Switch From Connection Ductility to Reinforcement Ductility With Curvature Reversal in Timber-Concrete Composites

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ABSTRACT

While multi-span continuity can be used to structurally enhance timber-concrete composites (TCCs), there has been scant research into the associated nonlinear load responses particularly of the resulting TCC zones under negative curvature. Consequently this paper presents tests to failure of TCC specimens using hardwood laminated veneer lumber joists and steel mesh connectors, one specimen (TP) under positive curvature, the other (TN) under negative curvature. It was found that the mesh connectors enabled high levels of slab-joist interaction not only in TP where the slab was almost uncracked, but also in TN where the slab exhibited pronounced cracking. Such distinct interaction enabled TN and TP to develop more than twice and six-times, respectively, the stiffness of the joist acting alone. Both TCC members exhibited encouraging ductility, the source of which switched from connection yield distributed along half the span in TP to steel rebar yield concentrated at midspan in TN. TP displayed deflection (global) and curvature (local) ductility near-plateaux over ranges close to or exceeding the corresponding elastic ranges, while for TN the ductility was manifest as low tangent stiffness regimes over deflection and curvature ranges generously exceeding the corresponding elastic ranges. A conspicuous residual hinge at midspan in TN and significant residual end slip in TP provided visual evidence of the ductility. These observations address the issue of TCC connection effectiveness in cracked concrete that has emerged from updating EC5. Crucially, the ductility of TN is predicated on the hardwood's high strain to fracture in flexure, which ensured that extensive rebar plasticity preceded failure of the timber.

KEYWORDS : timber-concrete composites, mesh connectors, tests, continuous beams, ductility.

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1. INTRODUCTION

1.1 Building on the Status Quo

Laminated veneer lumber (LVL), cross-laminated timber (CLT) and glulam are key forms of engineered timber with high quality control on dimensional tolerances, material properties and appearance that have inspired collaborations between architects and engineers to revolutionise the balance between the scale, shape, structural integrity, aesthetics and carbon footprint of buildings [1]. Engineered timber can be hybridised with other construction materials to improve structural performance, for example in timber-concrete composite (TCC) floors, which comprise thin concrete slabs shear connected to timber joists or panels. By resisting slip and separation between the slab and joist, the connections foster composite action to enhance load response. TCCs are 50% lighter and are more sustainable than reinforced concrete floors due to the thin slabs and because timber is both light and (if responsibly sourced) renewable. TCCs are also stiffer, stronger, more acoustically insulating, of higher fire rating and greater thermal mass than timber floors.

Development of TCCs in Europe started after the World Wars, when reinforcing steel was scarce and timber was used instead as exposed tension reinforcement to concrete slabs under positive bending [2]. Since then TCC systems have evolved worldwide, with some iconic examples having been built using various connection types and timber-concrete layouts. Hardwood LVL-concrete composite floors with coach screw connectors were used (2018) at the Anna Freud Centre in London, UK [3]. Softwood glulam-concrete composites with rectangular notch connections and hold-down screws were used (2012) for the floors of Austria's eight-storey life cycle tower one building [4], then alternatively with bird-mouth notches and hold-down screws (2014) for the oval classroom floors of Australia's Dr Chau Chak Wing building [5]. CLT-concrete composites with glued-in steel mesh connectors were used (2013) for a school dance floor in the UK [6] and for lecture theatre floors at the University of Massachusetts Design Building (2017) in the USA [7], while Laminated Strand Lumber-concrete composites with mesh connectors define the free floating staircase (2012) at the University of British Colombia's Earth Sciences building in Canada [5].

An adequate regulatory framework is needed to support wider use of TCCs in practice. To that end, a special working group has been convened [8] to develop a new section of Eurocode 5 focused on TCC design. This group is supported by a project team from within the European COST Action FP1402 that has just released a state-of-the-art report - or STAR [9] - on TCC design.

In its concluding Chapter, the STAR [9] highlights the important need for research on multi-span continuous TCCs. Now under load, a key consequence of continuity is that the negative moments induced along the zone over each internal support will often induce full-depth cracking of the concrete slab (Fig. 1(a)). This in turn adversely affects composite section flexural stiffness in that zone, but it also beneficially can enable steel yield in the regions of peak negative moment. Overall, relative to simply supported single spans, the continuity can be exploited to improve TCC floor performance under any given load for the following reasons :

- Every span experiences rotational restraint over each internal support, which improves stiffness.
- Positive moment demand drops but positive moment capacity remains high, so improving load capacity.
- In negative moment zones the above-described steel yield can strongly improve floor system ductility.

By these means continuity can lead to reduced material consumption over existing spans, or to economic design of longer floor spans, or via the increased ductility, to improved moment redistribution capability and enhanced general robustness of the floor system.

