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Timber Concrete Composite Floors

Technical Design Guide issued by Forest and Wood Products Australia



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Introduction

Timber concrete composite (TCC) floor systems are relatively new to Australia 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. In addition, most TCC structures exhibit partial (not full) composite action.

There are several design procedures for TCC structures. Among these, the Eurocode 5 (EC5) procedure¹ is relatively straightforward and has been successfully implemented in Europe. It uses 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.

This Guide presents a design procedure for TCC floor structures that is based on the Gamma method and *AS 1720.1 Timber structures Part 1: Design methods*.

The Eurocode 5 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. 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). While the latter must be defined by designers to meet the specific functional requirements of the floor under consideration, it is recommended that the serviceability guidelines in this document should be adopted as a minimum standard for TCC floors.

At the time of publication of this Guide, there is still uncertainty about some aspects of long-term deflection of TCC floors. As such, it is recommended that designers exercise caution when applying the design procedures contained in this document to floors exceeding 8 m in span, utilising the notched connections and crossed screws. This caveat restriction is due to a lack of research data at this stage to support the behaviour of floors and connections for spans exceeding 8 m. Some general considerations for manufacturing the notched connections are presented in this Guide.

Design of floor diaphragms for wind loading has been described in detail in WoodSolutions Technical Design Guide #35: *Floor Diaphragm*.

Fire resistance design is not covered in this Design Guide, for further information on Fire Design, please refer to WoodSolutions Technical Design Guide #15: *Fire Design*.

Design Requirements

The design procedure addresses performance requirements for the strength (normative) and serviceability (advisory or informative) limit states. 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 1170 Structural Design Action series (AS 1170).

The limit states that require checking are:

1. **Short-term ultimate limit state**, where the response of the structure to the maximum load is analysed. It generally corresponds to short-term exertion of the structure.
2. **Long-term ultimate limit state**, where the analysis focuses on the response of the structure to a quasi-permanent loading and avoiding failure due to creep of the timber member in particular. (Checking the end-of-life ultimate limit states corresponds to analysis and assessment of the durability/reliability of the structure.)
3. **Short-term serviceability limit state**, which corresponds to the instantaneous response of the structure to an imposed load.
4. **Long-term serviceability limit state** analysis considers time-dependent variations of the material properties; particularly creep, to identify the service life behaviour.
5. **1.0-kN serviceability limit state**: the instantaneous response to and imposed load of 1.0 kN at mid-span provides an indication of dynamic behaviour.

Design Procedure

The design procedure has three fundamental stages:

1. Identifying the characteristics of the TCC cross-section.
2. Evaluation of the strength capacity of the structure.
3. Assessment of the serviceability limit states.

3.1 Cross-section Characteristics

The free body diagram of a T-shape TCC module with partial composite action including internal and external loads is illustrated in Figure 3.1.

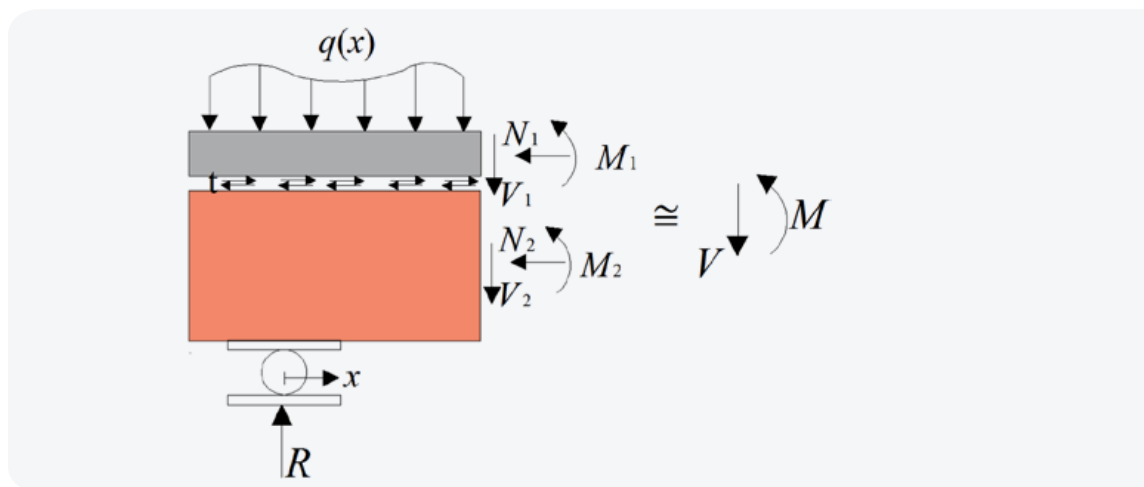


Figure 3.1: Internal and external loads applied on TCC system. Source: Moshiri, F².

The Gamma method in Eurocode 5 (part 1 annex B) has been recognised as the only specific design provision used to analyse TCC structures, with reasonable accuracy in determining the resultant stresses and deformations.

In the Gamma method, it is assumed there is no vertical separation between concrete slab and timber joist and plane sections remain plane, except for the discontinuity at the connection interface.

The Gamma method evaluates effective bending stiffness (EI_{ef}) using the shear bond coefficient (γ). The effective bending stiffness is used to check the design resistance of the connector and the stress values in the timber and concrete.

The effective bending stiffness depends on the shear bond coefficient of the concrete, where the shear bond coefficient of the concrete depends on the spacing and slip modulus of the connectors. Usually, the shear bond coefficient of regular connection is within the range of 0.1–0.4. A shear bond coefficient of 0 represents the layers which behave independently with no force couple resisted by the composite section ($EI_{ef} = EI_{min} = E_c I_c + E_t I_t$), while a shear bond coefficient of 1 indicates a fully composite beam with no slip in the interface ($EI_{ef} = EI_{max} = 4EI_{min}$).

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

$$(EI)_{ef} = E_c I_c + E_t I_t + \gamma_c E_c A_c a_c^2 + \gamma_t E_t A_t a_t^2 \quad (3-1)$$

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.

The second moment of area for concrete and timber components is:

$$I_c = \frac{b_c h_c^3}{12} \quad (3-2)$$

$$I_t = \frac{b_t h_t^3}{12} \quad (3-3)$$

where b and h are width and height of composite members, respectively. The tributary width of the concrete member (b_c) is assessed using Equations (3-4) and (3-5), which are derived from AS 3600:2009 *Concrete structures*, Section 8.8:

$$b_c = b_t + 0.2a \quad (\text{for T- beams}) \quad (3-4)$$

$$b_c = b_t + 0.1a \quad (\text{for I- beams}) \quad (3-5)$$

where a is distance between points of zero bending moment, which for continuous beams may be taken as $0.7L$.

The shear bond coefficients of concrete and timber components are:

$$\gamma_c = \frac{1}{1 + \frac{\pi^2 E_c A_c s_{ef}}{K_i L^2}} \quad (3-6)$$

$$\gamma_t = 1 \quad (3-7)$$

where E_c and A_c are modulus of elasticity (MOE) and section area of the concrete; s_{ef} represents effective spacing of the connectors; K_i is slip modulus of the connector; and L is span length of the beam.

The section areas of concrete and timber are:

$$A_c = b_c h_c \quad (3-8)$$

$$A_t = b_t h_t \quad (3-9)$$

The spacing for the commercially available connectors in Europe is within the range of 100 to 500 millimetres³. Based on the values for shear force at the interface, the spacing of connectors can be variable and an effective constant spacing (s_{ef}) can be assumed during calculation of shear bond coefficient⁴. The minimum, maximum and effective spacing for notched and screws connections (refer to Figure 3.2 and Figure 3.3) of the connections is given by:

$$s_{ef} = 0.75s_{min} + 0.25s_{max} \quad (3-10)$$

where s_{min} and s_{max} are the spacing at the beam ends and mid-span, respectively⁴. All connectors are evenly spaced within the end quarter spans, as indicated in Figure 3.2.

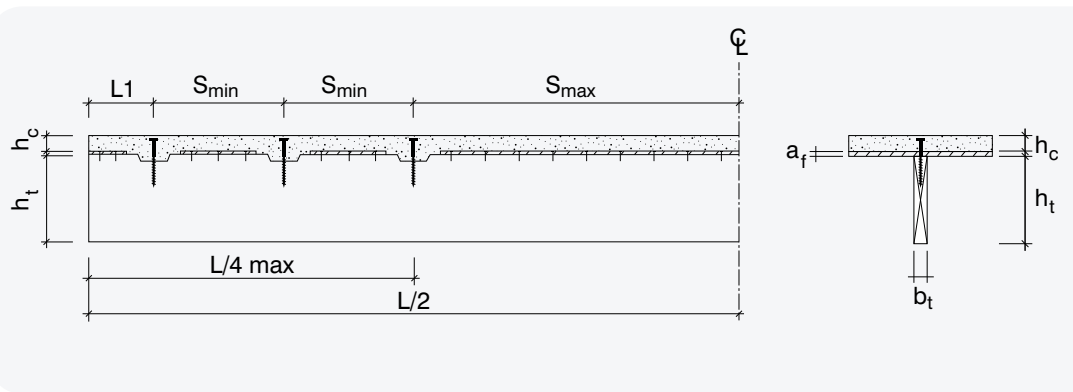


Figure 3.2: Connection-related distances and spacing for notched connections.

The notch shapes can be trapezoidal or triangular and should comply with the fabrication provisions and geometry specified in Section 5. The recommended minimum number of notches is three, at each end of the beam.

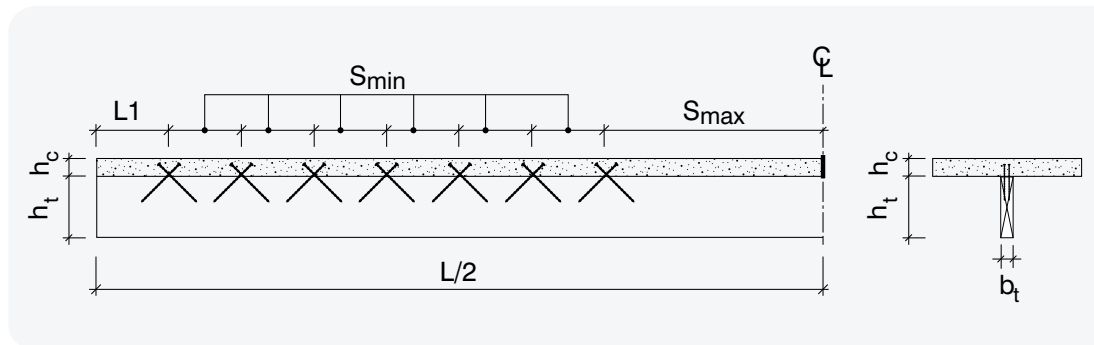


Figure 3.3: Connection-related distances and spacing for SFS connections.

The details of the connections are listed in Table 3.1.

