Experimental Tests of Notched and Plate Connectors for LVL-Concrete Composite Beams

David Yeoh[1]; Massimo Fragiacomo[2]; Mary De Franceschi[3]; and Andrew H. Buchanan[4]

Abstract: This paper reports the experimental results of symmetrical push-out tests performed on notched and toothed metal plate connectors for laminated veneer lumber (LVL)-concrete composite floor systems. The characteristic shear strength and slip moduli were evaluated for three types of connectors: (1) a 300-mm-long rectangular notch cut in the LVL joist and reinforced with a 16-mm-diameter lag screw; (2) a triangular notch reinforced with the same lag screw; and (3) two 333-mm-long toothed metal plates pressed in the lateral surface of two adjacent LVL joists. The shear force versus relative slip relationships are presented together with analytical prepeak and postpeak approximations which can be used to carry out nonlinear finite-element analyses of LVL-concrete composite beams. The failure mechanisms of the notched connections are also discussed. Analytical design formulas for shear-strength evaluation of notched connections derived in accordance with New Zealand Standards and Eurocodes are proposed based on four possible failure mechanisms. Good approximation was found if a slight modification of the Eurocodes formulas is introduced.

DOI: 10.1061/(ASCE)ST.1943-541X.0000288

CE Database subject headings: Composite beams; Wood; Concrete; Connections; Tests.

Author keywords: Composite structures; Timber; Concrete; Connections; Tests.

Introduction

Timber-concrete composite (TCC) floors consist of two parts, an upper concrete slab tied to a lower timber beam by means of shear connectors. The shear connectors transmit the shear and prevent or reduce the relative movement ("slip") between the lower edge of the slab and the upper edge of the timber beam depending on the efficiency of the connection. The concrete flange, mainly subjected to compression, takes advantage of the high compression strength and stiffness of concrete. The timber web, mainly subjected to tension and bending, benefits from the high tension strength to weight ratio of timber, particularly in the case of laminated veneer lumber (LVL). A wide range of shear connectors have been developed in the world particularly in Europe, and each of these connectors varies in its rigidity and strength. Ceccotti (1995) presented a large number of fasteners that can be used to connect the concrete slab to the timber and sorted them in different categories in relation to their degree of rigidity. The shear strength and stiffness (or "slip modulus") of the shear connectors at serviceability and ultimate limit state (ULS) are important parameters required for the design of a TCC floor.

Research on TCC connections was found as early as 1943 (McCullough 1943) where connections built from different metal fasteners and pipe dowels were tested. Shear transfer devices in the form of triangular plate spike were found to provide full composite action in a beam (Richart and Williams 1943) while on the contrary Pincus (1970) reported that mechanical shear fasteners such as nails developed less than 50% composite action between the timber and concrete T-beam. Pincus also confirmed that nails epoxied to timber were able to achieve full composite action up to failure. Pillai and Ramakrishnan (1977) carried out connection shear tests on a series of 3–5-mm-diameter nails and reported that the arrangement of the nails at an inclination of 45° with the head pointing toward the closest support resulted in higher strength and lower slip.

To date, many studies on TCC connection and its performance have been done (Fragiacomo et al. 2007; Gutkowski et al. 2004; Lukaszewska et al. 2007, 2009; Aicher et al. 2003; Clouston et al. 2005). Fragiacomo et al. (2007) reported results of tests to failure and under sustained load of a proprietary head stud connector screwed to the timber marketed by the "Tecnaria SpA" (www.tecnaria.it). The connector was found to perform well both in the short and long terms. Notched connection reinforced with dowel or metal anchor that allows tightening after the concrete curing with the advantage of reducing the gap between the concrete and timber caused by the concrete shrinkage within the notch was introduced by Gutkowski et al. (2004).

Lukaszewska et al. (2007) in an effort to develop a fully demountable TCC system with concrete slab prefabricated off-site chose seven types of connector to build 28 asymmetrical push-out specimens. Among these connectors, three were investigated for the first time: (1) a steel tube with a welded flange embedded in the concrete slab and a hexagon head coach screw; (2) a modified

[1]Senior Lecturer, Faculty of Civil and Environmental Engineering, Universiti Tun Hussein Onn Malaysia, 86400 Parit Raja, Batu Pahat, Johor, Malaysia (corresponding author). E-mail: david@uthm.edu.my

[2]Associate Professor of Structural Design, Dept. of Architecture, Design, and Urban Planning, Univ. of Sassari, Palazzo del Pou Salit, Piazza Duomo 6, 07041, Alghero, Italy. E-mail: fragiacomo@uniss.it

[3]Environmental Engineer, ADA Studio, Tolmezzo, Italy; formerly, Graduate Student, Dept. of Civil and Environmental Engineering, Univ. of Trieste, Italy. E-mail: mary.def@virgilio.it

[4]Professor of Timber Design, Dept. of Civil and Natural Resources Engineering, Private Bag 4800, Univ. of Canterbury, Christchurch 8140, New Zealand. E-mail: andy.buchanan@canterbury.ac.nz

Note. This manuscript was submitted on August 8, 2009; approved on July 23, 2010; published online on August 2, 2010. Discussion period open until July 1, 2011; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Structural Engineering***, Vol. 137, No. 2, February 1, 2011. ©ASCE, ISSN 0733-9445/2011/2-261–269/$25.00.

J. Struct. Eng. 2011.137:261-269.

steel tube with two welded flanges and a hexagon head coach screw in conjunction with a notch cut in the timber beam; and (3) a mechanical connector consisting of a pair of folded steel plates embedded into the concrete slab and connected to the glulam beam by means of annular ringed shank nails. Due to their simplicity and inexpensiveness, the first and third connector types were used in prefabricated TCC beams tested to failure (Lukaszewska et al. 2009). Nailplate or toothed metal plate is another connection system extensively used in timber construction due to the ease of assembly and reasonably good performance. Aicher et al. (2003) reported tests on two series of nailplates embedded in the concrete and nailed to the timber with equal width and length dimensions of 114×266 mm and compared the results with timber-timber connection. The conclusion was that a nailplate used in TCC beams is approximately 1.5 times stronger and 2.5–3 times stiffer than if used in timber-timber connections. Another metal plate type of connection such as a continuous steel mesh glued slotted into the timber beam was proposed by Clouston et al. (2005).

