DEVELOPMENT OF A DESIGN PROCEDURE FOR TIMBER CONCRETE COMPOSITE FLOORS IN AUSTRALIA AND NEW ZEALAND

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ABSTRACT

Medium rise commercial and multi-residential buildings (up to eight stories) represent significant markets that the timber industry can potentially penetrate. This is possible with the availability of advanced engineered wood product and 'new generation' composite structures. From the mid 2000's, the University of Technology, Sydney (UTS), in partnership with universities and industry key-players in Australia and New Zealand – overseen by Structural Timber Innovation Company (STIC) – has been active in investigating innovative structural systems that utilise timber and provide a competitive alternative to steel and concrete solutions.

Timber concrete composite (TCC) solutions have been gaining a lot of attention in Australia and New Zealand over the last few years. To address this emergence, researchers at UTS have focused on identifying and optimising TCC connections and outlining robust design procedure. This paper puts forward design guidelines that comply with Australian codes[1] and give consideration for ultimate limit state (ULS) and serviceability limit state (SLS) design requirements. Fabrication provisions are also provided in order to secure a sound and successful implementation of TCC floor solutions.

1. INTRODUCTION

Timber concrete composite (TCC) floor systems are relatively new to Australia and New Zealand and satisfactory performance requires a rigorous design procedure addressing both ultimate and serviceability limit states. TCC structures have a degree of complexity since they combine two materials that have very different mechanical properties and respond in different ways to their environment. The capabilities of both materials are used efficiently with the concrete member experiencing compression whilst the timber member takes tension and flexural actions. Most TCC structures exhibit partial (not full) composite action and this adds to the complexity of the system.

Several design procedures are discussed in the literature. Amongst these, the Eurocode 5 (EC5) [9] procedure is relatively straightforward and has been successfully implemented in Europe. It utilises a simplification for modelling the complex timber - concrete interaction known as the "Gamma coefficients" method, which manipulates properties of the concrete member to predict the cross-section characteristics of the structure. The Gamma method has been integrated in the design procedure put forward in this paper.

This paper starts with identifying the scope and major considerations of the design procedure, ensuring the procedure agrees with 1720.1–2010 [3]. It continues with short background information and a discussion of extensive R&D programs on connections and derivation of their characteristic properties. This empirical characterisation forms a critical aspect of the design procedure. An outline of the design requirements is listed before the design procedure is detailed. The design steps are logically arranged and address considerations for ultimate (ULS) and serviceability (SLS) limit states. The guidelines also include fabrication provisions and are provided to assist the successful implementation of TCC solutions. Further provisions related to uncertainties are listed before the conclusions.

1.1 SCOPE

The EC5 approach has been adopted as the underlying basis for the design procedures presented in this document; modified to comply with current design codes and practices in Australia[1]. It comprises normative parameters for the strength and safety (ultimate limit state) and informative guidelines for appearance, deflection limits and comfort of users (serviceability limit states). Whilst the latter must be defined by designers to meet the specific functional requirements of the floor under consideration, it is recommended that the guidelines in this document should be adopted as a minimum standard for TCC floors.

The design procedure capitalises on the results of a comprehensive research program currently in progress at UTS. It includes the latest developments and findings and has evolved from a previous version [10]. At the time of writing, there is still some uncertainty about aspects of long term deflection of TCC floors and the design procedures contained in this document are limited to floors not exceeding 8 m in span, utilising the notched connections prescribed in Table 1, and Figures 5 and 6.

[1] With minor modifications the same approach is relevant for New Zealand.

Provisions for connection manufacturing are also provided in Section 6.

1.2 BACKGROUND

Timber construction has a long history in Australia, both in residential and non-residential buildings. Numerous industrial storage buildings from the 1800's and recent residential dwellings are testimonies of successful applications of timber structures. The contributors to these successes have been the widespread availability of both high-quality native hardwoods and two-by-four framing material from softwood plantations. Today, wall frames and roof trusses dominate the use of timber in buildings. Interestingly, the engineered wood products emerging in the latter part of the 1900's have hardly penetrate the commercial and industrial building market whilst they have been very successful in the residential market.

In 2007 a major R&D project commenced at UTS. In mid 2008 the scope of the project was extended with the formation of the Structural Timber Innovation Company (STIC)[2], which is a research consortium that includes government bodies, industry groups and universities in Australia and New Zealand. Both projects aim at promoting and increasing the use of timber in multi-dwelling and non-residential buildings. From this broad scheme, UTS is leading the investigation of floor structures, focusing on innovative solutions and/or technologies, and developing efficient systems for floors, such as timber concrete composites and stressed-skin panels.