Type of Connection:	L_1 (mm)	S_{min} (mm)	S_{max} (mm)
trapezoidal	300 + length of bearing	$S_{min} \geq 300$	$L_1 + S_{min} \cdot \text{no. connections} \geq L/4$
triangular	280 + length of bearing	$S_{min} \geq 280$	$L_1 + S_{min} \cdot \text{no. connections} \geq L/4$
SFS - VB-48-7.5x165	300	$100 \leq S_{min} \leq 300$	$L/2 - L_1 + S_{min} \cdot \text{no. connections}$

Table 3.1: Details of the connections.

For a T-shape TCC module as shown in Figure 3.4, the distance between centroid of timber component and centroid of TCC section (a_t) and the distance between centroid of concrete component and centroid of TCC section (a_c) are given by:

$$a_c = \frac{\gamma_t E_t A_t H}{\gamma_c E_c A_c + \gamma_t E_t A_t} \quad (3-11)$$

$$a_t = \frac{\gamma_c E_c A_c H}{\gamma_c E_c A_c + \gamma_t E_t A_t} \quad (3-12)$$

The height factor (H) is defined by:

$$H = \frac{h_c}{2} + a_f + \frac{h_t}{2} \quad (3-13)$$

where h is the height of timber and concrete members and a_f is thickness of the formwork (if present).

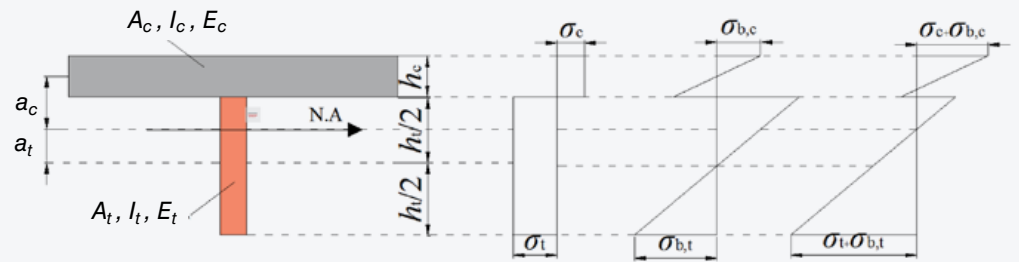


Figure 3.4: T shape TCC section - parameters and stress distribution of a partially composite beam. Source: Moshiri, F².

The stiffness values for each connection are derived from:

$$K_{ser} = \frac{0.4R_m}{v_{0.4}} \quad (3-14)$$

$$K_{ef} = \frac{K_{ser}}{j_2} \quad (3-15)$$

$$K_u = \frac{0.6R_m}{v_{0.6}} \quad (3-16)$$

This design guide uses three connection types that have been extensively tested to derive characteristic properties for both strength and stiffness. Two of these connection types require the fabrication of a notch in the timber beam (Figure 3.5), which are referred to as 'trapezoidal' and 'triangular' notched connections reinforced with vertical coach screw. The third type is un-notched, and uses proprietary SFS screws (Figure 3.6).

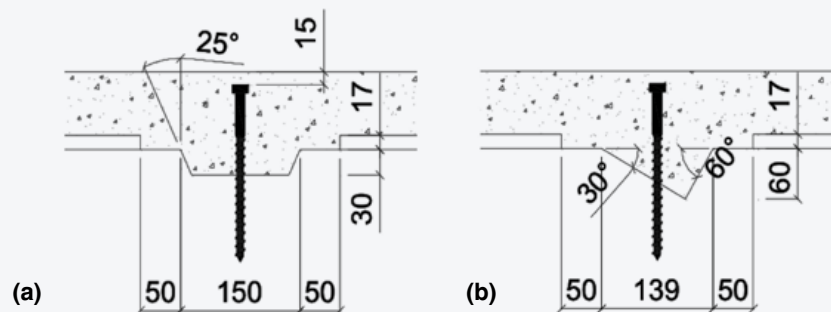


Figure 3.5: Notched connections – trapezoidal (a) and triangular (b).

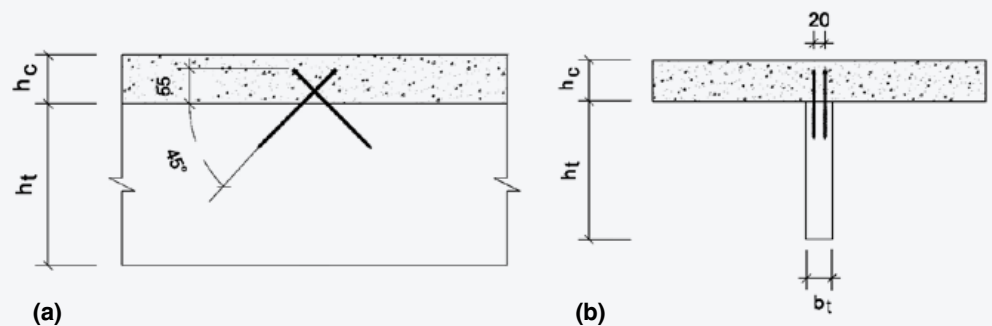


Figure 3.6: Cross SFS screw connection detail – profile (a) and cross-section (b).

The SFS screw connector developed in 1992 is recognised as a proprietary system for TCC structures, either in the construction of new flooring systems or the rehabilitation of existing timber floors in Europe. The double-headed screw consists of two parts with a diameter of 6 mm as an anchor in the concrete, and another threaded 165 mm long with a diameter of 7.5 mm as an anchor in the timber (total length of 220 mm).

The use of SFS screws is prescriptive and limited to the specific type of screws tested, which were VB-48-7.5x165, inclined at 30° to 45° as shown in Figure 3.6. The characteristic properties for SFS screws (per connection – one screw pair) are as follows:

SFS screws inclined at 45°

$$\begin{aligned} Q_k &= 33 \text{ kN} \\ K_{serv} &= 70 \text{ kN/mm} \\ K_{ult} &= 44 \text{ kN/mm} \end{aligned}$$

SFS screws inclined at 30°

$$\begin{aligned} Q_k &= 37 \text{ kN} \\ K_{serv} &= 55 \text{ kN/mm} \\ K_{ult} &= 44 \text{ kN/mm} \end{aligned}$$

Details of the equations used to generate these values are given in Appendix A2.

The characteristic strength of notched connections is the same and can be derived from Figure 3.7.

Although the creep behaviour of TCC floors is quite complex, the 'creep component' for long-term deflections is modelled using the j_2 factor. This is consistent with AS 1720.1, which uses a simplified multiplier on the initial short-term deflection. A value of j_2 between 3.0 (controlled) and 4.0 (variable environment) is currently recommended for indoor applications (Table 3.2) where j_2 represents stiffness modification factor – load duration.

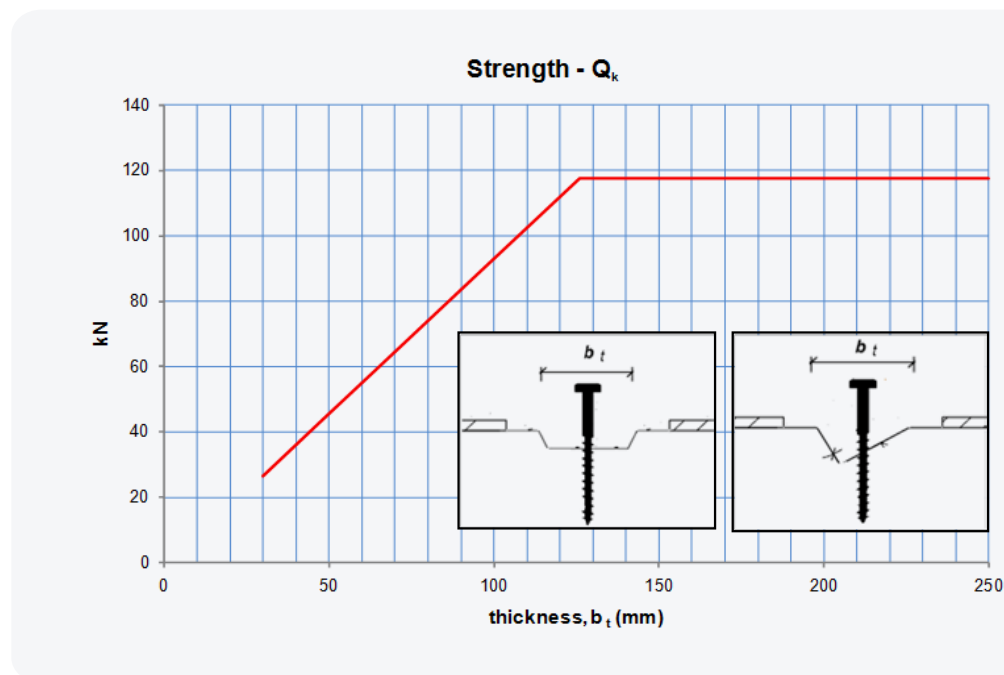


Figure 3.7: Characteristic strength of notched connections.

The characteristic stiffness of notched connections for serviceability and ultimate limit states can be derived from Figure 3.8 and Figure 3.9, respectively. Details of the equations used to generate these figures are presented in Appendix A2.

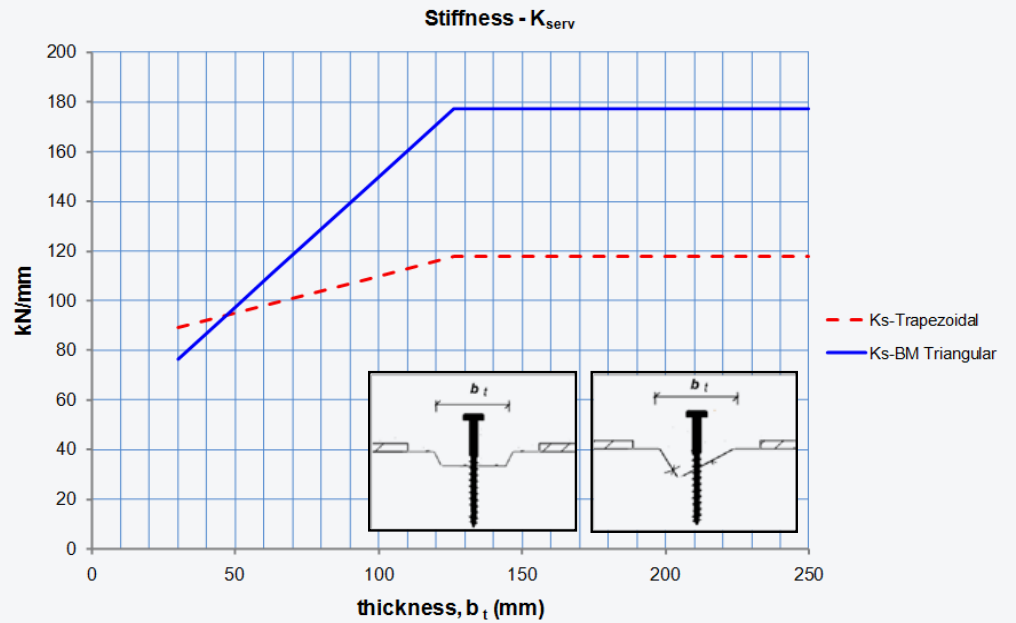


Figure 3.8: Characteristic stiffness (K_{serv}) of notched connections.