In the significantly cracked negative moment zones high performance connections are essential, to amplify composite action between the timber joist and the steel reinforcement, and in the process to mobilise tension stiffening in the cracked concrete. Now therein lies an important issue, because research and practical applications to date [2] have focused almost exclusively on single span simply supported TCCs, where only positive moments and hence only positive curvatures have developed, thereby embedding the connections largely in uncracked concrete. For situations in which reverse – negative moment – curvatures develop and induce pronounced, full-depth cracking of the slab, little is known about the behaviours of TCC connections. It is thus not surprising that the STAR [9] asks whether such cracks influence connection properties and indeed whether results for simply supported single span TCCs are transferrable to continuous TCC systems.

Given this lack of test data for TCC connections in negative curvature zones, it is prudent first to consider connections with established track records of high performance in positive curvature zones. To that end various reviews [2, 5, 9, 10] identify nails, screws, dowels, notches without or with hold-down screws, perforated steel plates or steel meshes glued-in to the timber, or solely gluing as the key connections investigated to date. Within this spectrum of connections, multiple studies (e.g. [2, 6, 7, 11-21]) have identified that glued-in steel mesh or plates possess excellent levels of slip modulus, longitudinal shear strength and ductility, while other connection types such as dowels can be even more ductile [2, 15].

Nežerka [12] discussed the merits of glued-in steel mesh/plate connectors in terms of their observed wood or steel shear failures (against the wood splitting failures induced by nail or screw connectors), and in terms of the simplified transformed TCC section calculations that stem from their near-zero slip characteristic. The above-cited experimental and predictive studies in which these outstanding properties of glued-in connections were observed have covered cast in-situ and prefabricated TCCs, softwood glulam and LVL, and different grades of concrete. Moar [21] stated that, given these excellent properties, the cost and curing time of the epoxy-based glues should be optimised to extract best value from these connections. This superlative mechanical performance of glued-in steel mesh / plate connections along positive curvature zones bodes well for, but must still be tested in the cracked negative curvature application.

If zero slip is closely approximated in negative curvature zones up to ultimate then, as Fig. 1(b) shows, the timber's extreme tension fibre strain is very near in magnitude to the steel reinforcement strain even after extensive plasticity of this steel induces pronounced cracking of the concrete. This implies that meaningful steel rebar-based ductility can be achieved only if the timber retains its flexural integrity up to extreme tension fibre strains generously exceeding the steel's yield strain. Hardwood LVL satisfies this criterion because hardwoods are of high flexural strengths and because LVL engineering enhances both the magnitudes of and quality control on those strengths. Now historic planting means that hardwoods will be increasingly available over the next 50 years [22]. Also recent advances in machinery have enabled development of hardwood LVL, but few studies have been reported thus far into hardwood LVL-concrete composites. One such study [4] entailed multi-span continuous members with coach screw connectors, while another two [23, 24] entailed positive curvatures and used notch connections which also perform to a high standard. It is now timely to build on these studies by focusing on LVL-concrete composites with either glued-in steel mesh / plate connectors or notch connections in both positive and negative curvature zones.

In positive curvature zones, the steel reinforcement strains are typically well below yield. Since timber is a brittle material any ductility in these zones would come from the connections. Experimental and numerical studies [25-28] have looked at the influence of this connection ductility on overall TCC member behaviour. Fig. 1(a) shows that this connection ductility in positive curvature zones is usually distributed along the span between zero and peak moment (because the vertical shear force on the section and hence the longitudinal shear force on the connections is peak in this zone), whereas in the reversed curvature zones the mechanism of ductility switches to steel rebar yield and is concentrated over short lengths in the peak moment zone (where the rebar stresses are highest). It is instructive to understand the structural implications of this ductility mechanism change and other variations in behaviour between these zones of opposite curvature.

1.2 Aim and Objectives of the Present Study

The load response of a multi-span continuous TCC will depend on the load type along with the relative flexural stiffness, strength and ductility distributions between the positive and negative curvature zones. This, along with the preceding discussion, mean that it is prudent to take a step back from the continuous members and instead, as an initial activity, to understand the structural actions in these zones of opposite curvature. To that end the overall aim of this study was to gain experimental insight into the structural characteristics of hardwood LVL-concrete composites with glued-in steel mesh connectors in positive and, separately, negative curvature zones.

The specific objectives were to :

- Observe the full-range effectiveness of the connections in negative curvature zones.
- Compare TCC stiffnesses, strengths, ductilities and failure modes for positive and negative curvatures.
- Establish the levels of predictability of the observed and measured load responses.

This last point is important, because while high stiffness connections simplify predictive calculations by justifying an assumption of zero interface slip, these calculations can still be complex due to the constitutive nonlinearities (concrete compression softening, steel yield, connection yield, etc) expected at mature stages of TCC response, as failure is approached.

The following sections of this paper describe the details of, the positive and negative curvature tests on and the significance and predictability of the results from hardwood LVL-concrete composite T-beam specimens. Then, in closing, the paper draws some key conclusions from the above study and makes some suggestions for further work.