In recent years, a comprehensive investigation on TCC connections has started at the University of Canterbury, New Zealand with the aim to develop a semiprefabricated LVL-concrete composite systems for medium- to long-span floors (Yeoh et al. 2008a,b). Research carried out and ongoing includes performance of connections and full-scale beams in the short and long terms, susceptibility to vibrations, and tests under repeated loading behavior of such medium- to long-span floor system. The load-bearing capacity of composite systems markedly depends on the level of composite action that is developed by the shear connectors. Since the cost and constructability of the system depend on the ease of production of these shear connectors, it is crucial to develop connectors that are stiff and strong yet easy to manufacture and assemble. This paper reports the outcomes of short-term (failure) push-out tests carried out on the three best connectors selected based on the outcomes of previous research (Deam et al. 2007; Yeoh et al. 2009). The objective of these tests was the evaluation of the characteristic strength and stiffness (also termed as slip modulus) values to be used in design of composite floors. The paper reports the experimental program, discusses the experimental and analytical behaviors of the connectors, and presents an analytical model for the shear-strength evaluation of the connectors.

Background of the Short-Term Push-Out Research Program

The short-term push-out research program was carried out in different phases at the University of Canterbury from 2006 to 2008. The choice of using a notched connection reinforced with a lag screw was based on early work reported by Deam et al. (2007) where small LVL-concrete composite blocks incorporating different types of connectors were tested to failure and their performance compared to each other. Connectors investigated included round and rectangular concrete plugs with and without screw and steel pipe reinforcement, proprietary SFS screws (SFS 1999; SFS Intec 2010), lag screws with different diameters, sheet brace anchors, and framing brackets. The comparison clearly showed the best performance of the rectangular concrete plug reinforced with a lag screw. Such a connector type was then successfully implemented by Deam et al. (2008) in LVL-concrete composite beams, some of which were prestressed with unbonded tendons. Following those former investigations, Yeoh et al. (2009) carried out a

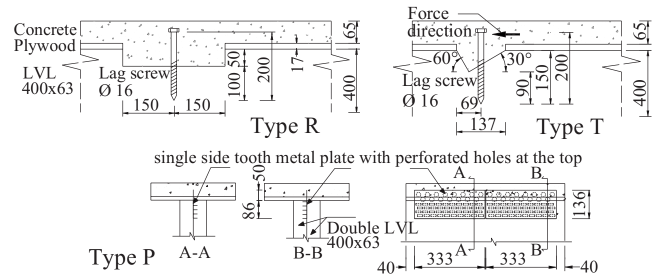

Fig. 1. Three types of connection (R, T, and P) tested in push-out tests (dimensions in millimeters)

parametric experimental study in order to investigate the effect of notch geometrical variations such as depth and length, presence of lag screw reinforcement, and size and penetration depth of lag screw on the strength and stiffness performance. Factors related to the ease of production, labor, and material costs were carefully considered in order to achieve an optimized notch shape that provides the best compromise between structural efficiency and labor cost. The next phase of the research, described in this paper, commenced at the end of year 2007 where three types of connections were chosen and tested in order to evaluate their characteristic strengths and mean slip moduli at serviceability and ULSs. Materials used were 400×63-mm LVL with a mean Young's modulus of 11.3 GPa [Carter Holt Harvey (CHH) 2007], Grade 35 normal weight, and 650 microstrain low shrinkage concrete with Eclipse admixture, both concrete with average densities of approximately $2{,}405\ \mathrm{kg/m^3}$ and obtained from a commercial batching plant, high tensile steel reinforcement, and 16-mm-diameter lag screws. A three-dimensional finite-element model of the selected connections using ANSYS software package (ANSYS 2007) is currently under development at the University of Stuttgart, Germany, with the purpose to predict the strength and stiffness of the connections (Yeoh et al. 2008a).

Experimental Program

The three best types of connection displayed in Fig. 1 were tested in shear to determine their characteristic strength and slip moduli: (1) $300(l) \times 50(d) \times 63(w)$-mm rectangular notch reinforced with one 16-mm-diameter lag screw (Type R); (2) 30 and 60° $137(l) \times 60(d)$-mm triangular notches reinforced with one 16-mm-diameter lag screw (Type T); and (3) two $333(l) \times 136(d) \times 1(t)$-mm staggered toothed metal plates (Type P), where l, d, w, and t=length, depth, width, and thickness, respectively. Nine specimens were tested per type of connection while another three specimens with the triangular notched connection were built and tested in the weak direction, i.e., with the notch inverted (60 and 30°) with respect to the direction of the shear force (Type TT). A total of 30 specimens were constructed. The cutting of the notches in the LVL, fabrication of interlayer plywood as permanent formwork, and other edge forms were done manually in the laboratory as oppose to an automated machine cut which is only practical for an industrial manufacturing scale. Fig. 2 shows the test setup of the symmetrical push-out test carried out in accordance with EN 26891 [Comité Européen de Normalisation (CEN) 1991] under a 1,000-kN Avery make universal testing machine. It must be pointed out that Specimens Types R, T, and TT had only one LVL joist, whereas Specimens Type P had two LVL joists to sandwich the two toothed metal plates (see Fig. 1). The connections were

J. Struct. Eng. 2011.137:261-269.

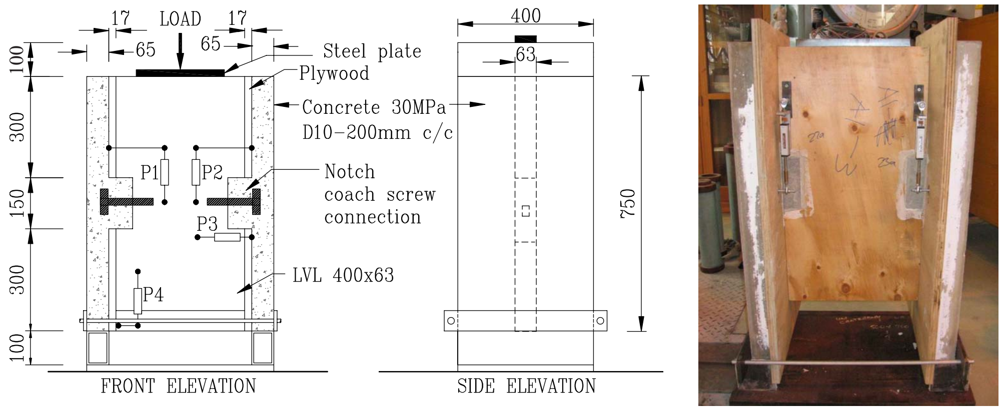

Fig. 2. Symmetrical push-out test setup (dimensions in millimeters)

