Reverse Shear Transfer Within Beech LVL-Concrete Composites Via Singly Inclined Coach Screw Connectors

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ABSTRACT

Double-shear tests are reported on beech LVL-concrete composite connections based on coach screw connectors singly inclined at either 45° or 90°. On different specimens with the same screw orientation the longitudinal shear force was applied either in forward or reverse, because in practice concrete shrinkage, moisture-induced timber expansion and oscillatory (e.g. seismic) or moving loads can induce reversal of the force on the connection. The test data show that relative to the 90° screw connections, sloping the screws to 45° in tension only marginally affected longitudinal shear strength but led to a five-fold increase of slip modulus and to a significant drop in ductility, while sloping the screws the other way to 45° in compression only marginally affected slip modulus but led to an almost four-fold drop in longitudinal shear strength and to a substantial increase in ductility. The specimens tested within each group showed good consistency of shear strength and (except the 45° tension screw specimens, despite their consistent strengths) of failure mode, but high variability of slip modulus. Comparisons with previously tested timber-concrete composite (TCC) connections based on other screw types and layouts suggest good performance of the present connections. The gamma method applied to a given TCC T-section under load shows that the present alternate connections lead to quite different depths of uncracked concrete and so to significant variation of midspan deflection. In closing, it is recommended that both forward and reverse shear testing becomes a protocol for singly inclined coach screw-based TCC connections.

KEYWORDS : timber-concrete composites, connections, LVL, testing, slip modulus, ductility.

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1. INTRODUCTION

1.1 Literature Review

The slip modulus, longitudinal shear strength and ductility of timber-concrete connections often strongly influence the deflections, vibration characteristics, load capacity and stress redistribution capabilities of timber-concrete composite (TCC) floors, Fig. 1. It is thus important to quantify these connection properties. To that end multiple studies [1-5] show that, by comparison with test data, existing standards predict these properties to quite variable degrees of accuracy. Thus, into the foreseeable future, testing will be the primary means of characterising connection behaviour.

TCC connections based on mechanical connectors have been extensively characterised in experimental studies to date. These include nails or nail-plates [1, 5-7], vertical or inclined screws [6, 8-12], dowels [4, 13, 14], perforated steel plates or steel meshes bonded into the timber [3, 15-17], and notches [3, 6, 18-22]. The tests have considered both normal and lightweight concretes, short-term and long-term behaviours, and the presence or absence of a formwork interlayer. In one study [21], rectangular notched connections reinforced with coach screws were found to outperform toothed metal plate connections under fatigue loading for application to road bridges. Other forms of mechanical connector include a "flowering" steel cylinder installed using a cartridge-operated gun [23], others which performed as well with prefabricated slabs as with cast in-situ slabs [24], and a hybrid cylindrical connector comprising an ultra-high performance fibre reinforced concrete (UHPFRC) annulus surrounding a solid circular steel core [25].

Adhesive (hence chemical) connections have also been investigated [26-30] as a means to avoid drilling into the timber, and to improve TCC connection (and so member) stiffness by reducing interface slip. Both dry slab-on-wet adhesive and wet concrete-on-wet adhesive options have been studied. Key fabrication variables (e.g. the pouring height for the concrete in the wet-on-wet option), connection performance under combined hygrothermal and mechanical loads, and long-term behaviour (up to 4.5 years) have all been experimentally verified in these studies.

A review [31] of relevant experimental research concluded that adhesive connections and connections comprising long notches with dowels are of superior slip stiffness and strength but are also brittle, while metal plate connectors and connections comprising short notches with dowels give excellent stiffness, excellent strength and moderate ductility. Finally, the review highlighted that dowel type connectors can combine good stiffness and strength with excellent ductility.

Each of the key structural properties of TCC connections has come under scrutiny in targeted studies. For example, longitudinal shear strength was found [14] to be palpably higher for the concrete-timber-concrete double shear test configuration than for the timber-concrete-timber alternative. Moreover, high ductility of the connections was shown [32, 33] to lead to user-friendly analysis approaches for TCC members at ultimate, and also in one case [34] to enable up to 87% and 144% increases in ultimate load and midspan deflection, respectively. Finally, the slip modulus deduced from testing of TCC connections was found [35] to need a 60% reduction in cracked concrete zones of a FE model, to enable reliable prediction of deflection for the associated TCC member.

