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Experimental behaviour of LVL–concrete composite floor beams at strength limit state

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ABSTRACT

This paper reports the outcomes of short-term collapse tests performed on eleven laminated veneer lumber (LVL)–concrete composite floor T-beams. Different variables such as span length (8 and 10 m), connection and concrete types, and design level (well- and under-designed, in terms of connector numbers) were investigated. During 4-point bending tests, mid-span deflection, connection slips and strains were measured. Connection types investigated include triangular and rectangular (150 mm and 300 mm long) notches cut in the timber and reinforced with a coach screw, and modified toothed metal plates pressed on the edge of the LVL joists. All of the beam specimens were designed using the effective bending stiffness or γ-method, in accordance with Annex B of Eurocode 5. The same method was used for an analytical–experimental comparison of the beam's performance at ultimate (ULS) and serviceability (SLS) limit state.

All well-designed beams provided more than 95% composite action even though there were relatively few connectors (e.g. six 300 mm long notches on the 8 m span beam). The ULS and SLS live load capacity of the beams was found to be approximately 90% of that of a fully composite beam. Correction factors providing a 15% increase for deflection and a 13% reduction of the effective bending stiffness are proposed for calculations using the transformed section method for all well-designed beams, i.e. beams designed using the γ-method according to Annex B of Eurocode 5. Although the γ-method was found to be significantly underestimate the ULS strength, it provided an accurate prediction of the short-term deflection. In terms of the connection type, the 300 mm rectangular notches provided the best performance, with high stiffness and strength beyond the ULS load level, and requiring fewer connectors along the beam. The triangular notch was found to be a viable alternative, with more connectors but was easier and faster to cut than a rectangular notch. Metal plate connectors provide a practical construction possibility, but the beam stiffness was found to rapidly deteriorate beyond the ULS load level.

1. Introduction

Timber–concrete composite (TCC) systems are a construction technique used to improve the strength and stiffness of existing timber floors as well as for new construction such as multi-storey buildings and short-span bridges. The combination of the two materials, exploits their best qualities, with the timber positioned in the tension region of the composite section and the concrete in the compression region. The presence of timber, due to its lower density in comparison with reinforced concrete, decreases the total weight of this flooring system, giving several advantages over reinforced concrete floors, including better efficiency in terms of load for a given self-weight, better seismic performance, and a lower carbon footprint. Compared to a wood-only floor, the concrete topping increases the thermal mass and fire resistance, improves the acoustic separation, and enhances the in-plane rigidity, which is particularly important in seismic regions. All the aforementioned advantages can be achieved only if the composite system is structurally effective, with a stiff and strong shear connection system. A wide range of connection systems is available, each with a different level of rigidity [1]. Seven types of connectors were tested in shear by Lukaszewska et al. [2], out of which the best two systems were chosen to build five fully prefabricated TCC floors tested to failure under 4-point bending [3,4].

A semi-prefabricated LVL–concrete composite system has been developed at the University of Canterbury, New Zealand, comprising "M" section panels built with laminated veneer lumber (LVL) beams acting as floor joists and a plywood interlayer as

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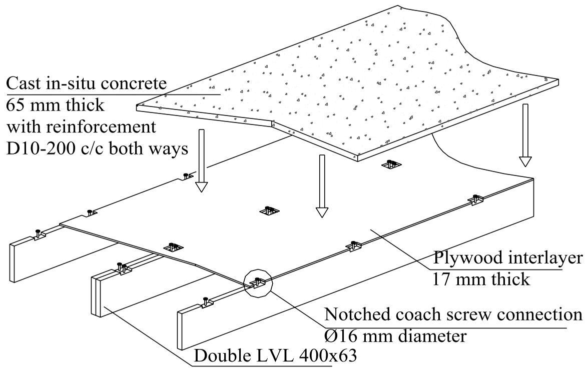

Fig. 1. Proposed semi-prefabricated LVL–concrete composite system (dimensions in mm).

permanent formwork (Fig. 1). The panels can be prefabricated off-site and then transported to the building site, craned into position and connected to the main frame with specially designed joist hangers [5,6]. The steel mesh is laid above the panels to provide shrinkage control for a 65 mm thick cast in situ concrete slab. The panels can be either propped while the concrete cures or alternatively pre-cambering the LVL joists to minimize deflection. Such pre-cambering effect is only possible during the cutting of the LVL in the manufacturing process. The connection system has notches cut from the LVL joist and reinforced with coach screws to increase the shear strength and provide more ductile behaviour. These notches are cut in the beams before the plywood interlayer is nailed on. The outcomes of the experimental push-out test carried out on different shear connectors is described in [7,8], whilst tests to failure of TCC beams prestressed with unbounded tendons are discussed in [9]. The design of LVL–concrete composite system is discussed at length in [5].

This paper reports the outcomes of the experimental tests to failure performed on eleven full-scale T-beams that were representatives of semi-prefabricated LVL–concrete composite floor strips. The specimens were 8 and 10 m long, and had different connection systems. The experimental results are critically discussed and compared with an analytical design method that accounts for the flexibility of the connection system.

2. Concept of composite action

The interconnection of a timber beam web member with an upper concrete flange produces a degree of composite action as illustrated in Fig. 2. Two extreme limits of stiffness can be identified: (1) a lower limit, termed 'no composite action' as in Fig. 2(c), where there is no horizontal shear force transfer between the two layers, which results in large interlayer slip and deflection; (2) an upper limit, termed 'fully composite action' as in Fig. 2(a), with complete shear force transfer between the two layers, no interlayer slip and a small deflection. The flexural behaviour of a real composite system is usually intermediate between these two limits, and termed 'partial composite action'. In this case, the amount of interlayer slip and deflection in the composite beam are significantly affected by the strength and stiffness of the interlayer connection system. The degree of composite action (DCA) can be 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); Δ_F, measured experimentally, signifies the deflection of the composite beam with the actual flexible connection [10].

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

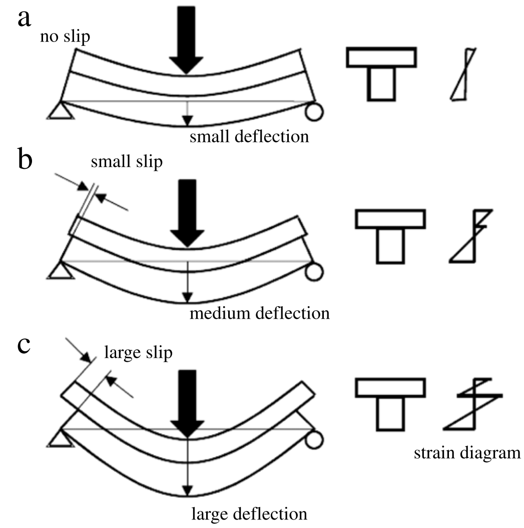

Fig. 2. Flexural behaviour of composite beam: (a) full composite action; (b) partial composite action; (c) no composite action.

The maximum *DCA* is desirable to increase the stiffness and the strength of the composite beam. This may require, however, the use of many connectors, and an uneconomical system. A compromise between structural efficiency and cost must therefore be found. The desired characteristics for the proposed semi-prefabricated LVL–concrete composite system are: (1) medium to long span, from 6 to 12 m, (2) the minimum number of connectors, to minimize construction cost, (3) high *DCA*, and (4) acceptable deflection in the long term. The choice of a strong and stiff connection is therefore crucial to meet these requirements, since stiffer connections reduce the deflection of the composite system. Very stiff connections, in fact, ensure complete composite action of the timber beam and concrete slab, with little or no slip at the beam–concrete interface and a small deflection.