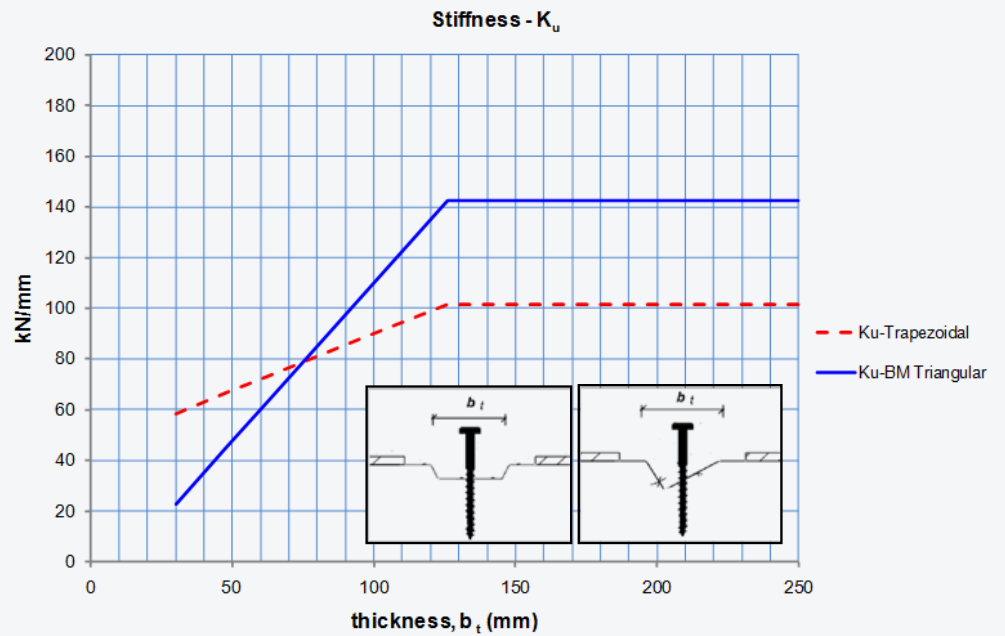


Figure 3.9: Characteristic stiffness (K_u) of notched connections.

Since the test data is not yet available for the thicknesses exceeding 126 mm, the characteristic properties are assumed to be constant beyond this point.

3.2 Strength of the TCC section – Concrete and Timber Members

3.2.1 Strength Requirements for Flexural Action

The load combinations and factors for the ultimate limit state (ULS) must comply with the relevant provisions of AS 1170. 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, 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.
- Strength of the connection interface

3.2.2 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 Figure 3.4. Equation (3-16) should be checked for the upper and lower fibres of the concrete member and for the lower fibre of the timber member.

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 stress. If some portion of the concrete member experiences 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.

$$\frac{\sigma_n}{f_n} + \frac{\sigma_b}{f_b} \leq 1.0 \quad (3-17)$$

where f_n and f_b are axial (tensile or compressive) and bending strength of composite members (timber and concrete), respectively, while σ_n and σ_b represent effective axial and bending stresses, respectively.

The general expression for bending stress of composite member (timber or concrete) is defined in:

$$\sigma_{b,i} = \pm \frac{1}{2} \frac{E_i h_i M^*}{(EI)_{ef}} \quad (3-18)$$

Specifically, the bending stresses in the concrete and timber members, $\sigma_{b,c}$ and $\sigma_{b,t}$, respectively, are:

$$\sigma_{b,c} = \pm \frac{1}{2} \frac{E_c h_c M^*}{(EI)_{ef}} \quad (3-19)$$

$$\sigma_{b,t} = \pm \frac{1}{2} \frac{E_t h_t M^*}{(EI)_{ef}} \quad (3-20)$$

where M^* is the design moment due to factored load and $(EI)_{ef}$ is the effective bending stiffness of composite section.

The bending moment capacities for concrete (ϕM_u) and timber (ϕM) can be written as, respectively:

$$\phi M_u = \phi f'_c \frac{2(EI)_{ef}}{\gamma_c E_c h_c} \quad (3-21)$$

$$\phi M = \phi k_1 k_4 k_6 k_9 k_{12} f'_b \frac{2(EI)_{ef}}{\gamma_t E_t h_t} \quad (3-22)$$

where f'_c is characteristic strength of concrete in compression while k factors and ϕ are modification and capacity factors as defined in AS 1720.

Each capacity determined for concrete and timber (ϕM and ϕM_u) must be greater than the design moment, M^* . The design moment, M^* is derived from loading requirements and boundary conditions of the TCC structure.

$$\phi M_u > M^* \text{ and } \phi M > M^* \quad (3-23)$$

The axial (in-plane) stress of concrete or timber is predicted by:

$$\sigma_{c/t,i} = \pm \frac{\gamma_i E_i a_i M^*}{(EI)_{ef}} \quad (3-24)$$

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

$$\sigma_{c,c} = - \frac{\gamma_c E_c a_c M^*}{(EI)_{ef}} \quad (3-25)$$

$$\sigma_{t,t} = \frac{\gamma_t E_t a_t M^*}{(EI)_{ef}} \quad (3-26)$$

Assessment of the axial stress is derived from the flexural action. However, the (corresponding) design axial force can be determined from:

$$N_c^* = \sigma_{c,c} A_c \quad (3-27)$$

$$N_t^* = \sigma_{t,t} A_t \quad (3-28)$$

and the allowable axial forces of concrete (ϕN_u) and timber (ϕN) are defined as:

$$\phi N_u = \phi f'_c A_c \quad (3-29)$$

$$(\phi N) = \phi k_1 k_4 k_6 k_{11} f'_t A_t \quad (3-30)$$

where f'_c and f'_t are characteristic axial strength of concrete and timber in compression and tension, respectively, while k factors and ϕ are modification and capacity factors as defined in Section 4, AS 1720.1.

If the depth of the timber member exceeds 150 mm, the characteristic tension strength must be reduced or modified in accordance with the Manufacturer's specifications.

3.2.3 Flexural Shear Strength

In the absence of structural reinforcement in the concrete member, the flexural shear strength (ϕV) is provided by the timber member, therefore:

$$(\phi V) \geq V^* \quad (3-31)$$

where for rectangular sections:

$$(\phi V) = \phi k_1 k_4 k_6 k_{11} f'_s \frac{2A_t}{3} \quad (3-32)$$

where f'_s is characteristic shear strength timber parallel to the grain while k factors and ϕ are modification and capacity factors as defined in AS 1720.1.

Some conditions (for example, use of a deep notch at the support of a beam) may require reducing the shear plane area by using the net area of the (beam) cross-section. AS 1720.1 has specific requirements for such conditions.

3.2.4 Bearing Strength

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

$$(\phi N_p) \geq N_p^* \quad (3-33)$$

in which:

$$(\phi N_p) = \phi k_1 k_4 k_6 k_7 f'_p A_p \quad (3-34)$$

3.3 Strength of the Composite Cross-Section – Connection Capacity

The connection (a machined notch containing a screw fastener or cross SFS screws) transfers the shear force occurring between the concrete and timber elements when loaded under flexure. Since the actual mechanics of this force transfer are relatively complex, a prescriptive approach that defines connection capacities (based on empirical test data) is specified in Section 5.1.

3.3.1 Shear Strength of the Connection

Assessment of the connection strength (N_j) includes assessment of the strength of the first connection due to V_{max}^* near to the support, and the connection located at the quarter-span area due to $V_{L/4}^*$.

$$(\phi N_j) \geq Q^* \quad (3-35)$$

where Q^* is lateral shear force and the connection strength (N_j) is calculated by

$$(\phi N_j) = \phi k_1 k_4 k_6 Q_k \quad (3-36)$$

where Q_k is characteristic capacity of a fastener while k_i and ϕ are modification and capacity factors as defined in AS 1720.1.

The effective shear force in the connection located near the support is:

$$Q_{(V_{max}^*)}^* = -\frac{\gamma_c E_c A_c a_c s_{min}}{(EI)_{ef}} V_{max}^* \quad (3-37)$$

where S_{min} is the minimum spacing of shear connections and the effective shear force in the connection located at the 'quarter' span:

$$Q_{(V_{L/4}^*)}^* = -\frac{\gamma_c E_c A_c a_c s_{max}}{(EI)_{ef}} V_{L/4}^* \quad (3-38)$$

3.3.2 Shear Strength of the Timber

The shear strength of the timber (ϕN_v) and tangential shear action in the area located between the support and the first connection (V^*) is checked as follows:

$$(\phi N_v) \geq V^* \quad (3-39)$$

where:

$$(\phi N_v) = \phi k_1 k_4 k_6 f'_s (b_t l_s) \quad (3-40)$$

where f'_s is characteristic shear strength timber parallel to the grain while b_t and l_s are width and length of the horizontal shear plane for the timber member.

3.4 Serviceability – Deflection

The load combinations and factors for the serviceability limit states (SLS) are defined in AS 1170. Serviceability of the TCC structure is assessed by checking the deflections against the limits defined to suit the functional requirements of the building being designed. In the absence of any specific limits, the following are recommended:

- Short-term 0.7Q only, limited to $L / 300$
- Short-term Point load deflection (Q), limited to 2.0mm
- Long-term $G + 0.4Q$, limited to $L / 250$
- Long-term G only, limited to $L / 300$

where Q and G are and imposed and permanent actions while L is the span.

The effective stiffness $(EI)_{ef}$ of the structure is defined earlier. Where deflection is deemed to be critical, a 5th percentile estimate of E , should be used.

The mid-span deflection under uniformly distributed load (Δ) is assessed using:

$$\Delta = \frac{5(G^* + \phi w_{imp}^*) L^4}{384(EI)_{ef}} \quad (3-41)$$

where w_{imp}^* and G^* are imposed design load and design self-weight while ϕ is creep coefficient of timber.

The mid-span deflection under a point load is assessed by:

$$\Delta = \frac{P^* L^3}{48(EI)_{ef}} \quad (3-42)$$

where p^* is design action for point load action.

The value of ϕ and $(EI)_{ef}$ are defined to suit the loading condition and duration. The creep component for long-term deflections is modelled using the j_2 factor, which is stiffness modification factor of connection for load duration. This is consistent with AS 1720.1, which uses a simplified multiplier to the initial short-term deflection.

3.4.1 Instantaneous Short-Term Deflection

- a) Imposed load only – deflection check under uniformly distributed load using Equation (3-41).
- b) 1.0 kN load (vibration check) – deflection check under point load using Equation (3-42).

The unit point load deflection criterion should be applied to the light-frame floors design when the vibration due to walking is an issue⁵. The unit point load deflection criterion is shown in Figure 3.10. For floors with spans below 3 m, the limit for unit load deflection is ≤ 2 mm and the deflection limit for span ≥ 3 m decreases exponentially as shown Figure 3.10. The deflection of about 0.6 mm under the application of 1 kN point load is acceptable to all floor spans.