1.2 Aims and Objectives of the Present Study

The aim of the study reported in this paper was to characterise the quasi-static behaviours of TCC connections based on beech LVL, a recent advance in engineered timber. The objectives were to :

- Understand the fabrication issues arising from driving large diameter coach screw connectors parallel to each other into beech (a hardwood) LVL, so that all screws are at a single inclination.
- Experimentally determine, separately under forward and reverse longitudinal shear, the constitutive behaviours of the TCC connections based on these singly inclined coach screws.

This study builds significantly on the above discussed and other experimental investigations into TCC connections, for the following key reasons, namely :

- Hardwood LVL is a recent innovation which represents engineered timber with the best possible structural timber properties, best consistency of those properties and highest efficiency (double that of glulam) of use of original wood. Indeed in all three senses LVL is superior to glulam, owing to the circumferential peeling of the very thin LVL layers off the original log, giving little waste and minimising the influence of defects on overall section properties, as opposed to the longitudinal machining of the same log to give the much thicker, more defect-sensitive layers and more waste for glulam. Given this recency of hardwood LVL, it is not surprising that only very few studies, including two [36, 37] on beech LVL-concrete composite slabs and one [38] on beech LVL-concrete composite beams, have been reported using this advance in engineered timber. Hence the study presented in this paper advances previous work because that earlier work by necessity could only employ hardwood or softwood original logs or glulam, or softwood LVL, but not hardwood LVL. Given the high and consistent strength of beech LVL, the use of large section coach screws is highly advisable, in order to draw out as much capacity as possible from this connection.

- Coach screws are installation-friendly fasteners which lead to connections with good structural properties. Now if the dominant design load for any given TCC floor is a UDL, it is prudent to use these screws in a singly inclined layout symmetrically about midspan, with the screw-heads pointing towards the supports, because that layout endows the resulting connections with high slip stiffness and strength. It must then be recognised that in service, the connections may well experience longitudinal shear reversal due to shrinkage of the concrete (if it is cast wet around the connectors), also due to moisture-induced expansion of the timber, possibly due to oscillatory seismic loading or to lorries moving along a TCC bridge deck, etc. Given the asymmetry of this connection about the vertical, such reversal induces a different constitutive behaviour of the connection. For that reason, it is strongly advisable to test singly inclined screw TCC connections alternately under longitudinal shear in opposite directions. This has influenced the testing regime adopted in the present study.

In what follows the hardwood LVL-concrete connection specimens and the reverse shear tests to which they were subjected are described, the resulting constitutive behaviours are quantified using the recorded test data, and the connections are compared. The paper then closes with some suggestions for further work.

2. TEST SPECIMENS

2.1 Local Details

Twelve double-shear beech LVL-concrete connection specimens, each with a central LVL stub flanked by concrete slabs, were fabricated and tested in this study. All connection specimens included the following features, namely :

- 400 mm deep x 200 mm wide sections of GL70 Baubuche beech LVL joists.
- M12 coach screws of 180 mm length.
- Concrete of a specified C32/40 nominal grade although, as seen later, the actual concrete was stronger.

Within the twelve specimens three screw inclination angles to the slab-joist interface were employed, namely 90^{o}, 45^{o} with the screws in tension and 45^{o} with the screws in compression. For each inclination angle, four specimens were used to check the extent of repeatability of results. Henceforth, the specimens are identified according to inclination angle and to the nature of the axial force (Tension, Compression) in the screw as set out in Table 1.

