Timber-Concrete Composite Connections and Beams

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Abstract

The paper reports the outcomes of an extensive research project on timber-concrete composite (TCC) floors at the University of Canterbury, New Zealand. Two phases of the entire programme are described (1) push-out tests on different notched connection systems and (2) first month monitoring of TCC floor beams after construction. A semi-prefabricated TCC floor system that is economical, practical and easy to construct is proposed. An analytical model for strength evaluation of a typical notched connection reinforced with coach screw is presented. Four best connection details were selected and used to design and construct TCC beams for application in medium to long-span office floors. The composite beams are being and will be tested under short- and long-term loading. The experimental results of the first month monitoring of beams after construction are reported and compared with a uniaxial finite element model which was specially developed for long-term and collapse analysis of TCC beams. Overall, the validations were found to be within good accuracy except for some cases with acceptable experimental deviations. Other parameters observed were different construction variables and type of concrete.

1. Introduction

The timber-concrete composite (TCC) floor is a construction technique which has become quite common in many countries. A concrete slab mechanically connected to its supporting timber joists using either notches cut from the timber or suitable mechanical fasteners enables a number of advantages: (1) retaining the original timber structures and simultaneously increasing its stiffness and strength, (2) developing a rigid floor diaphragm, and (3) enhancing the acoustic separation, thermal mass, and fire resistance of the floor. The materials in TCC are effectively utilised in terms of strength performance where the timber web is mainly subjected to tension and bending, the concrete flange is mainly subjected to compression, and the connection system subjected to shear. A stiff and strong connection system is crucial in order to achieve a suitable bending strength and stiffness of the TCC. Hence, a minimum relative slip between the bottom fibre of the concrete slab and the top fibre of the timber beam, and a high composite efficiency are paramount. Some ductility is desirable since both timber and concrete exhibit quite brittle behaviour in tension and compression, respectively, and the plasticization of the connection is the only source of ductility for the TCC system[1-2]. However, the connection system needs to be inexpensive to manufacture and install in order to make TCC beams competitive with other prefabricated or precast construction systems.

A semi-prefabricated floor system is currently under investigation at the University of Canterbury. The proposed system comprises of "M" section panels built with LVL beams which act as floor joists and a plywood interlayer as permanent formwork (see Figure 1). 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 designed joist hangers. The connection system has

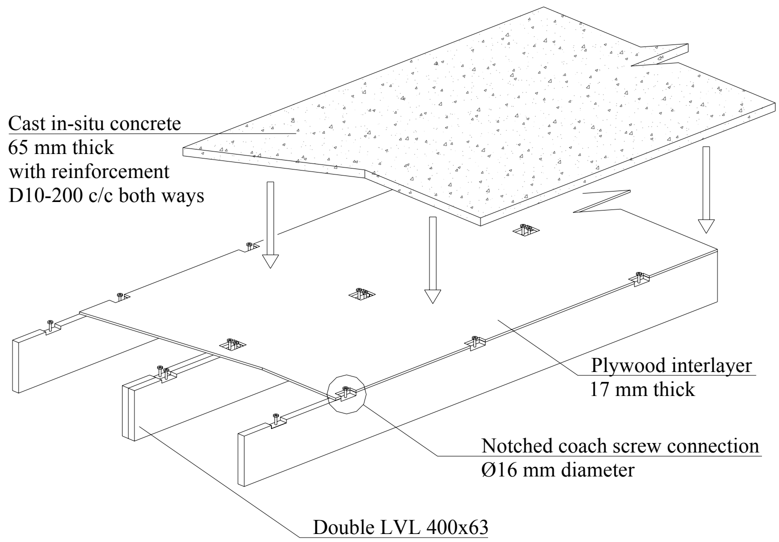

Figure 1 : "M" section semi-prefabricated TCC system

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.

