip does occur but it is smaller in magnitude than the slip developed in the beam with no composite action. Thus the case of partial composite action falls between the limits of no composite action (worst performance) and complete composite action (best performance). Thanks to the composite action, less deflections and larger resistance can be achieved with respect to the only-timber. Thus providing a connection between timber and concrete improves the structural performance at both serviceability and ultimate limit states. Since larger degree of composite action will lead to the better structural performance, it is important to use stiff connection systems.

2 ADVANTAGES AND DISADVANTAGES

The TCC system was originally developed in Europe (Italy, Germany, Switzerland, France, etc.) for strength and stiffness upgrading of existing buildings. The possibility to keep the existing wood floor of historical buildings is, in fact, a significant benefit in ancient buildings of important architectural value. This is made possible by pouring a thin layer of concrete slabs measuring 50 to 75 mm thick on the existing wood floor, normally built from large section timber joists capable to carry the extra weight of the concrete. Flexible connections in the form of nails, screws or bolts drilled into the existing floor joists provide the composite action. The concrete topping, in fact, strengthens and stiffens the existing timber floor, allowing the structure to resist larger loads. Important advantages of TCC over timber-only floors are: (i) retaining the original timber structures and simultaneously increasing its stiffness and strength; (ii) developing a rigid floor diaphragm, which is important for earthquake-prone regions; (iii) enhancing the acoustic separation, thermal mass and fire resistance of the floor; and (iv) reducing

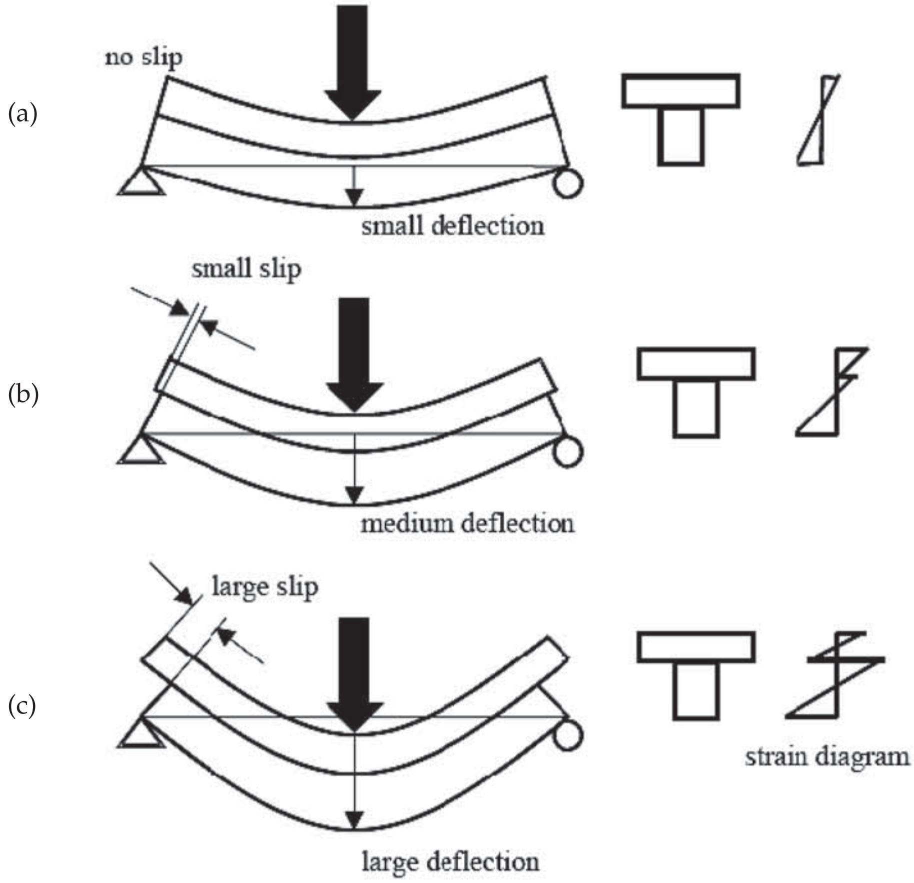

Figure 2: Definitions of composite action – (a) complete composite action, (b) partial composite action, and (c) no composite action.

the susceptibility to vibration. Thanks to the many benefits with respect to timber-only floors, the TCC structure is currently used also for new construction and may represent a possible solution for multi-storey timber buildings. Notable benefits should be highlighted with respect to the more traditional reinforced concrete slabs: the minor self weight, the aesthetic appearance of wood, and the better behaviour of the composite section compared to reinforced concrete structures, with all sustainable benefits of wood.

Despite the indisputable merits of the TCC structures, there are still some issues that reduce the diffusion of such a technique. First of all, the use of the TCC structure is often prevented by the larger labour cost needed. What mostly affects the total construction cost is the connection system. The performance of the TCC floor is significantly influenced by the behaviour of the connection system. Stiff and strong shear connectors are required to provide optimal structural efficiency resulting in a minimum relative slip between the bottom fibre of the concrete slab and the top fibre of the timber beam. Some ductility is desirable since both timber and concrete exhibit quite brittle behaviour in tension and compression, respectively, and the plasticisation of the connection is the only source of ductility for the TCC system (Frangi & Fontana, 2003; Ceccotti et al, 2006). However, the connection system needs to be inexpensive to manufacture and install in order to make TCC floors competitive with other construction systems such as steel and precast concrete floors.

3 PROPOSED SEMI-PREFABRICATED TCC FLOOR SYSTEM

Floors are a crucial part of multi-storey timber buildings. An increasing range of TCC systems has been developed, including cast-in-situ, semi-prefabricated and fully prefabricated floors. Concrete slabs prefabricated off-site that incorporate shear fasteners are being developed in Sweden (Lukaszewska & Fragiacomo, 2008; Lukaszewska et al, 2007). Those slabs are then connected with the timber joists on the building site, providing the possibility of constructing fully demountable solutions. Fully prefabricated TCC panels have also been developed and used in Germany (Bathon, 2006).

A semi-prefabricated floor system is currently under investigation at the University of Canterbury (UC), New Zealand. The proposed system comprises "M" section panels built with laminated veneer lumber (LVL) beams, which act as floor joists and a plywood interlayer as permanent formwork (see figure 3). The panels can be prefabricated off-site and then transported to the building site, craned into position and connected to the main frame with specially-

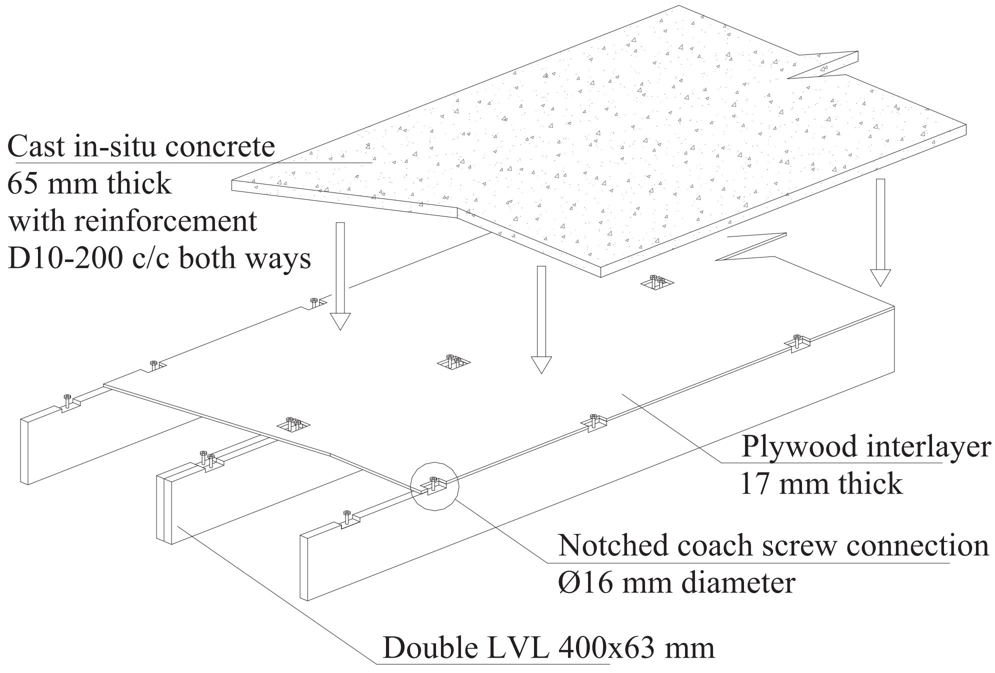

Figure 3: Proposed semi-prefabricated TCC floor system.

