Design of timber-concrete composite beams with notched connections

Massimo Fragiacomo[(1)] and David Yeoh[(2)]

[(1)] University of Sassari, Dept. of Architecture, Design and Urban Planning, Alghero, Italy

[(2)] Universiti Tun Hussein Onn Malaysia, Faculty of Civil and Environmental Engineering, Batu Pahat, Johor, Malaysia

1 Introduction on design of timber-concrete composite beam

Timber-concrete composite (TCC) structures must be designed so as to satisfy both serviceability (SLS) and ultimate limit states (ULS) in the short- and long-term (the end of the service life). The ULS is checked by comparing the maximum shear force in the connection, the maximum stress in concrete, and the combination of axial force and bending moment in timber with the corresponding resisting design values. The most important serviceability verification is the control of maximum deflection, which is used also for an indirect verification of the susceptibility of the floor to vibration, as suggested by Australian/New Zealand Standard 1170 Part 0 [1].

Two problems have to be addressed when evaluating stress and deflection of a TCC beam: (1) the flexibility of connection, which leads to partial composite action and, in general, does not allow the use of the transformed section method in design; and (2) the time-dependent behaviour of all component materials, i.e. creep, mechano-sorption, shrinkage/swelling, thermal and moisture strains of timber and concrete, and creep and mechano-sorption of the connection system.

To account for the first problem, two approaches have been proposed: the linear-elastic method [2] and the elasto-plastic method [3]. The linear-elastic method is based on the assumption that all materials (concrete, timber and connection) remain within the linear elastic range until the first componenent (generally, either the timber beam or the connection) fails. This is appropriate in many cases of technical interest, particularly for TCC with very strong and stiff connectors such as notches cut in the timber and filled with concrete. A linear-elastic analysis is generally carried out for the short-term (instantaneous) verifications according to the approach suggested by Ceccotti [2], which is based on the use of the gamma method reccommended in the Annex B of the Eurocode 5 [4]. According to the gamma method, an effective bending stiffness, $(EI)_{ef}$, given by Eq. (1), is used to account for the flexibility of the timber-concrete shear connection. A reduction factor γ, which ranges from 0 for no composite action between the timber and concrete interlayers to 1 for fully composite action (and rigid connection), is used to evaluate the effective bending stiffness:

$$(EI)_{ef} = E_1 I_1 + E_2 I_2 + \gamma_1 E_1 A_1 a_1^2 + \gamma_2 E_2 A_2 a_2^2 \tag{1}$$

where subscripts 1 and 2 refer to concrete and timber elements, respectively; E is the Young's modulus of the material; A and I are the area and the second moment of area of the element cross-section; a is the distance from the centroid of the element to the neutral axis of the composite section; and γ is the shear connection reduction factor. Using the effective bending stiffness, the maximum stresses in bending, tension and compression for both the timber and concrete elements, and the shear force in the connection can then be calculated [2]. In Eq. (1), γ_1 is calculated from Eq. (2) and γ_2 is taken as one:

$$\gamma_1 = \frac{1}{1+\dfrac{\pi^2 E_1 A_1 s_{ef}}{Kl^2}} \qquad \gamma_2 = 1 \qquad (2)\quad(3)$$

where s_{ef} is the effective spacing of the connectors assumed as smeared along the span of the floor beam; l is the span of the TCC floor beam; and K is the slip modulus of the connector. For verifications at ULS and SLS, different values of slip moduli, K_u and K_s, are used, defined by Eqs. (4) and (5), respectively. Such a difference between K_u and K_s arise from the shear force-relative slip relationship of the connection, which is generally non-linear [2,5]. These stiffness properties of connector are evaluated through experimental push-out shear test (Figure 1) carried out as recommended in EN 26891 [6]:

$$K_u = \frac{0.6F_m}{\nu_{0.6}} \qquad K_s = \frac{0.4F_m}{\nu_{0.4}} \qquad (4)\quad(5)$$

where F_m is the mean shear strength obtained from a push-out test, $\nu_{0.4}$ and $\nu_{0.6}$ are the slips at the concrete-timber interface under a load of 40% and 60% of the mean shear strength F_m, respectively. Figure 1 displays a typical experimental set-up of a push-out test, carried out at the University of Canterbury, New Zealand, to investigate the mechanical properties of notched connectors between LVL joists and concrete slabs [7].