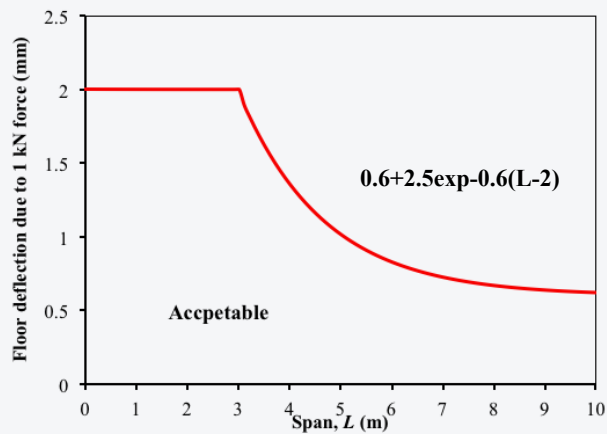


Figure 3.10: Design criterion for light-frame floors.

The shrinkage and creep effect of the concrete member and the creep of the timber are neglected. Thus, $\phi = 1.0$ and $(E)_{ef}$ is approximated as defined in Equations (3-1).

3.4.2 Long-Term End-of-Life Deflection

- Permanent (G) and imposed load (Q) – deflection check under uniformly distributed load using Equation (3-41)
- Permanent load only – deflection check under uniformly distributed load using Equation (3-41)

The shrinkage and creep of the concrete member and the creep of the timber are accounted for. There are two approaches for predicting the long-term deflection due to creep effects.

Method 1 (simplified method) is based upon empirical data obtained from long-term deflection measurements of TCC floor beams collected by researchers at the University of Technology Sydney (UTS). This estimates the gross deflection behaviour and involves multiplying the short-term EI_{ef} by a j_2 factor from Table 3.2.

Instantaneous Live load	1.0
Long-term loads in a controlled environment	3.0
Long-term loads in a variable environment	4.0

Table 3.2: Simplified Method – Recommended values of the creep factor j_2 .

Method 2 (rigorous method) involves consideration of both the concrete shrinkage and timber creep separately (using the j_2 factors in Table 3.3), as noted in Equations (3-43) to (3-46), below.

Load	j_2
Instantaneous Live load	1.0
Long-term loads in a controlled environment	2.0
Long-term loads in a variable environment	3.0

Table 3.3: Rigorous Method – Recommended values of the creep factor j_2 .

The effective (apparent) stiffness $(EI)_{ef}$ of the composite cross-section is given by:

$$(EI)_{ef} = E_{c,lts} I_c + E_{t,lts} I_t + \gamma_{c,lts} E_{c,lts} A_c a_c^2 + \gamma_{t,lts} E_{t,lts} A_t a_t^2 \quad (3-1)$$

where the term lts refers to modified value for long-term service. and I_t refer to Equations (3-2) and (3-3) and the gamma functions modified for long-term service are given by Equations (3-6) and (3-7):

$$\gamma_{c,lts} = \frac{1}{1 + \frac{\pi^2 E_{t,lts} A_c s_{ef}}{K_{eff} L^2}} \quad (3-6)$$

$$\gamma_{t,lts} = 1 \quad (3-7)$$

a_c and a_t are obtained from Equations (3-11) and (3-12) as:

$$a_c = \frac{\gamma_{t,lts} E_{t,lts} A_t H}{\gamma_{c,lts} E_{c,lts} A_c + \gamma_{t,lts} E_{t,lts} A_t} \quad (3-11)$$

$$a_t = \frac{\gamma_{c,lts} E_{c,lts} A_c H}{\gamma_{c,lts} E_{c,lts} A_c + \gamma_{t,lts} E_{t,lts} A_t} \quad (3-12)$$

where:

$$E_{c,lts} = \frac{E_c}{(1 + \varepsilon_{cs})(1 + \phi_{cc})} \quad (3-43)$$

$$E_{t,lts} = \frac{E_t}{j_2} \quad (3-44)$$

And:

$$\varepsilon_{cs} = k_1 \varepsilon_{cs,b} \quad (3-45)$$

$$\phi_{cc} = k_2 k_3 \phi_{cc,b} \quad (3-46)$$

where

ϕ_{cc} is design creep factor (concrete)

$\phi_{cc,b}$ is basic creep factor (concrete)

ε_{cs} is design shrinkage strain (concrete)

$\varepsilon_{cs,b}$ is basic shrinkage strain (concrete)

for H refer to Equation (3-13) and for b_c refer to Equations (3-4) and (3-5).

3.5 Serviceability – Dynamic Behaviour

In addition to the 1 kN point load vibration check above, a more rigorous dynamic assessment can be carried out based on the fundamental frequency of the TCC floor – noting that this formula predicts the behaviour of single span beams with different types of boundary conditions and continuous beams having a maximum of three spans. The formula will generally be conservative as a prediction of the floor system behaviour. Prediction of the first fundamental frequency of simply supported TCC floor beam is based on an empirically derived methodology, which is summarised in the formula below:

$$\text{Nat Freq (Hz)} = C_B \times \left(\frac{EI_{ef}}{L^4 \times m} \right)^{0.5} \quad (3-47)$$

where m is the mass per unit length (t/m if EI is in kNm², or kg/m if EI is in Nmm²), L is the clear span in metres (for continuous beams, the longest span should be used) and C_B is the frequency coefficient, which depends upon the number of spans and boundary conditions.

Table 3.4 lists the values of C_B for a single span. Frequency coefficient (C_B) is the same for both pin–pin and pin–roller boundary conditions whereas higher frequency is expected for pin–pin compared to pin–roller boundary condition.

No. of Spans	End condition	Values of C_B
Single	Pinned/pinned (simply supported)	1.57
	Single Fixed/pinned	2.45
	Fixed both ends	3.56
	Fixed/free (cantilever)	0.56

Table 3.4: Values of C_B for a single span. Source: Wyatt, T⁶.

It is essential that designers define the serviceability limits for deflection and dynamic performance to meet the intended functional requirements of the floor under design. Currently accepted design methods for timber floors, such as AS 1684 Residential timber-framed construction Part 1 Design Criteria, are generally based upon the assumption that acceptable performance of the floor is considered to occur when the fundamental frequency exceeds 8 Hz. However, this is a simplification and recent studies such as Hamm⁷ indicate that lower frequencies in the 3.5 to 5.5 Hz range may also be acceptable. Several modules coupled together would most likely have greater natural frequencies when several units are coupled together in a two-way system.

A more comprehensive assessment of the dynamic performance of the floor where the dynamic performance is deemed to be critical can be undertaken based on quantifying a 'Response Factor'⁸. This method is based on concrete and steel-concrete composite floor design, but is considered to be equally applicable to TCC floors. However, the method will normally require the use of finite element modelling to establish the dynamic parameters of the floor such as natural frequencies, mode shapes and damping.

Acoustic Performance

From a review of existing knowledge on acoustic performance of timber floors, it is clear that both airborne and impact sound insulation requirements can be fulfilled by applying suitable treatments and proper detailing to the timber floors. It is important to understand the difference in the factors affecting the airborne and impact sound insulation to address the acoustic performance of a floor.

A number of best practice guidelines based on existing knowledge of acoustic performance of timber floors are summarised below:

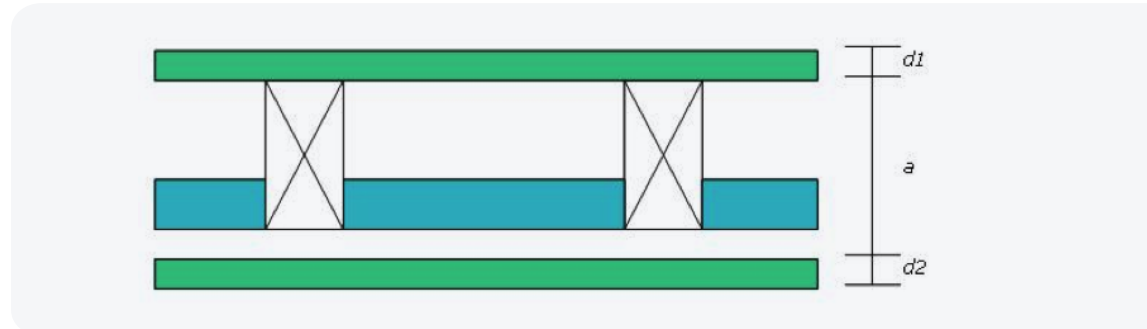


Figure 4.1: Double layer floor with good acoustic properties.

4.1 Guidance on Improving the Airborne Sound Insulation

1. Larger spacing and separated layers of double layer floor: The ceiling boards should not be directly connected to the floor joists. Ceiling should be separated from the floor joists (distance 'a' in Figure 4.1) by either providing resilient support for the ceiling board or separate joists for ceiling boards supported on the walls.
2. The ceiling boards should have a minimum density of 10 kg/m². Although single layer ceiling boards provide adequate airborne sound insulation, it is preferable to use two ceiling boards with staggered joints for better sound insulation performance.
3. The floor cavity between the subfloor and ceiling should be filled with sound absorbing material (mineral fibre). The material's type and density is dependent on the floor's construction. The BCA may require the mineral wool to be non-combustible.
4. Increasing the mass of the joist may not improve the airborne sound insulation of timber floors. The thickness of sound absorbing material, arrangement of resilient channels and depth and spacing of joist has some effect in the airborne sound insulation behaviour but it is not as significant as the effect of having ceiling boards separated from the joists.
5. A combination of sub-floor with a mass of 20 kg/m² and 150 mm thick sound absorbing material with ceiling boards supported on resilient metal channels have been reported to give good airborne sound insulation for timber floors.
6. Thin, heavyweight, and non-rigid layers, or asymmetric construction ($d_1/d_2 =$ approximately 2) options, are suitable to get satisfactory acoustic properties⁹.

4.2 Guidance on Improving the Impact Sound Insulation

1. Increasing the mass or by separating the ceiling from the floor joists can impact sound insulation of timber floors.
2. Good impact sound insulation can be achieved for floors constructed with a sub-floor layer (e.g. particle board, gypsum board), separated ceiling using resilient channels and sound absorbing material in the floor cavity. The requirements for the density of floor boards and insulation material are same as that for airborne sound insulation.

3. A floor with a mass of at least 200 kg/m^3 has been reported to have adequate impact sound insulation. However, mass alone may not be sufficient and attention also needs to be given to the floor finish and ceiling treatment
4. Providing soft floor topping can reduce high frequency impact sound transmission. Hard floor toppings such as concrete, marble, tile and hardwood lead to problems with high frequency impact noise. If a hard floor topping is unavoidable, a floating floor on a resilient layer should be used.
5. A top floor layer should be installed with a resilient under layer and should not be screwed directly to the timber joist (Figure 4.2).
6. Addition of transverse stiffeners can improve the high frequency impact insulation of the floor, but reducing the joist spacing may not always improve this.