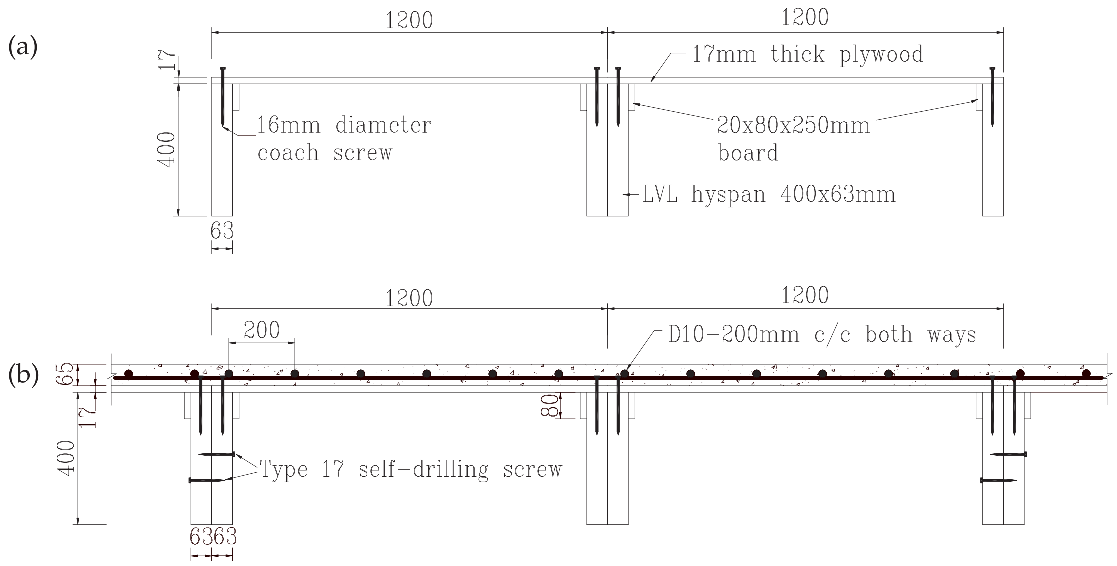

Figure 4: (a) Semi-prefabricated "M" section panel and (b) semi-prefabricated panel connected to adjacent panels with concrete slab (dimensions in mm).

designed joist hangers. Steel mesh is laid above the panels to provide shrinkage control for a 65 mm thick cast-in-situ concrete slab. The panels can be propped while the concrete cures. The connection system has notches cut from the LVL joist, and reinforced with a coach screw to provide more ductile behaviour during failure and to increase the shear strength. These notches are cut into the beams before the plywood interlayer is nailed on.

The 2400 mm wide "M" section panel is built with a single 400 × 63 mm LVL joist on each outer edge and a double LVL joist in the centre. The span of between 8 and 10 m requires six to eight connectors along the length of each joist to provide adequate composite action. Each panel weighs approximately 8 kN, resulting in a lightweight component that is easy to transport and crane. Figure 4 shows the sections of a single panel and how it is joined to the adjacent panels. The design is based on the effective bending stiffness method (the so-called "γ-method") as recommended by Ceccotti (1995) in accordance with the Eurocode 5 (CEN, 2004). A detailed worked example is found in Fragiacomo et al (2007a).

Advantages of this solution include: (i) ease of transport and lifting of the panels due to low weight; (ii) construction of a monolithic concrete slab with better in-plane strength and stiffness, and no need for additional connections between adjacent panels; (iii) high strength and stiffness achievable with reduced number of connectors, thanks to the effectiveness of the notched connection detail; (iv) medium to long-span floors, in the range 6 to 12 m; and, therefore, (v) a system capable of competing with traditional precast concrete solutions. One disadvantage is the need to introduce a "wet" component (the fresh concrete) on the building site, where all other components are "dry" for a multi-storey timber building.

4 CONNECTION PUSH-OUT TESTS

An experimental parametric study is essential for the optimisation of the notch shape so that the best compromise between labour cost and structural efficiency is achieved. Connection push-out tests were carried out separately both at the UC and at the University of Technology Sydney (UTS), Australia.

At UC, the tests were conducted in two phases in accordance with EN 26891 (CEN, 1991), where the connections are loaded in shear and the load-slip relationship recorded using a load cell and potentiometers P1, P2, P5 and P6 (see figure 5; potentiometers P5 and P6 are at the same location as P1 and P2, but on the opposite face of the specimen). A total of 15 different types of connections (A1 to H4) were identified in the first phase, with two of each connection type tested, numbering a total of 30 specimens, as presented in table 1. Variations of the typical notched connection, including the length, depth and shapes (dovetail, triangular, rectangular) of the notch, are detailed in figure 6. Coach screws of 12 and 16 mm diameters were also inserted in the centre of the notches in some cases, while in other cases no coach screw was used. The depth of penetration of the coach screw into the LVL and the end distance of the notch from the LVL were also varied. Slightly modified toothed metal plate fasteners (see figure 6(b)) that are pressed in the lateral side of two adjacent 400 × 63 mm LVL joists were also investigated and compared with the notched connections. In the second phase of the push-out test, three types of connection were tested for the characteristic values of strength and stiffness. Here, a total of nine specimens per type of connection were tested and the results presented in table 2. The details and results of the push-out tests for both the phases are discussed in the following sub-section.

At UTS, a series of push-out tests were performed involving different variations in notched connections such as: (i) square rectangular notch of 90° facets; (ii) bird-mouth or triangular notch; (iii) slant notch with 15°, 25°, 35° and 45° facets; and (iv) curve notch with a radius (see figure 7). The strength results of the tested connections are presented and discussed in the following sub-section. Comparisons were made between the results obtained at UC and UTS for connections that are identical.