Fig. 2, 3 and 4 show details of the C90, C45T and C45C specimens. Some points to note are :

- In all specimens, four screws were driven into the LVL stub at 120 mm spacing along a single line on each side, and in a plane normal to that of the lamellae and the wood fibres.
- In practice prefabricated slabs may be used with grouted connections directly over the screws. In order to strike a balance between these practical details and what is realisable in a laboratory environment, it was decided to employ freshly cast slabs in these specimens, with the method explained in the next bullet point used to allow for the above reality.
- In order to try and represent possible boundaries between precast slabs and grouted connections in practice, 15 mm x 15 mm section timber battens, nailed to the LVL stub either side of the line of screws, were used along the slab-joist interface of each connection.
- Slab width and thickness were constant at 600 mm and 95 mm across specimens, but slab height changed from 620 mm for C90 and C45C specimens to 740 mm for C45T specimens. This reflected the need for adequate slab volumes to surround the screws in the specimens.
- U-shaped steel bars looped transversely around the screws alternately from both sides of the slab, reflecting a detail to anchor precast slabs to the screws alternately from both sides.

The combination of a hardwood and a minimum embedment length for the screws meant that the possibility of pullout of the screws from the joists was minimised. It was also important to ensure adequate anchorage of the heads of the screws well within the concrete, near the tops of the slabs and as far as possible from the slab-joist interface. The choice of a hardwood meant that both the modulus and the relevant strengths (in bending, compression and tension) of the timber species exceeded (by over 30% on modulus) those of the commonly used glulam softwoods such as spruce.

2.2 Fabrication of Connection Specimens

First, the M12 coach screws were driven (by way of pre-drilled holes) into both sides of the twelve LVL stubs, but without the softwood timber battens included at that stage. Once this was completed, each screw of each specimen was held and shaken to check that it was not loose and then gently tapped with a hammer for further confirmation.

Subsequently, waterproof plywood formwork was fabricated on both sides of each stub, so that concrete slabs could be cast around the screws projecting from both these sides. This formwork was developed so that it could be rotated to enable casting of the second slab, once the first slab had sufficiently hardened soon after its own casting. Fig. 5 shows the upper face of this rotating formwork for example stubs of each screw inclination. The by then-installed softwood timber battens (strips) are also evident in the images. Note also the concrete chairs used to locate the rebar grids at the correct heights within the formwork. This rotating formwork concept was previously used to good effect in an earlier TCC project [39].

Poker vibrators were used to facilitate casting of ready-mixed concrete into the top voids of these formworks around the screws. Fig. 6 shows the trowelled surfaces of some of the first freshly cast specimens. The concrete in these upper voids was given 3 days to harden, after which every item of formwork was inverted (using lab cranes) for ready-mix casting into the reverse void. During both sets of casts, many concrete cube and cylinder samples were taken. All freshly cast specimens were covered with damp hessian over-laid by polythene sheeting and left to cure for at least 4 weeks before testing of the connections occurred.

In preparation for testing, striking of the formwork revealed high quality concrete with a good surface finish on the slabs of all specimens. This for example can be seen for the two slabs of the specimen shown in Fig. 7. No hairline cracks were observed on any of the cured slabs prior to testing, suggesting that shrinkage effects were probably minimal.

3.0 MATERIAL PROPERTIES

Manufacturer-supplied properties for the beech LVL material are provided in Table 2. From crushing and split tests performed on the cubes and cylinders, the individual, average and CoV compressive and tensile strengths of Table 3 were obtained. Note the high consistency of results (low CoV) and the high-strength concrete (compressive 51.9 N/mm^2) achieved.

4.0 TESTING STRATEGY

Double shear testing of specimens was performed in a 500 kN capacity DARTEC machine. Fig. 7 shows a specimen under test. Roller and pin supports were used under the slabs. Load was applied onto the top section (horizontal) face of the LVL stub halfway between the slabs, using a steel plate and thin rubber pad to spread the load onto that top section. At mid-height of each set of four screws (hidden in Fig. 7), slip was measured at the slab-joist interfaces using calibrated potentiometers. In Fig. 7 the right potentiometer facing the viewer is labelled POT, the other on the left is also visible. Each POT was fixed at one end to an L-shaped aluminium plate screwed into the timber, and at the other end to another L-shaped plate glued to the concrete and oriented at right angles to its counterpart on the timber. The plates can be seen in Fig. 7. There were two other POTs at the same elevation on the reverse face of the joist. With two POTs at each joist-slab interface, the mean of two slips was used for each group of four screws.

