EXPERIMENTAL BEHAVIOUR AT ULTIMATE LIMIT STATE OF A SEMI-PREFABRICATED TIMBER-CONCRETE COMPOSITE FLOOR SYSTEM

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SUMMARY

An extensive experimental programme aimed to develop multi-storey timber buildings for non-residential applications has been undertaken at the University of Canterbury, New Zealand. A performance requirement, among others, is that the floors must be medium to long-span, so as to allow open spaces. In addition, a prefabricated solution is preferred in order to take advantage of the prefabrication process, as commonly done in precast reinforced concrete and steel consruction. Due to those requirements, a semi-prefabricated timber-concrete composite system has been developed. This paper reports the results of the tests to failure performed on 11 full-scale beam specimens representative of semi-prefabricated timber-concrete composite floor strips. Four connection types (metal plates pressed onto the LVL beams, triangular notches, large and small rectangular notches cut from the LVL beams) were selected based on the outcomes of previous research. The experimental tests demonstrated that a small number of inexpensive connectors are able to provide a stiff connection system. The high degree of composite action achieved makes the proposed system suitable for medium to long-span floors.

INTRODUCTION

Timber-concrete composite system is a construction technique used for strength and stiffness upgrading of existing timber floors and new construction such as multi-storey buildings and short-span bridges. By combining two different materials it is possible to exploit their best qualities since the timber is positioned in the tension region of the composite section while the concrete is used in the compression region. The presence of timber, due to its lower density in comparison with reinforced concrete, decreases the weight of this flooring system, implying several advantages: (1) higher efficiency in terms of load carried per self-weight; (2) better seismic performance derived by less structural mass; and (3) lower carbon footprint of the building when compared with a concrete, due to the advantage of carbon stored in the timber. The advantages given by the concrete slab are: (1) larger thermal mass and fire resistance; (2) better acoustic separation; and (3) good structural performance in seismic regions since the floor behaves as a rigid diaphragm. All the aforementioned advantages can only be achieved if the composite system is structurally effective by means of a stiff and strong shear connection system. A wide range of connection systems are available, each with different level of rigidity. The choice of the connection system markedly affects the behaviour of timber-concrete composite floors. Ceccotti [1] presented a variety of connections, sorted out from the most flexible to the most rigid according to their stiffness . Seven types of connection were tested under direct shear by Lukaszewska et al. [2], out of which the best two systems were used to build a timber-concrete composite system that is fully prefabricated [3]. A semi-prefabricated timber-concrete composite system is proposed at the University of Canterbury comprising of “M” section panels built with laminated veneer lumber (LVL) beams acting as floor joists and a plywood interlayer as permanent formwork (see 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. 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 propped while the concrete cures. The connection system has notches cut from the LVL joist and reinforced with a coach screw to provide more ductile behaviour during failure and to increase the shear strength. These notches are cut into the beams before the plywood interlayer is nailed on. An extensive research programme of this system involving connection push-out tests and full scale T-beam tests in both the short- and long-term is currently on-going [4-5]. This paper reports the outcomes of the experimental test to failure performed on 11 full-scale specimens representative of semi-prefabricated timber-concrete composite floor strips. The specimens were 8 and 10m long, and had different connection systems.