4.1 Results and discussion

The relationship between shear force and relative slip for the first phase of push-out test performed in UC is presented in figure 8 for the selected specimens most representative of the different connector shapes. The results in terms of shear strength (F_{max}), secant stiffness (also defined as slip modulus) at serviceability limit state or 40% ($K_{s,0.4}$), at ultimate limit state or 60% ($K_{s,0.6}$) and at collapse or 80% ($K_{s,0.8}$) of the strength are summarised in table 1 as an average of the values measured on two specimens. The strength F_{max} is defined as the largest value of shear force monitored during the test for slips not larger than 15 mm (Fragiacomo et al, 2007a). In order to provide some information on the post-peak behaviour and the level of ductility, the ratio $\Delta 2/\Delta 1$ is introduced, defined as the ratio of strength difference at peak and at 10 mm slip ($\Delta 2$), to the strength at peak ($\Delta 1$), reported in table 1. The lower the $\Delta 2/\Delta 1$ ratio, the better the post-peak behaviour and the higher the ductility. For definition purpose, a ratio below

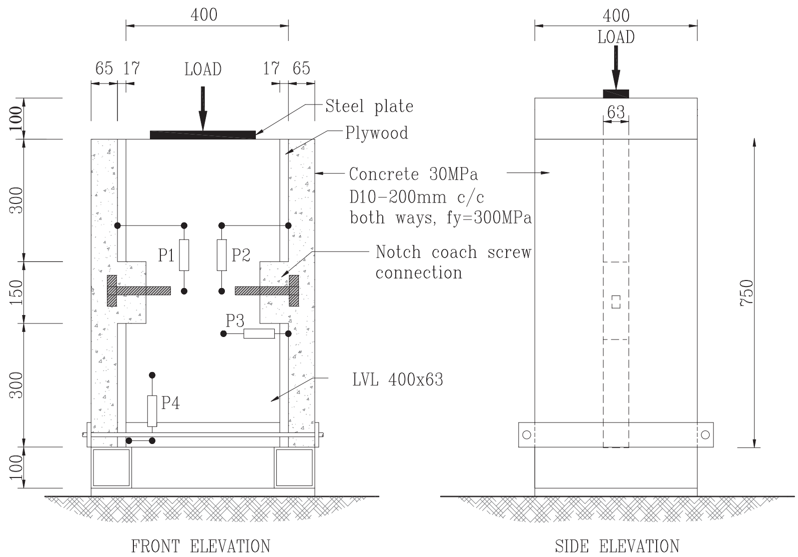

Figure 5: Symmetrical push-out test setup (dimensions in mm).

Table 1: Strength and stiffness values from first phase push out test at UC.

Connection type **length × depth × width (mm)**	F_{max} **kN** **Exp. Anal.**	$K_{s,0.4}$ **kN/mm**	$K_{s,0.6}$ **kN/mm**	$K_{s,0.8}$ **kN/mm**	**Δ2/Δ1** **(%)**
A1: Rectangular notch 150×50×63 Coach screw ϕ16	73.0 68.5	80.2	75.4	61.7	35.5
A2: Rectangular notch 50×50×63 Coach screw ϕ16	46.0 49.1	38.2	34.5	27.5	13.3
A3: Rectangular notch 150×25×63 Coach screw ϕ16	71.8	112.8	102.2	76.1	26.1
90d-150/25-CS[a] (identical to A3)	68.9 N/A	N/A	N/A	N/A	N/A
B1: Rectangular notch 150×50×63	48.3 56.7	104.7	59.3	41.3	73.9
C1: Rectangular notch 150×50×63 Coach screw ϕ12	66.0 66.3	77.9	74.5	62.3	38.8
C2: Rectangular notch 150×50×63 Coach screw ϕ16 depth 140 mm	84.2 87.8	211.2	145.0	95.5	36.5
D1: Doves tail notch 150×50×63	20.5	51.1	28.1	33.5	37.0
E1: Triangular notch 30°_60° 137×60×63	40.2	100.8	57.3	37.9	34.1
E2: Triangular notch 30°_60° 137×60×63 Coach screw ϕ16	82.6	122.8	104.0	75.4	36.5
B-60d/60-CS[a] (identical to E2)	66.48 N/A	N/A	N/A	N/A	N/A
F1: Rectangular notch short end 150×50×63 Coach screw ϕ16	74.4	92.7	91.1	73.6	49.0
G1: Rectangular notch LSC 150×50×63 Coach screw ϕ16	68.8	67.0	66.9	56.1	49.3
H1: Rectangular notch double LVL 150×50×126 Coach Screw ϕ16	128.2	217.9	183.1	119.1	42.1
H2: Double toothed mp 650 mm	163.9 163.4	377.6	275.9	127.4	44.0
H3: Double toothed mp 325 mm	81.1 81.7	480.0	508.4	53.4	33.3
H4: Double toothed mp 150 mm	47.9 37.7	54.3	38.7	31.2	37.5
[a] These specimens were tested at UTS					

50% would be considered as a ductile connection or otherwise a brittle connection.

The strength of connection is influenced significantly by the length of the notch. This is observed in a 50 mm length notch (A2 = 46 kN), which exhibited approximately 60% of the strength of a 150 mm notch (A1 = 73 kN). Similar agreement is also found when comparing the notches without coach screws but have different length at the mouth of the notch, ie. B1, rectangular notch 150 mm length (48.3 kN strength); E1, triangular notch with 137 mm length (40.2 kN

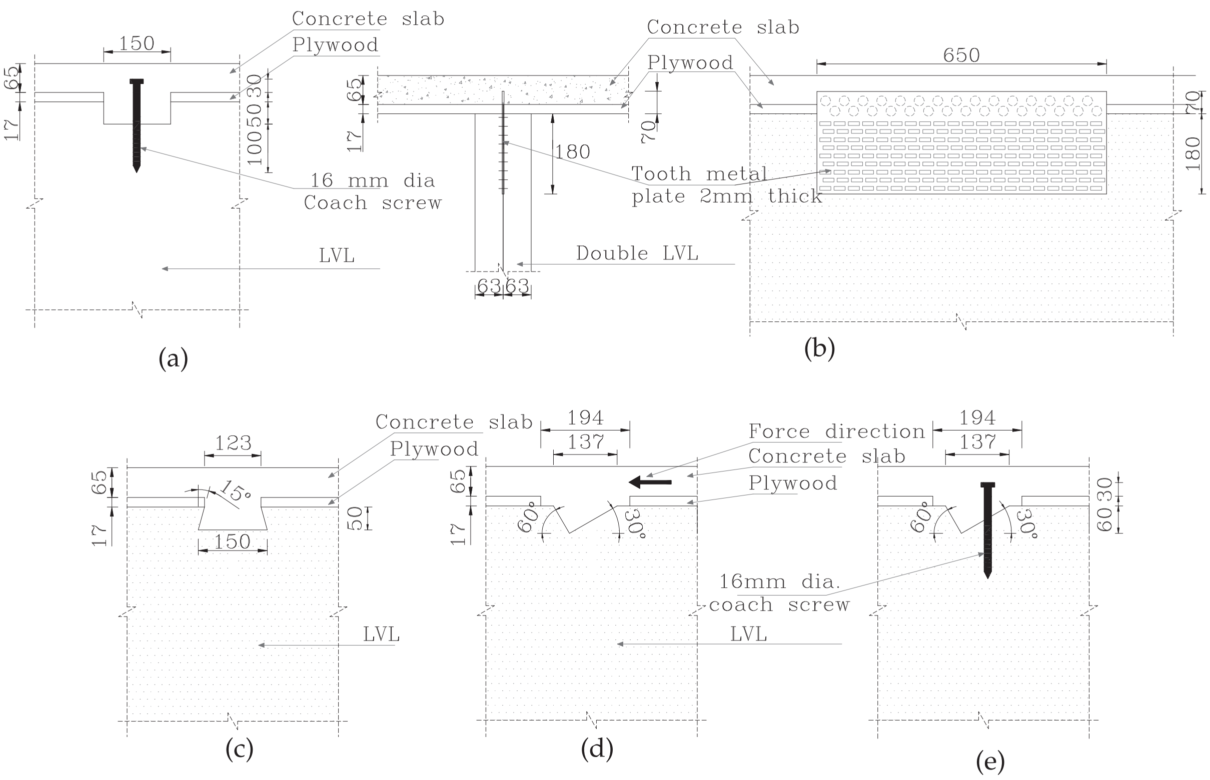

Figure 6: (a) Typical notched coach screw connection, (b) section and elevation of toothed metal plate specimen, (c) dovetail notch, (d) triangular notch and (e) triangular notch with coach screw (dimensions in mm).