loaded in shear and the load-slip relationship recorded using a load cell and 50-mm potentiometers (P1, P2, P5, and P6) (Fig. 2, Potentiometers P5 and P6 are at the same location as P1 and P2 but on the opposite face of the specimen). The connections were loaded at a rate of $0.2F_{est}$ kN/min in shear with the load applied onto the LVL web section of the specimen until the connection failed. The loading protocol requires an initial estimate of the strength of the specimen F_{est} which was determined on the basis of experience, preliminary tests, or calculation. This was then adjusted for the second specimen using the new actual F_{est} from the first tested specimen. The specimen was first loaded to $0.4F_{est}$ and held for 30 s, then unloaded to $0.1F_{est}$ and maintained for 30 s. Thereafter the specimen was loaded to failure or to a maximum slip of 20 mm, whichever occurred first. The purpose of the initial loading-unloading phase was to eliminate any internal friction in the connection and to ensure that any initial slip or slack present in the connection does not affect the final results. The slip measurements were recorded for each test specimen using potentiometers that were mounted adjacent to the connections. The slip at maximum load F_{max}, defined as the shear strength, was also recorded.

Results

EN 26891 [Comité Européen de Normalisation (CEN) 1991] provides specifications for the derivation of the connection shear strength and secant slip moduli at 40% [assumed as the serviceability limit state (SLS) load level], 60% (assumed as the ULS load level), and 80% (near the collapse load level) of the shear strength. Load-slip curves are presented for all connections in Fig. 3. Table 1 shows the average values of shear strength (F_{max}) and secant slip moduli at 40% ($K_{s,0.4}$), 60% ($K_{s,0.6}$), and 80% ($K_{s,0.8}$) of the shear strength. The standard deviation (σ), coefficient of variation (COV), and characteristic strength (R_k) at 5th percentile

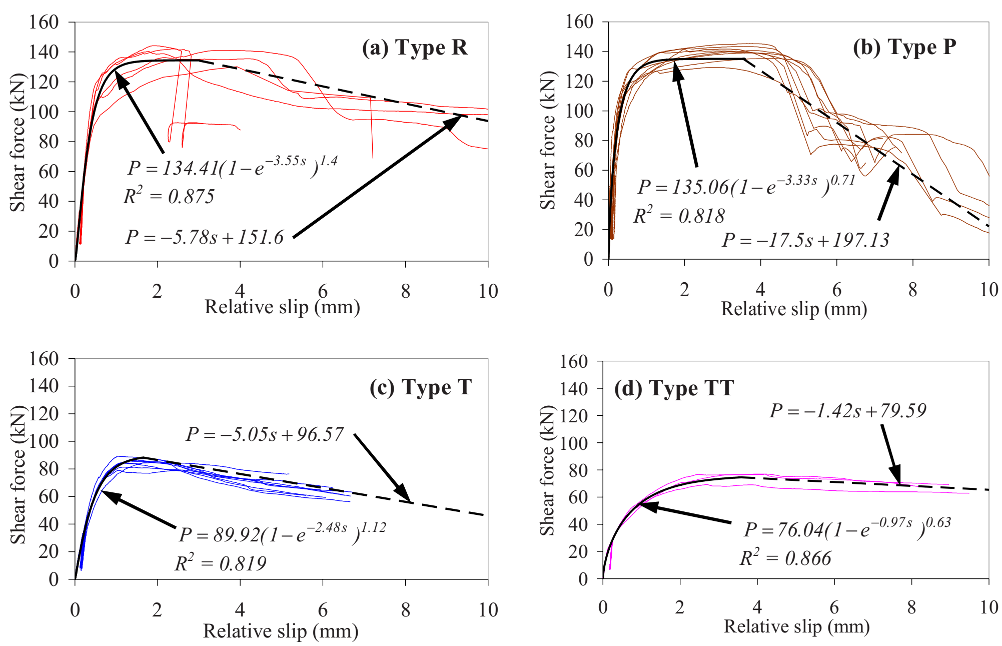

Fig. 3. Single connection experimental load-slip curves with analytical pre- and postpeak best-fit curves for connections: (a) Type R; (b) Type P; (c) Type T; and (d) Type TT

Table 1. Shear Strength and Secant Slip Moduli Values for a Single Connector

Type of connection (P_2/P_1%)	Values	Slip moduli $K_{s,0.4}$ (kN/mm)	$K_{s,0.6}$ (kN/mm)	$K_{s,0.8}$ (kN/mm)	Shear strength F_{max} (kN)
Type TT (1-LVL) (12.6%) ductile	Range	107.2–113.5	65.3–89.0	44.9–53.7	69.2–77.0
	Average [R_k]	109.8	78.9	50.4	74.3 [61.7]
	σ (COV%)	3.3 (3.0)	12.3 (15.6)	4.8 (9.5)	4.4 (6.0)
Type T (1-LVL) (49.7%) low ductile	Range	128.2–176.7	121.7–168.3	94.3–140.4	79.0–89.2
	Average [R_k]	145.8	138.8	115.9	84.8 [70.4] {2}
	σ (COV%)	13.5 (9.3)	12.7 (9.1)	12.1 (10.4)	3.1 (3.7)
Type R (1-LVL) (33.9%) fairly ductile	Range	216.9–286.0	205.4–282.2	113.7–258.8	130.1–144.2
	Average [R_k]	247.2	241.4	194.2	138.9 [115.3] {1}
	σ (COV%)	27.4 (11.1)	28.0 (11.6)	51.2 (26.4)	5.2 (3.7)
Type P (2-LVL) (80.7%) brittle	Range	249.3–589.5	239.3–510.6	182.3–362.6	129.3–145.4
	Average [R_k]	463.7 [2]	394.6	256.8	139.3 [115.6] {3}
	σ (COV%)	132.0 (28.5)	100.3 (25.4)	63.1 (24.5)	5.0 (3.6)

Note: { } strength rank for 2-LVL; [ ] adjusted by 0.83 for characteristic strength R_k.

for each tested connection are also reported. In order to quantify the postpeak behavior and the type of failure (whether ductile or brittle), the ratio P_2/P_1 is introduced, where P_2 refers to the difference of strength at peak and at 10-mm slip and P_1 the strength at peak, as reported in Table 1. The lower the P_2/P_1 ratio, the better the postpeak behavior and the higher the ductility. For definition purposes, a ratio below/above 50% would be considered as a fairly ductile/brittle connection.