3. Experimental programme

3.1. Beam specimens

The 'M' section semi-prefabricated LVL–concrete composite system had 2400 mm breadth and was built with a single 400 × 63 mm LVL joist on each outer edge and a double LVL joist in the centre (Fig. 3(a)). The M section was reduced to the inner 'T' section, made from a double LVL joist with a 1200 mm wide flange shown within the broken lines in Fig. 3(a) and alone in Fig. 3(b). This 'T' section was further scaled down to a single LVL joist with a 600 mm wide flange for the test beams (Fig. 3(c)). Each beam specimen was designed and constructed by careful selection considering a number of parameters: (1) the type of connection, (2) the number of connectors, (3) the span length, (4) the type of construction, and (5) the type of concrete. Construction variables include the number of days the prop was left in place at mid-span (0, 7 and 14 days), and whether the notches were cast at the time of the concrete placement or grouted 7 days later (in the case of beam A2, see Table 1).

Eleven beam specimens were designed for 8 and 10 m spans, built, and tested to collapse under 4-point bending load. Table 1 provides a description of the beam specimens. Beam G1 was a reference beam built from double LVL joists and a 1200 mm wide flange (Fig. 3(b)) while all other beam specimens had single LVL joist and 600 mm wide flange (Fig. 3(c)). Beam F1 was an exception as it required a pair of LVL joists to sandwich the toothed metal plate connections, giving a double LVL section with 1200 mm

Table 1
Details of the beam specimens tested to collapse.

Beam specimen	Span and {Flange breadth} (m) (mm)	Number and type of connectors	Design level	Time the prop was left in place
A1 (indoor)	8 {600}	6-R150	Under	7d
A2 (indoor)	8 {600}	6-R150	Under	10d
B1 (indoor)	8 {600}	10-R150	Well	7d
B2 (indoor)	8 {600}	10-R150	Well	7d
C1 (outdoor)	8 {600}	10-T	Well	7d
C2 (indoor)	8 {600}	10-T	Well	No
D1 (outdoor)	8 {600}	6-R300	Well	7d
E1 (indoor)	10 {600}	6-R300	Under	7d
E2 (indoor)	10 {600}	6-R300	Under	7d
F1 (outdoor)	8 {1200}	8-P	Under	7d
G1 (indoor)	8 {1200}	10-R150	Well	7d

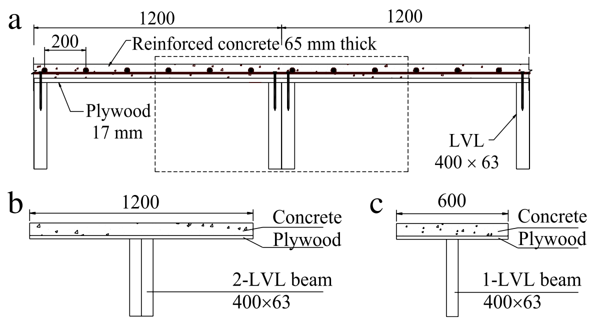

Fig. 3. (a) Semi-prefabricated "M" section panel; (b) Reduced T section; (c) Further reduced T section (dimensions in mm).

flange (Fig. 3(b)). Eight beams (A1, A2, B1, B2, C2, E1, E2, G1) were constructed indoors and three beams (C1, D1, F1) outdoors. Beams A1, B1, B2 and C2 were first subjected to the quasi-permanent service load $G + 0.4Q$ according to the AS/NZ Standard [11] for 3 months prior to a collapse test that was part of a separate long-term behaviour investigation [12]. Beam A2 was cast with pocket notches which were grouted on day 7 with high strength low shrinkage SIKA 212 grout [13]. The prop on this beam was removed at day 11 (only 3 days after the pocket grouting) when the manufacturer's specifications indicated that the grout would have achieved sufficient strength.

Four types of connectors (Fig. 4) were used to construct the composite beam specimens: (1) 150 mm long rectangular notches reinforced with a coach screw (R150), (2) 300 mm long rectangular notches reinforced with a coach screw (R300), (3) Triangular notches reinforced with a coach screw (T), and (4) Modified toothed metal plates (one pair) pressed in the edge of the LVL joist (P). These connectors were chosen on the basis of the outcomes of a parametric experimental study, which included push-out tests to failure carried out on 15 different connector types [12]. The average and characteristic shear strengths, R_m and R_k, and secant slip moduli $K_{0.4}$, $K_{0.6}$, and $K_{0.8}$ at 40%, 60% and 80% of the collapse shear load, respectively, are given in Table 2 as a result of the push-out tests to failure carried out on the four connector types [7].

Table 2
Average shear strength and secant slip moduli values for a single connector [7].

Type of connection	Secant slip moduli (kN/mm)			Shear strength (kN)	
	$K_{0.4}$	$K_{0.6}$	$K_{0.8}$	R_k	R_m
R150 (1-LVL)	80.2	75.4	61.7	60.6	73
T (1-LVL)	146	139	116	70.4	84.8
R300 (1-LVL)	247	241	194	115	139
P (2-LVL)	464	395	257	115	139

All of the beams were designed at ultimate (ULS) and serviceability (SLS) limit state in accordance with the design procedure suggested by Ceccotti [1]. This procedure, also known as the 'γ-method', is based on the use of the formulae for composite beams with flexible connections provided by Annex B of the Eurocode 5 [14] for the evaluation of the effective bending stiffness. Each quantity is calculated using the secant slip moduli $K_{0.4}$ and $K_{0.6}$ for ULS and SLS verifications in the short term, respectively. For verification of the long-term performance, the effect of creep is accounted for by dividing the elastic moduli of the concrete and LVL, and the slip modulus of the connector, by a factor of one plus the corresponding creep coefficient, in accordance with [5]. The characteristic shear strength R_k is used for ULS verifications of the connection. For each beam configuration, two different numbers of connectors were identified, with each number corresponding to a design level: well- or under-designed, depending on whether the ULS and SLS performances were satisfied or not. The most critical design criterion for the well-designed beams was SLS in the long term, followed by shear strength of connection at ULS in the short and long term. In the under-designed beams, the demand of shear force in the most stressed connector was about 30% more than the design resistance at ULS, for both short and long terms. The design imposed load was $Q = 3$ kN/m^2 for office buildings and the total permanent load was $G = G_1 + G_2 = 3$ kN/m^2, where $G_1 = 2$ kN/m^2 and $G_2 = 1$ kN/m^2 for the self-weight and the superimposed permanent load, respectively. The design level variations were used to investigate the actual strength and composite action achievable by the beam specimens, and to verify the accuracy of the analytical γ-method used in design.