The design is based on the effective bending stiffness method (the so-called "γ-method") as recommended by Ceccotti[3] in accordance with the Eurocode 5[4]. A detailed worked example can be found in Fragiacomo et. al[5].

This paper reports two phases of the investigations carried out: (1) push-out test on different notched connection systems and (2) first month monitoring of TCC floor beams after construction. The purpose of the first phase was to identify the parameters affecting the mechanical properties (shear stiffness and strength) and, ultimately, to optimize the connection detail. Geometrical variations included shape of the notch (rectangular, triangular and dove-tail), depth and length of the notch, use or not of a coach screw, diameter of the coach screw, and the embedment length into the timber. In addition, toothed metal plate connections were also tested since this system is considerably easier to construct. The behaviour of all connections was characterized in terms of shear strength and stiffness at strength and serviceability limit state by testing small timber-concrete composite blocks with the connection loaded in shear (push-out specimens) to failure. Simplified analytical formulae based on the New Zealand Standard for the design of the notch under all possible failure mechanisms were developed and compared with the experimental results. The second part of the paper reports the first outcomes of the long-term monitoring of the beams during and after construction which is a continuing phase after some extensive push-out connection investigations. The mid-span deflection for propped and unpropped construction methods over one month is compared to a finite element model. Further ongoing research involves tests to failure and long-term tests of full scale concrete-LVL composite beams, dynamic vibration tests of composite beams, and tests under repeated loads of composite beams.

2. Push-Out Tests on Connections

In order to optimize the notch shape with the best compromise between labour cost and structural efficiency, an experimental parametric study was carried out. The performance of different connector shapes listed in Table 1 was evaluated through experimental push-out shear tests performed on small LVL-concrete composite blocks (see Figure 2a). Variations of the typical notched connection (see Figure 2b) included the length, depth, and shape (dovetail, triangular and rectangular) of the notch. Coach screws of 12 mm 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 that are pressed in the lateral side of two adjacent 400 × 63 mm LVL joists were also investigated and compared with the notched connections[6]. A total of 15 different types of connection were selected. Two push-out specimens were then constructed for each connection type, for a total of 30 specimens. The push-out tests were performed in accordance with EN 26891[7] 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 2a).