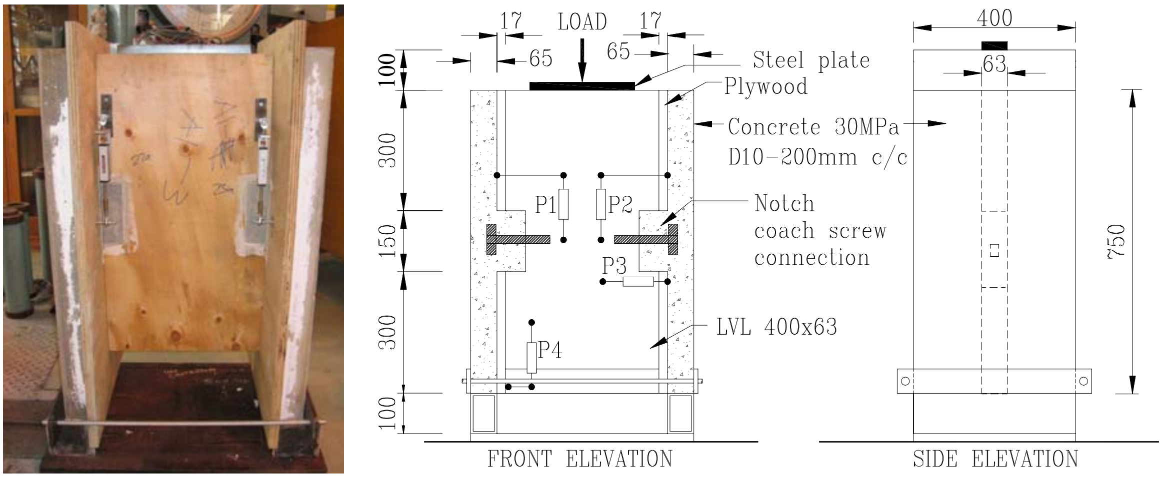

Figure 1: Symmetrical push-out test set-up (dimensions in mm)

The elasto-plastic solution [3] has been proposed specifically for cases where the failure of the TCC is attained after extensive plasticization of the connection system, so as to allow for redistribution of the shear force from the most stressed to the less stressed connectors along the beam. This is fairly common where the connectors are low strength, low stiffness and high ductility, such as for mechanical fasteners. The failure load is evaluated by assuming a rigid-perfectly plastic behaviour of the connection.

For verifications in the long-term, the ‘Effective Modulus Method’ recommended by Ceccotti [2] is used to account for the effect of creep of the different materials. The effective moduli of concrete, E_1, and timber, E_2; and slip modulus of connector, K in Eq. (1) are replaced with their respective effective moduli $E_{1,eff}$, $E_{2,eff}$ and K_{eff} given by:

$$E_{1,eff} = \frac{E_1}{1+\phi_1(t,t_0)} \qquad E_{2,eff} = \frac{E_2}{1+\phi_2(t-t_0)} \qquad K_{eff} = \frac{K}{1+\phi_f(t-t_0)} \qquad (6)\ (7)\ (8)$$

where $\phi_1(t,t_0)$, $\phi_2(t-t_0)$ and $\phi_f(t-t_0)$ are, respectively, the creep coefficient of concrete, timber, and connector, t and t_0 are, respectively, the final time of analysis (the end of the service life, usually 50 years) and the initial time of analysis (the time of application of the imposed load). A detailed description of the design of TCC at ultimate and serviceability limit states, with emphasis on the influence of creep in the long-term, including two worked examples, is provided in [8].

The approach discussed above neglects the effect of environmental strains caused by the different thermal expansion and shrinkage of concrete and timber on the internal forces and the deflection of TCC, resulting in an underestimation of the deflection at the end of service life. To resolve this issue, rigorous [9] and approximated [10] closed form solutions were derived to account for the effects of environmental strains and drying shrinkage of concrete on TCC. Such formulas were compared to each other [11] showing good accuracy, and then used to estimate the influence of different environmental conditions, type of exposure, and size of the timber cross-section on the design of TCC beams [12,13]. A significant influence on the design was found, particularly for TCC systems with solid timber decks and rigid connections, and for TCC floors with narrow timber joists exposed to outdoor, sheltered environmental conditions [13].

2 The notched connection

A wide range of connection systems have been developed in different parts of the world and throughout the century. The connectors can be metal or timber fasteners, or notches cut in the timber and filled by concrete. Based on their arrangement along the beam, the connectors can be categorized in discrete/continuous, and vertical/inclined. They can also be categorized in glued/non-glued, and prestressed/non-prestressed, based on the way they are inserted in the timber.