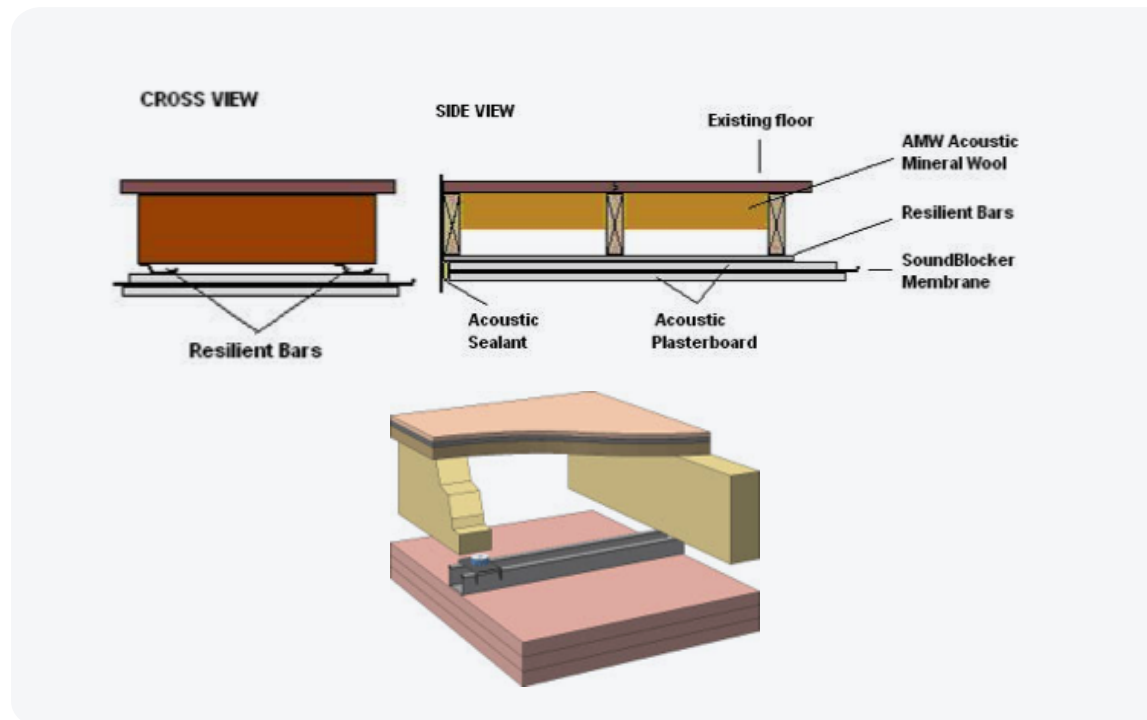


Figure 4.2: Acoustic improvement methods.

Manufacturing Provisions

5.1 Coach Screw and Notch Connection

The characteristic properties for the notched connections specified in this Design Guide (Section 3.1) apply to notched connections with screw fasteners manufactured in accordance with the specified geometries and dimensions in Table 5.1. The connections tested to characterise their properties used laminated veneer lumber (LVL) as the timber joist; however, any timber product with joint group classification of at least JD4 as per AS 1720.1 can be used. Where two pieces of timber are vertically laminated to make a thicker beam, each piece must include a coach screw at each notch (See Figure 4.1). A wide beam made from two pieces of timber must have two coach screws as specified in Table 5.1.

- (a) l_p refers to the depth of penetration of the threaded portion of the coach screw into the timber joist.
- (b) The pre-drill holes for the coach screws should be a diameter of 1 mm less than the root diameter of the coach screw and not exceed the root diameter of the screw.
- (c) The pre-drill holes should extend to the penetration depth l_p only.

5.2 Coach Screws

Coach screws suitable for TCC floors would come in lengths of around 100, 120, 130, or 150 mm. The length of coach screw that is clear of the timber is to be about 55 mm. Coach screws are hot dipped and galvanised, and their protection type and application as a corrosive resistant fastener are specified in WoodSolutions Design Guide #5: *Timber service life design – design guide for durability* (Section 8).

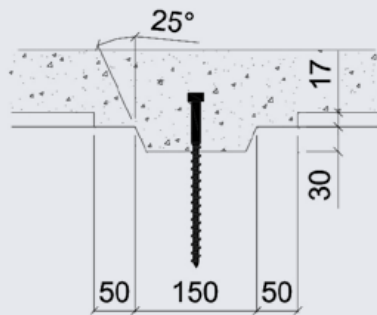
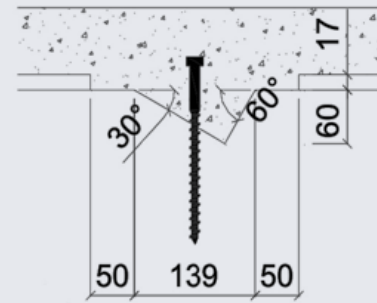
Connection types with geometry and dimensions (mm)	For beam thickness 50 mm or less	For beam thickness more than 50 mm
	Coach screw Ø: 12 mm (shank diameter) and l_p : 80 mm or at least the length of the thread.	Coach screw Ø: 16 mm and l_p : 100 mm or at least the length of the thread.
		

Table 5.1: Manufacturing provisions for notches.

5.3 Cross SFS Screws

The characteristic properties for the cross SFS screws inclined at 30° to 45° specified in this Design Guide (presented in Section 3.1) apply to SFS screws connections manufactured in accordance with the specified geometries and dimensions in Figure 5.1.

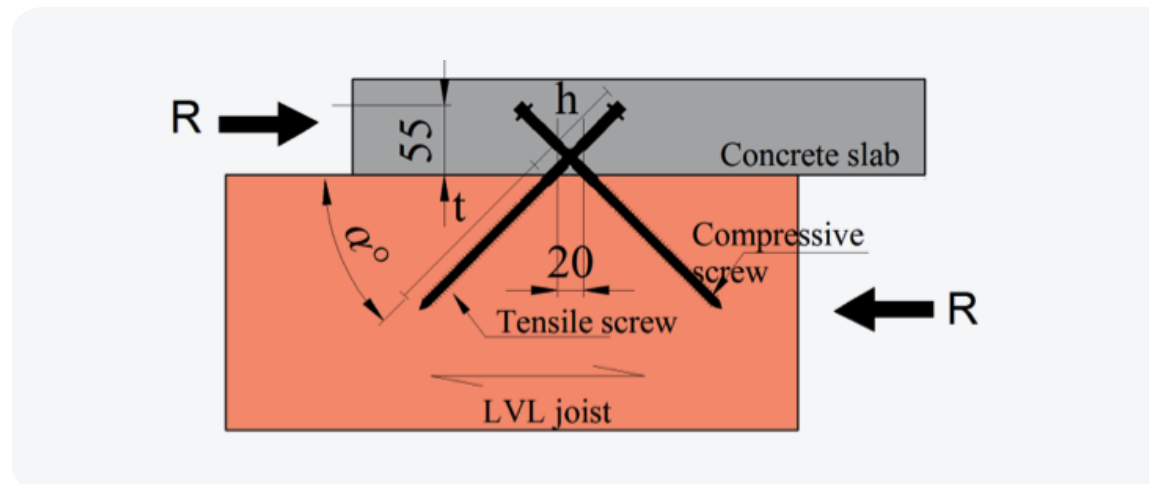


Figure 5.1: Manufacturing provisions for cross SFS screws inclined at 45°.

Source: Moshiri, F².

Screws were inserted so that the thread was in the timber. This meant a vertical length of approximately 55 mm was embedded in the concrete. (In the case of presence of the interlayer between timber and concrete members, screws have less length within the timber as screws also had to penetrate through the interlayer.)

The threaded part of a pair of screws should be installed laterally reversed at an angle of $\pm 30^\circ$ and $\pm 45^\circ$ with embedding length of 120 and 142 mm as shown in Figure 5.2. The inclination angles were measured from the horizontal.

The connections tested to characterise their properties used laminated veneer lumber (LVL) as the timber joist; however, any timber product with joint group classification of at least JD4 (as per AS1720.1) can be used. Where two pieces of timber are vertically laminated to make a thicker beam, each piece must include a pair of screws. A wide beam made from two pieces of timber must have two pairs of screws as specified in Figure 5.1.



Figure 5.2: SFS screws installed at an angle of $\pm 30^\circ$ (left) and $\pm 45^\circ$ (right).

Source: Moshiri, F².

SFS screws are made of mild carbon steel and their protection type and application as a corrosive resistant fastener are specified in WoodSolutions Design Guide #5: *Timber service life design – design guide for durability* (Section 8).

6

Provisions for Holes in Timber Joists

Installation of building services may require introduction of holes or penetrations through the timber joists. The following limits on details of the holes are prescriptive and are based on AS 1684. In situations where a larger penetration is required, advice must be sought from the manufacturer of the timber beam products being specified.

6.1 Size

- a) For span/depth ratio greater than 10, the maximum hole diameter shall be limited to 50 mm.
- b) For span/depth ratio less than or equal to 10, the maximum hole diameter shall be limited to 25 mm.
- c) Additionally, when depth (height) of the timber joist (h_t) is less than 200 mm, in addition to the limits prescribed in a) and b), the diameter of holes must not exceed $h_t/4$.

6.2 Spacing

The clear spacing between adjacent holes must be not more than 3 holes per 1.8 m.

Edge Distance

- a) The clear distance of a hole from the joist edge should be at least 50 mm when the depth of the timber joist (h_t) is greater than or equal to 200 mm.
- b) For timber joist where $h_t < 200$ mm, the clear distance to the hole from the edge should be at least $h_t/3$.

The depth of the timber joist only should be used to calculate the span/depth ratio.

Concluding Notes

The design procedure presented in this report is adapted from the design procedure of Eurocode 5 and is modified to suit local practices and reflect research and development recently undertaken in Australia and New Zealand.

The design methodology adequately addresses 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 dynamic response and the long-term performance of the structure.

Adapting the design procedure to suit Australian practices has been a challenging exercise and, where assumptions have had to be made due to uncertainties, these have erred towards being conservative.

A

Appendix A

A1 Commentary & Background Information

A1.1 Introductory Comments

The design procedure presented in this report is based on an extensive review of the published research combined with numerical investigation and laboratory testing.

This Appendix presents further information to improve understanding of some of the considerations and assumptions made in the design procedure.

The informative material discussed in the Appendix addresses:

- Loading conditions
- Connection behaviour
- Tributary width of the concrete
- Behavioural assessment of a notched connection.

A1.2 Loading Conditions

To comply with the loading provisions of the AS 1170 series, any TCC structure must be designed to resist a series of combined uniformly distributed loadings. Figure A1.1 depicts the free-body diagram for a beam under such loading action.