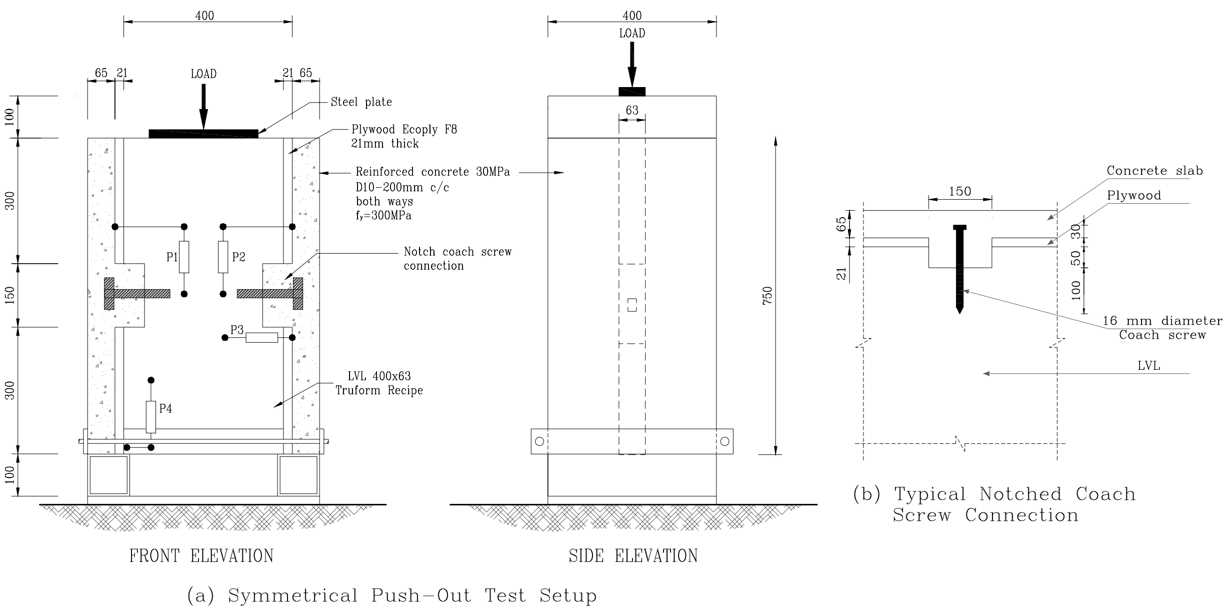

Figure 2 : Push-out test setup and typical notched coach screw connection (dimensions in mm)

2.1 Results and Discussion

The relationship between shear force and relative slip is presented in Figure 3 for the 15 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 40% ($K_{S,0.4}$), 60% ($K_{S,0.6}$) and 80% ($K_{S,0.8}$) of the strength[7] are summarized in Table 1. The strength F_{max} is defined as the largest value of shear force monitored during the test for slips not larger than 15 mm[7]. In order to provide some information on the post-peak behaviour and, therefore, on the ductility level, the ratio $\Delta 2/\Delta 1$ between the difference in strength at peak and at 10 mm slip, $\Delta 2$, and the peak strength, $\Delta 1$, is reported in Table 1. The lower the $\Delta 2/\Delta 1$ ratio, the better the post-peak behaviour and the higher the ductility.

The most important factors affecting the connection performance were found to be the length of the notch (compare F_{max} for specimens A1, 73kN and A2, 46kN) and the presence of a coach screw (compare F_{max} for specimens A1, 73kN, and B1, 48.3kN). Generally, all of the specimens failed by shear in the concrete (see photo in Figure 3), hence a longer length of notch is necessary to improve the shear strength. The only source of ductility was provided by the coach screw, which also significantly increased the resistance. The presence of a coach screw and its depth of penetration into the timber (compare $K_{s,0.4}$ for specimens A1 with 100mm penetration, 80kN/mm, and C2 with 140mm penetration, 211kN/mm) significantly enhanced the stiffness of the connection. Further findings and discussions on other connections including numerical model of the connections can be found in Yeoh et. al[6].

Connection Type (***length*** **×** ***depth*** **×** ***width***) **mm**	F_{max} Exp.	kN Anal.	$K_{s,0.4}$ kN/mm	$K_{s,0.6}$ kN/mm	$K_{s,0.8}$ kN/mm	$\Delta 2/\Delta 1$ (%)
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
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 140mm	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
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 sided toothed metal plate 650 mm	163.9	163.4	377.6	275.9	127.4	44.0
H3: Double sided toothed metal plate 325 mm	81.1	81.7	480.0	508.4	53.4	33.3
H4: Double sided toothed metal plate 150 mm	47.9	37.7	54.3	38.7	31.2	37.5

Table 1 : Shear strength and stiffness values for 15 different connection systems

As a result of the observations from the experimental tests to failure, and taking into account the ease of construction, the four most promising connection systems were selected for the design of TCC floor beams in the next phase of the research: (1) 150 × 25 mm rectangular notch reinforced with 16 mm diameter coach screw; (2) 300 × 50 mm rectangular notch reinforced with 16 mm diameter coach screw; (3) 150 mm long triangular notch reinforced with 16 mm diameter coach screw; and (4) toothed metal plate connector. The choice of the 300 mm length of connection (2) was based on the length of notch being the most important parameter to obtain a strong connection.

3. Analytical Model for Strength Evaluation of Connection

A simplified analytical model for strength evaluation of the notched connection is presented in Equations (1) to (4). The formulae were verified with the experimental results and were found to predict the failure load within acceptable range in most cases (see Table 1). The model is based on the control of all possible failure mechanisms that may occur in the connection region. The notched connection is regarded as a concrete corbel protruding into the LVL joist subjected to shear and bending moment coming from the shear load applied on the connection[6].