Table 2: Strength and stiffness values of connections tested in second phase of push-out test.

Type of connection	Values	Slip moduli			Strength
		$K_{s,0.4}$ (kN/mm)	$K_{s,0.6}$ (kN/mm)	$K_{s,0.8}$ (kN/mm)	F_{max} (kN)
Triangular 60°_30° 137*l* CS	Range	128.2-176.7	121.7-168.3	94.3-140.4	79.0-89.2
	Average	145.8	138.8	115.9	84.8 [66.48]
	σ	13.5	12.7	12.1	3.1 {49.7%}
Rectangular notch 300*l*×50*d*×63*w* CS	Range	216.9-286.0	205.4-282.2	113.7-258.8	130.1-144.2
	Average	247.2	241.4	194.2	138.9 [92.45]
	σ	27.4	28.0	51.2	5.2 {33.9%}
Toothed metal plate 2×333*l* 1 mm thick staggered	Range	249.3-589.5	239.3-510.6	182.3-362.6	129.3-145.4
	Average	463.7	394.6	256.8	139.3
	σ	132.0	100.3	63.1	5.0 {80.7%}

[ ] These specimens were tested at UTS; { } Measure of ductility in ratio Δ2/Δ1

strength); and D1, dovetail notch with 123 mm length (20.5 kN strength).

The presence of coach screw also affects significantly both the strength and stiffness of connection. Coach screw increases the strength of a connection in the range of 1.5 to 2 times to that without coach screw. For instance, connection E2 with coach screw is 2 times stronger than E1 without coach screw. Figure 8(a) shows a similar trend by comparing connections A1 and B1. The initial stiffness as shown in figure 8(c) is not enhanced significantly by a coach screw (compare E1 and E2), however, the coach screw is important to prevent the stiffness from deteriorating after the attainment of the serviceability limit taken as 40% of the maximum shear force. It appeared that the only source of ductility was provided by the coach screw, which also significantly increased the resistance.

It is observed in two cases for B1 and E1 that the stiffness after the attainment of the serviceability limit and the post-peak behaviour markedly degraded in

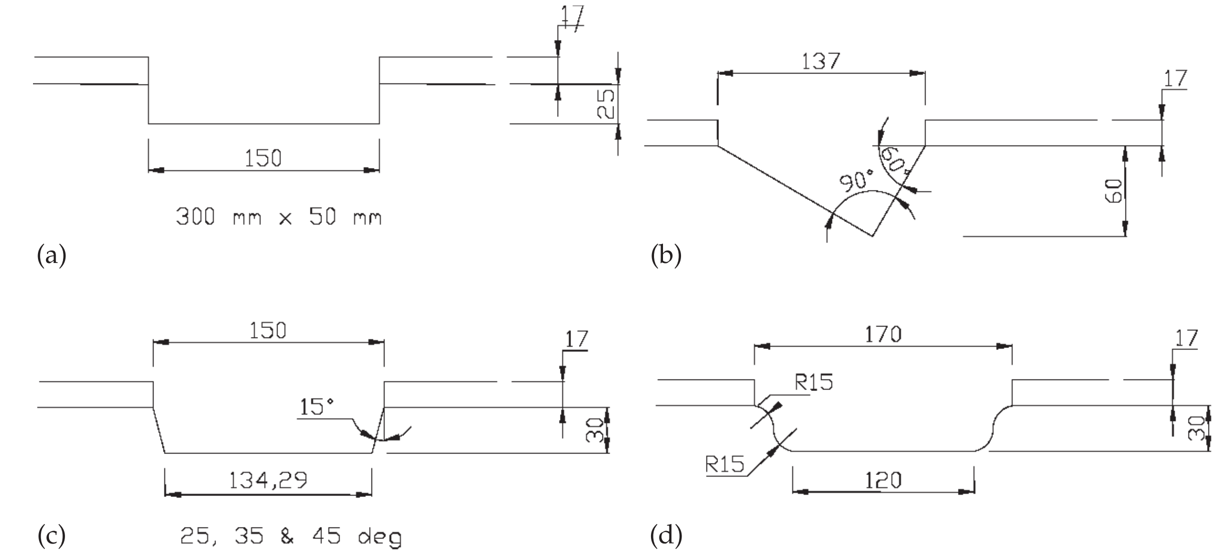

Figure 7: Detailing of the shear connections tested at UTS – (a) square notch (90° facets), (b) bird-mouth, (c) slant notch (15°, 25°, 35° and 45° facets) and (d) curve notch.

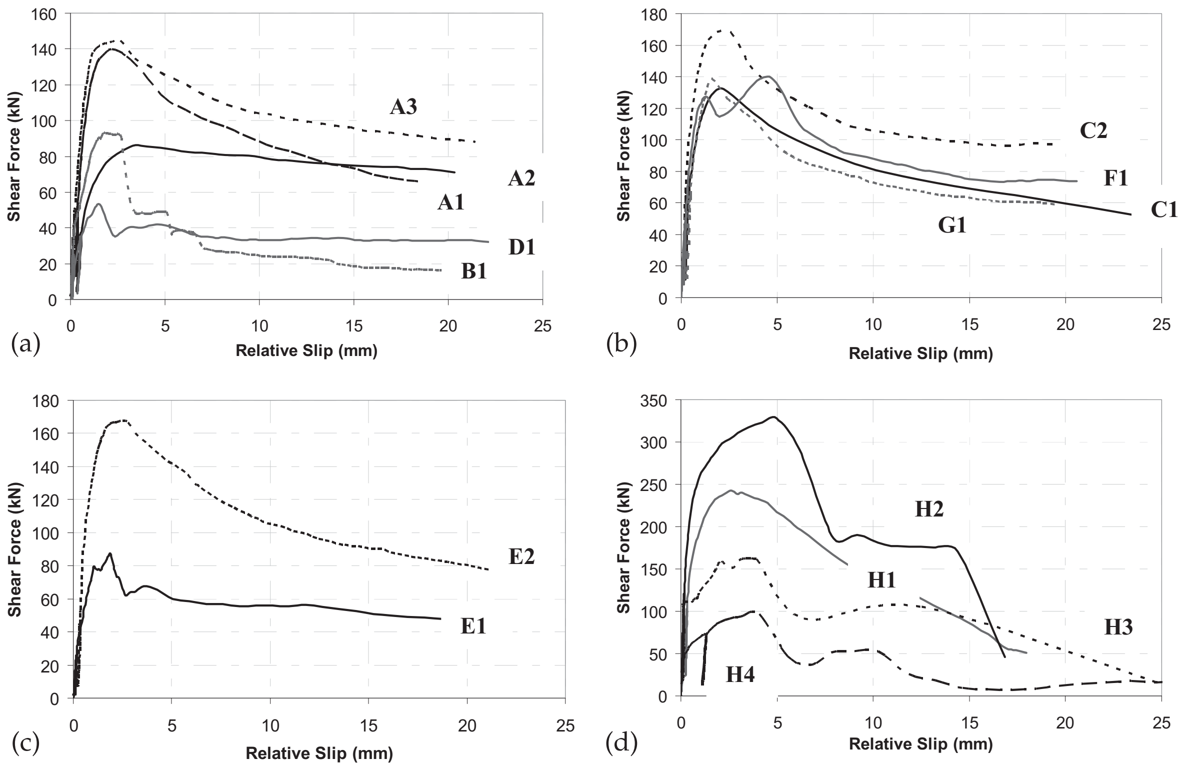

Figure 8: Relationship between shear force and relative slip for 15 connection systems tested in first phase push-out test at UC.