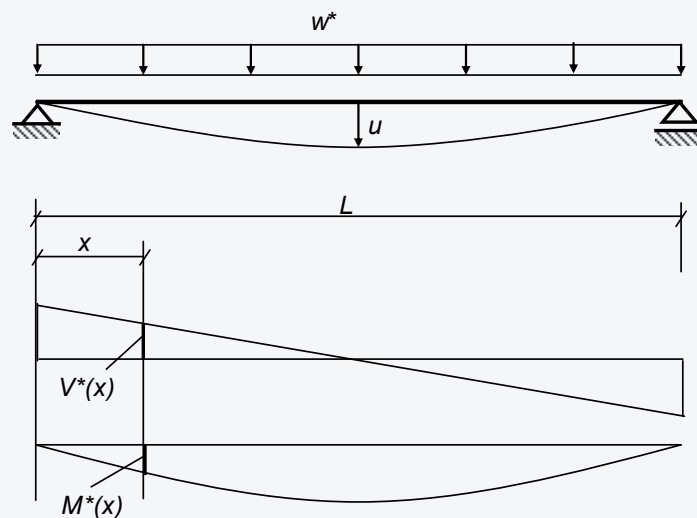


Figure A1.1: Free-body diagram of the TCC beam. Source: Yeoh, D, et al.¹⁰

Further design requirements may include checking the structural behaviours of TCC structures under pad or point loading.

A1.3 Connection Behaviour

The behaviour of the connection is important for the design of composite sections such as TCC structures and both the strength and flexibility of the connection must be considered. In Figure A1.2, possible states or extents of composite action are depicted:

- a) full - no slip between the member
- b) partial - some slip impediment
- c) zero composite action.

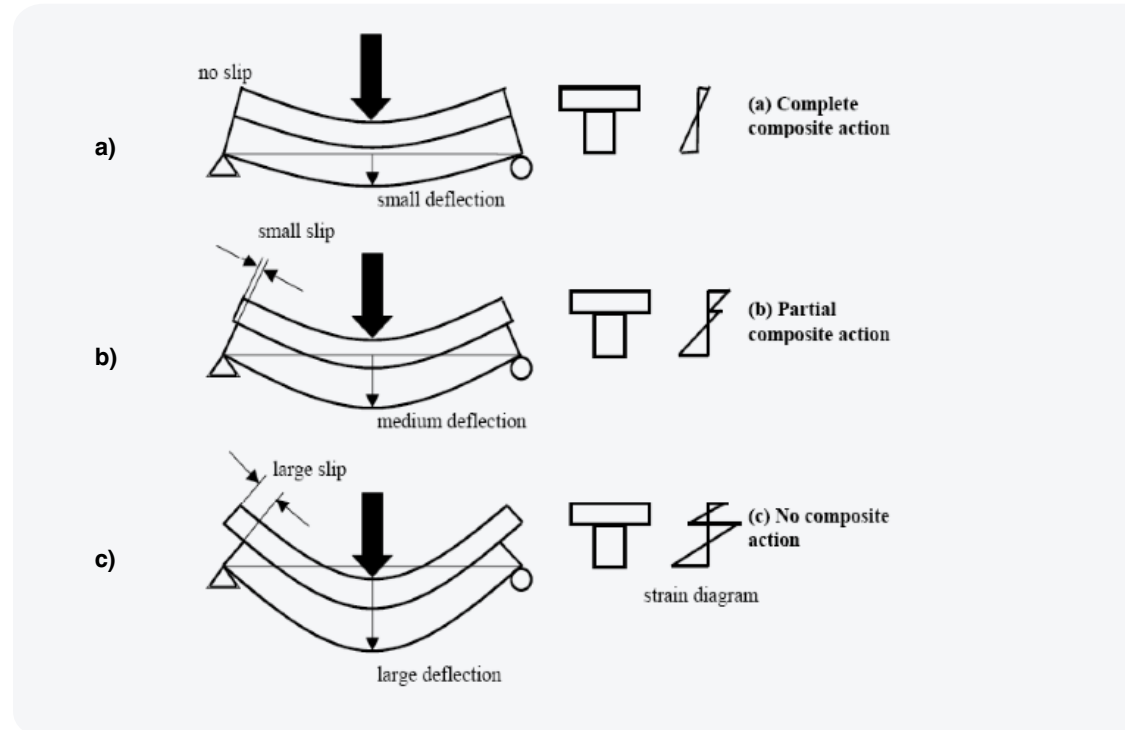


Figure A1.2: State of the composite action. Source: Yeoh, D.¹¹

TCC structures constructed with notched and coach screw connections exhibit partial composite action, since some measure of slip occurs between the layers of the cross-section^{12,13}. By optimisation of the section proportions (timber–concrete depth ratio), the concrete member can be sized to remain completely under compression stresses.

In an attempt to account for partial composite action, Möhler¹⁴ devised a series of formulae for linear-elastic inter-layers. Unfortunately, such assumptions do not accurately model the non-linear behaviour of TCC connections. A more accurate method was proposed by Cecotti¹⁵, which characterises the connection behaviour with two values of the stiffness modulus – respectively at 40% and 60% of the ultimate load capacity of the connection (refer to Figure A1.3) – which approximates both the performance at the serviceability and ultimate limit states.

To date, the structural properties of the connection types specified in this document rely on databases established by comprehensive laboratory investigations¹⁶. To increase the level of reliability of the design of TCC structures, more testing will be required on specific selected types of connection. This future work will contribute to increasing the confidence in the design and will maximise use of available connection structural properties.

The connection transfers the shear force between the members under flexure. This transfer is relatively complex and has been characterised to some extent by experimental investigations undertaken in 2009 at the University of Technology Sydney (UTS) by Agus, Gerber and Crews 2009¹⁷. The stiffness parameters for design are:

- K_{ser} for short-term serviceability
- K_{ef} for long-term serviceability
- K_u for the ultimate state.

Note: Refer to Section 3 for characteristic properties of connections.

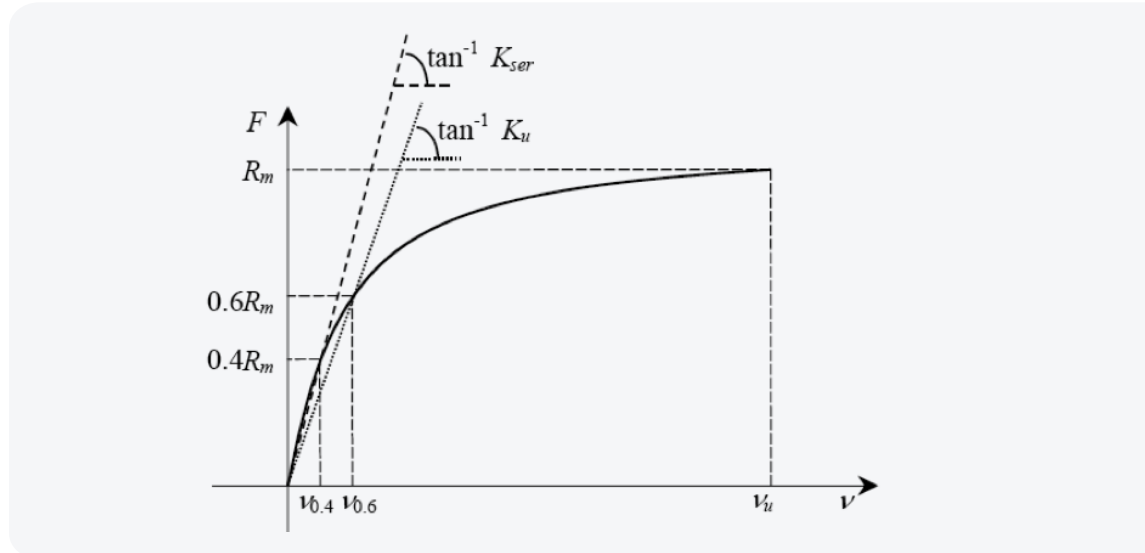


Figure A1.3: Slip moduli of the connections for limit states. Source: Yeoh, D., et al.¹¹

A1.4 Tributary Width of the Concrete Member

A T-beam (Figure A1.4) structural form is commonly used for TCC floors. The tributary width models the effects of shear lag. An accurate estimation for this is essential to fully use the properties of the concrete member and to achieve a safe design.

In Australia, two evaluations are proposed, one in AS 3600 and the other in AS 2327.1 2003 *Composite structures – Part 1: Simply supported beams*. For constructions that do not comply with the shape and dimension requisites of AS 3600 and AS 2327.1, the tributary width must be assessed to suit the construction technique used and include aspects such as the geometry, dimensions, proportion. In particular, the shear lag effect and buckling stability must be investigated.

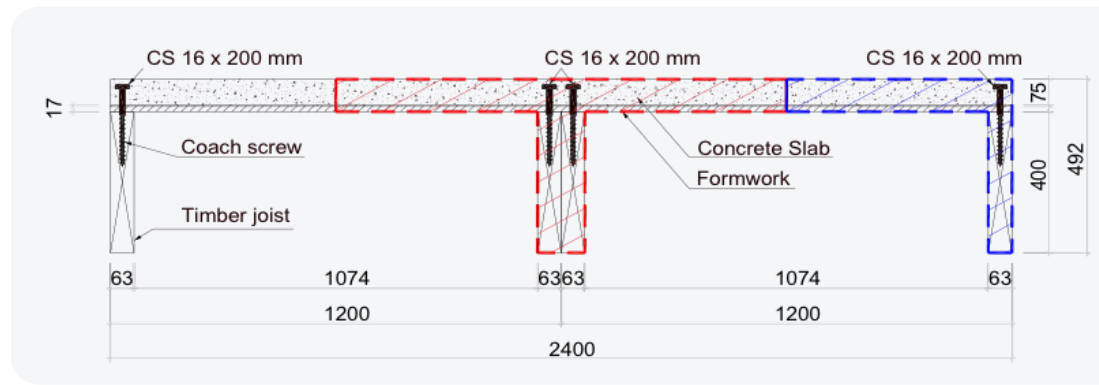


Figure A1.4: Division and tributary width of TCC element.

In AS 3600, the tributary width of the concrete member shall comply with (3-4) for T-beams and (3-5) for L-beams. Also, the tributary width must not exceed the beam spacing.

An alternative to AS 3600 for the assessment of the tributary width of the concrete member can be found in AS 2327.1.

- the tributary width:

$$b_c = b_{e1} + b_{e2} \quad (\text{A1-1})$$

in which, for an edge beam, b_{e1} and b_{e2} are:

$$b_{e1} = \min \left\{ \left(\frac{L_{ef}}{8} \right) \text{ or } \frac{b_1}{2} \text{ or } \left(\frac{b_{sf1}}{2} + 8D_c \right) \right\} \quad (\text{A1-2})$$

$$b_{e2} = \min \left\{ \left(\frac{L_{ef}}{8} \right) \text{ or } b_2 \text{ or } \left(\frac{b_{sf1}}{2} + 6D_c \right) \right\} \quad (\text{A1-3})$$

and for an internal beam, b_{e1} and b_{e2} are:

$$b_{e1} = \min \left\{ \left(\frac{L_{ef}}{8} \right) \text{ or } \frac{b_1}{2} \text{ or } \left(\frac{b_{sf1}}{2} + 8D_c \right) \right\} \quad (\text{A1-4})$$

$$b_{e2} = \min \left\{ \left(\frac{L_{ef}}{8} \right) \text{ or } b_2 \text{ or } \left(\frac{b_{sf1}}{2} + 8D_c \right) \right\} \quad (\text{A1-5})$$

where

b_{e1} and b_{e2} are measured effective on each side of the centre-line of the timber beam.