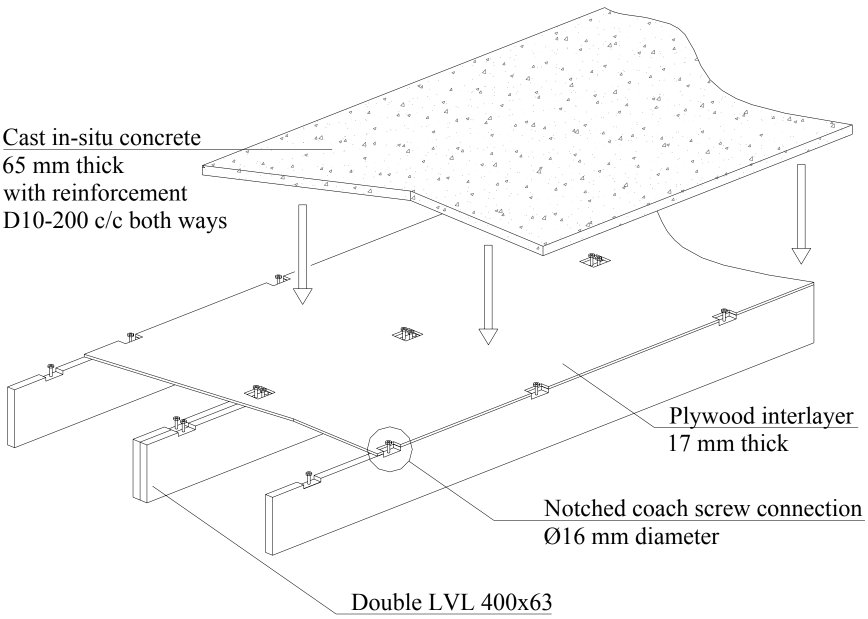

Fig. 1 Proposed semi-prefabricated timber-concrete composite system

CHOICE OF CONNECTION AND COMPOSITE ACTION

Four best types of connection were identified and used to construct the timber-concrete composite beams presented in this paper: (1) 150mm rectangular notched coach screw; (2) 300mm rectangular notched coach screw; (3) triangular notched coach screw; and (4) toothed metal plate. The four connections were selected based on the highest strength and stiffness values out of 15 types of connections that were tested in an experimental push-out parametric study [5].

The characteristics that are desired for the proposed semi-prefabricated timber-concrete composite system are: (1) medium to long span, from 6 to 12m; (2) minimum number of connectors, so as to minimize construction cost; (3) high level of composite action; and (4) minimum deflection in the long-term. The choice of a strong and stiff connection is therefore crucial to achieve the aforementioned requirements, since the stiffer the connection, the lesser the deflection of the composite system. Very stiff connections, in fact, ensure a complete composite action of timber beam and concrete slab, with no slip at the interface and small deflection (Fig. 2c). In the absence of connection no composite action is achieved, resulting in both large slip and deflection (Fig. 2a).

The degree of composite action (DCA) is expressed in percentage as given in Eq. 1 and defined as the ratio of the difference between deflection of beam with no connection (Δ_N, calculated theoretically) and with the actual flexible connection (Δ_F, measured experimentally) to the difference between deflection of beam with no connection (Δ_N, calculated theoretically) and with fully rigid connection (Δ_R, calculated theoretically) [6]. From a structural view point, it is better to implement a very stiff connection which results in little or no interlayer slip and minor deflection. Since this solution will incur in a large cost, a flexible connection with enough stiffness is adopted providing a good composite action can be achieved (Fig. 2b). Note that the composite action depends upon the load level and it usually decreases as the load increases. In the following, the degree of composite action was evaluated at the serviceability limit state (SLS) unless otherwise mentioned.