2. TEST SPECIMENS

2.1 Key Details

Three beam specimens were prepared and tested in four-point bending, each over a 4.6 m simply supported single span, see Fig. 2(a). Of these three one was a control specimen in the form of a 120 mm wide x 200 mm deep LVL joist acting alone, while the two other specimens were T-section TCCs of nominally identical details to each other and comprising the same LVL joist as the control specimen shear connected to a 70 mm deep x 500 mm wide slab, see Fig. 2(b). One TCC T-section specimen was loaded to failure under positive curvature by placing the T-section the right way up, while the other was tested under negative curvature by turning the T-section upside down, in both cases with the load applied downwards from above. Henceforth the LVL joist specimen, the TCC specimen under positive curvature and that under negative curvature will be termed LJ, TP and TN respectively. Key features of the specimens were as follows :

- The LVL joists were made of the hardwood beech with manufacturer quoted [29] flexural strength of 75 N/mm^2 and elastic modulus of 16.8 kN/mm^2. These give a timber flexural fracture strain above 4450 $\mu\varepsilon$, which exceeds the steel rebar's yield strain of 2500 $\mu\varepsilon$ by almost 80%. In the event, tests performed in this study confirmed the manufacturer's quoted elastic modulus at 10.3% moisture content, but found that the flexural rupture strain was above 6100 $\mu\varepsilon$ which both far exceeded the manufacturer's specification and was almost 2.5-fold the steel yield strain. Hence this LVL is an excellent candidate for allowing significant steel yield before timber fracture.
- On testing, the concrete possessed an actual average cube strength of 56.7 N/mm^2. An elastic modulus of 35 kN/mm^2 was assumed for this concrete.
- The connectors took the form of two longitudinal rows of mild steel raised mesh rectangular portions of designation 10-09 (0.9 mm thick metal [30]), bonded into 4 mm wide, 40 mm deep grooves in each LVL joist at 30 mm from the near edges of the joist. As shown in Fig. 2(c), each connector was a 400 mm long x 110 mm high rectangle, with 120 mm clear gaps between consecutive connectors along each row. Importantly, each connector comprised two identical rectangular mesh portions, laid as a pair into the groove. The bonding agent was a two-part epoxy adhesive of manufacturer-quoted [31] lap shear, flexural, compressive and tensile strengths of 18.3, 35, 85 and 17 N/mm^2 respectively and a setting time of 44 minutes, all at room temperature.

- As shown in Fig. 2(b) the longitudinal steel reinforcement comprised five bars, each 12 mm diameter, near the top of the slab and distributed uniformly across the width of the slab, with the middle bar located centrally between the two rows of connectors. These bars were structurally negligible in specimen TP, but strongly influenced structural action in specimen TN. In addition, an 8 mm diameter bar was laid transversely, halfway through each 120 mm gap between longitudinally adjacent connectors (Fig. 2(b), (c)). Hooks were used at the ends of all steel bars for extra anchorage.
- Instrumentation comprised electrical resistance strain gauges (ERSGs) bonded longitudinally to the LVL joists and to the steel rebars all at midspan. For each TCC specimen the ERSG G1, as shown in Fig. 2(b), was placed on the face of the joist furthest away from the slab, while ERSGs G2 and G3, also on Fig. 2(b), were placed on the joist laminations at 10 mm from the slab-joist interface. In addition, the middle and the two edge 12 mm diameter steel rebars which can be seen in the T-section of Fig. 2(b) were also strain gauged. For the control specimen LJ, gauges were placed at midspan on the extreme tension and compression faces. Moreover, displacement transducers were used to record midspan deflection for all specimens, and also to record end slip for both specimens TP and TN.

2.2 *Fabrication*

After forming the grooves into the two timber joists targeted for the TCC specimens, formwork was built around each such joist. Then, under room temperature conditions in the lab, the two-part epoxy adhesive was mixed and applied with a trowel into the grooves, only over the length of one connector (plus an extra 10 mm either side) at a time so as to reduce wastage. Each paired mesh connector was repeatedly fitted into and pulled out of its adhesive-filled length of groove until the adhesive had filled the holes in the lowest 40 mm depth of mesh. Finally, the paired mesh connector was fitted into the groove, taking care first to top up the groove with adhesive to minimise the chances of any voids in the final connection.

The adhesive was allowed seven days to cure, during which time the reinforcing steel bars were cut to length and placed in the formwork. Both items of formwork complete with reinforcement (before the rebars were finally in place on their chairs to give appropriate cover) and with the securely bonded-in connectors are shown in Fig. 3(a). Fig. 3(b) shows the local detail of two adjacent bonded-in paired mesh connectors, note the cured adhesive over the lowest 1 cm or so of the nearer paired connector.

Once the steel reinforcement grid had been formed, strain gauged and located using tying wire and small concrete block chairs appropriately placed within the formwork, the slabs were cast using ready-mixed concrete and poker vibrators. Fig. 3(c) shows the freshly cast slabs, which were then covered with polythene sheets and left to cure over four weeks before testing occurred.