Notches cut in the timber beam and reinforced with a steel screw or dowel, as illustrated in Figure 2, is by far one of the best connection for TCC with respect to strength and stiffness performance although it may not be altogether economical if the notches had to be cut manually [14,15]. The relative slip between the concrete slab and the timber beam can be prevented by direct bearing of the concrete within the notch on the timber of the beam, leading to a strong and stiff connection. Different notch geometries (rectangular, triangular, inverted trapezoidal) have been used with different timber materials (sawn timber, glulam, and LVL), with or without reinforcement.

Inveretd trapezoidal notches cut in a timber deck made from sawn timber were tested at Colorado State University, US [16]. The notches were reinforced with a metal anchor which can be tightened after 28 days from the concrete placement to eliminate any gap within the notch due to drying shrinkage of concrete and restore the tight fit at the concrete-timber interface. Rectangular notches, with or without a reinforcement made from lag screws, were tested at the University of Stuttgart, Germany, on a composite system made from board stacks [14]. Rectangular, triangular and trapezoidal notches cut in LVL joists, with and without a lag screw reinforcement, were tested at the University of Canterbury, New Zealand [15,17,18]. The length of the notch, the presence of a lag screw and its depth of penetration into the timber, were found to be the most important factors affecting the performance of the connection. It was found that the notch length affects the strength and stiffness of the connection while the lag screw improves the post-peak behaviour.

3 Experimental evaluation of mechanical properties of notched connections

The shear strength of a notched connection is an important mechanical property for the design of TCC floors at ULS. Such a quantity can be evaluated by testing to failure small TCC blocks (push-out tests, see for example Figure 1). The outcomes of an extensive experimental programme carried out at the University of Canterbury on several LVL-concrete push-out specimens (9 per connection type) is summarized in Table 1. The tests were carried out on rectangular and triangular notched connections reinforced with a lag screw (see connection details in Figure 2). The concrete slab was 600 mm wide and 65 mm deep, and the concrete had an average compressive strength of 45 N/mm^2. The LVL joist was 63 mm wide and 400 mm deep, and had a bending strength of 48 N/mm^2.

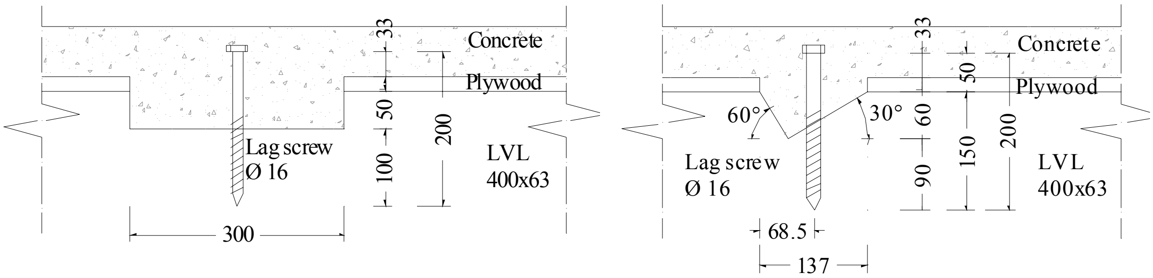

Figure 2: Details of the rectangular (left) and triangular (right) notched connections tested at the University of Canterbury (dimensions in mm) [7,18]

The outcomes of the tests as shear strength (mean value F_m and characteristic value F_k) and mean slip moduli for SLS and ULS verifications, K_s and K_u respectively, are listed in Table 1. More details on the experimental programme can be found in [7,18]. The quantity F_k is used for ULS control of the connection, whilst the quantities K_s and K_u are used in Eqs. (2) and (1) for evaluation of the effective flexural stiffness of the composite beam at SLS and ULS, respectively. From Table 1 it is fairly clear that there is no significant difference between K_s and K_u, hence only one value of the slip modulus could be used for both SLS and ULS verifications.

Table 1: Experimental values of the slip moduli and shear strength of rectangular and triangular notched connections, and analytical predictions of the mean shear strength.