Discussion

Connection Behavior

Similar mode of failures and behavior was observed for all the tested connections, as reported in Yeoh et al. (2009). Both the rectangular and triangular notched connections failed primarily due to the shear in concrete along the shear plane while plate tearing occurred in the toothed metal plate connection (Fig. 4). The lag screw enhanced the postpeak behavior of the rectangular and triangular connections as expressed by the calculated P_2/P_1 ratio (P_2/P_1=33.9 and 49.7%, respectively) in Table 1 and in the shear force versus relative slip plots [Figs. 3(a, c, and d)]. On the contrary, the toothed metal plate connection exhibited a ratio P_2/P_1=80.7% characterized by brittle behavior, as can be observed in Fig. 3(b) where a sudden load reduction after the attainment of the peak load is evident. Such a value of the P_2/P_1 ratio disagrees with the outcomes of the preliminary tests presented in Yeoh et al. (2009) where the range of 33–44% was obtained. Such a difference could be attributed to the reduction of the plate thickness from 2 to 1 mm, the use of single sided teeth instead of double sided teeth, and the use of two separate staggered plates instead of a continuous plate.

Tearing of the plate was the failure mechanism detected in the toothed metal plate connection. The high slip recorded for this connection could be attributed to the progressive tearing of the plate and possibly the slippage of the teeth from the LVL [Fig. 3(b)]. The ultimate load was reached with a relatively gradual increase of strength due to the steel plate ductility and the reinforcement in the concrete slab. Subsequently, two load reductions were observed for almost all the tested specimens corresponding to the progressive tearing of the first plate and then of the second plate.

The rectangular notched connection resulted in the stiffest connection with the additional benefit of a fairly ductile behavior due to the presence of the lag screw. However, the behavior of the specimens was not homogeneous due to the segregation of the concrete in some of the notches being a manufacturing defect due to insufficient compaction and brittle failure in the LVL in two specimens. As such, they were disregarded in the statistical evaluation of strength and slip moduli. The triangular notched connection showed less stiffness and strength. The maximum strength and stiffness decreased, whereas ductility improved when the triangular notched connection was tested in the opposite or weak direction (Type TT specimens). The triangular notched connection was also found to possess a relatively stable postpeak behavior as observed in a gradual and uniform reduction of load after failure for all the tested specimens [Fig. 3(c)].

The degree of composite action of all the connection types (R, T, and P) in a TCC floor beam was assessed in a separate research program reported in detail in Yeoh et al. (D. Yeoh, et al., "Experimental limit state behavior of LVL-concrete composite floor beams," J. Eng. Struct. Elsevier, submitted, 2010). In this program, a total of 11 8–10-m TCC floor beams with a minimum of six numbers of connections along the span were designed, constructed, and tested to collapse under four-point bending. The experimental degree of composite action was quantified as percentage as given in Eq. (1), 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);

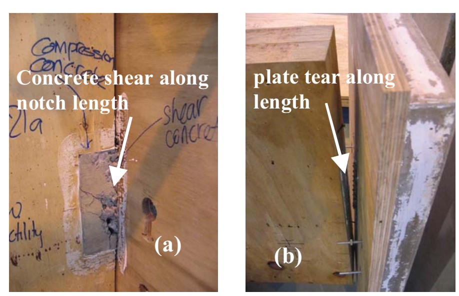

Fig. 4. (a) Connection Type R—concrete shear along length; (b) Connection Type P—plate tearing

Table 2. Compressive Strength of Concrete

Cast	f_c (N/mm^2)	$f_{c,\mathrm{ave}}$ (N/mm^2)	R (N/mm^2)	σ (N/mm^2)
1	45.1	45.4	0.51	0.45
	45.7			
2	46.5	47.4	1.66	1.48
	48.2			
3	43.6	42.2	2.81	2.50
	40.8			
Mean value		45.0		1.48

and Δ_F, measured experimentally, signifies the deflection of the composite beam with the actual flexible connection (Gutkowski et al. 2008)

$$DCA = \frac{\Delta_N - \Delta_F}{\Delta_N - \Delta_R} \times 100 \qquad (1)$$

All the beams exhibited a high level of composite action at service limit state between 86.8 and 99.9%. Only a variation of less than 10% was found when compared with the analytical degree of composite action calculated using the gamma method (Ceccotti 1995). This compares to a composite action of between 54.9 and 77% achieved by Gutkowski et al. (2008) who tested multiple 3.51-m span TCC layered beams connected with notch shear/key anchor details. The TCC beams built from Connection Types R, T, and P which have been designed to fulfill all ultimate and SLS inequalities showed a high degree of composite action within less than 10% to those of a fully composite beam. Analytical-experimental comparisons of load capacity at ULS and SLS in the short term were also reported in literature (D. Yeoh et al. 2010, unpublished).

Characteristic Strength R_k

The characteristic strength of the connection R_k derived in Table 1 is required for the design of a TCC beam at ULS. Here, only nine specimens per connection type were used to determine the characteristic strength as compared to the conventional practice of 30 specimens because of budget restrictions. Since the notched connection types failed by shear in the concrete, a ratio which relates the characteristic strength of the connection to the characteristic compressive strength of the concrete calculated using the mean strength obtained from the concrete cylinder tests is proposed based on the Australia-New Zealand concrete standard (SNZ Standards New Zealand 2006). Concrete is assumed to follow a normal distribution and due to its homogeneity as opposed to timber in the statistical analysis, the level of confidence is assumed to be 90% (while for timber it is 75%). From the concrete compressive cylinder tests performed on three series of two cylinders (corresponding to three concrete castings), a *ratio coefficient* for the prediction of the characteristic strength of a notched connection is determined. These values when compared with characteristic strengths derived using the approach suggested by Section D7.2, Eurocode 0 [Comite European de Normalisation (CEN) 2002] whether using a 5% characteristic factor for 9 or 30 specimens, are more conservative. As such, the use of nine specimens for the determination of characteristic strength is considered as an acceptable compromise between accuracy and cost.