3.0 TESTING STRATEGY

The loading setup was designed to ensure that, within the *highly cracked region* of the slab in specimen TN, as many as possible of the connections were subjected to *significant longitudinal shear*. A traditional four-point bending arrangement (defined by loads at third points along the span) would have defeated this objective, because the most highly cracked zone (the entire middle third zone of peak moment) would also be a zone of zero vertical shear, and so the connections in that highly cracked zone would be exposed to negligible longitudinal shear. A three-point bending arrangement is better suited because the most highly cracked zone, namely the zone of peak moment, is also a zone of high vertical – and so of high longitudinal – shear.

To those ends, as shown in Fig. 2(a), each beam specimen was tested over a 4.6 m simply supported span in near three-point bending, using two applied loads at only 200 mm centres from each other around midspan, rather than a single load applied at midspan (the ideal three-point bending arrangement) for stability reasons. A single servo-hydraulic actuator, located directly above midspan of each beam specimen, applied one load which was then distributed to the two locations along each specimen via a short spreader beam arrangement. Each test was conducted in displacement control at a rate of 2 mm per minute. Load, strain and displacement data were recorded at 1 Hz into electronic data loggers during all tests.

As shown in Fig 4(a) and Fig. 4(b), specimens TP and TN were tested with the T-section the right side up and upside down respectively. This ensured that the slab was largely uncracked in TP, but heavily cracked in the midspan zone to create conditions conducive to steel yield in TN. The alternative of keeping TN the right side up would have meant an upward load from underneath and hold-down supports at the ends, which would have been much more challenging to achieve.

Testing was paused and the load temporarily held if there was a need to reset a displacement transducer or to inspect and visually record in some detail any newly observed damage to the specimen. In the approach to failure the displacement control rate was reduced to 1 mm per minute.

4.0 TEST RESULTS

4.1 Failure Loads, Switch of Ductility Mechanism With Curvature Reversal and Strain Response

Table 1 summarises the failure loads and modes of the three specimens. Due to its compliance, control specimen LJ activated the actuator's stroke before specimen failure occurred. Hence, the failure load for LJ in Table 1 has been estimated assuming flexural failure at the extreme tensile fibre at midspan at a fracture strain of 6100 $\mu\varepsilon$, which was the recorded flexural fracture strain for specimen TP.

The Table shows that, with respect to LJ, composite action under positive curvature (specimen TP) increased the load capacity by 113%, while that under negative curvature (specimen TN) boosted load capacity by 45%. Both TN and TP exhibited ductile modes of failure. In what follows it is shown that the source of this ductility was yield of the longitudinal steel reinforcement in the midspan zone for TN, but switched to yield of the steel mesh connectors in longitudinal shear along an entire half span for TP.

Fig. 5 shows three images of the unloaded TN after failure. The first, Fig. 5(a), shows TN's residual deflected shape. The pronounced curvature evident around midspan is the leftover of a "plastic hinge" which formed due to extensive yield of the steel reinforcement in that peak moment zone while TN was under load. Such conspicuous hinging was a physical manifestation of significant ductile behaviour. Fig. 5(b) shows that the TN slab developed many wide transverse cracks in the midspan zone, further evidence of considerable steel yield locally. Fig. 5(c) zooms in on one side of TN in the midspan zone. It shows that the joist experienced flexural fracture in its tensile lower portion (highlighted within the dashed ellipse), immediately adjacent to the slab. This fracture occurred under load and constituted the ultimate failure mode of TN. Also evident in Fig. 5(c) are delamination within the LVL joist and separation between the slab and joist, both of which were observed only while TN was being unloaded after failure. These may well have been secondary effects, due to vertical tension developing in the glued-in steel mesh connectors as the joist tried to pull away from the slab during that unloading phase. Notice, in Fig. 5(b), longitudinal cracking of the concrete around the steel mesh connectors, concentrated around midspan and probably another secondary effect.

Still on TN, Fig. 6 compares the recorded variations with load of midspan strains at the extreme compressive face of the LVL (gauge G1 in Fig. 2(b)) and at a level within the LVL only 10 mm away from the slab-joist interface (the average of gauges G2 and G3 in Fig. 2(b)). The plots show almost bi-linear behaviour. The first linear regime is of high gradient and extends from zero load to approximately 60 kN, beyond which there is a distinct reduction in gradient which defines the second linear regime and was due to yield of the steel reinforcement. The significant slope reduction signifies that, relative to the first regime, much smaller load increments were needed during the second regime to induce given strain increments. This in turn indicates a sharp stiffness reduction in the midspan zone local to the strain gauges.

On Fig. 6 it is seen that at 100 kN applied load the joist's peak tensile strain (G2-G3 Av) exceeded 4600 $\mu\varepsilon$, over 85% beyond the rebar yield strain (and of course the rebars developed even higher strains), while at the joist's extreme compression fibre a compressive strain (G1) exceeding 7800 $\mu\varepsilon$ developed. The capability of the LVL to develop such large strains was crucial, otherwise buckling or fracture of the LVL laminae at low strains would have precluded steel yield in and so ductility from the rebar. Note that during the top 10% or so of the load the joist strain gauges (and even before that the rebar gauges) malfunctioned, which explains why the plots of Fig. 6 peak at about 100 kN, rather than at the 111.4 kN maximum load carried by specimen TN.