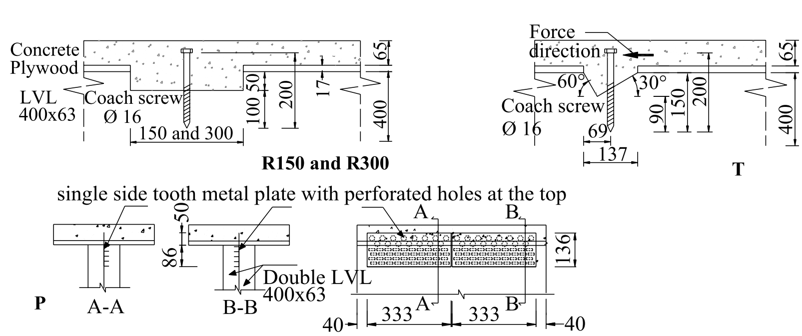

Fig. 4. Four types of connectors used to construct the composite beam specimens (dimension in mm) [7].

Table 3
Experimental mean properties of concrete.

Beam specimens	Concrete type	Slump (mm)	Shrinkage at 28d ($\times 10^{-6}$)	Compressive strength (MPa)	
				At 28d	At day of beam test
A1, A2, B1, C2	CLSC G35	150	436	49.6	58.0
C1, D1	CLSC G35	170	512	42.6	54.4
E1, G1	CLSC G35	190	667	41.5	48.2
F1	CLSC G35	220	–	43.4	54.4
B2	SLSC G35	100	474	28.0	38.8
E2	NWC G25	200	602	25.4	31.0
Pocket	SIKA 212		394	–	80.3

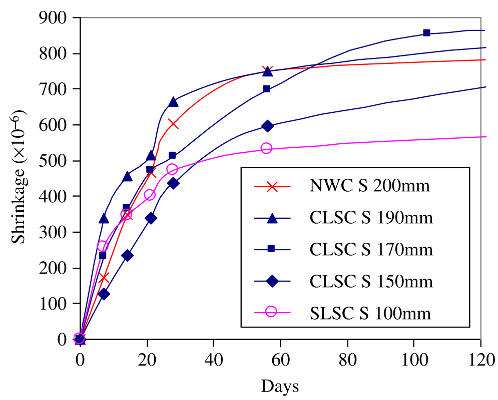

Fig. 5. Shrinkage of concrete mixes with different slump (S).

3.2. Materials

Three different types of concrete were carefully selected as shrinkage was expected to cause significant deflection of the composite beam in the long term, due to the high stiffness of the connection. A commercially available low shrinkage concrete (CLSC) was used for all the beams apart from beams B2 and E2 which were built using, respectively, a special low shrinkage concrete (SLSC) and a normal weight concrete (NWC). Both CLSC and NWC were supplied by a commercial batching plant. The CLSC specifications given to the supplier were: 35 MPa characteristic strength, 650 microstrain shrinkage at 28 day with Eclipse admixture, 13 mm aggregate size and 120 mm slump workability. The SLSC was batched in the laboratory with a 35 MPa characteristic strength mix design using limestone aggregates, which produced a lower drying shrinkage. The NWC was a 25 MPa characteristic strength concrete originally delivered for another project. Standard concrete material tests such as the slump test, the cylinder compressive strength test and the drying shrinkage test were conducted based on NZ3112 [15] for each batch of concrete. Some CLSC specimens had more than 120 mm slump. This compromise was accepted in order to reflect the actual construction scenario in the research. Fig. 5 shows a comparison of the shrinkage measured on the different concrete mixes and their slump. A significant part of the shrinkage occurred in the first 50 days after casting. It is evident that concrete mixes with high slump have also high shrinkage. The mean values of these quantities are summarized in Table 3. The average compressive strength of CLSC at 28 days and then at the day of the beam test were 44.3 MPa and 53.7 MPa, respectively, with coefficients of variation of 8.22% and 7.58%. The measured average density of CLSC was 2405 kg/m^3 while the average Young's modulus at the day of beam test can be estimated as 33.4 GPa based on the NZS 3101 [16] equation.

The LVL was the 400 $\times$ 63 mm Truform recipe, with a mean Young's modulus of 11.3 GPa [17]. For LVL members subjected to combined bending and tension, which is the case for the LVL joists in a composite floor, a strength domain given by Eq. (2) can be assumed [14,18]:

$$\left(\frac{\sigma_b}{f_b}\right) + \left(\frac{\sigma_t}{f_t}\right) \leq 1 \tag{2}$$

where σ_b, σ_t signify the flexural and tensile stress components due to the load (strength demand), and f_b, f_t signify the bending and tensile strengths of LVL, respectively (strength capacity). Eq. (2) can be rearranged to express the inequality in terms of the maximum tensile stress in the bottom fibre of the LVL beam:

$$\sigma_{\max} = \sigma_b + \sigma_t \leq f_m\,(M/N) = \frac{1 + \frac{\sigma_t}{\sigma_b}}{\frac{f_t}{f_b} + \frac{\sigma_t}{\sigma_b}} f_t \tag{3}$$

where $\sigma_{\max}$ and f_m signify the strength demand and strength capacity, respectively. The average strength capacity f can be calculated by assuming the average values of f_b and f_t for LVL, estimated as 46.8 MPa and 33.4 MPa, respectively, on the basis of the statistical properties measured by the manufacturer on small test specimens and corrected for the size effect [19]. The stress ratio σ_t/σ_b depends on the M/N ratio in the LVL joist, which is affected by the stiffness ratios between concrete and timber, and by the slip modulus of the connection system. Using the γ-method for each tested beam, the stress ratio σ_t/σ_b was found in the range from 0.77 to 0.91. By substituting those values inside Eq. (3), a mean LVL strength f_m of 39.5 MPa was obtained for the beam specimens under investigation.

3.3. Experimental set-up

All beams were tested approximately 4–5 months after their construction. Every beam was simply supported and subjected to 4-point, quasi-static bending test to failure using a 400 kN load controlled hydraulic actuator (Fig. 6). The loading protocol followed during the test was similar to that recommended for connection testing [20], with the beam was first loaded to 0.4F_{est}, held for 30 s, unloaded to 0.1F_{est}, held for 30 s and finally loaded up to the collapse of the beam at a constant rate of 0.2F_{est} per minute. The estimated failure load, F_{est}, of each composite beam was predicted using the γ-method. The load applied on the beam (2P) and deflection at mid-span ($\Delta_{\max}$) were measured for every beam. The relative slip between concrete slab and LVL beam (Δ_H) was measured at every connection location. The strains of LVL and concrete across the section were measured for all beam specimens at mid-span and for selected beams also at one-third of the span. During the test, the following observations were made: (1) the presence of visual cracks in the connections, (2) time and level of load when the first crack was detected, either audibly or visually, (3) nature and mode of failure, and (4) condition of connection prior to failure and after collapse.

4. Results and discussion

Two types of failure mechanisms were detected: (1) tensile fracture of the LVL under the loading points at one-third of the span (Fig. 7(a)) with no apparent sign of failure in connections, for well-designed beams, and (2) for under-designed beams, failure of the

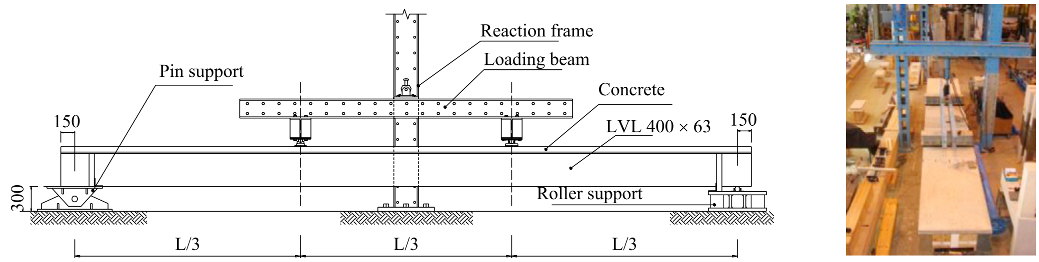

Fig. 6. Typical 4-point bending test set-up with photo (right) of a beam under the reaction frame ready to be tested (dimensions in mm).