Type of connection	K_s [kN/mm]	K_u [kN/mm]	F_k [kN]	F_m [kN]	F_m [kN]	F_m [kN]	F_m [kN]
	Experim.	Experim.	Experim.	Experim.	Analytic NZS	Analytic EC	Analytic EC*
Rectangular	247.2	241.4	115.3	138.9	186.4	99.1	140.3
Triangular	145.8	138.8	70.4	84.8	94.0	70.7	83.4

4 Analytical evaluation of the shear strength of notched connections

A simplified analytical model for strength evaluation of notch connections reinforced with lag screws is proposed in Eqs. (9) to (12). The formulas were compared with the experimental results and were found to predict the failure load with acceptable accuracy in most cases. The connection is regarded as a concrete corbel protruding into the laminated veneer lumber (LVL) joist subjected to the shear at the concrete-timber interface. The lag screw acts as reinforcement for the concrete corbel, and contributes to the shear transfer from timber to the concrete. The model is based on the control of all possible failure mechanisms that may occur in the connection region (see Figure 3) [19]: (1) failure of concrete in shear in the notch; (2) crushing 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 LVL beam; and (4) failure of LVL in crushing parallel to the grain at the interface with the concrete corbel. Analytical design formulas in accordance with New Zealand Standards and Eurocodes were derived. By comparing the outcomes from the different standards, it was found that the New Zealand Standards method overestimates the maximum shear strength, while the Eurocode method is quite conservative with the actual experimental results in between (see Table 1). An alternative approach based on the introduction of a reduction factor β^* to be used in the Eurocodes formulas was then derived and compared with the experimental results, showing the best accuracy.

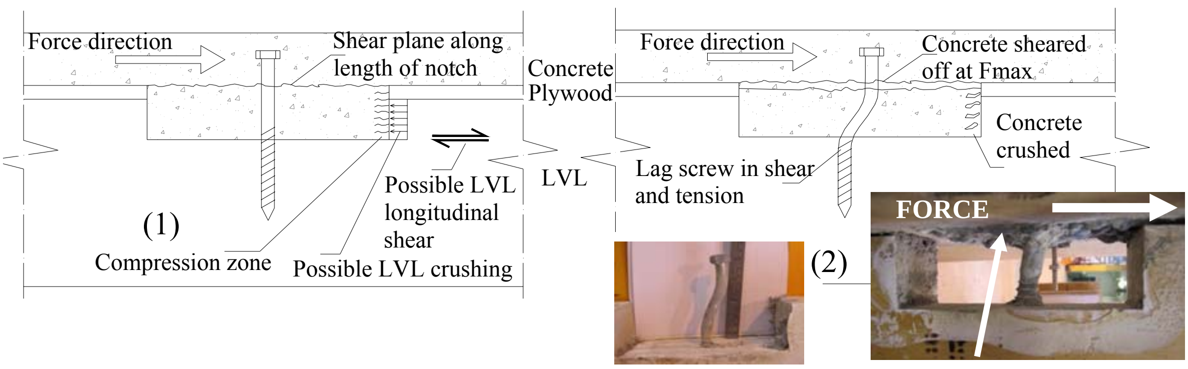

Figure 3: Experimental failure mechanisms and behaviour of a rectangular notch connection reinforced with a lag screw

4.1 Strength evaluation model according to New Zealand Standards (NZS method)

The corresponding formulas, reported herein after, were derived in accordance with provisions from New Zealand Standards for both timber [20] and concrete structures [21] based on the aforementioned four possible failure mechanisms of the notched connection:

$$F_{conc,shear} = 0.2 f_c^{'} bl + nk_1 pQ \qquad F_{conc,crush} = f_c^{'} A_c \qquad (9)\ (10)$$

$$F_{LVL,shear} = k_1 k_4 k_5 f_s Lb \qquad F_{LVL,crush} = k_1 f_c bd \qquad (11)\ (12)$$

where $F_{conc,shear}$ is the nominal shear strength of concrete for a notched connection reinforced with a lag screw, $F_{conc,crush}$ is the nominal compressive strength of concrete in the crushing zone, $F_{LVL,shear}$ is the nominal longitudinal shear strength of LVL between two consecutive notches or between the last notch and the end of the timber beam, and

$F_{LVL,crush}$ is the compressive strength of LVL in the crushing zone. $f_c^{'}$ is the compressive strength of concrete, b and l are the breadth of the LVL joist and the length of notch, respectively, n is the number of lag screws in the notch, k_1 is the modification factor for duration of loading for timber, p is the depth of penetration of lag screw in the timber, and Q is the withdrawal strength of the lag screw in Eq. (9). A_c is the crushing zone effective area, i.e. $b \times d$ in Eq. (10) where d is the depth of the notch. k_4 and k_5 are the modification factors for load sharing (taken as 1.0 for material with properties of low variability such as LVL), f_s is the LVL strength for longitudinal shear, and L is the shear effective length, i.e. the distance between two consecutive notches or between the last notch and the end of the timber beam in Eq. (11). f_c is the compressive strength of LVL parallel to the grain in Eq. (12). The design value of the shear strength is obtained by using the characteristic values of material strengths $f_c^{'}$, Q, f_s and f_c in Eqs. (9) to (12), and by multiplying the minimum among the four values of strength by the strength reduction factor ϕ.