Fig. 7 shows three post-failure images of the unloaded specimen TP, which give clues to the failure modes of that specimen. Fig. 7(a) shows longitudinal hairline cracks which developed in the concrete slab around the steel mesh connectors. This occurred along one entire half span of TP. Fig. 7(b) shows that TP exhibited significant (about 7 mm) residual slip at the end of that half span. These two observations show that ample longitudinal shear yield was a distinct feature of TP's failure. Fig. 7(c) shows flexural fracture and some delamination of the LVL joist in the midspan zone, which together comprised the ultimate failure mode of TP.

Fig. 8(a) shows the variation with load of slip at the end of TP's longitudinally cracked half span. It is seen that after an initial linear regime during which the slip remained low up to a peak of 0.1 mm at the maximum load of 164.1 kN, a near - ductility plateau developed in which the slip increased to almost 9 mm while the load hovered around the peak value. This extensive slip after attainment of peak load exposes plasticity of the steel mesh connectors as the source of

TP's ductility. In the next section of this paper, constitutive behaviour and axial equilibrium are applied to the recorded midspan strain data at peak load, to show that the estimated longitudinal shear force carried by the connectors in one half span did in fact closely approximate the capacity of all the connectors in that half span combined. This provides further evidence of connection shear yield. Finally, it was observed that no slip was recorded at the other end of TP, indicating some degree of asymmetry (cause unknown) of this specimen about midspan.

Fig. 8(b) compares the recorded variations with load of midspan strains in TP at the locations previously discussed, namely on the joist's extreme tension face (gauge G1 in Fig. 2(b)), also on the joist at 10 mm from the interface with the slab (the average reading from gauges G2 and G3 in Fig. 2(b)), and finally on the steel rebars (from which the average reading was also used). It is seen that the initial linear behaviour occurred up to peak load, then gave way to a well-defined near-ductility plateau, though this time of course due to connection yield. Notice the considerable post-peak redistribution due to connection yield, as evidenced by strain reversals in the steel rebar and in the timber near the slab-joist interface.

For comparison, Fig. 8(b) also shows the variations with load of the extreme tensile and compressive fibre strains recorded for specimen LJ at midspan. These plots confirm that linear behaviour prevailed for LJ up to the peak applied load, and also that the peak tensile strain (approximately 5000 $\mu\varepsilon$) recorded for LJ was almost 20% below the fracture strain recorded for the timber from testing specimen TP. As previously stated, the absence of failure in LJ during the test prompted an estimate of its failure load based on the recorded TP fracture strain.

Finally, note that while the joist midspan strains represented in Fig. 6 for TN and Fig. 8(b) for TP were each recorded from two levels which were almost symmetric about the mid-depth of these joists, the strain plots are themselves distinctly asymmetric about the zero strain vertical axes of the Figures. This asymmetry is pronounced for TN, even more so for TP. This implies that, alongside local bending of the joist, significant axial forces also developed in these joists at midspan, which in turn implies the development of notable longitudinal shear forces in both the TP and – especially – TN connections, a point which will be further examined later in this paper.

4.2 Effectiveness of Connections in Positive and Negative Curvature Zones

For different load levels applied during the tests, Fig. 9(a) and (b) show the through distributions of longitudinal strains recorded at midspan for TP and TN respectively. On each set of plots three levels in the TCC section are represented, namely the joist fibres both furthest away from the slab and at 10 mm from the interface with the slab, along with the steel rebar. For this purpose both the TP and TN T-sections have been placed with the T-table topside, to facilitate comparison between the two specimens.

Fig. 9(a) shows that the TP through-depth strain distributions were almost perfectly linear with near-zero slip at all loads represented, which span the spectrum from 24% to 98% of the peak load, hence even at loads closely approaching failure. This confirms the observations from previous studies [7, 13 - 20] of the near-full interaction enabled by steel mesh connections in positive moment zones, where the concrete is largely uncracked.

Of particular interest, though, is Fig. 9(b)'s revelation that the slab-joist through-depth strain distributions deviate from continuous straight lines only by quite modest amounts, signifying that the mesh connections also enabled pronounced levels of interaction in the highly cracked negative curvature zones. This observation constitutes one of the few currently available experimental outputs to vouch for the idea that TCC connections can be highly effective in cracked concrete, thereby directly addressing an issue raised in the FP 1402 STAR [9]. It is this high degree of mesh connection effectiveness in the negative curvature zone that has underpinned the ability for significant yield of the steel rebar before failure of the TN joist, by creating a load path from the timber via the mesh and through the cracked concrete into the rebar.

In Fig. 9(b) there are no plots above 57.3 kN because the TN rebar gauges malfunctioned beyond that load. However the 57.3 kN plot shows that there was no slab-joist interface kink in the strain line even when the rebar had already exceeded the steel yield strain. In future tests, more rebar gauge readings will be used to further investigate this point at more advanced stages of the yield regime, when wider cracks will have opened up in the concrete.