the absence of coach screw. The size of coach screw was found to only affect the strength and not the stiffness as seen in A1 and C1, while the penetration depth in excess of 20 mm increased the strength slightly but caused a large increase in stiffness, as can be noticed by comparing C2 and A1 (see figure 8(b)). The depth of notch has no effect on both the strength and the stiffness properties (compare A1 and A3). Generally, all of the specimens failed by shear in the concrete (see figure 9(a)), hence a longer length of notch is essential to improve the shear strength. The triangular shaped notch demonstrated similar performance to that of a rectangular notch having a nearly equal length (compare F_{max} for specimens A1, 73 kN, and E2, 83 kN), thus making it one of the more viable options as it is much easier to manufacture.

The double-sided 2 mm thick toothed metal plate connection (specimens H2, H3 and H4) exhibited a ductile plate tearing failure with high strength and stiffness, as presented in figure 9(b). The strength of this connection can be easily determined from the plate's yield strength and length. Furthermore, the connection demonstrated an encouraging result as shown in figure 8(d) making it by far the

Figure 9: (a) Rectangular notched connection failure – shear in concrete length, and
(b) toothed metal plate connection failure – plate tearing along length of plate.

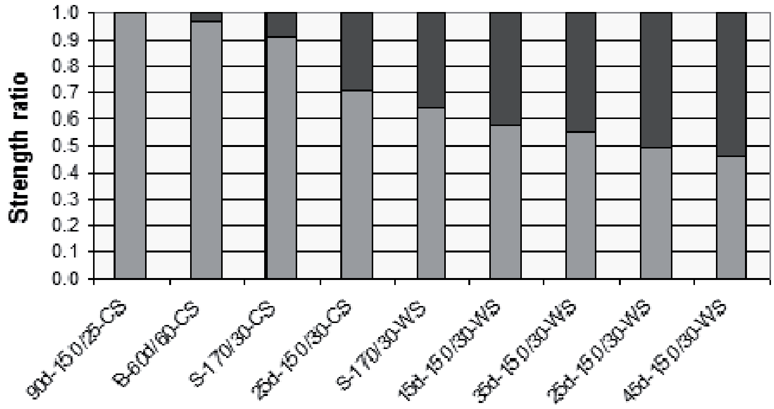

Figure 10: Strength comparison of push-out tests at UTS.

most appreciated connection type apart from the rectangular and triangular notches with coach screw connections.

Similar agreement concerning the use of a coach screw was found in the test results of UTS. A comparison of relative strength of each of the connection types is presented graphically in figure 10 (coach and wood screws are labelled as CS and WS in the specimen name, respectively), where the strength of each connection is expressed as a percentage of the strength achieved for the strongest connection (90d-150/25-CS) – which corresponds to 100%. The notation given for the type of connection can be read as, for instance 90d-150/25-CS: 90d for 90° facet, 150 for notch length, 25 for depth of notch and CS for coach screw. Other notations used are B for bird mouth and S for slanted facet. It can be clearly seen that the connections with a CS achieve higher strength than that with WS. In addition, two distinct groups of performance bands can be identified; the first one includes 90d-150/25-CS, B-60d/60-CS and S-170/30-CS (these three series offering high strength), while the second one includes the slanted-facet connection, with these series achieving about 50% of the strength of 90d-150/25-CS. More details of the investigations carried out at UTS are presented in Gerber et al (2008).

Figure 11 illustrates the failure mechanism of a typical notched coach screw connection experimentally observed during most of the tests. In general, a shear plane begins to form at $0.6F_{max}$. Thereafter, the coach screw starts to act in tension until two plastic hinges were developed. At that stage, the coach screw transfers most of the shear of the connection by rope effect. Further information on analytical and numerical model of the connections can be found in Yeoh et al (2008).

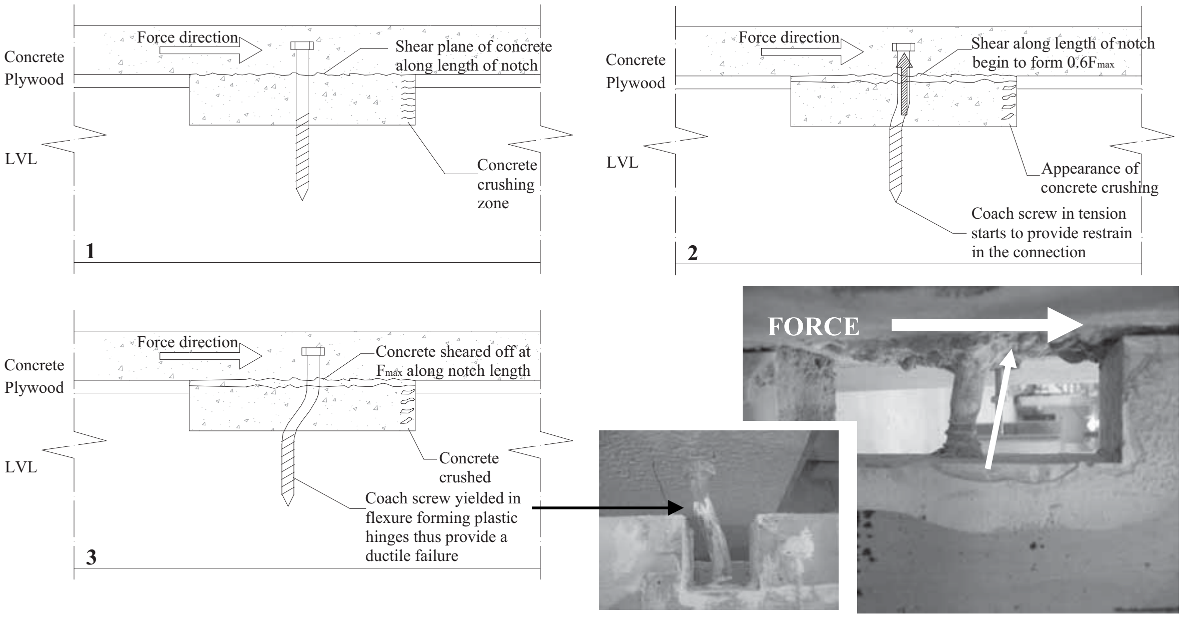

Figure 11: Experimental failure mechanism of notched connection with coach screw

Based on the observations from the experimental tests to failure, and taking into account the ease of construction, the four most promising connection systems were found to be: (i) 150×25 mm rectangular notch reinforced with 16 mm diameter coach screw; (ii) 300×50 mm rectangular notch reinforced with 16 mm diameter coach screw; (iii) 150 mm long triangular notch reinforced with 16 mm diameter coach screw; and (iv) 2×333 mm toothed metal plate connector. The latter three connections were tested in the second phase of push-out tests at UC and the results are presented in table 2. The results indicate that both the 300 mm rectangular notch of single LVL and toothed metal plate connections performed the best in equal strength.