The loading regime for each connection was taken from EN 26891 : 1991 [40]. Broadly, this entailed first loading up to 40% of the failure load, dwelling, then unloading down to 10% of the failure load, dwelling, and finally using load control up to 70% of the failure load, switching to displacement control above 70% of that failure load. Slips and loads were recorded by a data logger at 10 Hz throughout each test. Photos were taken during and after each test to carefully document the failure modes for all connection specimens.

5.0 TEST RESULTS

5.1 Slip Moduli and Longitudinal Shear Force Capacities

For each specimen, Table 4 summarises the longitudinal shear force capacities, failure modes and slip moduli at 40% failure load per four screw connector group. Two alternative definitions of slip modulus have been used, both based on advice in EN 26891 : 1991 [40]. Taking $\nu_{0.4}$ and $\nu_{0.1}$ as the slips at 40% and 10% respectively of the peak force, F_{max} , these slips are [40] :

$$k_i = 0.4F_{max} / \nu_{0.4} \quad \text{and} \quad k_s = 0.4F_{max} / (4[\nu_{0.4} - \nu_{0.1}]/3) \qquad (1)$$

These definitions render k_i the true secant modulus and k_s a modified secant modulus at $0.4F_{max}$. Note that F_{max} , the force on the four screws at either slab-joist interface, was half the peak force applied by the machine. Also, strictly speaking, EN 26891 : 1991 [40] defines F_{max} as either the peak force or the force at 15 mm slip. Now as will be seen later in this paper, the peak force occurred before 15 mm slip for the C90 and C45T specimens, justifying the use of the recorded peak force as the value of F_{max} for these specimens. Meanwhile for the C90 specimens, beyond 15 mm slip there was only a 10% further increase of force to failure. Moreover, this failure occurred while the force - slip characteristic was still on a distinctly upward trajectory, suggesting good structural integrity of the connection in that latter lead-up to failure. For these reasons F_{max} was again taken as the (this time post-15 mm slip) peak recorded force.

The naming of specimens in Table 4 follows that defined in Table 1, with the addition of "-*n*-A , B" to indicate both slab-joist interfaces A and B for specimen number *n* of the group concerned. Hence C90-3-A , B refers to both interfaces of the third specimen within the connection series with screws inclined at 90^{o}, while C45T-1-A , B refers to both interfaces of the first specimen within the connection series with tension screws inclined at 45^{o}.

The following key points emerge from Table 4, namely :

- The C90 specimens exhibited 169.6 kN longitudinal shear force capacity, over 3.5 times that of the C45C specimens (48 kN), but almost 90% that (194.1 kN) of the C45T specimens. Also, there was high consistency of shear strength between connections in any given group, evident from the low CoV values, all below 10%.
- By contrast, regarding k_i, the C90 specimens exhibited only 20% the slip stiffness of the C45T specimens and only 17% more slip stiffness than did the C45C specimens. Indeed, the C45T specimens were particularly stiff, at 494.9 kN/mm. Moreover, within each four-group of connection specimens, there was quite a palpable variation of slip stiffness, evident from the high CoV values in Table 4 ranging from 41% to 47%.
- The high consistency on strength might be due to the good quality control on the screws' yield stresses, which observations suggest might have governed the failures. The lesser consistency on slip modulus was also noted in a previous study [11] which used LVL based on another species of timber, and within which the variability was attributed to construction method, inconsistency in material properties and the overall nature of TCC connections.

5.2 Constitutive Characteristics

Fig. 8, 9 and 10 show the longitudinal shear force vs slip characteristics for the C90, C45C and C45T specimens respectively. On each set of axes there are eight plots, one for each of the two interfaces on each of the four specimens. The plots are labelled using the system of nomenclature defined in Table 1 and further described in the immediately preceding section.