2. REVIEW OF THE LITERATURE ON TCC TECHNOLOGY

First implementation of TCC technology goes back to the inter-war period and was motivated by the lack of construction materials, in particular steel [19]. During the following decades, TCC technology experienced a severe drop/lack of interest, to only regain attention in the 1980's when it was recognised as a viable and efficient structural solution for floors. As a result, research work started to intensify and continued during the 1990's. These research efforts resulted in the development and commercialisation of 'new' shear connectors, and the application of TCC solutions to multi-storey residential buildings and road bridges [7].

TCC solutions exhibit great strength and stiffness properties – efficiently utilising the concrete compression strength and the timber tensile strength. The structural performance of TCC elements is thus characterised by the strength of the members and the efficiency of their shear connectors in composite action. Works by Siess *et al*. were instrumental in identifying the role of the shear connectors in multi-layered composites and understanding their structural behaviour (both slip and strength) for predicting the degree of composite action. TCC structures also exhibit excellent acoustics and vibrational behaviour, provide adequate responses to fire and offer thermal mass that is usable for regulating the interior environment of a building [18].

The characteristics of TCC solutions make them attractive and competitive for markets such as multi-dwelling (multi-storey) and non-residential buildings (commercial and industrial), which are traditionally dominated by steel and reinforced concrete solutions. In addition to fierce competitors, current Australian building regulations also prejudice the access to these markets, since the use of timber is currently limited to buildings under 3 storeys in height [10]. In comparing TCC solutions with reinforced concrete slabs and timber joist floors, TCC structures are respectively lighter and more economical [1], and have higher structural capability (strength and stiffness), better heat resistance, more acceptable responses to vibrations and better sound insulation [17].

In the mid 2000's, extensive research programs on TCC solutions commenced in Australia and New Zealand [13, 16, 20, 21]. This initial work highlighted the critical roles of the shear connector in providing suitable composite action (stiffness and strength) and effect of the formwork on slip and strength [11, 12]. The research by Crews *et al*. identified that notched connections structurally outperformed nail-plates, and assessed the most beneficial shape and geometry of a notch. Computer modelling and laboratory work helped mitigate the formwork interference by identifying the most favourable dimensions of the formwork. This contributed immensely to the development of a thorough and reliable design procedure [14].

3. CONNECTION BEHAVIOUR AND CHARACTERISATION

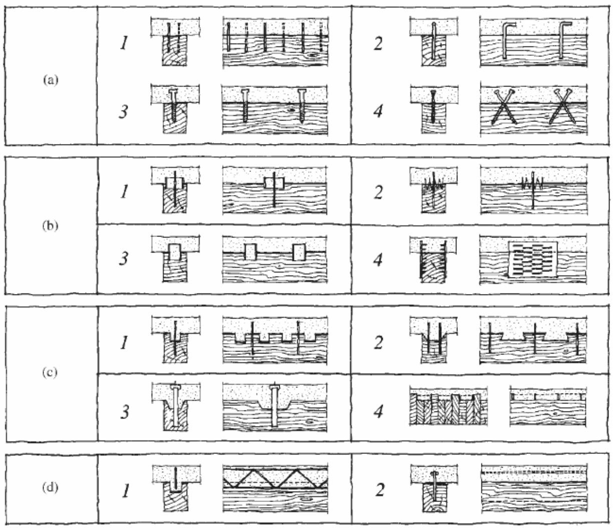
The structural behaviour of the connection is a significant parameter in the design of a TCC floor. The elastic properties of the connection are used for both limit states and for identification of the gamma coefficients in the design procedure.

3.1 CONNECTION BEHAVIOUR

An extensive (literature) review of shear connectors used in timber concrete composite structures, covering the period from 1985 to 2004, has been undertaken by Dias [15]. Elsewhere, Ceccotti also presents an overview of the timber-concrete connectors which are most commonly used to achieve composite action between the concrete and the timber members (Figure 1) [8].

The stiffness characteristics of some the shear connectors presented in Figure 1 are plotted in Figure 2. The load-slip plot in Figure 2 indicates that for this group of connector types, the stiffest connections are those in group (d), while the least stiff are in group (a).

[2] www.stic.co.nz



Notes on Figure 1:

(a1) nails; (a2) glued reinforced concrete steel bars; (a3, a4) screws; (b1, b2) connectors (split rings and toothed plates); (b3) steel tubes; (b4) steel punched metal plates; (c1) round indentations in timber, with fasteners preventing uplift; (c2) square indentations, with fasteners preventing uplift; (c3) cup indentation and pre-stressed steel bars; (c4) nailed timber planks deck and steel shear plates slotted through the deeper timber planks; (d1) steel lattice glued to timber; (d2) steel plate glued to timber.

Figure 1: Range of TCC connections [8].