The procedure is summarized in Table 2, where (1) range R = difference between the compressive strengths of the two specimens f_c at 28 days and (2) standard deviation σ is estimated as $0.89 \times R$ on the basis of two test specimens. The characteristic compressive strength of concrete can then be calculated as $f_{c,k} = f_{c,\mathrm{ave}} - 5.31\sigma = 37.2$ N/mm^2, where the coefficient $k = 5.31$ relates to the number of concrete castings (three castings in our case) tested (Owen 1963). Hence, the ratio coefficient of *0.83* is calculated by dividing the characteristic compression strength of concrete $f_{c,k}$ by the mean value of the compressive strength $f_{c,\mathrm{ave}}$. The ratio is expected to get close to 0.9 with a high number of specimens. This ratio coefficient, multiplied by the mean strength of the connection, provides the characteristic strength of the notched connections R_k presented in Table 1. Although the failure of the toothed metal plate connection was not triggered by shear in the concrete, the same ratio coefficient was conservatively used to derive the characteristic strength considering that metal plate is a material with much lower variability in strength compared to concrete. These characteristic strengths can then be used for the design of TCC beams.

Strength and Slip Moduli Comparisons

In order to make a direct comparison with the toothed plate connection which had a double LVL joist, the strength and slip moduli for the rectangular and triangular notched connections (Types R and T) were doubled based on preliminary push-out tests which involved push-out tests of a single notch with single LVL and a double notch (two times the width of a single notch) with double LVL (Yeoh et al. 2009). In this preliminary test, it has been found that the strength of a double notch was approximately two times the strength of a single notch while the slip moduli $K_{s,0.4}$, $K_{s,0.6}$, and $K_{s,0.8}$ were 2.71, 2.43, and 1.93 times larger, respectively (Table 3). As such, in terms of strength, the rectangular notched connection ranks the first ($115.3 \times 2 = 230.6$ kN), second the triangular notched connection ($70.4 \times 2 = 140.8$ kN), and third the toothed metal plate connection (115.6 kN), as presented in Table 1. The COV for strength was found to be in the range of 3.6–6%. However, the COV for the slip moduli exceeded 10% in most cases with the plate connection having a COV between 24.5 and 28.5%. By referring to the slip modulus at SLS ($K_{s,0.4}$), the triangular notched connection ranks the last

Table 3. Push-Out Tests on Single and Double Notched Connections (This Table Was First Published by The Institution of Engineers, Australia in the *Australian Journal of Structural Engineering* [Yeoh et al. (2009), Vol. 9(3), pp. 225–238]; It Is Reprinted with Permission)

Type of connection	$F_{\max}$ (kN)	$K_{s,0.4}$ (kN/mm)	$K_{s,0.6}$ (kN/mm)	$K_{s,0.8}$ (kN/mm)
Rectangular notch single LVL $150(l) \times 50(d) \times 63(w)$-mm coach screw ϕ16	73.0	80.2	75.4	61.7
Rectangular notch double LVL $150(l) \times 50(d) \times 126(w)$-mm coach screw ϕ16	128.2	217.9	183.1	119.1
Ratio of double/single notch	1.76	2.72	2.43	1.93

J. Struct. Eng. 2011.137:261-269.

Table 4. Comparison of Mean Strengths and Secant Slip Moduli for Different Connectors

Type of connection	Slip moduli (kN/mm) $K_{s,0.4}$	$K_{s,0.6}$	Shear strength F_{max} (kN)
ϕ16 lag screw only	29.0	6.3	46.4
150(l)-mm notch only	104.7	59.3	48.3
150(l)-mm NLS ϕ12	77.9	74.5	66
150(l)-mm NLS ϕ16	80.2	75.4	73
300(l)-mm NLS ϕ16	247.2	241.4	138.9

Note: NLS=notched connection reinforced with lag screw; l=length of the notch.

($145.8\times2=291.6$ kN/mm), second the plate connection (463.7 kN/mm), and first the rectangular notched connection ($247.2\times2=494.4$ kN/mm). A doubled triangular notched connection exhibited a slightly higher strength than the toothed plate connection but only approximately half the slip moduli of the toothed plate connection.

The length of the notch significantly affects the shear strength of the notched connection while the presence of a lag screw maintains a good stiffness after the attainment of the SLS load level ($0.4F_{max}$) and provides ductility in the postpeak stage. The 300-mm-long rectangular notched connection is by far the best connection in terms of strength, slip moduli, and postpeak behavior. On the other hand, the toothed metal plate connection has the advantage of not requiring any cut of the timber and thereby may allow the achievement of speed in construction and reduction in labor cost. Although the triangular notched connection is not as strong and stiff as the rectangular notch, it is nevertheless characterized by simpler construction which involves only two cuts of the timber. Such connection may be preferred particularly when computer-aided cutting machines are not available.

Influence of Lag Screw and Length of Notch on the Connection Performance

In order to investigate the effect of cutting a rectangular notch in the LVL, two additional push-out specimens with only one lag screw in each connection and no notches were built and tested. Comparisons were made between this connection, a $150(l)\times50(d)$-mm rectangular notch without lag screw, the same notch with 12- and 16-mm-diameter lag screws, and a $300(l)\times50(d)$-mm rectangular notch with a 16-mm lag screw, as reported in Table 4. The connection without a notch was the weakest in strength and stiffness while the 300-mm notch with lag screw connection was the strongest. The 300-mm notched lag screw connection was 3 times stronger and 8.5 times stiffer than the connection with just a lag screw. Hence, the importance of the concrete notch is emphasized here as a major contributor to both strength and stiffness. The absence of a notch brought about an approximately 80% reduction in slip modulus from 29.0 kN/mm at SLS ($K_{s,0.4}$) to 6.30 kN/mm at ULS ($K_{s,0.6}$). 20 and 40% reductions in slip modulus at ULS ($K_{s,0.6}$) and strength, respectively, were evident in the notch without a lag screw with respect to the same notch reinforced with a lag screw. The notch without a lag screw only achieved 80% of the stiffness of one reinforced with a lag screw. The 300-mm rectangular notch was found to be 1.9 times stronger and 3 times stiffer than the 150-mm rectangular notch. There was no significant difference in terms of strength and stiffness by changing the size of lag screw from 16 to 12 mm diameter (Yeoh et al. 2009). Regarding the notch length, the increment of shear strength was found to be roughly linear while the increment of stiffness varied exponentially.