The main points to emerge from these plots include the following, namely :

- While the C45C connections were of low strength, they all showed significant ductility in the form of distinct plateaux (exceeding a 10 mm slip range) on the force-slip characteristic.
- The C45T connections also exhibited distinct nonlinearity above 40% of their failure loads, but the associated ductility is variable. For example C45T-2 and C45T-3 show short ductility plateaux up to 1 - 2 mm beyond attainment of peak load, while the C45T-1 and C45T-4 curves show abrupt load drops (and so low ductility) while still on an upward trajectory, even though the failure loads are quite similar to those for C45T-2 and C45T-3.
- The C90 plots closely approximate bi-linear characteristics with the slopes of the second lines well below than the slopes of the first lines (but with these second slopes still not low enough to approximate ductility plateaux), with slope changes at 45% of the failure loads.
- Note the first unloading at 40% of the failure load. The secant slip modulus k_i described earlier is the gradient of the line from the origin to that point. The curves for the individual interfaces diverge from each other in that regime, which explains the variable slip stiffness between the four specimens of any one screw inclination as discussed earlier re Table 4.

Fig. 11 summarises the results by showing how the connections' normalised slip modulus (k_i) and, separately, their normalised longitudinal shear strengths varied with screw angle. For both these variables, normalisation has been done with respect to the value for C45C, since that connection was the most flexible and the weakest of the three. Fig. 11 immediately shows that, relative to the 90^o screw connections, sloping the screws to 45^o in tension had only a minor increasing effect on strength but quintupled the slip modulus, while sloping the screws to 45^o in compression had little effect on slip modulus but led to an almost fourfold drop in strength. Moreover, Figs 8 – 10 clearly show the progressive departure from the classic ductility plateau in the move from C45C through C90 to C45T.

5.3 Failure Modes

As stated in Table 4 and shown in Fig. 12, each C90 specimen failed at <u>one</u> side of the stub by fracture of all four screws there, owing to excessive yield of these screws just into the timber.

Fig. 12 show that the fractured screws experienced significant plasticity in flexure over short lengths protruding from the slab. Note the timber battens embedded in the slab. That the remaining bits of the screws left behind in the joist cannot be seen through the holes in the joist's surface in Fig. 12 is testament to the occurrence of fracture some distance into the timber.

As Table 4 and Fig. 13 show, the C45C specimens also failed consistently by fracture of the screws a short distance into the timber, accompanied by some crushing of the concrete near the screws, due to bearing of the screws on the slab. This time a fractured screw specimen can just be seen embedded in the portion of joist focused on in Fig. 13.

Unlike the consistency of failure mode observed for the above two sets of specimens, the C45T set exhibited changes of failure mode between specimens, although the failure load was consistent between specimens. Table 4 and Fig. 14 show that the C45T-1 failed by splitting of the LVL stub along one of its lamellae. C45T-2 and C45T-3 failed as did the other connection groups by fracture of the screws due to excessive plasticity just into the timber. Finally, Fig. 14 shows that C45T-4 failed by pullout of the screws from the slab at one slab-joist interface, the screws remaining intact and the immediately surrounding concrete remaining attached around these screws between the softwood battens but fractured from the rest of the slab. A branched splitting crack was further observed on the outside of that slab, see Fig. 14. Hence while this connection type possessed the highest longitudinal shear force capacity and by far the highest slip modulus, it also presented the most variability in failure mode.

6.0 FURTHER DISCUSSION OF RESULTS

6.1 Comparisons With Previously Tested Screw-Based TCC Connections

Blass et al. [6] reported the results of longitudinal shear tests on different TCC connections, including several based on self-tapping screws laid in X formation at ±45°. Fig. 15(a) shows the general layout of these specimens. Key features of the tests are as follows, namely :

- The partially threaded screw shown in Fig. 15(b) and spruce stub joists were used.
- Some specimens contained two X pairs of screws as shown, while others contained only one.
- Three interlayer thicknesses, namely 0 mm, 19 mm and 28 mm, were used in different specimens.
- The 70 mm slab thickness and 210 mm overall depth were constant across specimens, which means that the stub joist depth was reduced as the interlayer thickness increased.