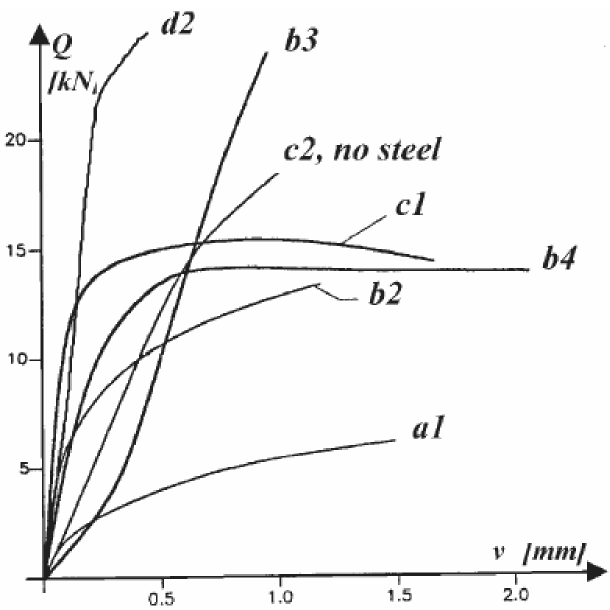


Figure 2: Schematic of load-slip behaviour of types of connection [8].

Connections in groups (a), (b) and (c) allow relative slip between the timber element and the concrete member, that is, the cross-sections do not remain planar under load – and the strain distribution is not continuously linear in the composite cross-section. Only connections in group (d) exhibit a planar behaviour, corresponding thus to fully composite action between timber member and the concrete slab. It can be assumed that TCC structures assembled with connectors from group (a) achieve 50% of the effective bending stiffness of TTC systems constructed with connectors from group (d) [7].

3.2 CONNECTION CHARACTERISATION

The behaviour and effectiveness of the tested shear connections were assessed based on their strength (failure load or maximum load), stiffness and failure mode. The strength of the connection specimens was defined as the maximum load that can be applied in the push-out tests before failure. Depending upon the failure mode, the connection specimens may have some load carrying capacity following the maximum load resulting in a ductile behaviour. The failure modes were therefore carefully documented in all tests. The connection stiffness or slip modulus, which represents the resistance to the relative displacement between the timber joist and the concrete slab, is one of the key parameters that define the efficiency of a shear connection. Stiffness for the serviceability limit state (SLS) and ultimate limit state (ULS) are essential to characterise a shear connection. The stiffness for SLS (K_{ser}) corresponds to the inclination of the load-slip curve between the loading start point (generally taken as 10% of failure load to overcome "settling in") and the 40% of the failure load. The stiffness for ULS (K_u) corresponds to the inclination of the load-slip curve between the loading start point and the 60% of the failure load. As a general rule, it can be assumed that $K_u = (2/3)\ K_{ser}$.

3.3 LABORATORY INVESTIGATION AT UTS – OBSERVATIONS AND STEPS TOWARDS SUITABLE CONNECTIONS

A number of shear connections have been tested using push-out tests on full scale specimens and load-deflection plots and stiffness for these connections have been determined. Parameters such as the type of connector, shape of notches, use of mechanical anchors and concrete properties have been investigated and analysis of this data has led to number of conclusions.

- Early research showed that the use of nail plates alone as shear connectors did not prove to be effective, whilst a combination of nail plates with either screws or concrete notches was more effective - especially incorporation of concrete notches.
- A number of concrete notch type shear connections were then tested such as trapezoidal, triangular type and polygonal notch and parameters such as slant angle, use of either coach screw or normal wood screw as mechanical fastener, inclination of the mechanical fastener, inclination of the slanting face and use of low shrinkage concrete were studied.
- Use of coach screws has the advantage of deeper penetration depth inside the concrete slab in comparison to normal wood screws due to their longer length. This resulted in a single coach screw providing higher shear capacity than a combination of four wood screws.
- Interesting results were obtained from the triangular type connections as these connections generally exhibited higher strength and stiffness than the trapezoidal notch connections and especially so for

triangular connections using 70-20 and 60-30 angle combinations.

- Polygonal notch connections were also found to be superior to the trapezoidal notch connections, however, the complex angle sequence makes such connections difficult to fabricate.
- On the other hand, triangular type connections are much easier to fabricate with a simple cutting sequence and do not need special tools for fabrication. Use of a slanted coach screw configuration in the triangular notch connections provided higher stiffness; however, the effect on characteristic strength was not significant, while steel plate placed on top of the coach screw did not provide any additional strength or stiffness. It should however be noted that the coach screws in the triangular notch provided only limited post peak plastic behaviour when compared to trapezoidal notch connections.
- The depth of the notch has a significant effect on both the stiffness and strength of the connections. Connections with 60 mm deep notch had superior strength and stiffness compared to the connections with 90 mm deep notch. Test results also showed that widening the slot dimension had a positive effect on strength and stiffness of the connections.
- The effect of the ratio of coach screw diameter to LVL thickness is one of the parameters that needs to be further investigated. Table 1 highlights the effect of the ratio of coach screw diameter to LVL thickness and suggests that there is no advantage to using 16mm diameter screws in 48mm (nominal 45 mm) thick LVL beams.