However, the toothed metal plate shows a stiffness value of 1.8 times the rectangular notch. This is by far the most appreciated connection. The downside of this mechanical fastener is the level of ductility measured at 80.7%, hence defined as brittle. This is contrary to the test result of such connection in the preliminary phase, which showed a ductility ratio in the range of 33-44%. Such phenomena could be attributed to the reduction of plate thickness from 2 to 1 mm, double-sided teeth to single-sided teeth, and continuous plate length to two separate pieces of plate in staggered position. Both the rectangular and triangular notches are termed ductile having a ductility ratio of less than 50%. The 300 mm rectangular notch is 1.9 times stronger and 3 times stiffer than the 150 mm rectangular notch, which was tested in the first phase (see A1 in table 1).

The significant difference of strength and stiffness values of the same connection tested in UC and UTS as presented in tables 1 and 2 is largely attributed by the strength and quality of concrete. For instance, honeycomb due to lack of compaction was observed in the 300 mm length rectangular notch in UTS. The mean compressive strength at UTS was 32.73 MPa, as opposed to 42.71 MPa at UC. The failures in the notches are predominantly due to concrete shear along the length of the notch and therefore the compressive strength of concrete is an important indicator.

5 COMPOSITE BEAM EXPERIMENTAL PROGRAM

An extensive experimental program on full-scale T-strips of TCC floor spanning 8 and 10 m is currently ongoing at UC in collaboration with UTS, which involves five phases: (i) short-term monitoring of beams outdoor and indoor, in unconditioned environment, where the deflections and strains of nine beams have been monitored for a period of 1 month after the concrete placement to investigate the effects of the construction process and the environmental changes; (ii) short-term monitoring of beams indoor in unconditioned environment, where four beams are being monitored for a period of 3 months with the quasi-permanent load condition G_k+0.4Q_k applied using water buckets after 28 days (see figure 12(a)) from the concrete placement in order to investigate the time-dependent behaviour during construction and the first months of life of the structure; (iii) repeated loading of selected beams under 2 million cycles, so as to investigate the possibility of using the proposed system for short-span bridges; (iv) test to failure of all the beams in (i) and (ii) under four-point bending static load (see figures 12(b) and 13); and (iv) long-term monitoring of three beams under quasi-permanent load condition for a period of 1 year followed by unloading for 3 months to assess the creep coefficient during loading and unloading periods.

The four most promising types of connectors for the beam specimens were identified using the

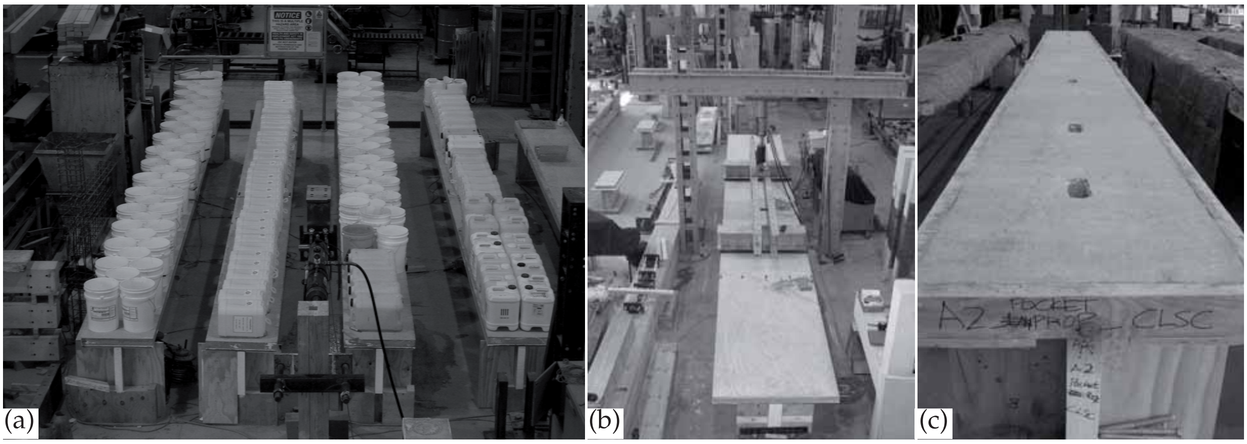

Figure 12: Full scale TCC T-beams at the UC – (a) arrows pointing to four beams under service loads using buckets of water, (b) an 8 m beam, 1200 mm width ready for collapse test at four point bending, and (c) arrows pointing to connection pockets in beam.

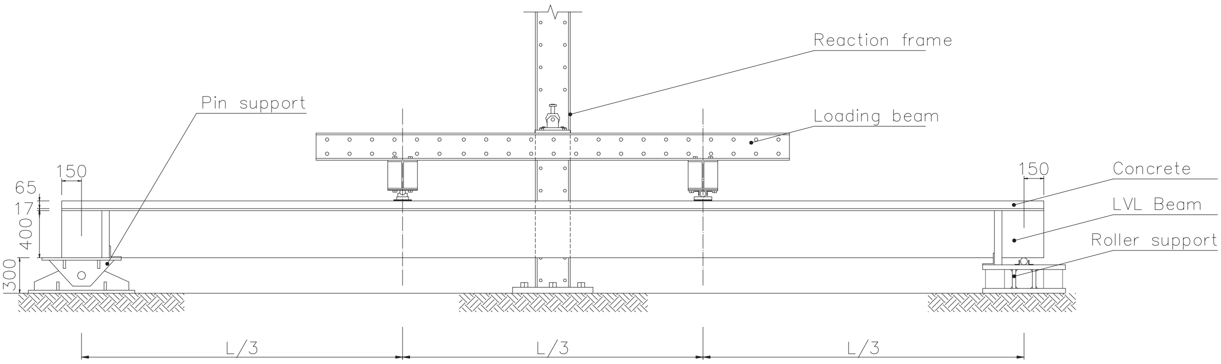

Figure 13: Four-point bending test setup for collapse test of TCC beams (dimensions in mm).

push-out tests detailed in the previous section. Different numbers of connectors corresponding to two scenarios, well-designed and under-designed according to the Eurocode 5 provisions, have been considered for each type of connection. Well-designed herein refers to full compliance of all inequalities at both the ultimate and serviceability limit state verifications, while under-designed refers to a beam design where the demand of maximum shear force in the connection exceeds approximately 1.3 times the shear force resistance of the connection at the ultimate limit state. The method of effective bending stiffness (also known as γ-method) for ultimate and serviceability limit state was adopted in the design, with the slip moduli $K_{s,0.4}$ at serviceability limit state and $K_{s,0.6}$ at ultimate limit state, and strength values F_{max} obtained from the aforementioned push-out tests for the selected connection type.

Two span lengths were tested: 8 and 10 m. Construction variables include the number of days of propping (0, 7 and 14) and curing (1 and 5), and whether the notches were cast at the time of the concrete placement or grouted 7 days later. The grouted notches required a void or pocket (see figure 12(c)) at the time of concrete placement that was filled later with high strength grout (with shrinkage compensation). The type of concrete was carefully selected as shrinkage may induce excessive deflection on the TCC beam due to the high stiffness of the connection (Fragiacomo et al, 2007a). A commercially available low shrinkage concrete (CLSC) of 35 MPa, 650 microstrain with special admixture (Eclipse), 13 mm size aggregate and 120 mm slump was used. Figure 14 illustrates a typical TCC T-strip beam with a 300 mm length notched coach screw connection.

6 FIRST MONTH MONITORING OF BEAMS

This section reports the first phase of the beam experimental program. Five beams were constructed outdoor, while another four beams constructed indoor. The deflections and strains at mid-span were monitored for all the beams during the first month after the concrete placement (see table 3). The aims of this short-term test are to investigate the effects of environmental changes and type of construction.