Analytical Approximation of the Shear-Slip Curves and Failure Mechanisms

The experimental shear force versus relative slip curves of each connection type tested were fitted with an average analytical curve comprising of a prepeak and a postpeak behavior (Fig. 3). The prepeak behavior fitted with the least-squares method is based on the nonlinear analytical model proposed by Ollgard et al. (1971) and described by Eq. (2), whereas the postpeak behavior is described by a linear curve with negative slope given by Eq. (3). The corresponding parameters are listed in Table 5. Such a nonlinear shear force-slip relationships can be used in advanced uniaxial finite-element beam models such as that developed by Fragiacomo et al. (2004) for nonlinear analyses to failure of TCC beams, as reported by Ceccotti et al. (2006)

$$P = P_{max}(1-e^{-\beta s})^{\alpha} \quad \text{for } s < s_p \tag{2}$$

$$P = as + b \quad \text{for } s_p < s < s_u \tag{3}$$

where s=relative slip; s_p=slip at P_{max}; s_u=maximum slip at the lower end of the postpeak curve; P_{max}=maximum load reached by the approximating curve; and α, β, a, and b=constants. It is important to remember that the toothed metal plate connection (Type P) is regarded as a double LVL while the other connections, triangular (Types T and TT) and rectangular notched (Type R, as a single LVL.

Fig. 5 shows the typical prepeak and postpeak behaviors of a tested connection, and Fig. 6 illustrates the corresponding failure mechanisms and behavior of a notched connection. In general, a shear plane begins to form at $0.6F_{max}$ as indicated by (1) in Figs. 5 and 6. At this stage, the concrete notch began to shear and crushing of concrete occurred due to the compression of the notched connection under the force of $0.6F_{max}$. As the load reaches the peak value, the concrete notch is almost completely sheared and the compression zone of the concrete becomes very obvious. Here, the lag screw starts to carry most of the load by

Table 5. Analytical Pre- and Postpeak Shear Forces versus Relative Slip Relationship for a Single Connector Corresponding to Eqs. (2) and (3)

Connector type	Prepeak behavior P_{max} (kN)	α	β (mm^{-1})	Postpeak behavior a (mm)	b (kN)	Slip s_p (mm)	s_u (mm)
TT	76.0	0.63	0.97	−1.42	79.6	3.58	10
T	89.9	1.13	2.48	−5.05	96.6	1.65	10
R	134.4	1.40	3.55	−5.78	151.6	3.01	15
P	135.1	0.71	3.33	−17.5	197.1	3.54	10

J. Struct. Eng. 2011.137:261-269.

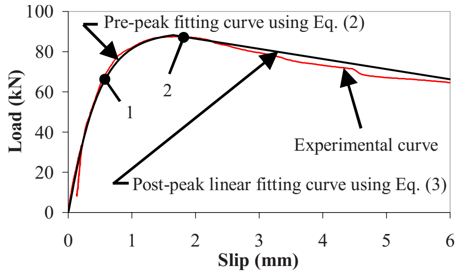

Fig. 5. Typical prepeak and postpeak behaviors

rope effect acting in shear and tension resulting in the formation of two flexural plastic hinges as the load decreases gradually with an increase in the slip [see (2) in Figs. 5 and 6]. The slope of the load descent highly depends on the size of the lag screw and the depth of the penetration in the case of a notched connection.

Derivation of Design Formulas for Strength of Rectangular Notched Connections Reinforced with Lag Screw

A simplified analytical model for strength evaluation of the notched connection is proposed in Eqs. (4)–(7). The notched connection is regarded as a concrete corbel protruding into the LVL joist subjected to shear coming from the shear load applied to the connection. The lag screw acts as reinforcement for the concrete corbel and contributes to the shear transfer from the timber to the concrete. The formulas were compared with the experimental results and were found to predict the failure load with acceptable accuracy in most cases. The model is based on the control of all possible failure mechanisms that may occur in the connection region [see also Kuhlmann and Michelfelder (2006)]: (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, as illustrated in Fig. 6 and discussed in the previous section. 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 Eurocodes method is quite conservative with the actual experimental results in between. 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.

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 (SNZ Standards New Zealand 1993) and concrete structures (SNZ Standards New Zealand 2006) based on the aforementioned four possible failure mechanisms of the notched connection

$$F_{\text{conc,shear}} = 0.2 f_c' bl + nk_1 pQ \tag{4}$$

$$F_{\text{conc,crush}} = f_c' A_c \tag{5}$$

$$F_{\text{LVL,shear}} = k_1 k_4 k_5 f_s L b \tag{6}$$

$$F_{\text{LVL,crush}} = k_1 f_c b d \tag{7}$$

where $F_{\text{conc,shear}}$=nominal shear strength of concrete for a notched connection reinforced with a lag screw; $F_{\text{conc,crush}}$=nominal compressive strength of concrete in the crushing zone; $F_{\text{LVL,shear}}$ =nominal longitudinal shear strength of LVL between two consecutive notches or between the last notch and the end of the timber beam; and $F_{\text{LVL,crush}}$=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. (4). A_c is the crushing zone effective area, i.e., $b \times d$ in Eq. (5), where d=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. (6). f_c is the LVL compressive strength parallel to the grain in Eq. (7). 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. (4)–(7) and by multiplying the minimum among the four values reported above by the strength reduction factor ϕ.