Whilst the variability of maximum load (strength) is considered to be acceptable, the variability of the characteristic stiffness properties highlights some of the uncertainty that is inherent in the performance of notched connections for TCC constructions. It is proposed to use the data generated to date, to refine connection performance and attempt to reduce that stiffness variability to lower levels that could lead to more efficient design of these types of floor structures.

3.4 EMPIRICAL CHARACTERISATION OF NOTCHED CONNECTIONS

The main results for both connection types (which are described schematically in Figure 3), are presented in Table 1.

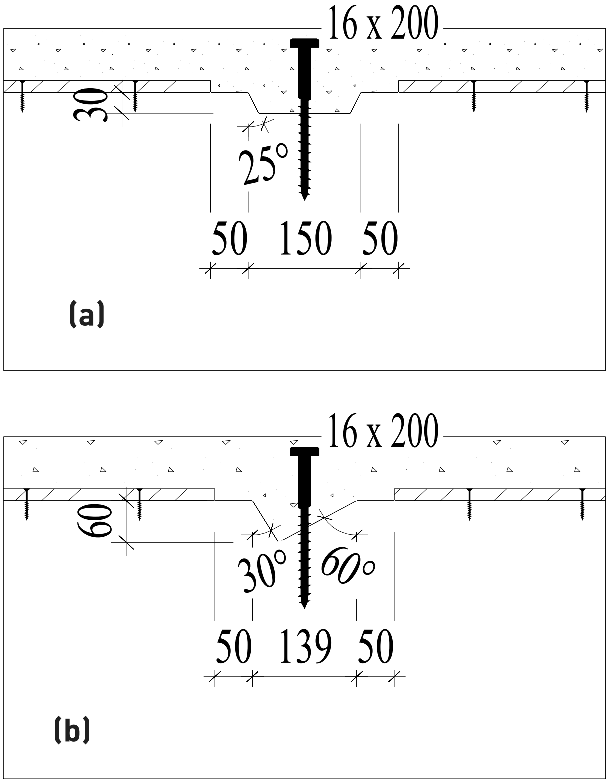


Figure 3: Notched connections – trapezoidal (a) and triangular (b) shapes (12 and 16 mm coach screw are used for 45 and 63 mm thick LVL respectively).

Table 1: Characteristic properties of notched connections – trapezoidal (T-Series) and triangular (B-Series) notch shapes.

Connection Description	Strength Q_k (kN)	K_{ser} (kN/mm)	K_u (kN/mm)
T1 – 48mm LVL, 16mm bolt	46 – 8.7%	87 – 20.5%	60 – 13.0%
T2 – 48mm LVL, 12mm bolt	46 – 6.6%	106 – 15.0%	87 – 17.9%
T3 – 63mm LVL, 16mm bolt	78 – 6.4%	109 – 19.3%	81 – 24.7%
T4 – 96mm LVL, 12mm bolt	89 – 10.0%	110 – 34.8%	93 – 39.3%
T5 – 126mm LVL, 16mm bolt	134 – 4.8%	124 – 41.3%	103 – 30.2%
B1 – 48mm LVL, 16mm bolt	55 – 8.1%	37 – 12.4%	36 – 15.2%
B2 – 48mm LVL, 12mm bolt	51 – 8.4%	115 – 48.4%	46 – 54.0%
B3 – 63mm LVL, 16mm bolt	66 – 7.7%	98 – 12.9%	74 – 27.7%
B4 – 96mm LVL, 12mm bolt	91 – 5.5%	156 – 19.8%	119 – 20.8%
B5 – 126mm LVL, 16mm bolt	120 – 11.6%	213 – 34.2%	150 – 22.7%

Notes: a) Integer = capacity; % = CV, b) Strength – 5th percentile based on a log normal distribution, c) Stiffness – 50th percentile.

4. DESIGN REQUIREMENTS

Load type and intensity, load combinations and modification factors for both the ultimate and the serviceability limit states have been defined in accordance with the AS/NZS 1170 series [4, 5].

The limit states that require checking can be summarised as follows:

1. the short-term ultimate limit state; where the structure response to the maximum load is analysed. It generally corresponds to short-term exertion of the structure.
2. the long-term ultimate limit state. This analysis focuses on the structure response to a quasi permanent loading and aims at avoiding failure due to creep of the timber member in particular*.
3. the short-term serviceability limit state. This corresponds to the instantaneous and short-term responses of the structure to an imposed load. The initial deflection is included in this verification.
4. the long-term serviceability limit state. This analysis aims to identify the service life behaviour of structure considering the time-dependent variations of the material properties; in particular creep. This assessment also takes into account the initial deflection.
5. the 1.0-kN serviceability limit state. This corresponds to the instantaneous response of the structure to an imposed point load of 1.0 kN at mid-span.