A simplified analytical model is used to evaluate the failure load associated with all the possible failure mechanisms of the connection, which are: (1) failure of concrete in shear in the notch; (2) failure of concrete in compression in the notch; (3) failure of LVL in longitudinal shear, between two consecutive notches or between the last notch and the end of the beam; and (4) failure of LVL in crushing parallel to the grain at the interface with the concrete corbel.

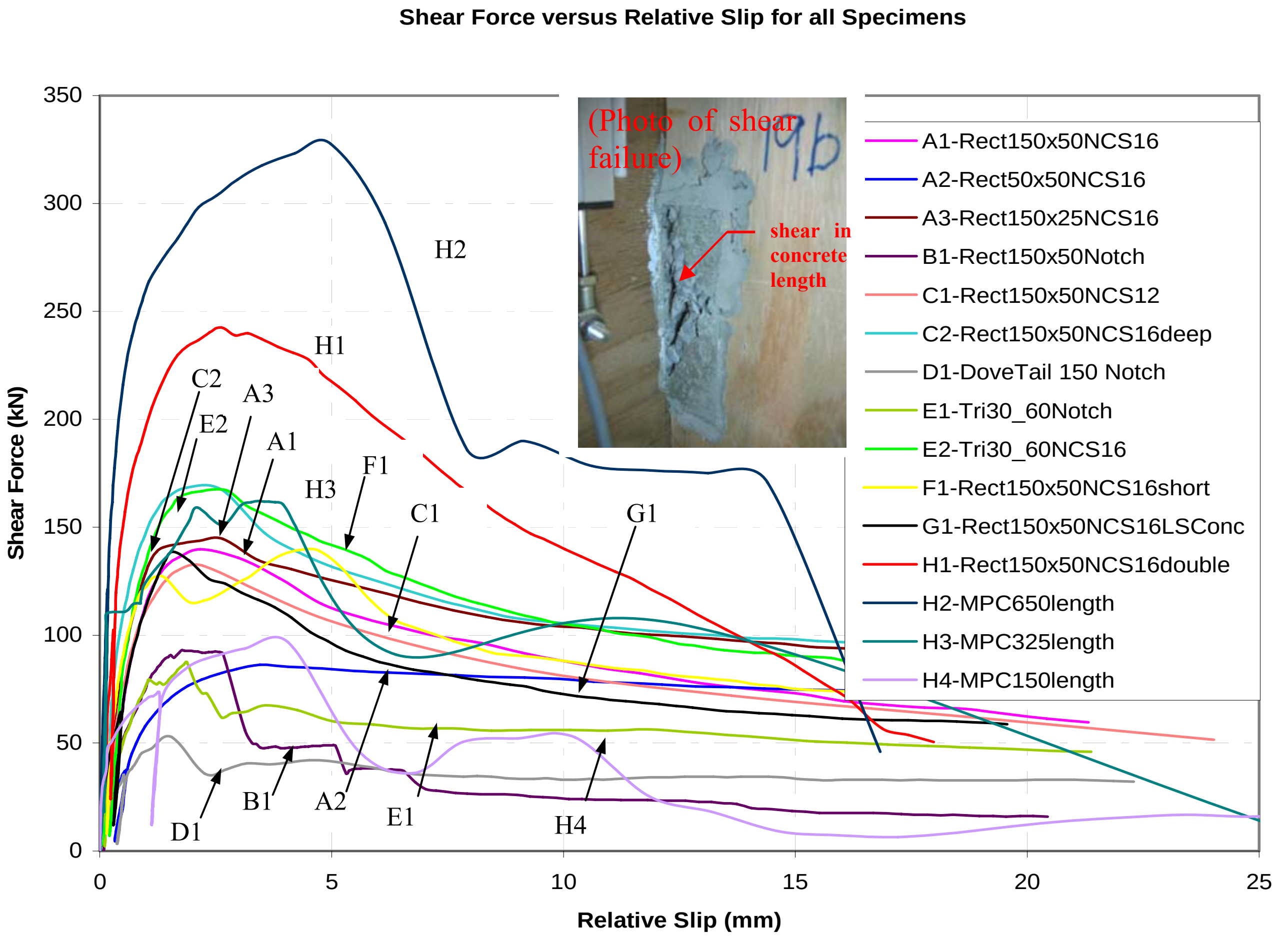

Figure 3 : Relationship between shear force and relative slip for 15 tested connection systems with photo of the shear failure in notched connection with coach screw

The corresponding design strengths are calculated in accordance with provisions of the New Zealand Standards for both timber and concrete structures[8-9]. The formulas are reported in the following:

1) Nominal shear strength of concrete for a notched connection reinforced with a coach screw:

$$F_{conc,shear} = 0.2 f_c^{'} bd + nkpQ_k \tag{1}$$

2) Nominal compressive strength of concrete in the crushing zone:

$$F_{conc,crush} = f_c^{'} A_c \tag{2}$$

3) Nominal longitudinal shear strength of LVL between two consecutive notches or between the last notch and the end of the timber beam:

$$F_{LVL,shear} = k_1 k_4 k_5 f_s Lb \tag{3}$$

4) Compressive strength of LVL at crushing zone,

$$F_{LVL,crush} = k_1 f_c hb \tag{4}$$

where f'_c is the compressive strength of concrete, b and d are the breadth and depth of notch, respectively, n is the number of coach screws, k_1 is the modification factor for duration of loading for timber, p is the depth of penetration and Q_k is the characteristic withdrawal strength of the coach screw in Equation 1. A_c is the crushing zone effective area, i.e. $b \times d$ in Equation 2. k_4 and k_5 are the modification factors for load sharing, f_s is the LVL characteristic shear stress, L is the shear effective length and b is the breadth of the LVL beam in Equation 3. f_c is the LVL characteristic compressive stress, and h is the depth of the notch. The design values of the shear strength is then obtained by multiplying the minimum among the four values reported above by the strength reduction factor ϕ.