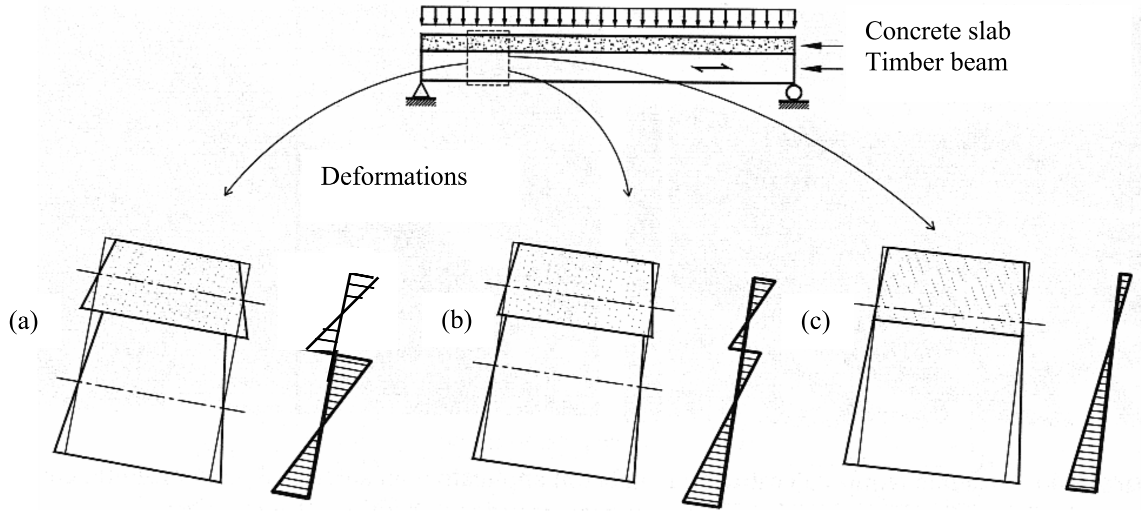

Fig. 2 Influence of the connection stiffness in a TCC beam subjected to flexure: (a) no composite action; (b) partial composite action; (c) complete composite action

$$DCA = \frac{\Delta_N - \Delta_F}{\Delta_N - \Delta_R} \quad (1)$$

The choice of the connection is therefore crucial because it greatly influences the structural performance and the overall cost of the proposed system.

BEAM DESIGN AND EXPERIMENTAL SET-UP

The overall research on full-scale timber-concrete composite beams comprised of fours phases: (1) short-term monitoring of beams outdoor and indoor, in unconditioned environment, where the deflections of 9 beams were monitored for a period of 1 month after the concrete placement, with the aim to investigate the effects of the construction process and the environmental changes; (2) short-term monitoring of beams indoor in unconditioned environment, where 4 beams were monitored for a period of 3 months with the service load applied after 28 days from the concrete placement; the purpose of this phase was to investigate the time-dependent behaviour during construction and the first months of life of the structure; (3) repeated loading of selected beams and test to failure of all the beams in (1) and (2) under four-point bending static load; and (4) long-term monitoring of 3 beams under service load for a period of at least 1 year, in order to assess the creep coefficient in the long-term. Only the test to failure of beams under four-point bending static load in phase 3 is presented in this paper.

The proposed ‘M’ section of semi-prefabricated timber-concrete composite system was reduced to a ‘T’ section considering only the middle part of the actual system. The ‘M’ section panel is 2400 mm wide built with a single 400 × 63 mm laminated veneer lumber (LVL) joist on each outer edge and a double LVL joist in the centre (Fig. 3a). Hence, the geometrical properties of the reduced ‘T’ section is the middle double LVL joist with a 1200mm wide flange as illustrated in Detail A of Fig. 3b. One out of all the other beams with notched connections was built with this sectional properties as a control beam while the other specimens were constructed with a further reduced section of a single LVL and a 600mm wide flange (Fig. 3c). The beam with toothed metal plate connection was an exception as it required 2 LVL sections to sandwich the plates and therefore the sectional dimension of this beam was a double LVL section with 1200mm flange.

All the beams were designed at ultimate limit state and serviceability limit state with different numbers of connectors corresponding to two scenarios, well-designed and under-designed. The method of effective bending stiffness (also known as γ-method) according to the Annex B of the Eurocode 5 provisions [7] for ultimate and serviceability limit state was adopted in the design, with the slip moduli $K_{s,0.4}$ at serviceability limit state and $K_{s,0.6}$ at ultimate limit state, and strength values, F_{max} obtained from more push-out tests performed on the selected connection types [8]. Well-designed beams refer to beams that fully comply with all inequalities at ultimate and serviceability limit states , while under-designed beams refer to beams where the maximum demand of shear force in the connection is about 1.3 times the resistance at ultimate limit state.

Each beam was designed and constructed by varying 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) and the curing time (1 and 5), and whether the notches were cast at the time of the concrete placement or grouted 7 days later. The type of concrete was carefully selected as shrinkage is expected to induce significant deflection on the composite beam due to the high stiffness of the connection. The concrete selected is a commercially available low shrinkage concrete (CLSC) of 35MPa characteristic strength, 650 microstrain shrinkage at 28 day with a special admixture known as Eclipse, 13mm size aggregate and 120mm slump. The LVL beams made with the Truform recipe were supplied by an Australasian LVL manufacturer. The mean bending strength and mean Young's modulus of the LVL were 58.4MPa and 13GPa respectively.