4.2 Strength evaluation model according to Eurocodes (EC method)

Based on the Eurocodes for both timber [4] and concrete structures [22], the shear strength of concrete for a notched connection reinforced with a lag screw when modelled as a corbel can be calculated using the following equation:

$$F_{conc.shear} = \beta 0.5 b_n l_n v f_c + n_{ef} (\phi_{cs} d_{ef} \pi)^{0.8} f_w \tag{13}$$

where β is the reduction factor of the shear force for load applied in proximity of the support of the notch regarded as a corbel, which should be assumed as 0.25 in accordance with Eurocode 2 [22] for the case under study; b_n and l_n are the breadth of the joist and the length of the notch, respectively; v is a strength reduction factor for concrete cracked in shear, assumed as 0.516; f_c is the compressive strength of concrete; n_{ef} is the effective number of lag screws, assumed equal to the actual number of screws in the notch if they are spaced enough; ϕ_{cs} is the diameter of the lag screw, d_{ef} is the pointside penetration depth less one screw diameter; and f_w is the withdrawal strength of the screw perpendicular to the grain. The other three failure mechanisms are governed by design equations similar to Eqs. (10) to (12), the only difference being that the coefficients k_4 and k_5 are replaced by the modification factor for system effect k_{sys}, assumed 1.0 for LVL, and the coefficient k_1 is replaced by k_{mod} which denotes the modification factor for duration of load and moisture content. The design value of the shear strength is then obtained by using the design values of the material strengths f_{cd}, f_{wd}, etc., which are obtained by dividing the characteristic values by the material strength coefficients, γ_m, in the design equations, and by taking the minimum of the so obtained four values of design strengths.

4.3 Modified reduction factor method (EC* method)

A new reduction factor, β^*, given in Eq. (14), was introduced to replace the existing reduction factor, β, in Eq. (13) in order to account not only for the loading distance but also for the length of the notch, l_n, which was found to have a significant effect in the experimental tests, and the diameter of the lag screw, ϕ_{cs}.

$$\beta^* = \frac{l_n - 2\phi_{cs}}{2 l_n} \tag{14}$$

4.4 Experimental-analytical comparisons

Table 1 provides a comparison of the experimental mean shear strength for the rectangular and triangular notched connections with the three analytical strength evaluation methods discussed above. For all connector types, the governing design formula was found to be Eq. (9) and Eq. (13) for concrete shear, which agrees well with the failure mechanism detected in the experimental tests. The EC method was found to be the more conservative than the NZS method while the EC* method shows a prediction very close to the experimental outcomes in all of the cases.

5 Collapse tests of TCC beams with notched connections

The formulas discussed above allow a reasonably accurate prediction of the shear strength of a notched connection, which can then be used in ULS verifications. More complex is to derive analytical formulas for the mean slip modulus of a notched connection. Such a quantity is needed in Eqs. (1) and (2) to calculate the effective flexural stiffness of the composite beam and, then, all other quantities such as deflections, stresses, etc. needed for ULS and SLS verifications. So far, no accurate formula was proposed for the prediction of the slip modulus, therefore experimental testing is currently the only possible way to evaluate this quantity. For the rectangular and triangular notched connections tested in New Zealand, the values are reported in Table 1.

On the other hand, notched connections were found to be fairly stiff and, therefore, it may be interesting to investigate the possibility to use the transformed section method, which assume fully rigid connection between concrete and timber and, therefore, does not require the slip modulus, in the design of TCC beams with notched connections. To this aim, reference to an extensive experimental programme carried out on full-scale composite beams is made in this paper.