4. Composite Beam Experimental Programme

The full-scale TCC floor beams experimental programme discussed herein comprises of 4 phases: (1) short-term monitoring of beams outdoor and indoor, in unconditioned environment, where the deflections of 9 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; (2) short-term monitoring of beams indoor in unconditioned environment, where 4 beams are being monitored for a period of 3 months with the service load applied after 28 days from the concrete placement in order to investigate the time-dependent behaviour during construction and the first months of life of the structure; (3) repeated loading of selected beams and test to failure of all the beams in (1) and (2) under four-point bending static load; and (4) long-term monitoring of 3 beams under service load for a period of 1 year and then 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 push-out tests mentioned in the preceding section. All the beams with different numbers of connectors corresponding to two scenarios, well-designed and under-designed according to the Eurocode 5[4] provisions, have been considered and designed at ultimate limit state and serviceability limit state for each type of connection.

Each beam has been designed and constructed by varying a number of parameters: (1) the type of connection, (2) the number of connectors, (3) the span length, (4) the type of construction, and (5) the type of concrete. Two span lengths were tested: 8 m and 10 m. Construction variables include the number of days of mid-span propping (0, 7 and 14) and curing (1 and 5), and whether the notches are cast at the time of the concrete placement or grouted 7 days later. The type of concrete was carefully selected as shrinkage is expected to induce significant deflection on the TCC beam due to the high stiffness of the connection. The concrete selected is a commercially available low shrinkage concrete (CLSC) of 35 Mpa strength, 650 microstrain shrinkage at 28 day with special admixture (Eclipse), 13 mm size aggregate and 120 mm slump.