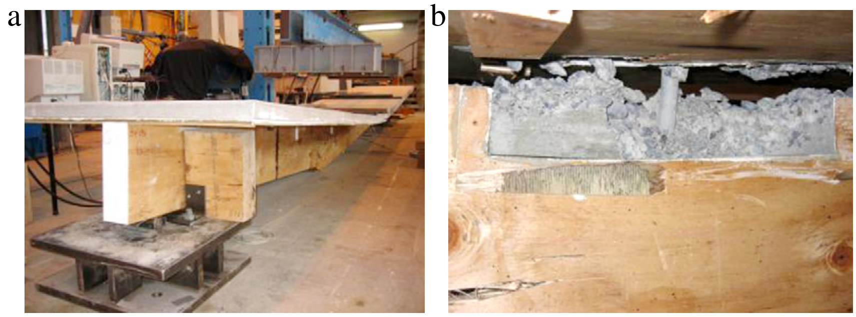

Fig. 7. Different types of failure mechanisms detected in the composite beams: (a) fracture in tension of LVL; (b) failure for concrete shear and crushing in 300 mm rectangular notch coach connection.

Table 4
Summary of collapse TCC floor beam results.

Beam	F_{max}	M_{exp}	w_{eq}		Δ_{max}	$K_{fi,beam}$	Load (kN)		DCA_{SLS} (%)		
	$2P_c$		Exp.	Anal			ULS	SLS	Exp	Anal	Ratio
	kN	kN m	kN/m	kN/m	mm	kN/mm	$2P_u$	$2P_s$		$K_{0.4}$	Exp/Anal
A1*si*	87.3	116	14.6	8.28	64.1	1.36	46.4	30.9	86.8	96.5	0.90
A2*s*[a]*i*	75.3	100	12.5	8.28	63.2	1.19	40.0	26.7	90.1	96.5	0.93
B1*s*[a]*i*	105	140	17.5	11.3	63.1	1.67	72.2	48.1	97.3	97.8	0.99
B2*si*	97.5	130	16.3	11.3	73.8	1.53	67.0	44.6	96.2	97.8	0.98
C1*so*	89.7	119	15.0	12.9	58.3	1.54	61.6	41.1	95.5	98.0	0.98
C2*s*[a]*i*	110	147	18.3	12.9	66.7	1.65	75.5	50.4	96.1	98.0	0.98
D1*s*[a]*o*	80.8	108	13.5	13.6	48.1	1.68	55.5	37.0	96.3	98.4	0.98
E1*si*	79.6	133	10.6	7.65	93.8	0.85	42.3	28.2	99.9	98.8	1.01
E2*si*	55.4	92.3	7.38	–	66.9	0.84	29.4	19.6	98.9	98.8	1.01
F1*d*[a]*o*	174	232	28.9	15.5	95.6	1.82	92.2	61.5	98.1	98.7	0.99
G1*di*	201	268	33.5	22.5	69.4	2.90	138	92.0	96.6	97.1	0.99

[a] Indicates beams not tested to complete destruction to allow for vibration tests; *s* for single LVL 600 mm wide flange; *d* for double LVL 1200 mm flange; *i* for beams constructed indoor; *o* for beams constructed outdoor.

shear connection and/or crushing of concrete with plasticization of the coach screw in the case of notched connections (Fig. 7(b)), or plate tearing in the case of metal plate connections. The failure pattern of notch connectors was similar to that detected in push-out tests [7], where concrete strength was found to be significantly influence the shear strength of the connection and, therefore, the load-bearing capacity of the composite beam. In most cases, the first crack sound was heard at approximately 60% of the collapse load, F_{max}, indicating the start of connection yielding which was followed by further plasticization and the screeching sound becoming louder. The failure hierarchy observed for under-designed beams was as follows: (1) crack sound in one or multiple connections as an early warning; (2) failure of the first connector, usually near the support; (3) consecutive failures of the other connectors towards the middle of the beam due to redistribution of the shear force; (4) when all connectors have failed, the load is resisted only by the LVL beams with zero composite action and eventually tensile fracture of the LVL.

The test results for the beams are summarized in Table 4. Several beams were not tested to complete destruction to enable vibration tests to be performed, which was a study under a separate project. The maximum or collapse total load, F_{max}, corresponding to the resultant of the point load, $2P_c$, and the maximum mid-span displacement at collapse, Δ_{max}, are reported in Table 4. The total load–mid-span deflection curves are displayed in Fig. 8 for all the beams where the F_{max} values for single LVL beams were doubled to allow immediate comparison with the double LVL beams with 1200 mm concrete flange width. In the same figure, also the upper limit of a full composite beam, the lower limit of no composite beam, and the case of only LVL beams with no concrete slab are plotted. The experimental equivalent uniformly distributed load, w_{eq}, in kN/m was calculated by equating the experimental maximum bending moment such that $wL^2/8 = (2P_c)L/3$. The corresponding w_{eq} analytical value was calculated using the γ-method [1] with connection secant slip moduli of $K_{0.6}$. The load w_{eq} is defined as the maximum load such that all LVL, concrete and connection pass the short-term verifications at ULS. The mean values of the mechanical properties (modulus of elasticity and strength) of the materials were used in the analytical prediction of w_{eq} to compare it with the experimental value. The experimental final beam stiffness, $K_{fi,beam}$, was calculated at the maximum or collapse load, F_{max}.

The derivation of load at ULS depends on the experimental failure mechanism. Based on the experimental maximum or

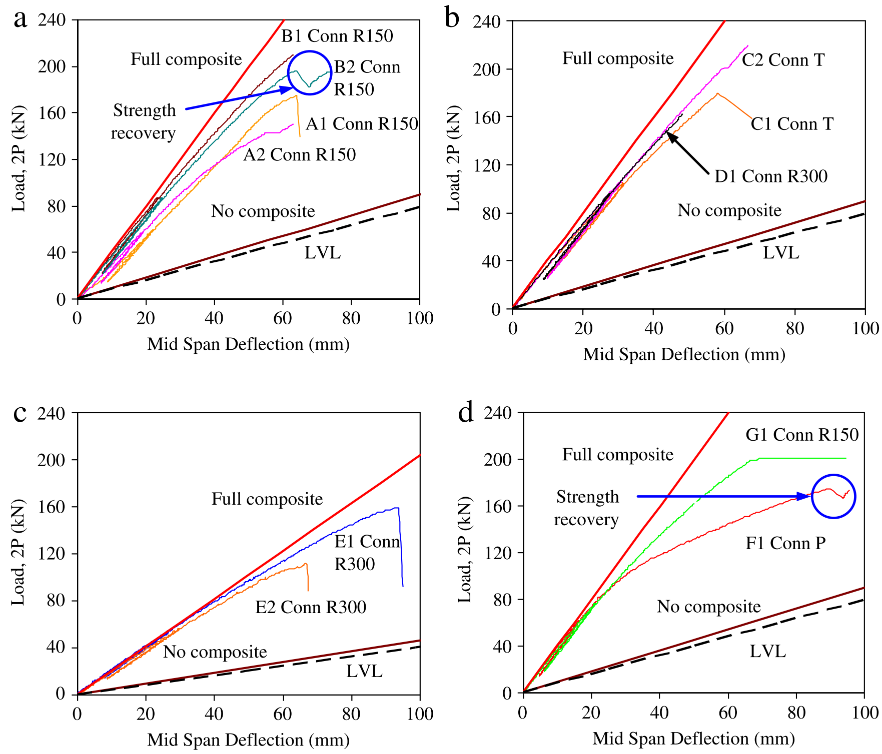

Fig. 8. Experimental load–deflection plots reflecting double LVL 1200 mm wide flange section for all beams (refer Fig. 4 and Table 1 for beam and connection description).