Table 5 compares the relevant properties of the beech LVL-concrete connections from the present study with those tested in the above earlier study. Note that on the slip modulus k_s is used, for consistency with the results presented in [6]. Results from the earlier study [6] are given per single X screw connection (multiplying the test result by 0.5 when the actual specimen entailed a double X layout), while for the present specimens the results are given for a single screw connection (multiplying the test results by 0.25 given the presence of four screws in each actual specimen). Note from Table 5 the average concrete cube strength of 33.5 N/mm^2 for the earlier specimens against 51.9 N/mm^2 for the present specimens, along with the use of softwood in the earlier tests against hardwood in the present tests, and finally two 7.3 mm diameter screws in X formation for the earlier tests against a single M12 screw in the presently reported work. Bearing these differences in mind, the following points emerge from a scrutiny of Table 5, namely :

- In the earlier specimens with no interlayer, the move from one X (A-SCH specimens) to two X (B-SCH specimens) screw layouts had little effect on the connection properties per single X.
- Still on the earlier specimens, the move from no interlayer (A/B-SCH specimens) to an interlayer (C/D-SCH specimens) caused slip modulus to drop by about a half and longitudinal shear strength to drop by about a third.
- The present specimens with no interlayer and a single M12 screw in C45C or C90 configuration were of similar slip moduli to the earlier specimens with an interlayer and two 7.3 mm diameter screws in X layout. However while the present C45C specimen was of comparable strength to the earlier specimens with an interlayer, the present C90 specimen was of considerably higher strength, exceeding that even of the earlier specimens with no interlayer by a factor of 2.
- The present C45T specimens exceed even the earlier specimens with no interlayer by a factor of almost four on slip modulus and approximately two on strength. The high increase in slip modulus in particular is very likely a combination of the change from softwood to hardwood, to a larger diameter screw and to a stronger (hence stiffer) concrete.

6.2 Comparisons With Eurocode and ETA Predictions

Eurocode 5 (EC5) proposes the following expression [41] for the slip modulus (this time k_i) of a TCC connection, namely :

$$k_i = 2 \text{ x } \rho^{1.5} \, d \, / \, 23 \qquad (2),$$

while the ETA [42] proposes the following expression for the same slip modulus, namely :

$$k_i = 780\, d^{0.2}\, l_{ef}^{\,0.4} \qquad (3),$$

using the symbols ρ = timber density, d = screw diameter to outside of threads, and l_{ef} = the penetration depth of the screw into the timber, with all dimensions in mm and ρ in kg/m^3.

Using values of ρ = 750 kg/m^3, d = 12 mm and l_{ef} = 120 mm, the predicted and experimental values are presented alongside each other in Table 6. The ratio between each prediction and the test data are also given. It is immediately apparent that the EC5 prediction correlates well with the C45C and C90 test results, but significantly under-predicts (by a factor exceeding five) slip modulus for C45T. Moreover, the ETA predictions under-predict the C45C and C90 stiffnesses by a factor of 2.5, and that for C45T by a factor exceeding 14. A key issue is that neither predictive expression allows for the reality of a screw angle dependency of the slip modulus. Fig. 11 clearly shows that, in the range of screw-slip angles between 90° and 45° in tension, this neglect of the angle dependency might require some revision.

6.3 Application of Gamma Method to TCC Beam

Based on the work of Möhler, Eurocode 5 [41] uses the Gamma method to calculate an effective flexural stiffness $(EI)_{eff}$ of the TCC section allowing for slip. Key features of the Gamma method are :

- It was developed for timber-timber hybrid members, and so cracking of the flange connected to the joist was not originally envisaged. In the present application both no cracking and partial cracking of the concrete slab are considered, using the method as presented by Blass et al. [6].
- A sinusoidal moment diagram is assumed. In the present study the UDL, a common load case for building floors, is assumed along the member. The resulting parabolic moment diagram is similar to a sinusoid, and so application of the method is justified.
- Linear elastic behaviour of the connections and materials is assumed, consistent with the importance of serviceability checks on TCC performance.