L_{ef} is the effective span of the beam calculated in accordance with Clause 5.3.3, AS 2327

b_{sf1} is effective width of composite beam top flange (0 for timber beam)

D_c is the overall depth of the concrete slab

b_1 , b_2 are centre-to-centre spacing of adjacent beams or distance from centre of timber beam to edge of slab outstand.

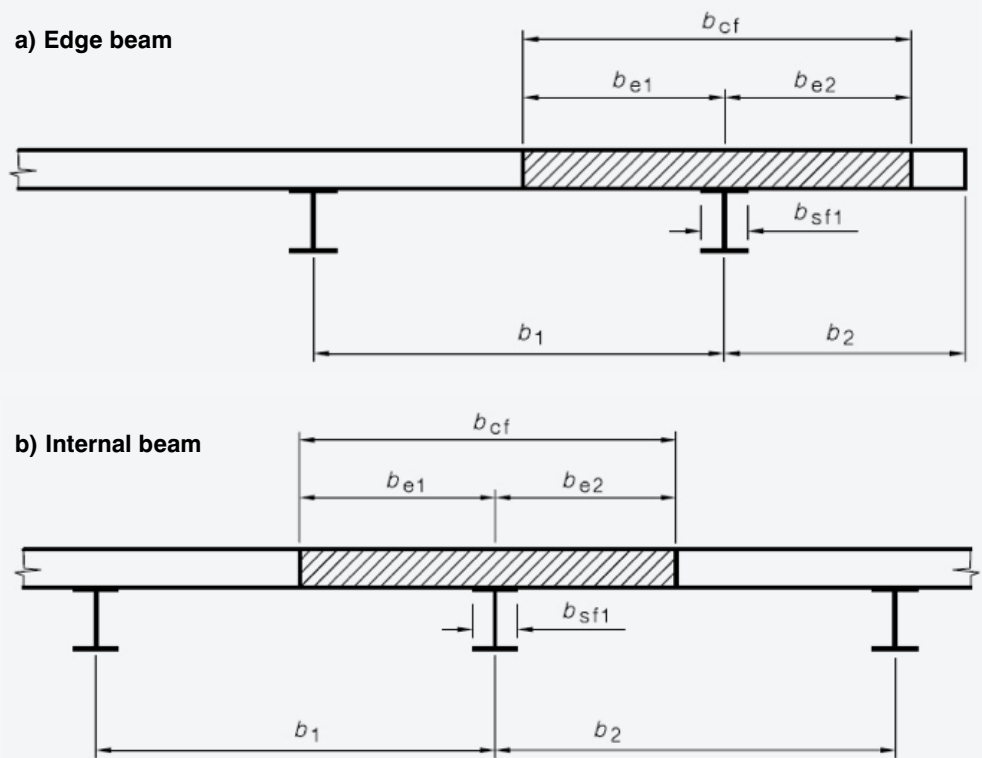


Figure A1.5: Tributary width of the concrete.

Source: AS 2327.1, *Composite structures, in Part 1: Simply supported beams*. 2003, Standards Australia: Australia.

A1.5 Behavioural Assessment of a Notched Connection

The behavioural response of a connection is very complex and in-depth analysis may provide a better understanding about the actual force transfer and flow in the connection. However, at the present time, a prescriptive approach has been adopted for design inputs of the connection types specified in Table 5.1 of this Guide.

The information in this section of the appendix is background to research that will improve understanding of the connections behaviour. Meanwhile, the experimental results presented in the body of the report are sufficient to carry out safe designs.

A1.5.1 Strength Requirements for the Connections

Analysis of connection behaviour must address critical areas such as the shear strength at the base of the concrete bulge, the shear strength of wood preceding the first notch and the crushing strength of the facet area (Figure A1.6).

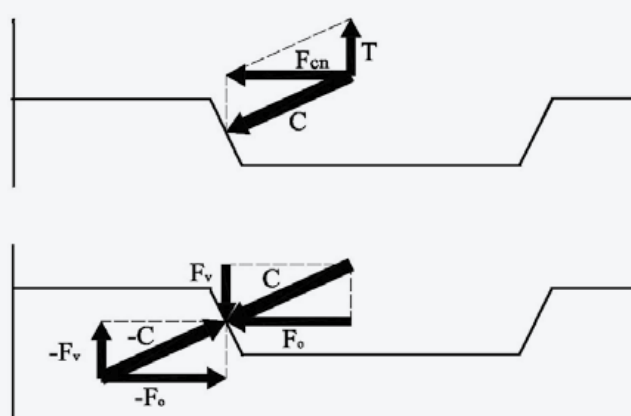


Figure A1.6: Local forces acting in a rectangular notch.

The investigation conducted at UTS¹⁷ identified that the shear strength (in-plane shear) of the concrete bulge and the wood portion behind the (first) shear key are critical for ensuring a safe design. The study has also identified that only the connection resisting the largest load action needs to be checked.

A1.5.2 Shear Strength of the Concrete Bulge (AS 3600)

The shear strength at the baseline of the concrete bulge (Figure A1.6) must be assessed:

$$\phi V_{uc} \geq V^* \quad (\text{A1-6})$$

where

$$\phi V_{uc} = \phi \beta_1 \beta_2 \beta_3 b_v d_o \left(\frac{A_{st} f'_c}{b_v d_o} \right)^{\frac{1}{3}} \quad (\text{A1-7})$$

where

$$\beta_1 = 1.1 (1.6 - d_o / 1000) \geq 1.1 \quad (\text{A1-8})$$

$$\beta_2 = 1.0 \quad (\text{A1-9})$$

$$\beta_3 = 1.0 \quad (\text{A1-10})$$

A_{st} = cross-sectional area of the coach screw.

b_v is width of the notch concrete

d_o is length of the notch concrete

A1.5.3 Shear Strength of the Timber

This check has been included in the design guidelines.

A1.5.4 Bearing Strength of the Timber

The bearing strength of the notch facet – compression contact area in the connection and location of the maximum shear force (refer to Figure A1.1) – must also be verified:

$$(\phi N_\theta) \cos(90 - \theta) \geq V^* \quad (\text{A1-11})$$

where:

$$(\phi N_\theta) \cos(90 - \theta) = \frac{(\phi N_l)(\phi N_p)}{(\phi N_l) \sin^2 \theta + (\phi N_p) \cos^2 \theta} \cos(90 - \theta) \quad (\text{A1-12})$$

and the parallel and perpendicular components are:

$$(\phi N_l) = \phi k_1 k_4 k_6 f'_l A_l \quad (\text{A1-13})$$

where

ϕN_l is design capacity in bearing parallel to the grain (timber)

ϕN_p is design capacity in bearing perpendicular to the grain (timber)

ϕN_θ is design capacity in bearing at an angle to the grain (timber)

θ is angle of the notch facet under compression,

f'_l is characteristic strength in bearing parallel to the grain

A_l is bearing area for loading parallel to the grain (timber)

for (ϕN_p) refer to (Equation 3.34).

A2.1 Connection Behaviour and Classification

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 accounted for in the identification of the Gamma coefficients in the design procedure. An extensive (literature) review of shear connectors used in timber concrete composite structures, covering the period from 1985 to 2004, has been undertaken by Dias¹⁸. Elsewhere, Ceccotti⁴ also presents an overview of the timber-concrete connectors (Figure A2.7) that are most commonly used to achieve composite action between the concrete and the timber members.

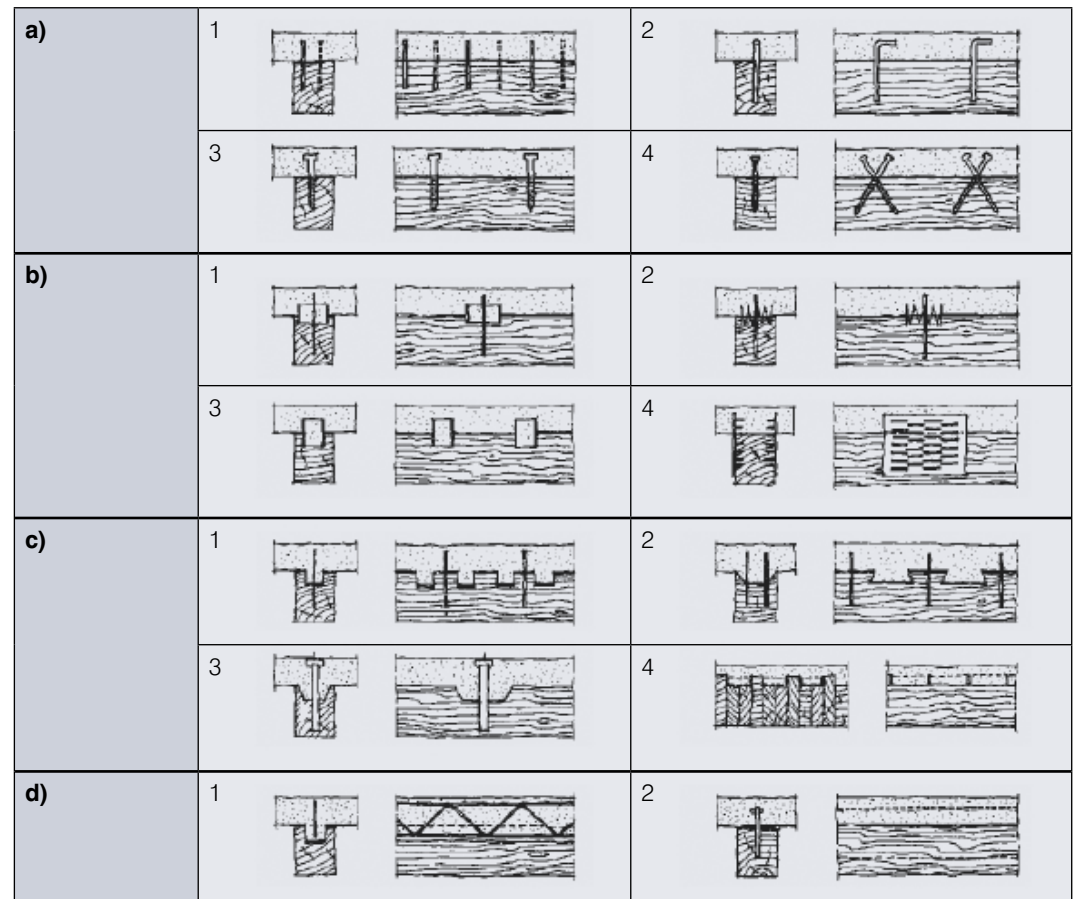


Figure A2.7: Range of TCC connections.