collapse load, $F_{\max}$, the ULS load was estimated using the formula [21] $2P_u = (f_d/f_m) \times 2P_c \times k_{\text{mod}} = 0.687(2P_c)$ for beams with fracture tensile failure (in the case of well-designed beams) or $2P_u = (R_k/R_m) \times 2P_c \times k_{\text{mod}}/\gamma_M = 0.531(2P_c)$ for beams with connection shear failure (in the case of under-designed beams). The SLS load was estimated by $2P_s = 2P_u/\gamma_Q = 0.458(2P_c)$ for well-designed beams and $0.354(2P_c)$ for under-designed beams. The properties were assumed as follows: LVL design strength, $f_d = 33.85$ MPa (from Eq. (3), assuming the design values of f_t and f_b provided by the manufacturer); LVL mean strength, $f_m = 39.45$ MPa; load duration modification factor, $k_{\text{mod}} = 0.8$ (where imposed load refer to an office building, for which a medium term load duration was considered); connection strength characteristic/mean ratio, $R_k/R_m = 0.83$ [7]; partial factor for connection, $\gamma_M = 1.25$; partial factor for variable action, $\gamma_Q = 1.5$.

5. Short-term performance at ULS

Fig. 9 presents analytical–experimental comparisons of load capacity at ULS in the short term in terms of imposed load for tested beams built from commercial low shrinkage concrete (CLSC), LVL-only and full composite beams. The analytical design imposed load in kN/m^2 was predicted such that all the ULS short-term inequalities were satisfied using the γ-method with connection secant slip modulus $K_{0.6}$ where concrete, LVL and connection strength design values were used. In all cases, the connection strength inequality was governing. The experimental live load, Q, in kN/m^2 was converted from $(2P_u)$ in Table 4 using the equivalence of the bending moments.

Important observations from Fig. 9 are:

1. All well-designed beams (B1; average of C1 and C2; and G1) exhibited an experimental load capacity very close to that of a fully composite section (approximately 90%). This is true for beams with a large degree of composite action (Table 4). Note that beam D1 was not tested to complete collapse so the 7.88 kN/m^2 is lower than its collapse load.
2. All experimental imposed load capacities were about 3 times larger than the analytical capacities for all under-designed and well-designed beams. In other words, the γ-method underestimated the short-term ULS capacity for all cases in this experiment. It is important to note that for all cases, the design governing condition was long-term deflection.

6. Short-term performance at SLS

The analytical and experimental capacities at SLS in terms of imposed load in kN/m^2 corresponding to the deflection limit of span/300 in the short term is given in Fig. 10 for the fully composite and LVL-only beams. The analytical imposed loads were predicted using the effective bending stiffness, EI, obtained from the γ-method with connection secant slip modulus $K_{0.4}$ and mean Young's moduli of concrete and LVL such that the aforementioned deflection limit was satisfied. The experimental imposed load was determined from the experimental load–deflection curve as the load corresponding to the deflection limit quoted above. The experimental load in kN was converted to kN/m and then to kN/m^2 using a deflection equivalent criterion, i.e. $5wL^4/384\text{EI} = Pa(3L^2 - 4a^2)/24\text{EI}$ where $a = L/3$, and then by dividing w by the flange width (600 or 1200 mm).

Important observations from Fig. 10 are:

1. In most cases, the analytical prediction underestimated the experimental imposed load by about 10%. The γ-method, therefore, provided an accurate and conservative prediction of the imposed load at SLS.
2. The experimental load capacities of well-designed beams were only 10% less than those for fully composite beams.
3. The actual design imposed load of 3 kN/m^2 is approximately one-quarter of the analytical and experimental imposed load capacity. This is due to deflection in the short term not being the

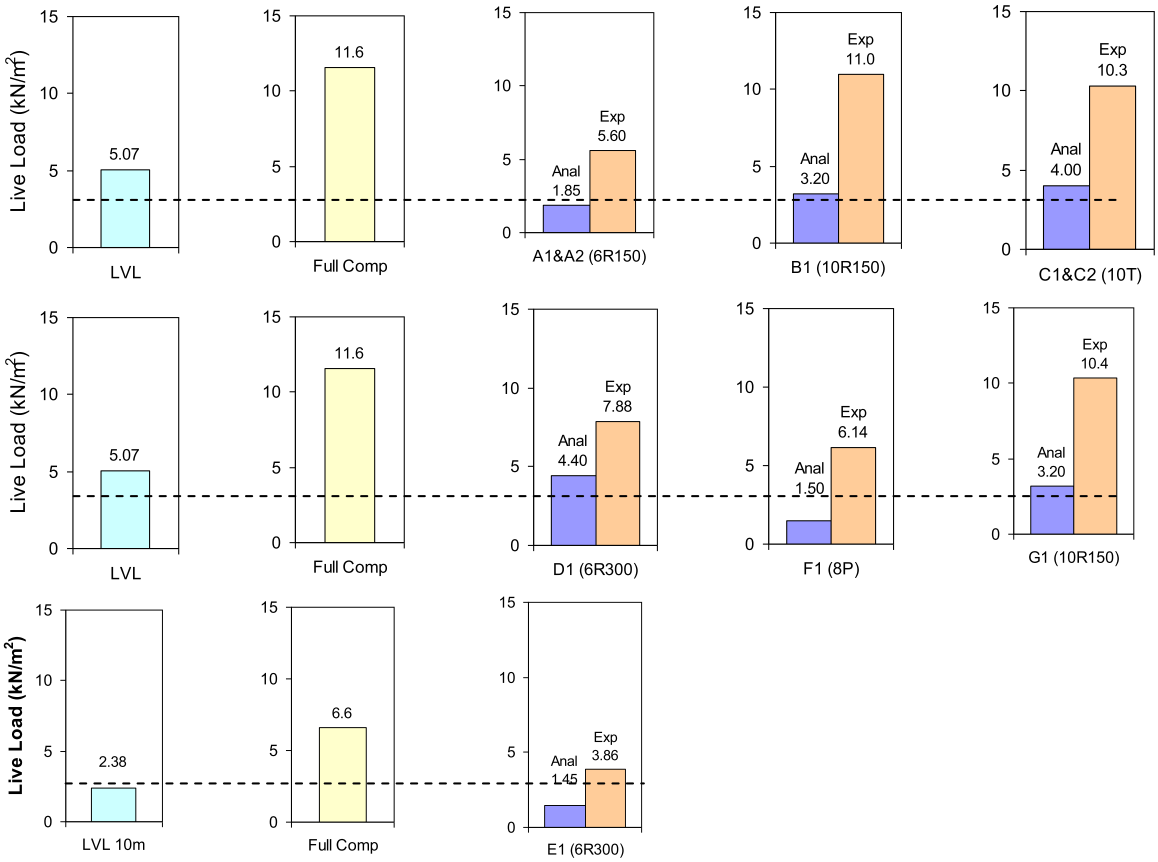

Fig. 9. Analytical–experimental short-term ULS imposed load capacity of tested TCC beams compared to LVL-only and full composite TCC. Dashed line shows the design live load (3 kN/m^2).

governing design criterion for the beams under investigation, which were governed mostly by deflection in the long term.