**Checking the end-of-life ultimate limit states corresponds to an attempt to analyse and assess the durability and reliability of the structure.*

5. DESIGN PROCEDURE

The design procedure has three fundamental stages:

1. The initial stage of the design procedure focuses on identifying of the characteristics of the TCC cross-section.
2. Assessment of the strength capacity of the structure is completed in the second stage of the procedure and is consistent with AS 1720.1–2010 [3]; whilst
3. The final stage deals with the serviceability limit state.

5.1 CROSS-SECTION CHARACTERISTICS

The effective (apparent) stiffness of the composite cross-section is:

$$(EI)_{ef} = E_c I_c + E_t I_t + \gamma_c E_c A_c a_c^2 + \gamma_t E_t A_t a_t^2 \qquad (1)$$

Note: The subscripts c and t refer to concrete and timber respectively, unless otherwise specified. The contribution of the formwork (if present) is neglected in the design.

where the section properties in Equation 1 are given by:

$$I_c = \frac{b_c h_c^3}{12} \qquad I_t = \frac{b_t h_t^3}{12} \qquad (2a); (2b)$$

$$\gamma_c = \frac{1}{1 + \dfrac{\pi^2 E_c A_c s_{ef}}{K_i L^2}} \qquad \gamma_t = 1 \qquad (3a); (3b)$$

Note: The connection stiffness coefficient K_{ser} is used for the serviceability design and K_u is used for the ultimate design.

$$A_c = b_c h_c \qquad A_t = b_t h_t \qquad (4a); (4b)$$

$$a_c = \frac{\gamma_t E_t A_t H}{\gamma_c E_c A_c + \gamma_t E_t A_t} \qquad a_t = \frac{\gamma_c E_c A_c H}{\gamma_c E_c A_c + \gamma_t E_t A_t} \qquad (5a); (5b)$$

The height factor "H" is defined by:

$$H = \frac{h_c}{2} + a_f + \frac{h_t}{2} \qquad (6)$$

Note: a_f is the thickness of the formwork.

where:

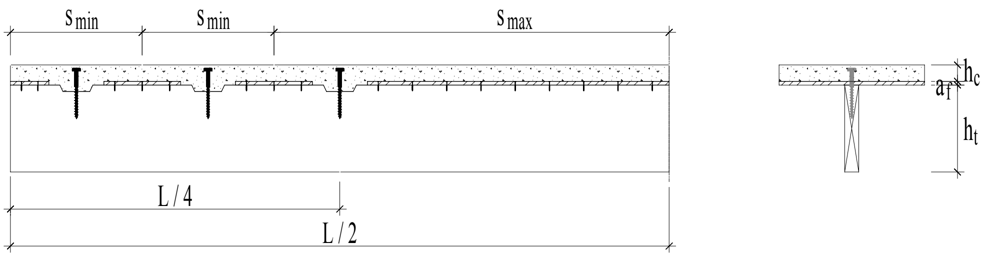
- the tributary width of the concrete member is assessed with AS 3600–2009, Section 8.8 [2]:

  $$b_c = b_t + 0.2a \qquad b_c = b_t + 0.1a \qquad (7a); (7b)$$

- the effective spacing (refer to Figure 4) of the connections is given by:

  $$s_{ef} = 0.75 s_{\min} + 0.25 s_{\max} \qquad (8)$$

  where all connectors are evenly spaced within the end quarter spans and s_{min} is about $2.h_t$ or typically 600 mm to 800 mm.

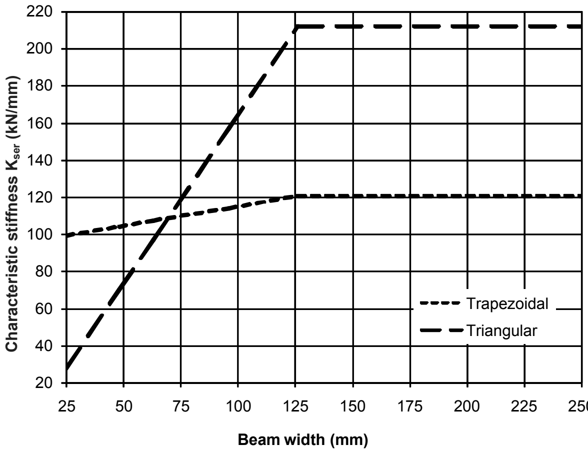


Note: The notch shapes can be trapezoidal or triangular and comply with the fabrication provisions provided under Section 6 of this procedure.

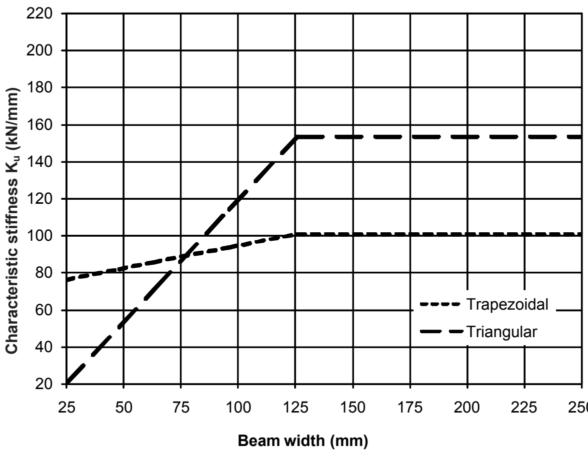
Figure 4: Connection-related distances.