The crux of the method is a gamma (γ) factor used to allow for the effects of the shear flexibility (k) of the connections, as follows :

$$\gamma = [1 + \pi^2 E_c\, A_c\, s/(kL^2)]^{-1} \qquad (4),$$

where E_c, A_c, s and L are respectively Young's modulus of the concrete, the cross sectional area of the structurally active concrete, the spacing (assumed constant in this example) of the connections along the T-section member, and the span of this member. In this study the concrete is assumed to be of zero tensile strength and so only the compression concrete is structurally active.

Now, in general, a gap may exist between the top of the joist and the base of the structurally active concrete due to cracking of the lower part of the slab and / or to the presence of a permanent formwork interlayer sitting on the joist. Blass et al. [6] allow for this possibility by allowing for a gap of depth d_g as shown in Fig. 16.

With reference to this Figure, the following expressions for a_t (depth from the section neutral axis to the mid-depth of the joist) and a_c (height from the section neutral axis to the mid-depth of the structurally active concrete) apply [6] :

$$a_t = \gamma E_c A_c (d_c + 2d_g + d_t) / 2 / (\gamma E_c A_c + E_t A_t) \quad (5),$$

$$a_c = [(d_c + 2d_g + d_t) / 2] - a_t \quad (6),$$

Finally, the effective flexural stiffness $(EI)_{eff}$ of the section is calculated as [6] :

$$(EI)_{eff} = (E_c I_c + \gamma E_c A_c a_c^2) + (E_t I_t + E_t A_t a_t^2) \quad (7)$$

This calculation approach has here been applied to the TCC beam of section shown in Fig. 17(a). Note that the 95 mm slab depth and 200 mm joist width of this section match those used for the C45C, C90 and C45T connections. Hence, in the present example it is assumed that these three different connections are used in alternate schemes at 200 mm spacing along the span.

The beam is assumed to be of 6 m simply supported single span and is subjected to a UDL of magnitude 4 kN/m. Two alternative calculations were performed, namely one assuming a fully uncracked slab consistent with the original spirit of the Gamma method, and another assuming a partially cracked slab with zero tensile strength of the concrete. For the latter approach, the following equation (after Blass et al. [6]) was used to check the stress (σ_{bc}) at the base of the active (uncracked) concrete zone.

$$\sigma_{bc} = E_c M (\gamma a_c - d_c / 2) / (EI)_{eff} \quad (8),$$

where M is the section moment. Now at the start of the calculations the depth of cracked concrete (if it exists) is not known, so an iterative method is needed to determine this depth. Essentially, when the iterations converge, the value of a_c calculated via Equations (5) and (6) leads to an effective section stiffness $(EI)_{eff}$ which, via Equation (8), enables a calculation of zero stress at the base of the active (uncracked) concrete. This iterative loop was performed in Excel using the goal seek function.

Once $(EI)_{eff}$ is known, the midspan deflection δ_{max} is calculated as follows :

$$\delta_{max} = 5wL^4 / [384(EI)_{eff}] \qquad (9),$$

where w is the load per unit length.

The resulting variations in deflection as a function of connection screw angle, and also for rigid connections, are shown in Fig. 17(b). It is seen that both the C90 and C45C connections lead to significantly higher deflections (77% and 88% more, respectively, for the cracked cases) than does the rigid connection. Also, there is a palpable difference between the uncracked and cracked results for each of the C90 and C45C connections. The C45T connection, by contrast, shows little difference between the uncracked and cracked cases, with only 18% more deflection than the rigid connection case. This again demonstrates the superior efficiency of the C45T connections.

For the partially cracked slab analyses Fig. 17(c) shows, as a function of screw angle, the predicted percentage of the full slab depth which is active (uncracked). It is seen that a rigid connection enables almost 3/4 of the full slab depth to be active, while the corresponding figure for the C45T connection is just over 2/3, only about 7% below the rigid connection value. However the C90 and C45C active slab depths, down to only about 1/2 the full slab depth, are both similar to each other and far below the rigid connection value. Thus the C45T connection mobilises far more effective structural participation from the slab than do the other two connection types. Note that in each case, relative to the rigid connection, the %age difference on deflection far exceeds that on slab depth. This is because the deflections are inversely proportional to the entire TCC section's effective flexural stiffness, which in turn is a nonlinear function of the active slab depth and of the associated changes in a_c and a_t. Finally, note that the timber and slab peak stresses as calculated by the Gamma method are within the capacities and linear regimes of the materials.