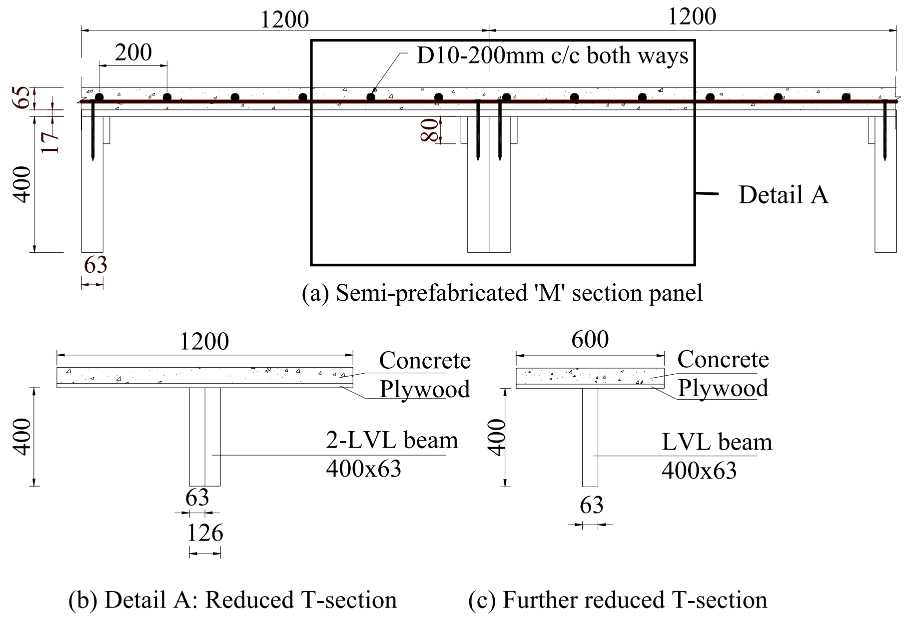

Fig. 3 (a) Semi-prefabricated "M" section panel; (b) Detail A of reduced T-section; (c) Further reduced T-section (dimensions in mm)

A total of 11 beams of two span lengths, 8m and 10m, were subjected to four point bending load until collapse (Fig. 4). The failure load, F_{est}, of each beam was first estimated using the γ-method. A 400kN capacity static ram was used to apply the load in displacement control. The same load protocol as in the push-out tests performed on the connection was used [5], where the beam was first loaded to $0.4F_{est}$, held for 30 seconds, unloaded to $0.1F_{est}$, held for 30 seconds and finally loaded up to the collapse of the beam at a constant rate of $0.2F_{est}$ per minute. In order to perform this loading and unloading regime at $0.4F_{est}$ and $0.1F_{est}$ respectively, some modifications had to be made on the displacement control software. The loading of the beam until failure required an average time of 15 to 20 minutes. The following properties were measured during the test for every beam: (1) total load applied on the beam by the ram , F (2) deflection at mid-span, Δ_{max}; (3) relative slip between concrete slab and LVL beam at every connection location, Δ_H; and, (4) strain of LVL and concrete across the section at mid-span and at one-third of the span. The existing deflections of each beam were noted prior to moving the beams to the reaction frame. This is because some of the beams were exposed for a certain period of time to outdoor environmental conditions and to the service load which induced an initial sag on the specimen . During the test, the following observations were made and recorded: (1) presence of visual cracks in the connections; (2) time and level of load in which the first crack were detected either by hearing or visually; (3) nature and mode of failure; and (4) conditions of connections prior to failure and after collapse.