In Fig. 9(a), (b) the neutral axis is the intercept of each strain plot with the zero strain vertical axis. This intercept is seen to be in the slab for TP and in the joist for TN. It is instructive, for each of specimens LJ, TP and TN, to observe the variation with applied load of neutral axis height (NAH), defined as the height of this intercept above the base of the joist. Fig. 9(c) shows the results, for which both the TP and TN T-sections are oriented with the T-table at the top. It is seen that for specimen TP the NAH was almost constant at just over 20 mm above the slab-joist interface up to peak load, while for TN the NAH hovered between approximately 12 mm and 24 mm above mid-depth of the joist including an observable reduction at 50+ kN probably due to steel yield, and finally for LJ a small but discernible migration downwards of the neutral axis from mid-depth of the joist is evident at higher loads. This last point will be picked up again in a subsequent section of this paper.

4.3 Trends in Behaviour Based on Test Data

Fig. 10(a) compares the load vs midspan deflection plots based on the recorded test data for all three specimens. Immediately apparent is the progressive enhancement in structural properties in proceeding from the original joist (LJ) to the TCC member under negative moment (TN) to the TCC member under positive moment (TP). Also note that while LJ shows a singly linear behaviour, TN exhibits first an uncracked linear regime followed by a slope reduction to a cracked linear regime and then a further significant slope reduction to a slightly nonlinear regime due to steel yield. This final, low slope, nonlinear regime for TN occurs over a wide range of deflections – from approximately 45 mm to 115 mm – and so shows clear scope for ductile behaviour. TP shows a singly linear and high stiffness regime up to peak load, after which the connection yield led to the pronounced near-ductility plateau.

Four key points from Fig. 10(a) are crucial and should be amplified, as follows :

- There is a distinct increase of member stiffness (slope) in proceeding from LJ to the cracked regime of TN, and a further significant stiffness increase from TN to TP. This means that the reinforcement in the cracked concrete zone was very effective at improving stiffness beyond that of the joist alone, while the presence of uncracked concrete (with little contribution from the reinforcement) was most effective at improving stiffness. In order to quantify these stiffness enhancements, Table 2 compares the gradients of the LJ, TN and TP lines. For each relevant line this gradient was calculated as $\Delta P/\Delta\delta$, where ΔP and $\Delta\delta$ signify the load and deflection increments, respectively, between two chosen points on the line. For LJ the entire line applies, while for TN the second stage (cracked but not yielded) line is used, and for TP the near-single gradient line between zero and peak load is applicable. Using this approach the Table shows that, relative to the joist acting alone (LJ), the cracked negative curvature specimen (TN) led to a 120% increase in stiffness while the positive curvature specimen (TP) induced a 456% stiffness increase. Note that in TN there were uncracked zones (where the steel rebar contributions were negligible) near the supports and cracked zones (where the steel rebar played a significant role) towards midspan, which rendered the stiffness increase from TN non-uniform along the span. For TP a more uniform stiffness increase along the span was expected. In both cases, clearly the high degree of interaction provided by the connections was instrumental in achieving those high stiffness enhancements.

- In order to emphasise the effectiveness of the connections in the negative moment zone, Table 2 also includes another stiffness variable $\Delta P/\Delta\delta$ which, by necessity, had to be predicted for the negative moment TCC member based on beam theory, but assuming zero interaction between the slab and the joist. Both an uncracked slab (where the steel rebar is ignored) and a cracked slab (which accounts for rebar contributions) have been considered in two alternative cases. Hence in this approach both the slab and joist are in pure flexure and develop identical curvatures, so that the joist's neutral axis is at its mid-depth while either the uncracked slab's neutral axis is at the slab's mid-depth or the cracked slab's neutral axis is as dictated by the layout and properties of its concrete and steel rebar. Table 2 shows that relative to LJ, the zero interaction layout gives only 47% and 22% stiffness increases assuming uncracked and cracked slabs respectively, as compared to the much higher 120% stiffness increase achieved in specimen TN. Thus the composite action that this connection induced was so significant as to stiffen the joist by 73% more than does an uncracked slab in zero composite action with the joist, despite the fact that this connection-induced composite action caused full cracking of the slab. This provides compelling evidence for the high degree of effectiveness of the mesh connections in the cracked negative curvature zones.
- The 45% increase (from 76.9 kN to 111.4 kN) and further 47% increase (from 111.4 kN to 164.1 kN) in load capacity from LJ to TN and from TN to TP respectively, were dictated by the loads at which the connections yielded in TP and the steel rebar yielded in TN. In future work, the connection's longitudinal shear strength could be manipulated to observe the effect on load capacity without compromising ductility.
- The 42.9 mm deflection range of TP's ductility near-plateau is almost 90% of the 50.2 mm first-yield deflection. For TN, there was a 55% drop in global tangent stiffness due to reinforcement yield, from 1.33 kN/mm before yield to 0.61 kN/mm after yield. Also, the lower stiffness applied over a 71.6 mm deflection range which approaches double the 42.2 mm deflection at which first-yield occurred. In future studies these statistics may be improved upon, but meanwhile they clearly show that both the nature and extent of the ductility available in both positive and negative curvature zones are quite encouraging.
- In Fig. 10(a) both the slope and the deflection range of the low gradient regime which defines TN's approach to peak load are manipulable by changing the steel rebar pattern in the negative curvature zone. This is because TN's low slope regime starts with steel yield and is

terminated by timber flexural fracture both in the midspan zone. In TP, by contrast, connection yield along the entire half span ending with timber fracture at midspan define failure. Hence positive curvature ductility may be influenced by manipulating the connection details.