A semi-prefabricated LVL-concrete composite system was developed at the University of Canterbury, New Zealand, comprising of 2400 mm wide M-section panels built with laminated veneer lumber (LVL) beams acting as floor joists and a plywood interlayer as permanent formwork (Figure 4a) [7]. For the purpose of experimental tests to collapse, the M-section was reduced in width from 2400 mm to a T-section of 1200 mm and 600 mm (Figure 4b). Nine beam specimens of 8 and 10 m span were designed, built and tested to failure under four-point bending, with the purposes of measuring the flexural stiffness, identifying the failure mechanisms, and assess the load-carrying capacity. Three types of notched connectors were used to construct the composite beam specimens: (1) Rectangular notches 150 mm long and 25 mm deep reinforced with a lag screw (R150); (2) Rectangular notches 300 mm long and 50 mm deep reinforced with a lag screw (R300) – see Figure 2; (3) Triangular notches reinforced with a lag screw (T) – see Figure 2. All beams had one LVL joist and a 600 mm wide concrete slab, except beam G1 which had two LVL joists and a 1200 mm wide concrete slab.

The beams were designed at ULS and SLS using the gamma method for two design levels: well-designed and under-designed, depending on whether all design inequalities at ULS and SLS were satisfied or not. The most critical design criterion for the well-designed beams was deflection at SLS in the long-term, followed by shear strength of connection at ULS in the short- and long-term. In the under-designed beams, the demand of shear force in the most stressed connector was about 30% more than the design resistance at ULS in the short- and long-term. An imposed load Q of 3 kN/m^2 for office buildings and a total permanent load $G = G_1 + G_2$ of 3 kN/m^2, with G_1 and G_2 signifying the self-weight and the superimposed permanent load, assumed as 2 and 1 kN/m^2, respectively, were considered in the design. The purpose for the variations in the design level was to investigate the actual strength and composite action achievable by the beam specimens, to verify the accuracy of the analytical gamma method used in design, and to explore the possibility to disregard the connection flexibility and use the transformed section method. The details of the beam tested are summarized in Table 2. More information can be found in [7].

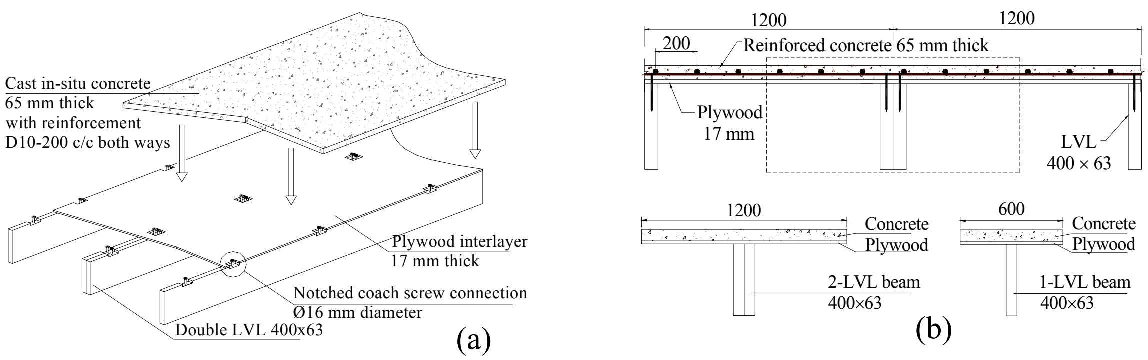

Figure 4: (a) Semi-prefabricated panels; (b) Reduced T-section (dimensions in mm)

Table 2: Details of the beam specimens, failure loads, and experimental (Exp), analytical (gamma method - Anal) and fully rigid (FuC) mid-span deflection at SLS load level (No. conn.=Number of connectors, R150=rectangular notch 150 mm long and 25 mm deep, R300=rectangular notch 300 mm long and 50 mm deep, T=triangular notch)

Beam	Span length [m]	No. conn.	Conn. type	Design level	Failure load $2P_{max}$ [kN]	Deflection Δ at SLS load level [mm]				
						FuC	Exp.	Anal	Exp/ FuC	Exp/ Anal
A1	8	6	R150	Under	87.3	15.6	22.7	17.5	1.45	1.30
A2	8	6	R150	Under	75.3	13.5	18.0	15.1	1.34	1.19
B1	8	10	R150	Well	105.0	24.3	26.5	26.1	1.09	1.02
B2	8	10	R150	Well	97.5	24.3	27.1	26.1	1.12	1.04
C1	8	10	T	Well	89.7	20.7	23.9	22.1	1.15	1.08
C2	8	10	T	Well	110.0	25.4	28.8	27.1	1.13	1.06
D1	8	6	R300	Well	80.8	18.7	21.1	19.7	1.13	1.07
E1	10	6	R300	Under	79.6	27.8	27.8	28.9	1.00	0.96
G1	8	10	R150	Well	201.0	23.2	25.9	25.5	1.12	1.02