5. First Month Monitoring of Beams

This section reports the first phase of the aforementioned extensive research programme. 5 beams were constructed outdoor while another 4 beams constructed indoor. The deflections at mid-span were monitored for all the beams during the first month after the concrete placement (see Table 2). Figure 4 displays a typical 8 m TCC T-beam with a 300 mm length rectangular notched connection. The aims of this short-term test are to investigate the effects of environmental changes and type of construction, and compare the experimental results with a purposely developed uniaxial finite element model.

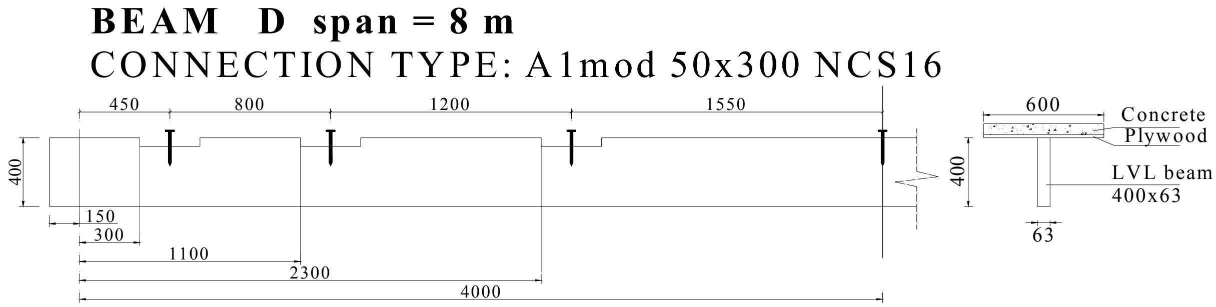

Figure 4 : A typical 8 m TCC T-beam with a 300 mm length rectangular notched connection (dimensions in mm)

Beam Notation and (Location)	**Connection and (Number of connectors) in mm**	**Span and (Width) in metre**	**Propped (Days) or Unpropped**	**Design level and (Concrete Type)**
A1 (Indoor)	25*d*x150*l* NCSϕ16 (6 numbers)	8 (0.60)	Propped (14)	Under-designed (CLSC)
C1 (Outdoor)	30º_60º TriNCSϕ16 (10 numbers)	8 (0.60)	Propped (7)	Well-designed (CLSC)
D1 (Outdoor)	50*d*x300*l* NCSϕ16 (6 numbers)	8 (0.60)	Propped (7)	Well-designed (CLSC)
D2 (Outdoor)	50*d*x300*l* NCSϕ16 (6 numbers)	8 (0.60)	Unpropped	Well-designed (CLSC)
E1 (Indoor)	50*d*x300*l* NCSϕ16 (6 numbers)	10 (0.60)	Propped (7)	Under-designed (CLSC)
E2 (Indoor)	50*d*x300*l* NCSϕ16 (6 numbers)	10 (0.60)	Propped (7)	Under-designed (NC)
F1 (Outdoor) double LVL	Plate_2x333*l* Staggered (8 numbers)	8 (1.20)	Propped (7)	Well-designed (CLSC)
F2 (Outdoor) double LVL	Plate_2x333*l* Staggered (8 numbers)	8 (1.20)	Unpropped	Well-designed (CLSC)
G1 (Indoor) double LVL	2x25*d*x150*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

Table 2 : Characteristics of beams monitored over 1 month

Deflection of LVL at mid-span was recorded using potentiometer, every five minute during concrete casting and subsequently every hour after the concrete has set. Relative humidity and temperature were automatically recorded with 4 key events noted overtime: (1) concrete placement, (2) concrete set, assumed as 6 hours after casting, (3) prop removal, and (4) 28 day.