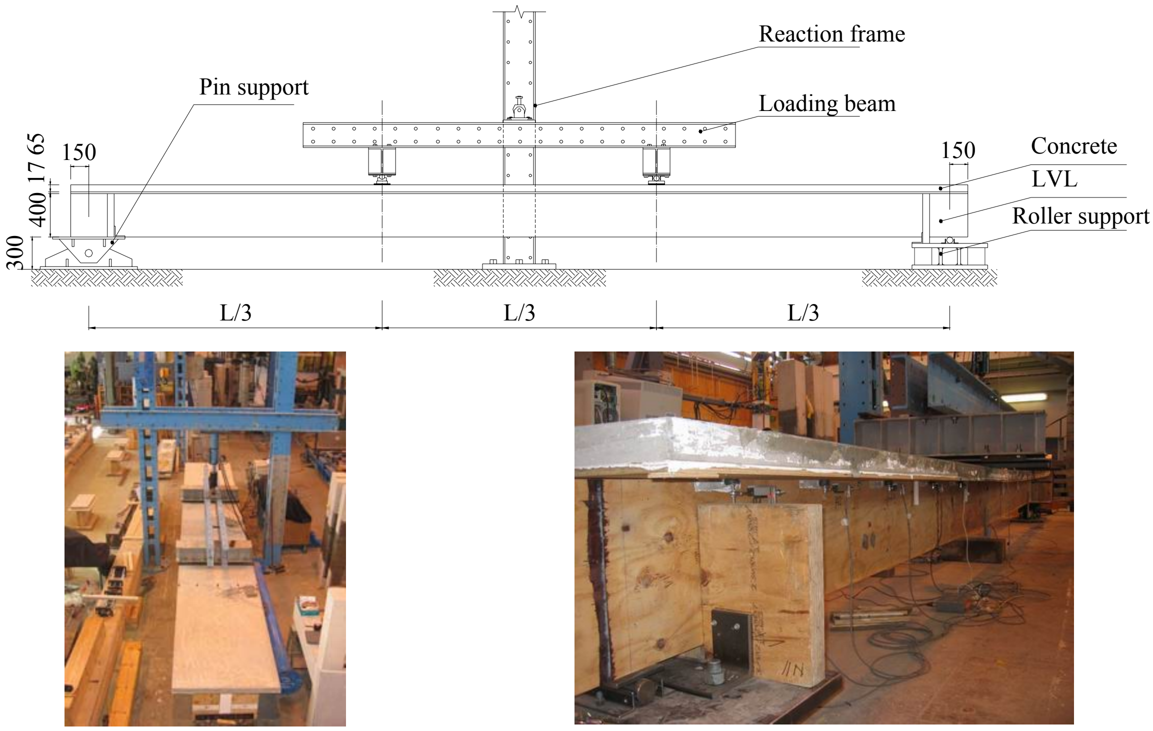

Fig. 4 Four-point bending test setup for collapse test (dimensions in mm)

EXPERIMENTAL AND ANALYTICAL RESULTS

The details of the beams tested such as beam notations, span length, type of connection and level of design, and the summary of results including the failure load F_{max}, experimental maximum bending moment M_{exp}, estimated equivalent uniform distributed load w_{max}, maximum deflection at midspan Δ_{max}, and composite action are presented in Table 1. The type of connection is denoted with the depth of the notch (*d*), the length of the notch (*l*), the diameter of the coach screw (ϕ). Triangular notches are denoted with “Tri”.

By comparing the control beam G1 with beam B1 which was the identical reduced half flange width version, it is found that the F_{max} of B1 (105.10kN) was approximately half the F_{max} of G1 (200.94kN) and the composite actions for G1 and B1 only differed by 5.23%. Based on this outcome, the F_{max} results of all the other reduced width beams can be doubled to evaluate the collapse load of the full width version beams, while the composite actions calculated are also representative of the full width version beams. The degree of composite action at collapse load for all the beams are in the range of 80.66% to 94.75% with an exception for beam F1 with a toothed plate connection that exhibitted a 65.52%.

Table 1 Experimental beam details and summary result of collapse test.