7.0 CONCLUSIONS

From the study presented in this report the following key points emerge, namely :

- Three sets each of four double-shear Baubuche LVL-concrete specimens were tested to failure. Four M12 coach screws along a line were used as the connectors at each concrete-timber interface. Each set of four connection specimens was defined by the inclination angle of the screws to the interface. The screw inclinations used were 90^o (C90 specimens), 45^o with the screws in compression (C45C specimens) and 45^o with the screws in tension (C45T specimens) when the connections were loaded. Essentially, there was longitudinal shear reversal between the C45C and C45T specimens.
- The two-stage fabrication process – first threading the screws into the Baubuche stubs, then casting the ready-mixed concrete around the screws – to create twelve hardwood LVL-concrete connection specimens was successful. Simple tests showed that the screws were firmly embedded in the timber. The cast concrete quality (including finish) was excellent.
- Ready-mix concrete was used for good quality control on concrete properties and to facilitate casting. The average cube compressive strength of 51.9 N/mm^2 exceeded the required strength for the specified C32/40 concrete. This should be borne in mind when viewing the connection test results provided below.
- Under load the C90 and C45C specimens failed consistently by fracture of the excessively plastified screws within the Baubuche LVL stubs, near one of the two timber-concrete interfaces in each specimen. By contrast, the failure modes of the C45T specimens were not consistent and changed from splitting of the LVL stub in one case, to fracture from excessive plasticity of the screws in two cases, to pullout of the screws from the concrete in the remaining specimen.
- Within each set of connections (including the C45T set, despite their variable failure modes) the failure loads were consistent, with low CoV values (below 10%) for all tests. This failure load, per line of four screws at each interface, changed from 48 kN for the C45C specimens to 169.6 kN for the C90 specimens to 194.1 kN for the C45T specimens.
- Within each set, the connection slip modulus at 40% of the failure load showed much more variability (and high CoV values of up to 47%) than did the failure loads. Per line of four screws, the C45T specimens exhibited by far the highest slip modulus of 494.9 kN/mm, compared to 97.7 kN/mm for the C90 specimens and 83.2 kN/mm for the C45C specimens.

- The force-slip plots show that the C45C specimens were very ductile, while the C45T specimens showed modest nonlinearity and the C90 specimens approximated bi-linear characteristics with the second line sloped distinctly above the horizontal.
- Given these stark differences in constitutive behaviour between the C45C and C45T specimens, it is strongly recommended that a protocol is defined to conduct both forward and reverse longitudinal shear tests on asymmetric TCC connections such as those based on singly inclined screws.
- Relative to other screw-based TCC connections tested in previous studies, the C45C connections are of similar slip modulus and shear strength, while the C90 connections are of higher strength but similar slip modulus, and the C45T connections are superior in both strength and slip modulus.
- The EC5-predicted slip modulus correlated well with the test results for the C45C and C90 connections, but was too low by a factor exceeding five for the C45T connection. There is a need for the EC5 formula to include a screw angle dependency.
- Application of the Gamma method shows that the C45T connections lead to structurally active compression concrete depths and midspan deflections not far off those for rigid connections, while the C90 / C45C connections induce active slab depths and peak deflections almost 23% / 25% less and about 77% / 88% more than the rigid connection values. Hence the C45T connections are far superior in activating the concrete and improving structural efficiency.

In future work different combinations of screw inclination angles could be investigated in order to gravitate towards appropriate balances between connection stiffness, strength and ductility under both forward and reverse longitudinal shears. In addition, both forward and reverse shears should be applied in a low cycle fatigue context (reflecting seismic action) on the same connections, to establish the extents of any hysteresis. Moreover, the high cycle fatigue properties of these connections (for possible application to road bridges) should also be studied.

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