Fig. 10(b) shows the midspan moment (M) vs curvature (κ) characteristics obtained for all three specimens based on the test data. The moment was calculated by applying equilibrium to the member under the recorded applied load, while the section curvature was determined as the gradient of the through-depth strain line in the LVL joist based on strain recordings from joist gauges G1, G2 and G3 shown in Fig. 2(b) and discussed earlier. The trends noted from the deflection plots, Fig. 10(a), which reflect the global behaviours of the specimens, are also broadly evident in these M - κ plots of Fig. 10(b) which reflect local section behaviour at midspan. In particular, the curvature range of TP's ductility near-plateau exceeds 150% of the first-yield curvature. For TN, there was a 67% drop in local section stiffness when reinforcement yield occurred, and the lower (yield regime) stiffness applied over a curvature range which is almost triple the curvature at which first-yield occurred..

Finally, for any applied load on TN or TP, the combined longitudinal shear force on all the connections in either half span was calculated via a three-step process. First, the timber material constitutive behaviour was used to convert the joist's midspan through-depth strain distribution into a corresponding midspan through-depth stress distribution. Second, these joist stresses were converted into a joist axial force at midspan by taking the product of average through-depth stress and the joist's cross sectional area. Third, longitudinal equilibrium of the joist in an exploded elevation of the structure requires that the longitudinal shear force carried by all the connections in either half span equates to this joist axial force at midspan.

The outcome of this exercise are the Fig. 10(c) plots showing the variations with load of the total longitudinal shear force developed by the connections within one half span, for each of TP and TN. It is immediately apparent that both sets of connections worked hard, clearly more so for the TP connections than for their TN counterparts. Indeed at applied loads common to these two specimens, the TP connections developed almost 80% more longitudinal shear force than was the case for TN. In addition, at the peak TP load of 164 kN, the longitudinal shear force demand per unit of connection very nearly equated to the average longitudinal shear failure load obtained from the connection tests. This is further compelling evidence that the TP connections exhibited longitudinal shear yield. By contrast, the maximum TN connection force from Fig. 10(c) was, per unit connection, well below the average capacity recorded from the connection tests, although the large deflections due to steel yield in the midspan zone may well have led to the longitudinal splits in the concrete around the connections in that zone as seen in Fig. 5(b).

Above 80 kN the TN connection force plot in Fig. 10(c) stops, because it is not clear how the higher strains translate into stresses in the compression zone. This is an important concern because Fig. 5(c) shows residual buckling of the upper lamellae at midspan of TN, clearly due to development of high compressive strains in the latter stages of that test, but it isn't clear at what stage of the test this buckling started. Also, note from Fig. 11 the relationship between the recorded strains at the extreme tension and compression fibres for specimen LJ at midspan. At lower loads the two are equal, but beyond 2000 $\mu\varepsilon$ the compressive strain progressively exceeded its tensile counterpart up to 10% at a tensile strain of 5000 $\mu\varepsilon$. This was very likely a symptom of compression softening, the nature of which is uncertain and needs further investigation. Meanwhile note that on Fig. 10(c) the peak TN connection force represented is not far off the tensile yield force of the five 12 mm diameter steel rebars.

4.4 Predictability of TCC Structural Characteristics

It is instructive to establish whether the tenets of beam theory may be used to predict the load responses of the three beam specimens of this study. This can have multiple benefits , as follows:

- It is a reliable means by which the effectiveness of this mesh connection in approaching full timber-concrete interaction can be established;
- It can potentially enable user-friendly analyses of TCCs that facilitate design of such beams;
- It may provide an efficient method for evaluating the structural enhancements, relative to the timber joist on its own, from different reinforcement layouts when the TCC section is subjected to negative moments.

To those ends, for all three test specimens, Table 3 compares the neutral axis height predicted using first moments of area with those deduced from the through-depth recorded strain distributions of Fig. 9. This height is with respect to the joist's bottom fibre in LJ and to the joist's furthest fibre from the slab in TN and TP. Table 3 shows that the ratio of experimental to predicted heights always exceeds 90%, suggesting that the connections enabled behaviour not far off full interaction even in the highly cracked negative curvature zones.

Finally, a flexural stiffness EI was calculated for each of LJ, TN and TP based on four different approaches, as follows:

- Prediction based on transformed section theory, which assumes full slab-joist interaction.
- Use of the slope of the relevant load vs midspan deflection experimental line from Fig. 10(a). If the beam is of constant section and the load is symmetric about midspan, then the flexural stiffness EI may be related to the applied load increment ΔP, the resulting midspan deflection increment $\Delta\delta$, the span L and the distance "a" of each load from the nearby support as :

$$EI = a(3L^2 - 4a^2)(\Delta P/\Delta\delta)/48 \tag{1}$$

The term $(\Delta P/\Delta\delta)$ in Eqn (1) is the slope of the relevant line from Fig. 10(a). The constant section requirement along the beam means that this is a global calculation for the member.