- the stiffness of the connection corresponds to (refer to Figure 5):

$$K_{ser} = \frac{0.4R_m}{v_{0.4}} \qquad K_u = \frac{0.6R_m}{v_{0.6}} \qquad \text{(9a); (9b)}$$



(a)



(b)

Note: The beam width also refers to the width of the notch.

Figure 5: Serviceability (K_{ser}) and ultimate (K_u) characteristic stiffness of trapezoidal and triangular notches.

5.2 STRENGTH OF THE COMPOSITE CROSS-SECTION – CONCRETE & TIMBER MEMBERS

The load combinations and factors for the ultimate limit state must comply with the relevant provisions of AS/NZS1170 series [4, 5]. The checks imposed on a structure under flexural action or flexural and axial actions are described in Sections 3.2 and 3.5 of AS 1720.1–2010 [3] respectively. These requirements apply to TCC floor structures as follows:

- bending strength – the concrete and timber members resist a combination of bending moment and/or axial force.
- flexural shear strength – the timber member resists the flexural shear force.
- bearing strength – the timber member resists the support action/reactions.
- interface strength.

5.2.1 Strength Requirements for Bending Strength

At the extreme fibres – upper and lower – the concrete and timber members experience compression and tension stresses which result in combined bending and axial stresses as defined in Equation 10. The check is completed for the upper and lower fibres of the concrete member and for the lower fibre of the timber member. [3]

$$\frac{N^*}{N_d} + \frac{M^*}{M_d} \leq 1.0 \quad \textit{(expressed as local and global stress ratios)} \qquad (10)$$

The general expression for bending stress is defined in Equation (11):

$$\sigma_{b,i} = \pm \frac{1}{2} \frac{E_i h_i M^*}{(EI)_{ef}} \quad \textit{(local stress)} \qquad (11)$$

Specifically, the stresses in the concrete and timber member respectively are:

$$\sigma_{b,c} = \pm \frac{1}{2} \frac{E_c h_c M^*}{(EI)_{ef}} \qquad \sigma_{b,t} = \pm \frac{1}{2} \frac{E_t h_t M^*}{(EI)_{ef}} \qquad \text{(11a); (11b)}$$

Equations (11a) and (11b) respectively identify the bending moment capacity:

$$M_{d,c} = \phi f_c' \frac{2(EI)_{ef}}{\gamma_c E_c h_c} \qquad (12a)$$

$$M_{d,t} = \phi k_1 k_4 k_6 k_9 k_{12} f_b' \frac{2(EI)_{ef}}{\gamma_t E_t h_t} \qquad (12b)$$

Note: The characteristic bending strength must be reduced for timber member exceeding depth of 300 mm or modified in accordance with the manufacturer's specification. The strength sharing factor (k9) must be taken as unity for EWP.

These capacities must be greater than the design moment M^*, which is derived from loading requirements and boundary conditions of the TCC structure. The axial (in-plane) stress is predicted using Equation 13:

$$\sigma_{c/t,i} = \pm \frac{\gamma_i E_i a_i M^*}{(EI)_{ef}} \qquad (13)$$

Specifically, the stresses in the concrete and timber member respectively are:

$$\sigma_{c,c} = -\frac{\gamma_c E_c a_c M^*}{(EI)_{ef}} \qquad \sigma_{t,t} = \frac{\gamma_t E_t a_t M^*}{(EI)_{ef}} \qquad \text{(13a); (13b)}$$

[3] An efficient design of a TCC cross-section occurs when the concrete member is fully under compressive stress and the timber member is mainly subjected to tensile and bending stress. If some portion (generally small) of the concrete member experience tension stress, this contribution is ignored in the design. It is also possible for the timber beam to experience compression, but this is not critical because the timber material exhibits adequate compression capacity.

Assessment of the axial stress is derived from the flexural action. However, Equations 13a and 13b can be manipulated to identify the (corresponding) design axial force:

$$N_c^* = \sigma_{c,c} A_c \qquad N_t^* = \sigma_{t,t} A_t \qquad \text{(14a); (14b)}$$

where the allowable axial capacities are defined as:

$$N_{d,c} = \phi f_c^{'} A_c \qquad N_{d,t} = \phi k_1 k_4 k_6 f_t^{'} A_t \qquad \text{(15a); (15b)}$$

Note: The characteristic tension strength must be reduced for timber member exceeding depth of 150 mm or modified in accordance with the manufacturer's specification.[4]

5.2.2 Strength Requirements for Flexural Shear Strength

In the absence of structural reinforcement in the concrete member, the flexural shear strength is provided by the timber member, therefore;

$$V_d \geq V^* \qquad (16)$$

where for rectangular sections:

$$V_d = \phi k_1 k_4 k_6 f_s^{'} \frac{2A_t}{3} \qquad (17)$$

Note: Some conditions, (for example use of a deep notch), may require reducing the shear plane area by using the net area of the (beam) cross-section.