Beam Notation	Span (m)	Flange (mm)	Connection type (numbers) (mm)	Design Level	F_{max} (2Pc) (kN)	M_{exp} (kNm)	w_{eq} (kN/m)	Δ_{max} (mm)	DCA % at SLS
A1	8	600	25*d*x150*l* NCSϕ16 (6)	Under	87.30	116.40	14.55	64.05	87.60%
A2	8	600	25*d*x150*l* NCSϕ16 (6)	Under	75.27	100.36	12.54	63.19	92.81%
Average					81.28	108.38	13.55	63.62	90.20%
B1	8	600	25*d*x150*l* NCSϕ16 (10)	Well	105.10	140.13	17.52	63.06	99.23%
B2 #	8	600	25*d*x150*l* NCSϕ16 (10)	Well	97.52	130.03	16.25	73.82	96.88%
C1	8	600	30º_60º TriNCSϕ16 (10)	Well	89.72	119.63	14.95	58.30	96.93%
C2	8	600	30º_60º TriNCSϕ16 (10)	Well	110.00	146.67	18.33	66.71	97.56%
Average					99.86	133.15	16.64	62.50	97.25%
D1	8	600	50*d*x300*l* NCSϕ16 (6)	Under	80.84	107.79	13.47	48.14	98.09%
E1	10	600	50*d*x300*l* NCSϕ16 (6)	Under	79.59	132.65	10.61	93.81	98.88%
E2 ##	10	600	50*d*x300*l* NCSϕ16 (6)	Under	55.36	92.27	7.38	66.91	99.04%
F1	8	1200	Plate_2x333*l* Staggered (8)	Well	173.60	231.47	28.93	95.64	98.51%
G1 Control	8	1200	2x25*d*x150*l* NCSϕ16 (6)	Well	200.94	267.92	33.49	69.38	98.60%

Note:
1. Unless otherwise stated, all concrete used to construct beams were the commercial low shrinkage concrete (CLSC).
2. M_{exp} is the experimental maximum moment.
3. w_{eq} is the equivalent unifom distributed load calculated from the experimental maximum moment

\# Special low shrinkage concrete (SLSC) using limestone aggregate with 500microstrain shrinkage

\## Normal weight concrete (NC) 25MPa

Although beam F1 was well designed, it developed a large deflection at failure resulting in a relatively low composite action. This could have been caused by slippage of the teeth in the LVL and yielding of the plate along the shear plane of the connection as the load approached the collapse level. The horizontal slip measured in beam F1 was in the range of 3.7-5.6mm as opposed to 0.2-2.3mm in the beams with notched coach screw connections. It must be pointed out, however, that under the load levels of 40%, 60% and 80% of the collapse load, corresponding to the load at serviceability limit state, ultimate limit state and $0.8F_{max}$ respectively, the degree of composite action was 99.42%, 97.63% and 83.52%. Fig. 5 presents the connections horizontal slips measured in beams D1 and F1 at F_{max} with their corresponding shear force obtained using the experimental shear force-slip relationship derived from the push-out tests. The slips of connections measured in beam F1 were already in the post peak region showing that the connections had already yielded. Although the force in both toothed metal plate and 300mm rectangular notched connections were in the high range exceeding 100kN, the slip in the notched connection was significantly lesser.