- Use of the slope of the relevant moment (M) vs curvature (κ) experimental line from Fig. 10(b). According to theory the flexural stiffness EI may be related to the applied moment increment ΔM and the resulting section curvature increment $\Delta\kappa$ as :

$$EI = \Delta M/\Delta\kappa \tag{2}$$

In contrast to the global deflection route outlined above, this calculation is based on behaviour local to the strain gauged section of the member.

- Use of the slope of the relevant load vs connection force experimental line from Fig. 10(c). If the beam is of constant section, then the flexural stiffness EI may be related to the applied load increment ΔP, the resulting connection force increment ΔF along half the span, the span L, the cross sectional area of the timber joist A_T, the elastic modulus of the timber E_T and the distance $\bar{y}$ between the joist section's centroid and the TCC section's neutral axis as :

$$EI = L\, E_T\, A_T\, \bar{y}\, (\Delta P/\Delta F)/4 \tag{3}$$

Note that the term $(\Delta P/\Delta F)$ in Eqn (3) is the inverse slope of the relevant line in Fig. 10(c). Key to this calculation is $\bar{y}$, which uses the experimentally determined TCC section's neutral axis as plotted in Fig. 9(c). Again the constant section requirement renders this a global calculation. Of course this calculation applies to the TCC members, but not to specimen LJ.

The results of applying these different approaches to each test specimen are presented in Table 4. Also presented in the Table is the ratio of each experimentally based *EI* value to the corresponding prediction. The following trends are evident from Table 4.

- The full prediction agrees well with the deflection and M-κ approaches particularly for specimens LJ and TP, where the full prediction is mostly within 8% and at maximum 13% away from the test-based values. For specimen TN the fully predicted *EI* value exceeds that based on the deflection and M-κ approaches by 20% and 26% respectively, which is still highly encouraging. This is clear evidence that the connections enabled near-full interaction in the uncracked positive curvature zones and quite high levels of interaction in the highly cracked negative curvature zones.
- The prediction is also below the connection force-based *EI* value by 19% for specimen TP, and exceeds the connection force-based *EI* value by 20% for specimen TN. Hence the predictions give the least successful comparisons with the connection force – based calculations, but even here the values are not wildly differing.
- For specimen TP the prediction is either only marginally above or is below the experimentally-based values, while for TN the predictions exceed all experimentally-based values. This again suggests almost full interaction and somewhat reduced, but still quite high interaction due to these mesh connections in positive and negative moment zones respectively.

5.0 CONCLUSIONS

From the experimental investigations discussed in this paper the main conclusions are that :

- Relative to a hardwood laminated veneer lumber (LVL) joist acting alone, the use of steel mesh shear connections between this joist and a concrete slab increased section stiffness by 520% under positive curvature with the slab almost uncracked, and by over 110% in highly cracked negative curvature zones.
- The high degrees of slab-joist interaction introduced by the mesh connections also led to ductile failure behaviour by longitudinal shear yield of the connectors in the positive curvature specimen and switching to yield of the reinforcing steel at midspan in the cracked negative moment zone. Clear visual evidence of these ductile failure modes came from

observations of a distinct residual plastic hinge for the negative curvature specimen and pronounced residual end slip for the positive curvature specimen.

- Recorded test data from the positive curvature specimen confirmed this specimen's ductile behaviour. The deflection ductility is marked by a near-plateau on the load-deflection plot and extends over a range almost equal in magnitude to the first yield deflection, while the curvature ductility is also defined by a near-plateau on the midspan moment-curvature plot and extends over a range which is 1.5 times the curvature at first yield.
- For the negative curvature specimen the ductility due to steel yield shows up as a 55% stiffness reduction on the load-deflection plot and extends over a deflection range double that of the first-yield deflection, while on the midspan moment – curvature plot it emerges as a 67% stiffness reduction over a range which is triple the first-yield curvature.
- The predicted flexural stiffnesses based on transformed section theory are mostly within 8% (and in one case 13%) of those based on the global load-deflection and local moment-curvature test data for the original joist and for the TCC member under positive curvature. For the TCC member under negative curvature this stiffness disparity between predicted and experimental sources was within 26%. When the test-based connection force data were included the disparity grew to 19% for the positive curvature TCC member. These results suggest that, if steel mesh connections are used, transformed section theory is highly applicable to the TCC member under positive curvature, somewhat less so but still with high levels of interaction for the TCC under negative curvature which caused significant cracking of the concrete. For other types of ductile connection which enable only partial composite action, transformed section theory will be inapplicable.

In future work other mesh connection and steel reinforcement layouts may be investigated to establish their influences in the stiffness, strength and ductility of TCC's under both positive and in particular in highly cracked negative curvature zones.

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