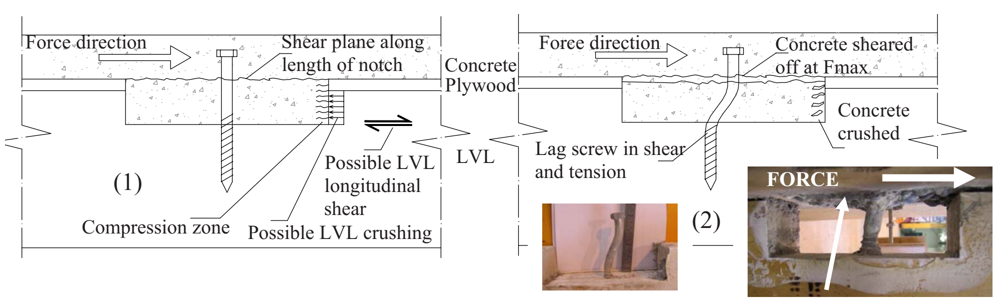

Fig. 6. Experimental failure mechanisms and behavior of a notched connection reinforced with a lag screw

Table 6. Experimental-Analytical Comparison of Connector Shear Strength

Type of connection	Mean strength (kN)			
		Analytical method		
	Experimental	NZS	EC	EC*
TT	74.3	94.0	70.7	83.4
T	84.8	94.0	70.7	83.4
R	138.9	186.4	99.1	140.3

Strength Evaluation Model according to Eurocodes (EC Method)

Based on the Eurocodes for both timber [Comite European de Normalisation (CEN) 2004b] and concrete structures [Comite European de Normalisation (CEN) 2004a], the shear strength of concrete for a notched connection reinforced with a lag screw when modeled as a corbel can be calculated using the following formula:

$$F_{\text{conc,shear}} = \beta 0.5 b_n l_n \upsilon f_c + n_{ef}(\phi_{cs} d_{ef} \pi)^{0.8} f_w \tag{8}$$

where β=reduction factor of the shear force taken as 0.25 which corresponds to the loading distance from the edge of the support in the case of the notch treated as a corbel; b_n and l_n=breadth of the joist and the length of the notch, respectively; υ=strength reduction factor for concrete cracked in shear, assumed as 0.516; f_c=compressive strength of concrete; n_{ef}=effective number of lag screws, assumed equal to the actual number of screws in the notch if they are spaced enough; ϕ_{cs}=diameter of the lag screw; d_{ef}=pointside penetration depth less one screw diameter; and f_w=withdrawal strength of the screw perpendicular to the grain. The other three failure mechanisms are governed by design equations similar to Eqs. (5)–(7), the only difference being that the coefficients k_4 and k_5 are replaced by k_{sys} being modification factor for system strength, which is assumed as 1.0 for LVL, and the coefficient k_1 is replaced by k_{mod} which refers to the equivalent 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 in the design equations and by taking the minimum of the four values of design strengths so obtained.

Reduction Factor β^* Method (EC Method)*

A new reduction factor β^* given in Eq. (9) was introduced as to replace the existing reduction factor β in Eq. (8) 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}. This method was found to be in close proximity with the experimental mean strength values

$$\beta^* = \frac{l_n - 2\phi_{cs}}{2l_n} \tag{9}$$

Table 6 provides a comparison of the experimental mean strength for the rectangular and triangular notched connections with the three analytical strength evaluation methods. For all connector types, the governing design formula was found to be Eqs. (4) and (8) 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 the cases.

Conclusions

The paper reports the outcome of experimental push-out tests carried out on three connector types for LVL-concrete composite beams. The connectors were $300(l) \times 50(d) \times 63(w)$-mm rectangular notches cut in the LVL and reinforced with a 16-mm-diameter lag screw, 30 and 60° $137(l) \times 60(d)$-mm triangular notches reinforced with the same diameter lag screw, and two $333(l) \times 136(d) \times 1(t)$-mm toothed metal plates pressed on the lateral surface of the LVL joist, where l, d, w, and t=length, depth, width, and thickness, respectively. The aim of the push-out tests was to determine the characteristic values of the shear strength and the mean values of the slip modulus, which are important design properties. To this purpose, 30 symmetric push-out specimens were constructed and tested to failure. It was found that the length of the notch significantly enhances the strength performance of the connection while a lag screw improves the slip modulus at ULS, the postpeak behavior, and enables a more ductile failure to take place. The 300-mm notch reinforced with a lag screw is 3 times stronger and 8.5 times stiffer than a connection without a notch but just with the lag screw and 1.9 times stronger and 3 times stiffer than a 150-mm reinforced notch connection. The 300-mm-long rectangular reinforced notch connection stands out as the best connection among those tested due to the high strength and slip moduli, while the 2×333-mm toothed metal plate connection appeared to be the most practical and labor cost effective since it does not involve any notching. However, this connection system requires a readily available hydraulic press of industrial size for this system to be used in floor construction. On the other hand, the triangular notch reinforced with a lag screw has the advantage of easier and faster construction requiring only two cuts. None of the notched connections exhibited a brittle failure due to the use of the lag screw, whereas a brittle failure was observed in the toothed metal plate connection characterized by tearing of the plate.

Analytical prepeak and postpeak approximations for the load-slip relationship of three selected connections were presented and related to the failure mechanism and behavior of the connections. Considering four possible failure mechanisms, analytical formulas for the strength evaluation of the notched connection were derived according to New Zealand Standards and Eurocodes. The formulas were found to predict the experimental failure load with acceptable accuracy in all cases, with the closest agreement achieved when a new reduction factor was introduced in the Eurocodes formulas to take into account the length of the notch and the diameter of the lag screw. The failure in the notched connections is primarily due to shearing of the concrete in the shear plane. Therefore the characteristic strength of the three selected connections was calculated using a ratio coefficient of 0.83 derived statistically from the cylinder compressive strength test results.

Acknowledgments

The technical support from Carter Holt Harvey (Mr. Warwick Banks and Mr. Hank Bier), MiTek NZ (Mr. Steve Coll and Mr. Antony Cook), Firth Concrete (Mr. Dene Cook and Mr. Cliff

J. Struct. Eng. 2011.137:261-269.

Davis), and Dr. James Mackechnie is gratefully acknowledged, together with the financial contribution provided by Carter Holt Harvey and by the New Zealand government through the FIDA funds. Special thanks to research assistants Ms. Marta Mazzilli and Ms. Jennifer Haskell and technicians at the University of Canterbury, Mr. Norm Pilling and Mr. John Maley.