Deflections and strains of LVL at mid-span were recorded using potentiometer and strain gauges, respectively, every 5 minutes during concrete casting, and subsequently every hour after the concrete had set. The strains on the LVL joist were measured at three locations along mid-span: at both side faces and lower fibre of LVL (see figure 15). Relative humidity

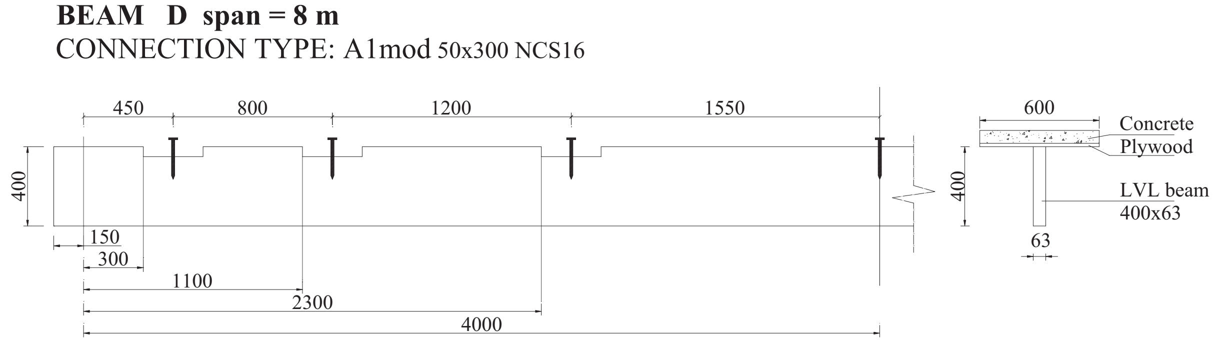

Figure 14: A typical 8 m TCC beam with a 300 mm length rectangular notched connection (dimensions in mm).

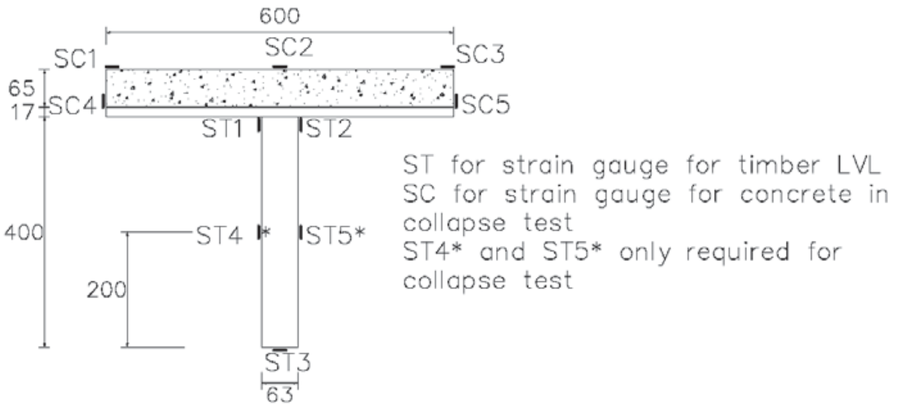

Figure 15: Strain gauges at mid-span (dimensions in mm).

Table 3: Short-term 1 month beams monitoring schedule.

Beam notation and (location)	Connection and (number of connectors) in mm	Span and (width) in m	Propped (days) or unpropped	Design level and (concrete type)
A1 (Indoor)	25*d*×150*l* NCSϕ16 (6 numbers)	8 (0.60)	Propped (14)	Under-designed (CLSC)
C1 (Outdoor)	30°_60° TriNCSϕ16 (10 numbers)	8 (0.60)	Propped (7)	Well-designed (CLSC)
D1 (Outdoor)	50*d*×300*l* NCSϕ16 (6 numbers)	8 (0.60)	Propped (7)	Well-designed (CLSC)
D2 (Outdoor)	50*d*×300*l* NCSϕ16 (6 numbers)	8 (0.60)	Unpropped	Well-designed (CLSC)
E1 (Indoor)	50*d*×300*l* NCSϕ16 (6 numbers)	10 (0.60)	Propped (7)	Under-designed (CLSC)
E2 (Indoor)	50*d*×300*l* NCSϕ16 (6 numbers)	10 (0.60)	Propped (7)	Under-designed (NC)
F1 (Outdoor) double LVL	Plate_2×333*l* Staggered (8 numbers)	8 (1.20)	Propped (7)	Well-designed (CLSC)
F2 (Outdoor) double LVL	Plate_2×333*l* Staggered (8 numbers)	8 (1.20)	Unpropped	Well-designed (CLSC)
G1 (Indoor) double LVL	2×25*d*×150*l* NCSϕ16 (6 numbers)	8 (1.20)	Propped (7)	Well-designed (CLSC)

Note: NCS = notched coach screw, CLSC = commercial low shrinkage concrete, NC = normal concrete

(RH) and temperature were automatically recorded with four key events noted overtime: (i) concrete placement, (ii) concrete set, assumed as 6 hours after casting, (iii) prop removal, and (iv) 28 day.

6.1 Results and discussions

Figure 16 reports the history of mid-span deflection for selected outdoor TCC beams (C1, D1 and D2) under unconditioned environment. Overall, the deflection plot in all the beams throughout the whole monitoring period followed a wave pattern with daily period according to the environmental fluctuations. The peaks of RH occurred at the times of the minimum daily temperatures. The fluctuation of deflection was found in all plots to be consistent with the peaks of RH and minimum values of temperature. Basically, the deflection fluctuation was within the range of 4 to 6 mm, and took place between day and night.

Deflection of unpropped beam (D2) increased 11 mm at time of casting. Uneven and soft outdoor grounds have caused invalid deflection in propped beams (C1, D1), which had to be corrected. Props were removed after 7 days in propped beams. An instantaneous 6 to 10 mm, deflection increment was recorded when the prop was removed, although the final deflection at 28 days was in the range of 5 mm less than the unpropped beams. On the whole, propping of beams at mid-span was important to minimise permanent deflection and enable initial composite action to be developed before sustaining the full self-weight of the concrete slab. Nevertheless, after the removal of props, deflection fluctuations in all beams follow a similar trend due to RH and temperature changes, which were also observed in unpropped beams. Figure 17 displays the indoor experimental-numerical comparisons in terms of mid-span deflection for selected TCC beams (E1, E2). The environmental fluctuations were not as prominent as in outdoor conditions and, therefore, the day-to-night deflection variations were insignificant. Low shrinkage concrete (in E1) was effective in reducing the total deflection by 5 mm at 28 days, when compared to normal weight concrete (in E2). The concrete shrinkage, in fact, increases the overall deflection of composite beams, especially when the connection is very stiff like in the case under study.

The temperature and RH experienced during these tests were not as adverse as it would be in many regions of Australia, which will impose high fluctuations. Therefore it is crucial that further tests be carried out to monitor the behaviour of the system under more severe conditions.