5.2.3 Strength Requirements for Bearing Strength

The bearing strength is provided by the timber member, therefore;

$$N_{d,p} \geq N_p^* \qquad (18)$$

in which:

$$N_{d,p} = \phi k_1 k_4 k_6 k_7 f_p^{'} A_p \qquad (19)$$

5.3 STRENGTH OF THE COMPOSITE ACTION (CONNECTION CAPACITY)

The connection (or notch) transfers the shear force occurring between the members under flexure. The actual mechanics of this force transfer are relatively complex. However a prescriptive approach that defines connection capacities (based on empirical test data – refer to Figure 6) that ensures the design procedure remains user-friendly, has been adopted for this document.

5.3.1 Shear Strength of the Connection

A global assessment of the connection strength is performed. It includes the assessment of the strength of the connection closest to the support, V_{max} (shear force

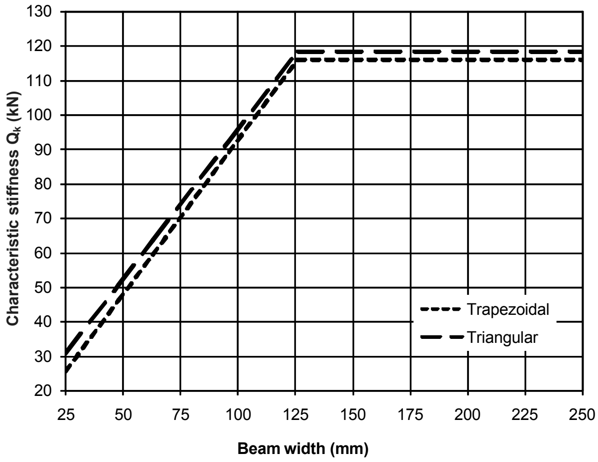


Figure 6: Characteristic strength (Q_k) of trapezoidal and triangular notches.

at the support), and the connection located at the quarter-span area, $V_{L/4}$ (shear force at the quarter span).

$$N_{d,j} \geq Q^* \qquad (20)$$

Note: Refer to Figure 6, for empirical strengths of the specified connections.

where:

$$N_{d,j} = \phi k_1 k_4 k_6 Q_k \qquad (21)$$

and the effective shear force in the connection located near the support equals:

$$Q^*_{\left(V^*_{\max}\right)} = -\frac{\gamma_c E_c A_c a_c s_{\min}}{(EI)_{ef}} V^*_{\max} \qquad (22)$$

and the effective shear force in the connection located at the 'quarter' span:

$$Q^*_{\left(V^*_{L/4}\right)} = -\frac{\gamma_c E_c A_c a_c s_{\max}}{(EI)_{ef}} V^*_{L/4} \qquad (23)$$

5.3.2 Shear Strength of the Timber

The shear strength of the timber – tangential shear action in the area located between the support and the first connection is assessed and checked as follows:

$$N_{d,l} \geq V^* \qquad (24)$$

where:

$$N_{d,l} = \phi k_1 k_4 k_6 f_s^{'} \left(b_t l_s\right) \qquad (25)$$

5.4 SERVICEABILITY VERIFICATION

The load combinations and factors for the serviceability limit states (SLS) are defined in the AS/NZS 1170 series [4, 5]. Serviceability of the TCC structure is undertaken by checking the deflections against the limits defined to suit the functional requirements of the building being

[4]Modification of the design approach enforces in AS 1720.1–2010 [3].

designed. In the absence of any specific limits the following are recommended:

- short term live load only, limited to span / 300 (initial deflection to be included),
- short term point load deflection (1 kN), limited to 2.0 mm,
- long term permanent and live loads (reduced), limited to span / 250 (initial deflection to be included),
- long term permanent load only, limited to span / 300 (initial deflection to be included).

Note: The effective stiffness $(EI)_{ef}$ of the structure is approximated as defined in Equations 1 to 9. Where deflection is critical E_{5th} should be used in $(EI)_{ef}$ calculation.

The mid-span deflection under uniformly distributed load is assessed as follows:

$$\Delta = \frac{5 j_2 \left(G^* + w^*_{imp}\right) L^4}{384 (EI)_{ef}} \quad (26)$$

Note: Whilst it is understood that the creep behaviour of TCC floors is quite complex, the "creep component" for long term defections is modelled using the j_2 factor. This is consistent with AS 1720.1–2010 [3], which uses a simplified multiplier to the initial short term deflection.