References

Aicher, S., Klock, W., Dill-Langer, G., and Radovic, B. (2003). "Nails and nailplates as shear connectors for timber-concrete composite constructions." *Otto-Graf J.*, 14, 189–209.

ANSYS. (2007). *Release 11.0 academic teaching advanced*, ANSYS Inc., Canonsburg, Pa.

Carter Holt Harvey (CHH). (2007). *New Zealand factory production data for LVL Truform recipe from January to May 2007*, CHH, Auckland, New Zealand.

Ceccotti, A. (1995). "Timber-concrete composite structures." *Timber engineering STEP 2*, 1st Ed., H. Blass et al., eds., Centrum Hout, The Netherlands, E13/1–E13/12.

Ceccotti, A., Fragiacomo, M., and Giordano, S. (2006). "Behavior of a timber-concrete composite beam with glued connection at strength limit state." *Proc., 9th World Conf. on Timber Engineering*.

Clouston, P., Bathon, L., and Schreyer, A. (2005). "Shear and bending performance of a novel wood-concrete composite system." *J. Struct. Eng.*, 131(9), 1404–1412.

Comite European de Normalisation (CEN). (2002). "Eurocode 0: Basis of structural design." *EN 1990*, Brussels, Belgium.

Comite European de Normalisation (CEN). (2004a). "Eurocode 2: Design of concrete structures—Part 1.1: General rules and rules for buildings." *EN 1995-1-1*, Brussels, Belgium.

Comite European de Normalisation (CEN). (2004b). "Eurocode 5: Design of timber structures—Part 1.1: General rules and rules for buildings." *EN 1995-1-1*, Brussels, Belgium.

Comité Européen de Normalisation (CEN). (1991) "Timber structures—Joints made with mechanical fasteners—General principles for the determination of strength and deformation characteristics." *EN 26891*, Brussels, Belgium.

Deam, B., Fragiacomo, M., and Buchanan, A. (2007). "Connections for composite concrete slab and LVL flooring systems." *Mat. and Struc., RILEM*, 41(3), 495–507.

Deam, B. L., Fragiacomo, M., and Gross, L. S. (2008). "Experimental behavior of prestressed LVL-concrete composite beams." *J. Struct. Eng.*, 134(5), 801–809.

Fragiacomo, M., Amadio, C., and Macorini, L. (2004). "A finite element model for collapse and long-term analysis of steel-concrete composite beams." *J. Struct. Eng.*, 130(3), 489–497.

Fragiacomo, M., Amadio, C., and Macorini, L. (2007). "Short- and long-term performance of the 'Tecnaria' stud connector for timber-concrete composite beams." *Mat. and Struc., RILEM*, 40(10), 1013–1026.

Gutkowski, R., Brown, K., Shigidi, A., and Natterer, J. (2004). "Investigation of notched composite wood-concrete connections." *J. Struct. Eng.*, 130(10), 1553–1561.

Gutkowski, R., Brown, K., Shigidi, A., and Natterer, J. (2008). "Laboratory tests of composite wood-concrete beams." *Constr. Build. Mater.*, 22(6), 1059–1066.

Kuhlmann, U., and Michelfelder, B. (2006). "Optimised design of grooves in timber-concrete composite slabs." *Proc., 9th World Conf. on Timber Engineering*.

Lukaszewska, E., Fragiacomo, M., and Johnsson, H. (2009). "Laboratory tests and numerical analyses of prefabricated timber-concrete composite floors." *J. Struct. Eng.*, 136(1), 46−55.

Lukaszewska, E., Johnsson, H., and Fragiacomo, M. (2007). "Performance of connections for prefabricated timber-concrete composite floors." *Mat. and Struc., RILEM*, 41(9), 1533–1550.

McCullough, B. (1943). "Oregon tests on composite (timber-concrete) beams." *Proc., American Concrete Institute*, 39, 429–438.

Ollgard, G., Slutter, G., and Fischer, W. (1971). "Shear strength of stud connectors in lightweight and normal weight concrete." *AISC Eng J*, 8, 55–64.

Owen, B. (1963). *Handbook of statistical tables*, Addison-Wesley, Reading, Mass.

Pillai, S. U., and Ramakrishnan, P. (1977). "Nail shear connectors in timber-concrete composites." *IE(I) Journal-CI*, 58, 34–39.

Pincus, P. (1970). "Behaviour of wood concrete composite beams." *J. Struct. Div.*, 96(7), 2009–2019.

Richart, F. E., and Williams, C. B. (1943). "Tests of composite timber-concrete beams." *Proc., American Concrete Institute*, 39, 253–262.

SFS. (1999). *SFS Verbundsystem VB Holz Beton Verbund (SFS connection system for timber-concrete composite structures)*, SFS Unimarket AG, Heerbrugg, Switzerland, PS50099D.

SFS Intec. (2010). "Fasteners for wood." ⟨http://www.sfsintec.biz/internet/sfsinten.nsf/PageID/fasteners_wood⟩ (December 2010).

SNZ Standards New Zealand. (1993). "Design timber structures." *NZS 3603*, New Zealand.

SNZ Standards New Zealand. (2006). "Concrete structures standard, Part 1: The design of concrete structures." *NZS 3101: Part 1*, New Zealand.

Yeoh, D., Fragiacomo, M., Aldi, P., Mazzilli, M., and Kuhlmann, U. (2008a). "Performance of notched coach screw connection for timber-concrete composite floor system." *Proc., 10th World Conf. on Timber Engineering*.

Yeoh, D., Fragiacomo, M., Ghafar, H., Buchanan, A., Deam, B., and Crews, K. (2008b). "LVL-concrete composite floor systems: An effective solution for multi-storey timber buildings." *Proc., Australasian Structural Engineering Conf.*.

Yeoh, D., Fragiacomo, M., Buchanan, A., and Gerber, C. (2009). "Preliminary research towards a semi-prefabricated LVL-concrete composite floor system for the Australasian market." *Australasian J. Struc. Eng.*, 9(3), 225–238.

J. Struct. Eng. 2011.137:261-269.