The experimental degree of composite action at SLS, DCA_{SLS}, was calculated using Eq. (1) with the experimental deflection obtained from the corresponding SLS load in Table 4 using the γ-method and the connection slip modulus $K_{0.4}$. All the beams exhibited a high level of composite action, between 86.8% and 99.9%. This is observed in Fig. 8, where all the load–deflection curves were in close proximity to the fully composite curve. In all cases, the analytical γ-method closely estimated the experimental values with between 1% and 10% difference.

By comparing the imposed load capacities at SLS of fully composite and experimental beams (Fig. 10), the difference was only less than 10% particularly for well-designed beams (B1, average of C1 and C2, D1, G1). This indicated that the transformed section method can be used with a suitable correction factor to design composite beams such as those investigated in this study that are characterized by a high degree of composite action (Table 4).

In an attempt to quantify this correction factor for a design at SLS, experimental beam deflections were compared with those expected for a fully composite section at the SLS load level ($2P_s$) in Table 5. The analytical deflection determined using the γ-method with the connection secant slip modulus $K_{0.4}$ was also included in the comparison. For the well-designed beams, the experimental deflection was 1.09–1.15 times the fully composite deflection, and 1.02–1.08 times the analytical deflection. Taking a conservative approach, this finding is indicative of a 15% increment correction factor to the deflection or, equivalently, a 13% reduction to the flexural stiffness (EI) calculated using the transformed section method.

Table 5
Deflection at SLS load ($2P_s$) of full composite (FuC), experimental and analytical beams built from commercial low shrinkage concrete (CLSC).

Beam	Deflection, Δ (mm)				
	FuC	Exp.	Anal.	Exp/FuC	Exp/Anal
A1	15.6	22.7	17.5	1.45	1.30
A2	13.5	18.0	15.1	1.34	1.19
B1	24.3	26.5	26.1	1.09	1.02
C1	20.7	23.9	22.1	1.15	1.08
C2	25.4	28.8	27.1	1.13	1.06
D1	18.7	21.1	19.7	1.13	1.07
E1	27.8	27.8	28.9	1.00	0.96
F1	15.5	16.6	16.2	1.07	1.02
G1	23.2	25.9	25.5	1.12	1.02

7. Comparisons among different beams

7.1. Effect of beam T-section reduction (beams B1 and G1)

Beam G1 was a reference beam with a double LVL joist and 1200 mm wide concrete flange. All other beams with notch connection were constructed with a reduced sectional geometry made of a single LVL joist and 600 mm wide concrete flange, and the same span and notch length. In order to confirm that this sectional reduction does not affect the actual strength and stiffness properties, the experimental results of beam B1 with 600 mm wide concrete flange was compared to beam G1. The stiffness, $K_{fi,beam}$, and collapse load, F_{max}, of beam B1 were doubled and found to be 15% and 5% larger than beam G1, respectively (see Table 4 and Fig. 8). The degree of composite action calculated for the two beams was less than 1% difference. The differences were deemed to be within acceptable limit considering possible variations in the

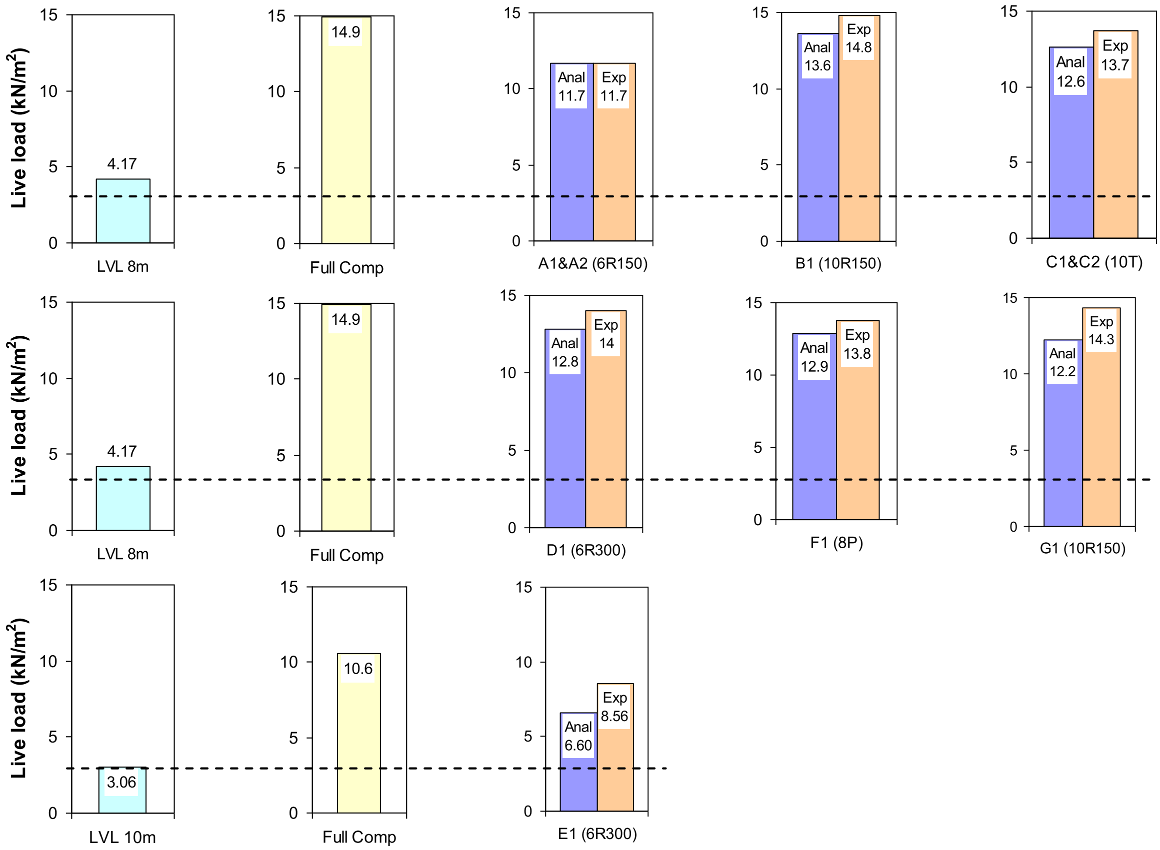

Fig. 10. Analytical–experimental comparison of imposed load capacity in the short term at SLS for tested TCC beams, LVL-only and fully composite TCC. Dashed line shows the design imposed load (3 kN/m^2).

concrete for the two beams. It can therefore be concluded that the single joist LVL composite beam with 600 mm wide concrete flange is fully representative of the entire semi-prefabricated composite panel 2400 mm wide, the load-bearing capacity and stiffness of which can be simply evaluated by multiplying by four the values measured on the 600 mm wide beam specimens.