Two types of failure mechanisms were observed: (1) fracture in tension of LVL under loading points at one-third of the span (Figure a) with no apparent sign of failure in connections, for well-designed beams; and (2) for under-designed beams, failure of connection in shear and/or crushing of concrete with plasticization of the lag screw in the case of notched connections (Figure b). The failure pattern of notched connectors was similar to that detected in push-out tests [7] where concrete strength was found to significantly influence the shear strength of the connection and, therefore, the load-carrying capacity of the composite beam. In most cases, the first crack sound was heard at approximately 60% of the collapse load F_{max} indicating the start of connection yielding which was followed by further plasticization as the screeching sound became louder. The failure hierarchy observed for under-designed beams was as follows: (1) crack sound in one or multiple connections as an early warning; (2) failure of the first connector, usually near the support; (3) consecutive failures of the other connectors moving towards the middle of the beam due to redistribution of the shear force; (4) when all connectors have failed, the load is resisted only by the LVL beams and final fracture of LVL in tension.

Figure 5: Different types of failure mechanisms detected in the composite beams: (a) fracture in tension of LVL; (b) failure for concrete shear and crushing in 300 mm rectangular notched connection

6 Results and discussion

Analytical-experimental comparisons of load-carrying capacity at ULS and SLS in the short-term in terms of imposed load for tested TCC beams and fully composite beams were performed. The analytical design imposed load in kN/m^2 was predicted such that all the ULS and SLS short-term inequalities were satisfied using the gamma method with connection slip moduli K_u and K_s, respectively, where concrete, LVL and connection strength design values were used. For under-designed beams, the connection strength inequality was governing followed by deflection in the short- or the long-term. The design of well-designed beams was governed by either deflection in the short- or long-term [7].

In the ULS comparison, it was found that all well-designed beams exhibited an experimental load-carrying capacity very close to that of a fully composite beam with rigid connection (approximately 0.9 times). This can be clearly appreciated from Figure 6, which displays a typical load-deflection curve of a well-design beam (in this case, specimen B2) and compare such a curve with the cases of fully composite (rigid connection) and non-composite (no connection) beam. In the SLS comparison, the analytical prediction underestimated the experimental imposed load by about 10%. This indicated that the gamma method provided an accurate and conservative prediction of the imposed load at SLS. Furthermore, the experimental load-carrying capacities of well-designed beams were only 10% less than that of fully composite beams implying that these beams have relatively high degree of composite action (87 to 100%) which was quantified according to Eq. (15):

$$DCA = \frac{\Delta_N - \Delta_F}{\Delta_N - \Delta_R} \times 100 \tag{15}$$

where Δ_N, calculated theoretically, signifies the deflection of the composite beam with no connection (lower limit); Δ_R, calculated theoretically, signifies the deflection of the composite beam with fully rigid connection (upper limit); and Δ_F, measured experimentally, signifies the deflection of the composite beam with the actual flexible connection. This further indicates that the transformed section method can be used with some correction factors to design composite beams with notched connections such as those investigated in this study characterized by a high degree of composite action.