5.1 Finite Element Modelling

A finite element (FE) program purposely developed for long-term and collapse analysis of timber-concrete composite beams has been used to model the first part of the long-term tests. The purpose of the numerical modelling was to calibrate the program on the experimental tests, which were performed over a limited period of 28 days, so as at a later stage to extend the results to the end of the service life (50 years) and to composite beams with different mechanical and geometrical properties. The uniaxial FE model is made from two parallel beams, the concrete slab and the timber beam, connected at their interface with a continuous spring system which models the connection and account for its flexibility (see Figure 5).

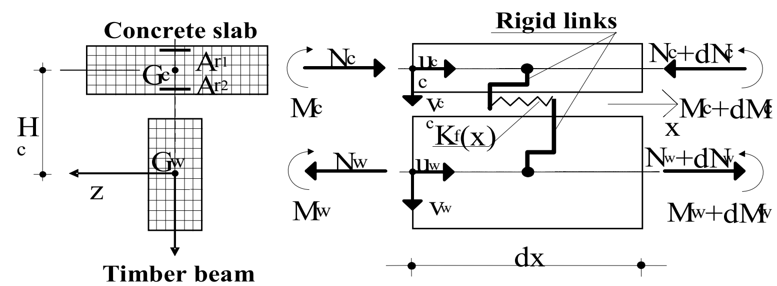

Figure 5 : Cross-section (left) and elevation (right) of the uniaxial FE model used in the numerical analyses

The materials can be modelled with their time-dependent behaviour for long-term analyses under constant sustained load. Concrete can be considered as a viscoelastic material in compression and in tension before cracking. Timber can be modelled as a hydro-viscoelastic material, where creep, mechano-sorption, shrinkage/swelling due to temperature and relative humidity variation of the environment, and dependency of the Young's modulus on moisture content can be taken into account using the Toratti's rheological model[10]. Creep and mechano-sorption can also be accounted for in the connection system. Further details on the model can be found in literature[11].

The shear force-relative slip relationship obtained from push-out tests and fitted with a power-type function was inputted at the connection locations in the FE model. The concrete cross-section was divided into 20 layers, while the timber cross-section was divided into 80 horizontal layers and 20 vertical columns. The mechanical properties of timber (E= 10.7 GPa) and concrete (E= 33 GPa, f_{cm}= 46 MPa, f_{ctm}= 3.4MPa) as measured from experimental tests or provided by the manufacturer were used. The actual relative humidity and temperature histories monitored during the tests were inputted to represent the environmental conditions.

5.2 Results and Discussions

Figure 6 reports the experimental-numerical comparisons in terms 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 relative humidity (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 relative humidity 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 day 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.