7.2. Effect of pocket notches (beams A1 and A2)

Beams A1 and A2 had the same connection design and were built indoors. The notched connections in beam A2 were left pocketed during the concrete pouring as opposed to the casting of the whole slab including the notches in the case of beam A1 and all the other beams. The pockets were grouted on the 7th day with high strength low shrinkage SIKA 212 grout which had a drying shrinkage of 394 microstrain and 80 MPa compressive strength on the day of beam testing as compared to 436 microstrain and 58 MPa of concrete used in beam A1, respectively (Table 3). Considering the achievable compressive strength, beam A2 was expected to perform better than beam A1. On the contrary, although the compressive strength and shrinkage properties of the pockets in beam A2 were better, the beam exhibited lower stiffness (12% less) and collapse load (14% less) compared to beam A1 (Table 4 and Fig. 8). Possible explanation for this phenomenon could be due to only one of each beam type tested and the statistical variability in the material properties. However, the actual reason for this difference is not fully known. Insufficient propping days (3 days according to SIKA 212 manufacturer recommendation) before the grouted notches developed enough strength could be a possible reason.

7.3. Effect of design level (beams A and B)

To investigate the effect of the design level in TCC beams, two types of beams were compared, both with a similar connection (R150): beams A, under-designed (with 6 connectors), and beams B, well-designed (with 10 connectors). The well-designed beams were approximately 1.2 times stiffer and stronger (collapse load, F_{max}) than the under-designed beams. A redistribution of shear force after the first connection yielding was evident in a well-designed beam because of the sufficient number of connectors in the beam. This is particularly evident in the load–deflection curve of beam B2 (Fig. 8(a)) where there was a recovery of strength after the load decreased at about 200 kN following the yielding of a connector. This is an important outcome as it ensures a moderate ductile behaviour of the composite beam which may allow sufficient time for evacuation in the case of an emergency. Such a recovery was not seen in the under-designed beams. The high degree of composite action exhibited by the well-designed beams implied that deflection is minimal.

7.4. Effect of connection type (beams B1 and C1; beams A1 and F1)

Beam B1 with 150 mm rectangular notch connection (R150) was compared with beam C2 with triangular notch connection (T). Both beams have the same number of connectors (10). No significant differences in strength, stiffness or composite action can be identified (Table 4). This shows that different types of notched connections used in TCC beams do not affect the structural performance as long as the connectors have shear strength that are close (see Table 2). The ductility in such a notched connection is highly dependent upon the coach screw, without which, the connection exhibited a brittle behaviour [7]. The influence of ductile connection on TCC beams is not the main focus of this paper. However, this subject is discussed in [22].

The beams with notched connections (in particular beam A1 with R150) had a similar strength to the metal plate connected beam (F1) (with 174.6 kN represented by doubling the collapse load of A1, and 174 kN for F1, Fig. 8(a) and (d)). Beam F1 showed slightly better initial stiffness (3.68 kN/mm) than A1 which, however, declined rapidly (1.82 kN/mm) after $0.6F_{\max}$. This behaviour was not observed in beams with notched connections and was likely due to the yielding and tearing of the metal plate connections that were more ductile than the coach screws in the notched connections. Consequently, past $0.6F_{\max}$, these connections slipped more than notched connections causing larger beam deflections. It was also observed that a metal plate connected beam, although under-designed, exhibited a sort of strength recovery, unlike under-designed notch connected beams. In order to improve the post-peak stiffness of beam F1 and prevent the final brittle failure of the plate connection due to tearing, it is recommended that the plate thickness be increased [7].

7.5. *Effect of notch length (beams B1 and D1; beams A and D1)*

Beams with rectangular notch connectors of different lengths but the same design level were compared: beam B1 with 10 notches 150 mm long (R150) and beam D1 with 6 notches 300 mm long (R300). Both beams had the same design level, i.e. they were designed for the same load. Both the stiffness and degree of composite action of the beams were almost identical (1.67 kN/mm and 97.3% for B1; 1.68 kN/mm and 96.3% for D1, respectively, in Table 4). The actual maximum load of the beams was not known as the test was stopped before collapse occurred (so they could be used for another project).

By comparing beams with the same number of notch connectors (six) and different notch length (150 mm in beams A, and 300 mm in beam D1), it is evident that the beam with the longer notch (D1) performed better in stiffness (30% more) and composite action (10% more). No actual maximum load can be compared since beam D1 was not tested to complete destruction. The use of longer notches is preferable to improve the performance of the composite beam as the length of the concrete notch itself increases the shear strength and stiffness of the connection as found in push-out tests [7].

7.6. *Effect of concrete compressive strength and concrete type (beams E1 and E2)*

Beam E1 was built with grade 35 low shrinkage concrete (measured $f_{cm} = 48$ MPa) and beam E2 with grade 25 normal concrete (measured $f_{cm} = 31$ MPa). Beam E1 (79.6 kN) exhibited 40% higher collapse load than beam E2 (55.4 kN) (Table 4 and Fig. 8) with the same stiffness and degree of composite action. Essentially, it was the concrete in the notched connections that provided the shear transfer capacity between the concrete and LVL. Therefore, this comparison indicates that compressive strength of the concrete is crucial to achieve beams with high strength performance.

The use of low shrinkage concrete is crucial to reduce the total shrinkage of the concrete slab along the entire span length of the beam, which if prevented by the stiff connection system, will induce a self-equilibrated stress distribution (Eigenstresses) in the concrete, LVL and connection. As such, additional deflection with potential serviceability problems may occur, causing possible problems of excessive deflection in the long term. Although the main focus of this paper is not on behaviour of TCC in the long term, it is important to highlight that the deflection after 1 year of a normal concrete TCC beam was 1.16 times larger than that of a TCC beam built from the same low shrinkage concrete reported in this paper [5]. This deflection increment was mostly due to the shrinkage effect of the overall slab. The shrinkage of the concrete within the notch did not cause any significant gap opening at the sidewall interfaces between the concrete and the LVL. Such a gap might have caused an increase in deflection due to the lack of contact between the concrete and the LVL, as until that gap is closed, the connector in the notch would rely only on the stiffness of the coach screw. In neither of the beams and push-out specimens [7] tested to failure, such an increase in flexibility due to the gap was noted, reinforcing the conclusion that no significant gap occurred in the connection due to drying shrinkage even in the case of normal concrete. Interest in the long-term behaviour of TCC and the effect of different concrete type (for example, lightweight concrete versus normal concrete) is found in [23].

7.7. *Effect of indoor and outdoor environmental exposure before collapse test (beams C1 and C2)*

Beams C1 (outdoor) and C2 (indoor) were compared. Note that beam C1 was left outdoors without any imposed load subjected to environmental changes (corresponding to spring and summer seasons) for 4–5 months. Beam C2 ($F_{\max} = 110$ kN, $K_{\mathrm{fi,beam}} = 1.65$ kN/mm) was found to be 20% stronger and 10% stiffer than beam C1 ($F_{\max} = 89.7$ kN, $K_{\mathrm{fi,beam}} = 1.54$ kN/mm). No difference was found in the degree of composite action.