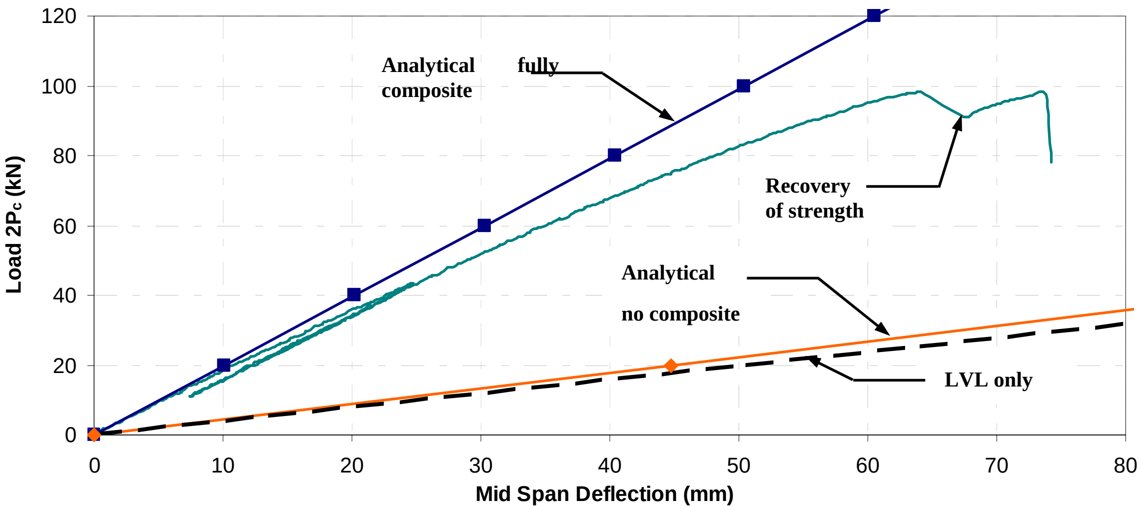

Figure 6: Typical experimental load $2P_c$ vs. midspan deflection curve for well-designed beams (specimen B2)

In an attempt to quantify this correction factor for design, the fully composite beam deflections (FuC) at SLS load level were compared, as presented in Table 2, with the experimental (Exp.) deflections, and with the analytical (Anal) deflections determined using the gamma method with the connection slip modulus K_s experimentally measured in push-out tests on connections. For the well-designed beams, the experimental deflection was 1.09 to 1.15 times the fully composite deflection, and 1.02 to 1.08 times the analytical deflection. Taking a conservative approach, this finding is indicative of a 15% increment correction factor to the deflection or, equivalently, a 13% reduction to the flexural stiffness (*EI*) calculated using the transformed section method (Eq. (1) with γ_1=1).

The method of the transformed section can therefore be used in design of TCC beams with notched connections. For evaluation of deflection at SLS, the flexural stiffness (*EI*) should be conservatively reduced by 13%. For connection design at ULS, it is suggested that no reduction in the flexural stiffness calculated with the transformed section be made, so as to overestimate the demand of shear force in the connection and carry out a conservative design. The connection strength capacity can then be calculated using the analytical formulas proposed in this paper. For timber and concrete design at ULS, the use of the flexural stiffness calculated with the transformed section method may be non conservative, therefore the use of the 13% reduction factor is recommended. It should be noted, however, that these ULS verifications are usually less critical than the ULS of connection and SLS of deflection. Hence, any possible approximation on the correction factor is less critical for such ULS verifications.

7 Conclusions and implications for future code developments

This paper discusses the design of timber-concrete composite beams with notched connections. Analytical formulas for the prediction of the shear resistance of notched connectors were derived, based on four possible failure mechanisms: (i) shearing of the concrete within the notch, (ii) compression of the concrete within the notch, (iii) shearing of the timber parallel to grain between two consecutive notches or from the first notch to the end of the beam, and (iv) crushing of the timber parallel to the grain at the interface with the concrete. The formulas, derived according to the New Zealand Standards and the Eurocodes, were validated against the results of an extensive experimental programme which involved several push-out specimens to failure carried out on small LVL-concrete composite blocks at the University of Canterbury, New Zealand. Such formulas can therefore be proposed in the new versions of the aforementioned regulations.

Based on an extensive experimental programme carried out at the University of Canterbury which involved tests to failure of full-scale LVL-concrete composite beams with notched connections, it was found only a minor difference between the experimental deflection at serviceability limit state and the analytical value calculated using the transformed section method, i.e. by neglecting the flexibility of the connection. This suggests a possible simplified procedure for design of timber-concrete composite beams with notched connections, i.e.: (1) calculation of the flexural stiffness (*EI*) of the composite section using the transformed section method; (2) calculation of the shear strength demand of the notched connection at ultimate limit using the flexural stiffness (*EI*); (3) comparison of the shear strength demand with the strength capacity of the notched connection evaluated using the proposed analytical formulas; (4) evaluation of the deflection at serviceability limit state by reducing the flexural stiffness (*EI*) by 13%; (5) calculation of the stresses in concrete and timber at ultimate limit state by reducing the flexural stiffness (*EI*) by 13%.

Although the aforementioned procedure was derived and validated on a particular type of composite floor made from LVL joists with rectangular and triangular notched connections, the procedure is general and can be applied to any type of composite structure with notched connection. Further analytical-experimental comparisons are however warranted, in particular to check the accuracy of the 13% reduction factor of the flexural stiffness used together with the method of the transformed section for different types of composite floors (for example, with solid deck).

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