7 CONCLUSIONS

The important issue with such a composite system addressed in this paper and in the currently ongoing research is the large deflections experienced over the service life of the structure. In order to minimise these deflections, it is recommended to use concrete with reduced shrinkage and propping at mid-span, which were done in this project. Another possible method of reducing the deflections is by precambering the floor joist, and in this case of precambering the LVL; it involves a modification in the cutting of the LVL at the factory. This paper has presented the preliminary outcomes of a broad experimental program ongoing at UC and UTS, also with the participation of overseas institutions such as the University of Sassari, Italy. This joint research program is aimed to develop a floor solution suitable for medium- to large-span floors in multi-storey timber buildings. The performance requirements of effective acoustic

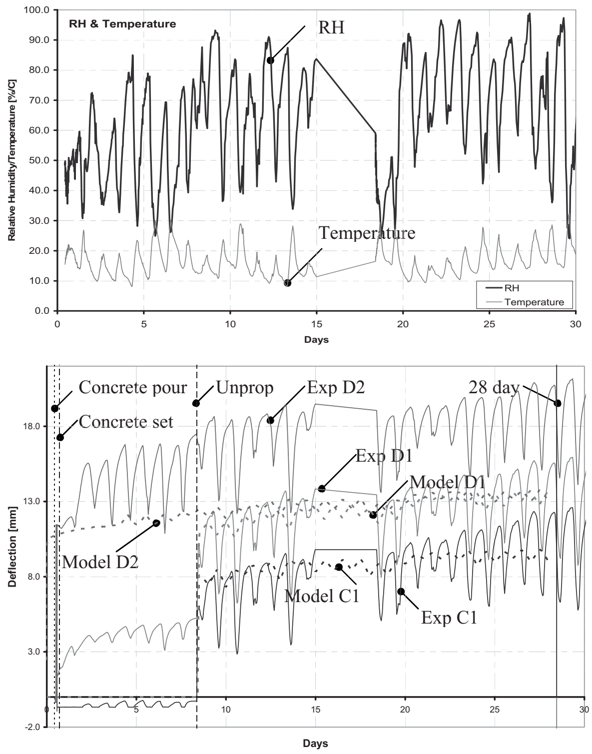

Figure 16: History of mid-span deflection for outdoor beams (bottom) with corresponding RH and temperature histories.

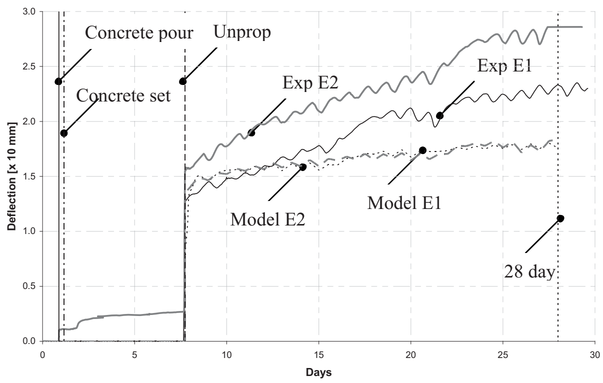

Figure 17: History of mid-span deflection for indoor beams.

separation, adequate fire resistance and reduced susceptibility to vibrations indicate that the use of a concrete topping is highly desirable. In order to exploit the stiffness and strength contribution of the concrete, a shear connection system should be used, so as to obtain composite action between the concrete topping and the timber beam. The solution under research is therefore a semi-prefabricated TCC system, where timber panels made from LVL joists and plywood sheets are prefabricated off-site, craned into position and used as permanent form for the concrete topping, which is poured on site. This solution has the advantages of the prefabrication and allows, at the same time, the construction of a monolithic floor due to the concrete topping poured on site.

Composite action is obtained by cutting notches from the LVL joists and relying on the bearing at the timber-to-concrete interface, or using tooth metal plates pressed on the side faces of the LVL joists. Different notch shapes have been investigated by performing push-out tests on small composite blocks, and the four more promising systems identified. The mechanical properties of the connectors (shear strength and slip moduli) needed for the design of the floor have then been evaluated. Based on those values, strips of 8 and 10 m composite floors for office buildings have been designed, constructed and tested. The tests, currently ongoing, include long-term, repeated and monotonic loading.

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DAVID YEOH

David Yeoh is a senior lecturer at the Faculty of Civil and Environmental Engineering of Universiti Tun Hussein Onn Malaysia, Malaysia. He has been in the academic and research field for the past 8 years, where he taught a variety of courses in the areas of structures and design at the undergraduate level, such as Theory of Structures, Strength of Materials, Civil Engineering Software Applications, Structural Steel Design, Reinforced Concrete Design and Structural Timber Design. His research interests are mainly in the areas of structural timber and concrete, where he has supervised more than 25 undergraduate theses and authored more than 20 national and international scientific papers. He is an active member of the Malaysian Standard Committee for Structural Use of Timber and has been directly involved on a working group for the drafting of several parts of the standardisations. He is the author of a book titled *Reinforced Concrete Design – A Problem and Solution Approach* published by McGawHill, Singapore. He is currently pursuing his PhD specialising in timber-concrete composite floors at the University of Canterbury, New Zealand, under the supervision of A/Prof Massimo Fragiacomo, Prof Andy Buchanan and Dr Bruce Deam.

MASSIMO FRAGIACOMO

Massimo Fragiacomo is Associate Professor of Structural Design at the Faculty of Architecture of the University of Sassari, Italy. Past work experience includes 3 years as Senior Lecturer at the University of Canterbury, New Zealand, where he taught Timber Engineering, and 6 years as Research Fellow at the University of Trieste, Italy, where he taught Design of Steel Structures. He graduated in Structural Engineering at the University of Trieste, Italy, with the maximum score, and was awarded a PhD in Design and Preservation of Structures by the University of Venice, Italy. He is author of about 120 scientific papers, 30 of which published in international journals listed by the Web of Science. His main areas of expertise include timber engineering, earthquake engineering, composite structures, finite element modelling and steel construction. He is active member in a number of international committees, such as the Cost Action E55 and the CIB-W18 on Timber Structures, and has been principal investigator for research projects of an overall value of several hundreds of thousands of Euros. Massimo has been invited to speak at seminars and professional courses on timber engineering organised by institutions in Europe, America and Australasia. He is also member of the scientific board for the *Journal of Structural Fire Engineering*, and member of the scientific committee for two international conferences on timber engineering.

ANDY BUCHANAN

Dr Andy Buchanan is Professor of Timber Design at the University of Canterbury, Christchurch, New Zealand. He is a structural engineer with special interests in timber engineering, fire safety and earthquake engineering. His current research includes design and structural performance of large timber structures, including innovative post-tensioned timber buildings designed to resist major earthquakes and fires. Andy has a BE (Honours) degree from the University of Canterbury (1970), a Masters degree from the University of California (1972), and a PhD from the University of British Columbia, Canada (1984), where he worked with Prof Borg Madsen. Prior to joining the University of Canterbury in 1987, he was a consulting engineer in private practice, involved in structural design of a wide range of buildings. He is the author of *Structural Design for Fire Safety* (John Wiley & Sons, 2001) and the *Timber Design Guide* (NZ Timber Industry Federation, 2007). He is past President of the New Zealand Timber Design Society, and Research Director of the Structural Timber Innovation Company Ltd.

CHRISTOPHE GERBER

Dr Christophe Gerber is a Research Fellow at the University of Technology Sydney (UTS). He is a structural engineer and has been an active researcher for more than a decade, with a particular interest in timber engineering and wood technology. His current professional interests include building sustainability, composite structures and bonding technology, structural analysis, and finite element analysis. Christophe graduated in structural engineering, major in timber construction, at the Berner Fachhochschule – School of Architecture, Civil and Wood Engineering, in Bienne, Switzerland, in 1997, and obtained his PhD at UTS in 2007. From 1997 to 2002, he worked in the R&D department of the Berner Fachhochschule. Between 2002 and 2006, he completed his PhD research at UTS. Since 2007, he works as a Research Fellow with Prof Keith Crews at UTS and is also involved in the academic program.