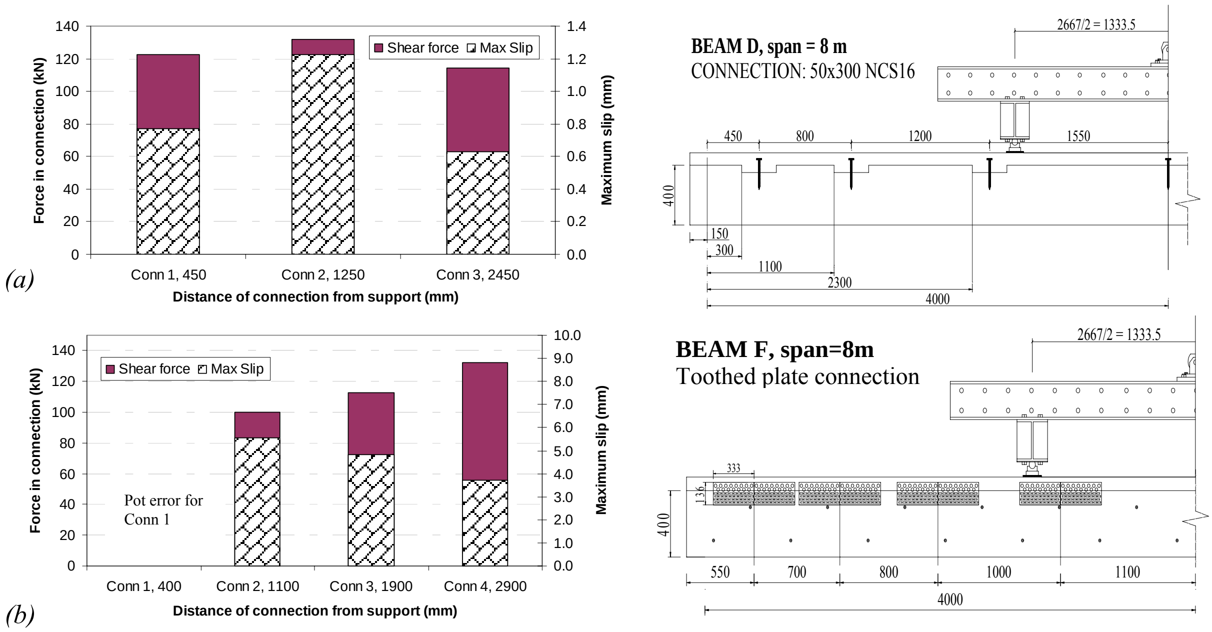

Fig. 5 Shear force in connections under the collapse load F_{max} and their corresponding maximum horizontal slip for beams (a) D1 and (b) F1, with location of the connectors along the beam length on the right-hand side.

Beams D1, E1 and E2 (which used a 300mm rectangular notched connection) showed a degree of composite action above 98% even though they were under designed compared to other under designed beams (A1 and A2 with 150mm rectangular notched connection) that only attained a degree of composite action of an average 90.20% (see Table 1). This proved that a 300mm rectangular notched coach screw connection is very effective. It was found that the strength of concrete significantly influenced the F_{max} of the beam, going from 79.59kN for specimen E1 with CLSC 35MPa characteristic compressive strength to 55.36kN for specimen E2 with NC 25MPa characteristic strength. Therefore, it is important to use higher strength concrete in order to increase the strength of the connections and, therefore, the load-bearing capacity of the composite beam. The use of triangular notched connection (beams C1 and C2) is also a viable option to the larger notch as it led to an average F_{max} = 99.86kN and composite action of 92.87%. Such values are similar to those obtained for the beam B1 with 150mm rectangular notched connection, with the advantage of the triangular notched connection being much easier to cut than the rectangular one.

Fig. 6 displays a typical experimental load $2P_c$ vs. midspan deflection curve for beam B2, together with two limit curves representing the behaviour of timber-concrete composite system with no connection ($K = 0$) and with fully rigid connection ($K = K_\infty$). These limit curves were determined using the γ-method [7] with $\gamma_1 = 0$ and $\gamma_1 = 1$, respectively. The analytical curve representing the load-deflection curve of the LVL beam alone is also plotted. The high degree of composite action can immediately be recognized by the experimental curve being in close proximity to the analytical fully composite curve.