8. Horizontal slip of shear connection

The relative slip between the concrete slab and the LVL joist was monitored during the tests at the connector location. The slips in beams D1 and F1 are presented in Fig. 11(a1) and (a2), respectively, together with the corresponding shear force plotted in Fig. 11(b1) and (b2). Beams D1 and F1 have three numbers of 300 mm rectangular notches (R300) and four pairs of modified metal plate connectors (P), as shown in Fig. 11(c1) and (c2), respectively, on each side of the span. The connections in each beam are numbered incrementally starting with one from the end to mid-span, for example, Conn 1 is located nearest to the end span or support and Conn 4 is located nearest to mid-span in beam F1 as illustrated in Fig. 11(2). The shear forces were obtained from load–slip curves measured in the connection push-out tests [7]. The amount of slip is indicated for different load levels at SLS, ULS and $0.8F_{\max}$. The largest slip occurred, normally, in the inner connector nearest to the outer edge of the point load, for example Conn 2 in beam D1.

Connection R300 in beam D1 behave relatively stiff even past the ULS load level. Connection P exhibited high initial stiffness, but slip markedly increased after the ULS load indicating yielding of the plate. Fig. 11(b2) shows that the shear force in all connectors of beam F1 reached a plateau and eventually dropped indicating complete failure in the connections and beam. Such behaviour was not observed in beam D1 since connection R300 has a different failure mode [7].

9. Conclusions

Short-term collapse 4-point bending tests were conducted on eleven, 8 and 10 m span laminated veneer lumber (LVL)–concrete composite floor T-beams. Several variables such as connection types, concrete type, and design level corresponding to number of connections were investigated. Mid-span deflections and connection slips were measured during the tests. The types of connectors were triangular and rectangular (150 and 300 mm long) notches cut in the LVL and reinforced with coach screws, and modified toothed metal plate connectors. Different concrete

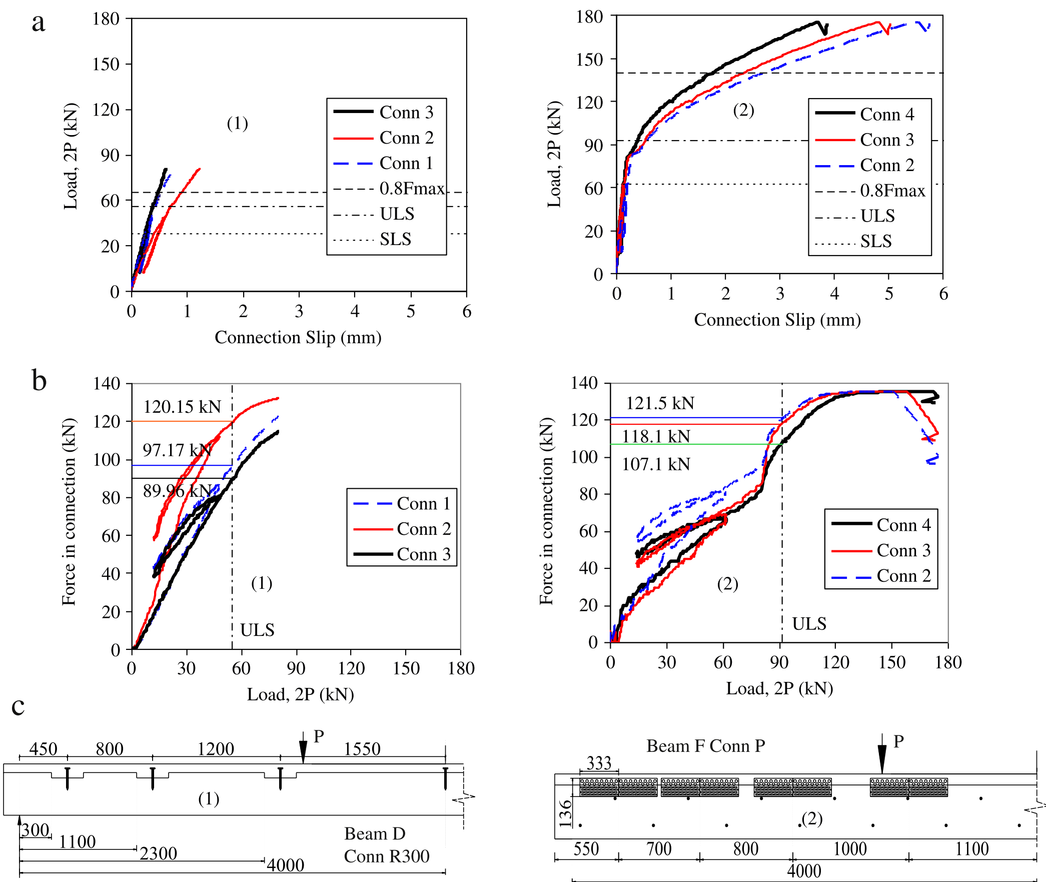

Fig. 11. Load-connection–slip curves (a), the corresponding shear force in connection (b), and position of connectors with respect to loading point where Conn 1 is located nearest to left support and Conn 4 nearest to mid-span (c), for (1) beam D1 with single LVL (connection R300), and (2) beam F1 with double LVL (connection P). (Refer to Fig. 4 for connection type.)

was used including normal weight, commercial low shrinkage, and special low shrinkage concrete. The effective bending stiffness method or γ-method according to Annex B of Eurocode 5 was used to design the beams under 3 kN/m^2 design imposed load and 1 kN/m^2 design permanent load in addition to the self-weight. Six beams were well-designed and five were under-designed. Well-designed beams refer to beams that fully comply with all design inequalities at ULS and SLS. Under-designed beams refer to beams where the maximum demand of shear force in the connection was about 1.3 times the resistance at ULS.

All well-designed beams exhibited more than 95% composite action regardless of the type of connection used. They also showed redistribution of shear force in the connectors thus enabling strength recovery in the event the outer connections fail. Therefore, a well-designed system is highly recommended.

The 300 mm long rectangular notch and coach screw connection is recommended for composite beams for two main reasons: (1) High stiffness and strength even beyond the ULS load level; (2) It requires fewer connectors along the beam and therefore has less cost than the triangular notched alternative. Although the triangular notch requires more connectors than a 300 mm notch for the same design level, it is easier and faster to cut, particularly if computer numerical control (CNC) machines are not available. Metal plate connections proved to be practical for construction, however a disadvantage was the quick decrease in stiffness beyond the ULS load level. This behaviour could be mitigated by increasing the plate thickness to postpone the brittle failure for tearing.

No significant difference was found in the short-term performance among beams with different shrinkage properties of concrete. However, the strength of concrete is important especially in notch connected beams since the concrete within the notches provides the shear transfer between the LVL and the concrete slab.

Comparisons of the experimental results and analytical predictions provided the following conclusions about the short-term performance at ULS and SLS:

1. All well-designed beams with a high degree of composite action exhibited experimental imposed load capacities at ULS and SLS very close to those of a fully composite beam (approximately 10% less).
2. Therefore, correction factors providing a 15% increase in the deflection and a 13% reduction of the effective bending stiffness, (EI), are proposed for calculating the deflection and strength using the transformed section method for all well-designed beams.
3. All experimental imposed load capacities at ULS are about three times larger than the analytical capacities for all under- and well-designed beams. In other words, the γ-method underestimated the short-term capacity at ULS in all cases.
4. In most cases, the analytical prediction underestimated the experimental imposed load at SLS about 10%. In other words, the γ-method provided a reasonably accurate prediction of the imposed load at SLS.

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