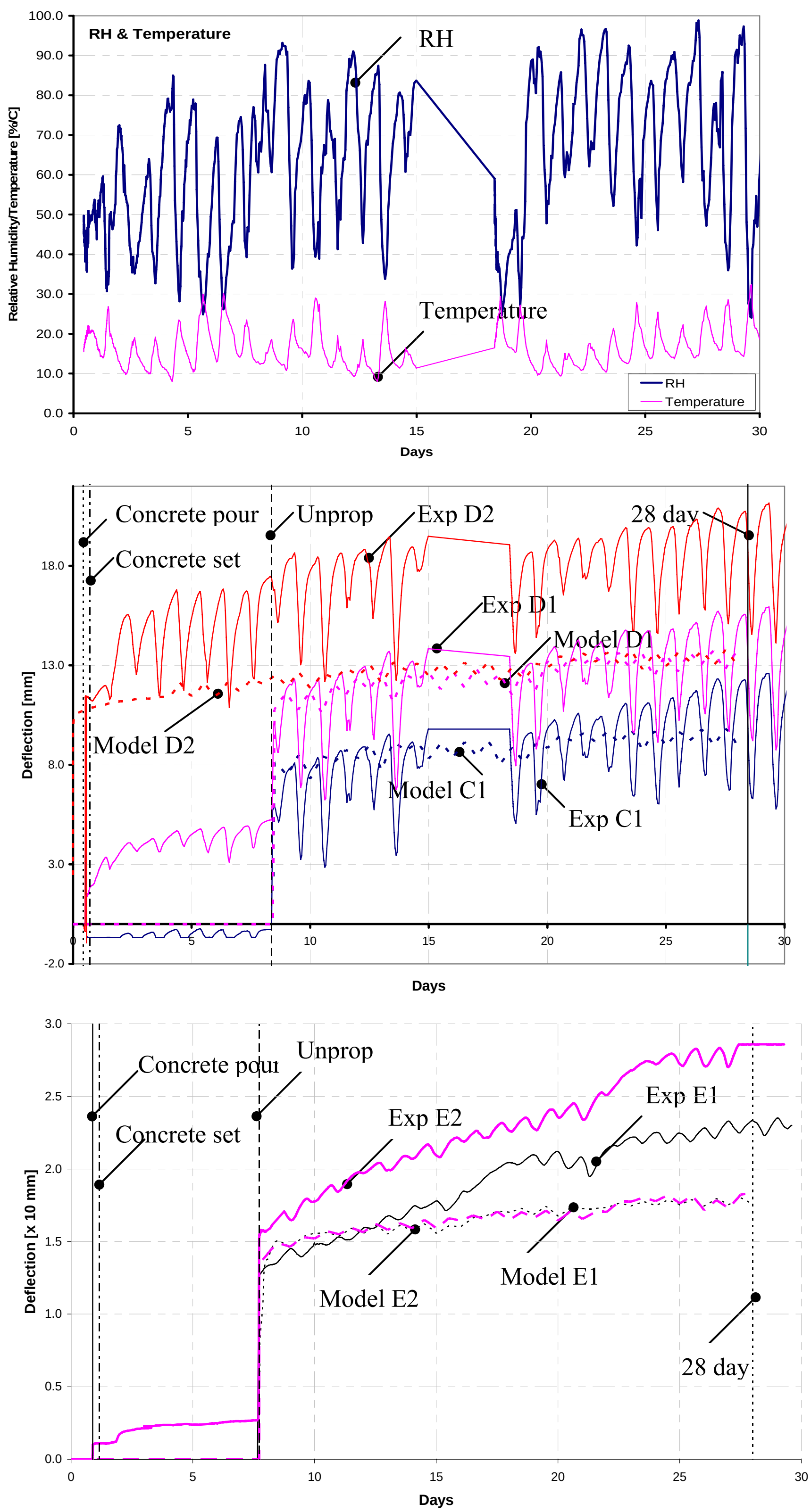

Figure 6 : Experimental-numerical deflection for(a) outdoor and (b) indoor beams with RH and temperature histories

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 6b 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 day 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 experimental-numerical comparisons show that the software can capture the experimental results with an overall good accuracy. In general, the deflection differences were less than 10 % for almost all specimens monitored over time. Based on these experimental validations, the software can be used to extend the experimental results to end of the service life (50 years) so as to control the deflection in the long-term, which could be critical for the design of long-span TCC beams.

6. Conclusions

The outcomes of two parts of the research involving push-out connection testings and first month short-term monitoring of TCC beams were presented in this paper. The most important factors affecting the connection performance were found to be the length of the notch and the presence of a coach screw. Based on the control of all possible failure mechanisms, analytical formulas for the shear strength of the notched connection were derived. The formulas were found to predict the experimental failure load within acceptable range in most cases. Primary observations from the first month TCC beams monitoring are: (1) Propping of beams at mid-span is crucial to minimise permanent deflection and enable the development of initial sufficient composite stiffness to sustain the full self-weight of the concrete slab; (2) Excessive shrinkage of concrete causes extra deflection, hence low shrinkage concrete is desirable in TCC to minimise any permanent deflection; (3) Extreme environmental fluctuations exert larger deflection variations in composite beams due to the different thermal expansion coefficients of timber and concrete; and (4) The peaks of deflection were consistent with the peaks of environmental relative humidity RH and with the minima of the environmental temperature.

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