The value of the j_2 factor is defined to suit the loading condition and duration (based on empirical test data – refer to Table 2).

The mid-span deflection under point load at mid-span is assessed with Equation 27 with the load sharing factor g_{41}, refer to AS 1720.1–2010, Section E8 [3]:

$$\Delta = \frac{P^* L^3}{48 (EI)_{ef}} \quad (27)$$

Table 2: Creep coefficient j_2.

Instantaneous and short-term Live loads	1.0
Medium-term Live loads (reduced)	2.0
Long-term Live loads (reduced)	3.0
Permanent loads only	4.0

5.4.1 Initial Deflection

The initial deflection at mid-span is calculated with Equation 28 considering the early shrinkage of the concrete member:

$$\Delta_{ini} = \frac{L \Delta_{c,rel}}{8\left(h_c + a_f + h_t\right)} \quad (28)$$

in which $\Delta_{c,rel}$ represents the length variation of the concrete member due to shrinkage after 28 days (autogenous and drying). ε_{cs} refer to AS 3600–2009, Clause 3.1.7 [2].

$$\Delta_{c,rel} = \varepsilon_{cs(28)} L \quad (29)$$

5.4.2 Instantaneous Deflection (Live Loads Only)

The shrinkage and creep effect of the concrete member and the creep of the timber is neglected (j_2 = 1.0).

(a) imposed load deflection check under uniformly distributed load, from Equation 26,

(b) deflection under 1.0 kN (vibration check), from Equation 27.

5.4.3 Short-term, Long-term and End-of-life Deflection

The shrinkage and creep of the concrete member and the creep of the timber are accounted for (*j2* refer to Table 2). The deflection is calculated for:

(a) permanent and imposed load (deflection check under uniformly distributed load), and

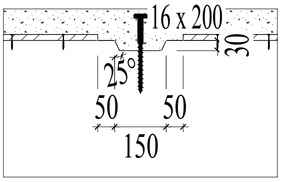
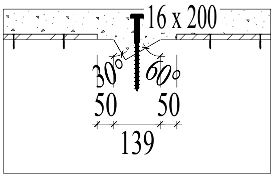
(b) permanent load only (deflection check under uniformly distributed load)

using Equation 26.

6. MANUFACTURING PROVISIONS

For the safety of the design and compliance with the design procedure, the notched connections must be manufactured in compliance with directives provided in Table 3. The manufacturing requirements include geometrical and dimensional aspects.

Table 3: Manufacturing provisions for the notches.

<table>
<tr><th>Connection types with geometry and dimensions in mm</th><th>For beam thickness 50 mm or less</th><th>For beam thickness more than 50 mm</th></tr>
<tr><td>
</td><td rowspan="2">Coach screw Ø: 12 mm and l_p: 80 mm or at least the length of the thread.</td><td rowspan="2">Coach screw Ø: 16 mm and l_p: 100 mm or at least the length of the thread.</td></tr>
<tr><td>
</td></tr>
</table>

7. PROVISIONAL COMMENTS

The Gamma procedure is a robust and safe design approach. However, there are several aspects of it that are not yet fully understood. These parameters include:

- shear strength of the concrete notch – effect of the coach screw,
- flexural shear strength of the beam – effect of deep notch and use of the net area of the shear plane,
- short-term serviceability – initial deflection and effect of concrete type and its curing,
- long-term deflection – identification of the creep coefficient j_2 (global creep),
- distribution and spacing of the notch – understanding of s_{min} and s_{max}.

A better understanding of these parameters may contribute to the enhancement of the accuracy of the design and the optimisation of TCC solutions. One of these parameters, that is, the long-term deflection is already under investigation at UTS. It is anticipated that UTS will examine the other parameters in a near future.

Further work will also focus on making the design procedure more user-friendly wherever possible whilst preserving the safety and functionality of the design.

8. CONCLUSION

Timber concrete composite (TCC) solutions have generated a great deal of interest in Australia and New Zealand over the last decade. This can be related to their satisfying the ultimate and serviceability expectations from modern architecture and design, which can help them capture important markets such as medium rise commercial and multi-storey residential buildings (up to eight stories). TCC structures also represent a genuine alternative to steel and concrete solutions.

TCC structures utilise the mechanical properties of both materials even though the composite action is only partial. As a result, they exhibit manifold complexity. The design methodology presented in this paper has adequately addressed the complexity of TCC structures, including the partial composite action provided by the connection, and imposes a comprehensive series of strength checks on the cross-section components, and serviceability checks with consideration of the long term performance of the structure.

The proposed design procedure is an adaptation from the design procedure of EC5 and modifications have been made to suit local practices. Adapting this procedure to suit Australian practices has been a challenging exercise and where assumptions have had to be made due to uncertainties, these have erred on being conservative. These assumptions are also areas for further research in order to address the uncertainties associated with them. The design procedure thus reflects and capitalises on research and development recently undertaken and in progress in Australia and New Zealand.

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