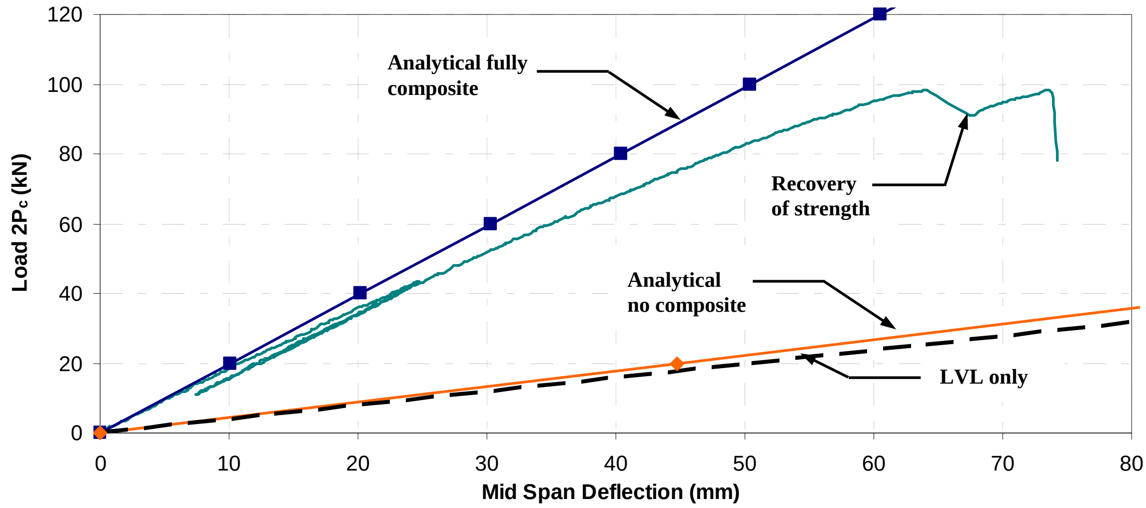

Fig. 6 Typical experimental load $2P_c$ vs. midspan deflection curve for beam B2

(a) *(b)*

Fig. 7 (a) Collapse of beam due to LVL tension fracture, (b) 300mm rectangular notched coach screw connection failure

The collapse of all the beams occurred due to fracture in tension of the LVL beam under the loading point at L/3 of the span (see Fig. 7a) with connector failed or largely plasticized. All notched connectors failed for shear and crushing of concrete, as it is displayed in Fig. 7b and in accordance with the outcomes of the push-out tests [5]. It is therefore not surprising that the concrete strength was found to significantly affect the load-bearing capacity of the composite beam as it does influence the shear strength of the connection. The first crack sound, which could be the beginning of yield in the connection, was heard at approximately $0.6F_{max}$ for most of the beams tested. In beams that were well designed and had a sufficient number of connectors, a recovery of strength was evident after the first connection failed (see Fig. 6). This is an important outcome as it ensures a moderate ductile behaviour of the composite beam which allows sufficient time for evacuation in the case of an emergency. The failure hierarchy of a timber-concrete composite system as observed in the tests performed is as follows: (1) crack sound in one or multiple connections as an early warning, (2) failure of connections starting from the support and moving inward, (3) after failure of the first outer connector, the shear load carried by the first connector is redistributed among the other connectors, leading to consecutive failures of the other connectors, (4) when all connectors have failed, the load is supported by only the LVL beams with zero composite action and final fracture of LVL in tension.

CONCLUSIONS

The paper presents the outcome of a full-scale experimental collapse test on semi-prefabricated timber-concrete composite beams using selected connections. The timber panel, made from LVL joists, plywood sheets, and with preassembled connectors, is prefabricated off-site, transported on site, craned into position and propped. The concrete slab is then poured on top of the panel. Rectangular and triangular notches cut from the LVL and reinforced with a coach screw were used as connectors. Toothed metal plates pressed in the LVL beams were also investigated as a possible alternative. The 300mm rectangular notches were showed to be more favourable than the toothed plate connection considering the following factors: (1) better degree of composite action achieved, (2) minimum horizontal slip in connection at failure, and (3) higher load-bearing capacity, F_{max} achieved. Alternatively, the triangular notched connection is a suitable option as it performed approximately the same as a 150mm rectangular notched connection, but it is significantly easy to construct. The use of stronger concrete enables better connection performance and, therefore, larger load-bearing capacity of the composite beam. The failure of the composite beams occurred with fracture of LVL in tension after failure of the connection with crushing and shear of concrete in the notches. Limited redistribution of shear force was noted from the outer to the inner notches. Although this redistribution is not enough to define the behaviour of the composite beam as ductile, it provides some advice should a collapse load be approached by the structure in the case of an emergency. Based on the high values of load-bearing capacity and degree of composite action achieved in the experimental tests, it can be concluded that the proposed semi-prefabricated composite system can be effectively used for medium to long-span floors.

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