Different connections shown in Figure A2.7 are as follows:

(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 prestressed steel bars; (c4) nailed timber plank deck and steel shear plates slotted through the deeper timber planks; (d1) steel lattice glued to timber; (d2) steel plate glued to timber.

The stiffness characteristics of some of the shear connectors presented in Figure A2.7 are plotted in Figure A2.8. The load-slip plot indicates that for this group of connector types, the stiffest connections are those in group (d), while the least stiff are in group (a). 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 TCC systems constructed with connectors from group (d)¹⁵.

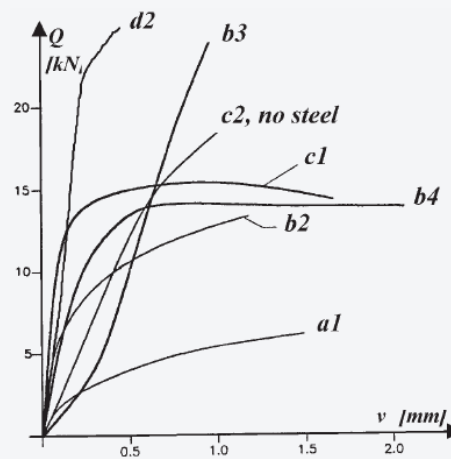


Figure A2.8: Schematic of load-slip behaviour of types of connection.

Source: Ceccotti, A., ed.¹⁵

A2.2 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 defining 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 $K_u = (2/3) K_{ser}$.

A2.3 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, while 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.

- 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.
- The effect of the ratio of coach screw diameter to timber joist thickness is one of the parameters that need to be further investigated. Table A2.1 highlights the effect of the ratio of coach screw diameter to LVL thickness and suggests that there is no advantage in using 16 mm diameter screws in 48 mm thick LVL beams.

While the variability of maximum load (strength) is considered to be acceptable, the variability of the characteristic stiffness properties highlights some of the uncertainty 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 stiffness variability to lower levels that could lead to more efficient design of these types of floor structures.

A2.4 Empirical Characterisation of Notched Connections

The main results for both connection types are presented in Table A2.1.

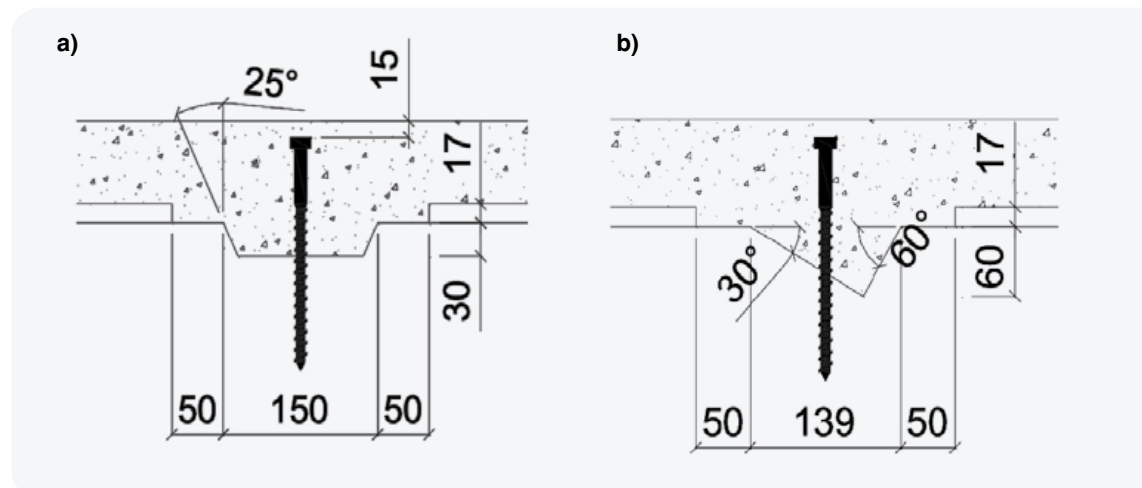


Figure A2.9: Notched connections – trapezoidal a) and triangular b) (45 and 63 mm thick LVL with 12 and 16 mm coach screw, respectively).

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

Table A2.1: Characteristic properties of notched connections – trapezoidal (T-Series) and triangular (B-Series) notch shapes.

Integer = capacity; % = CV

Strength – 5th percentile based on a log normal distribution

Stiffness – 50th percentile

A2.5 Equations for Characteristic Properties of Connections

These Equations describe the sloping sections of the curves in Figures 3.7 to 3.9.

Trapezoidal Notch and Coach Screw:

$$Q_k = 0.95 \times (\text{thickness}) - 2 \quad (\text{A2-1})$$

$$K_{serv} = 0.3 \times (\text{thickness}) + 80 \quad (\text{A2-2})$$

$$K_u = 0.45 \times (\text{thickness}) + 45 \quad (\text{A2-3})$$

Triangular Notch and Coach Screw:

$$Q_k = 0.95 \times (\text{thickness}) - 2 \quad (\text{A2-4})$$

$$K_{serv} = 1.05 \times (\text{thickness}) + 45 \quad (\text{A2-5})$$

$$K_u = 1.25 \times (\text{thickness}) - 15 \quad (\text{A2-6})$$

where thickness is in mm, and must be between 30 mm and 126 mm.

The maximum values for Q_k , K_{serv} and K_u for trapezoidal notch are 118 kN, 118 kN/mm and 102 kN/mm while the same for the triangular notch are 118 kN, 177 kN/mm and 143 kN/mm, respectively.

It is noted that the test data is not yet available for thicknesses exceeding 126 mm.

A3 Worked Example – 8 m TCC Floor Span by 5 m Bearer

TCC work example prepared by Arup, Sydney Building Structure.

Calculation of timber concrete composite floor capacity in accordance with the Design Guide

The calculations were initially carried out by spreadsheet but have been presented here, written in full, to demonstrate the calculation process.

A3.1 Material Input

Timber type: LVL 11

Timber modulus	$E_t = 11000 \text{ MPa}$
Timber density	$\rho = 620 \text{ kg/m}^3$
Timber bending strength	$f'_b = 48 \text{ MPa}$
Timber tensile strength	$f'_t = 30 \text{ MPa}$
Timber shear strength	$f'_s = 6.0 \text{ MPa}$
Concrete modulus	$E_c = 31000 \text{ MPa}$
Concrete density	$\rho_c = 2500 \text{ kg/m}^3$
Concrete compressive strength	$f'_c = 32 \text{ MPa}$
Concrete thickness	$h_c = 80 \text{ mm}$
Plywood thickness	$a_f = 15 \text{ mm}$

A3.2 Loading Input

Super imposed dead load	$\text{SDL} = 1.0 \text{ kPa}$
Live load	$\text{LL} = 4 \text{ kPa}$
Acceleration due to gravity	$g = 9.81 \text{ m/s}^2$
Concrete selfweight	$C_w = h_c \times \rho_c \times g = 1.96 \text{ kPa}$
Formwork selfweight	$F_w = a_f \times \rho \times g = 0.09 \text{ kPa}$

A3.3 Geometric Input

Joist Span	$L = 8 \text{ m}$
Spacing	$S = 600 \text{ mm}$
Beam Depth	$h_t = 400 \text{ mm}$ (Therefore 3 No. fit into a 1200 mm billet without wastage)
Beam Width	$b_t = 90 \text{ mm}$
Concrete Thickness	$h_c = 80 \text{ mm}$
Beam Selfweight	$B_w = b_t \times h_t \times g \times \rho = 0.219 \text{ kN/m}$
Concrete Effective Width	$b_c = \min(S, b_t + 0.2 \times L) = 600 \text{ mm}$

A3.4 Joist Ultimate Strength Checks

A3.4.1 Required capacity

$$W^* = 1.2 \times (B_w/S + C_w + F_w + \text{SDL}) + 1.5 \times \text{LL} = 10.10 \text{ kPa}$$

$$M^* = w^* b_c L^2/8 = 48.5 \text{ kNm}$$

$$V^* = w \times S \times L/2 = 24.2 \text{ kN}$$

A3.4.2 Section properties

$h_c = 75.9 \text{ mm}$; (Reduced effective concrete thickness due to concrete tension at Ultimate Limit State, ULS)

$$A_c = h_c \times b_c = 45540 \text{ mm}^2$$

$$A_t = h_t \times b_t = 36000 \text{ mm}^2$$

$$I_c = \frac{b_c h_c^3}{12} = 21.86 \times 10^6 \text{ mm}^4$$

$$I_t = \frac{b_t h_t^3}{12} = 480.0 \times 10^6 \text{ mm}^4$$

A3.5 Worked Example (a): 8 m Floor with Trapezoid Notches

The K factor below has been determined from Figure 3.8 of the Design Guide

$$K_f = 100 \text{ kN/mm}$$

Refer to Section 3 of the Design Guide for further details.

A3.5.1 Ultimate Limit State Checks

Assuming that there are 4 shear connectors in each half beam span, as shown in the Figure A3.1.

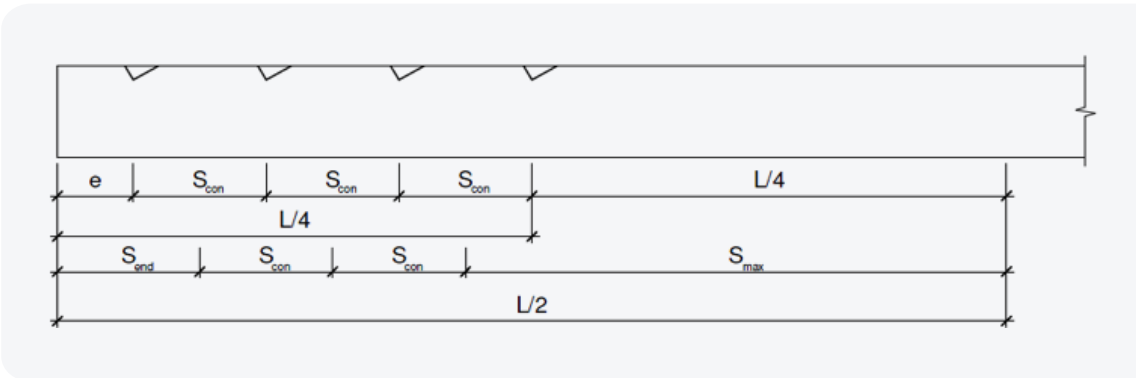


Figure A3.1: Notched connection arrangement.

Number of Connectors; $n = 4$

$$e = 320 \text{ mm}$$

$$s_{con} = (L/4 - e)/(n-1) = 560 \text{ mm}$$

$$s_{end} = s_{con}/2 + e = 600 \text{ mm}$$

$$s_{min} = \min(s_{end}, s_{con}) = 560 \text{ mm}$$

$$s_{max} = L/4 + s_{con}/2 = 2280 \text{ mm}$$

$$s_{ef} = 0.75 \times s_{min} + 0.25 \times s_{max} = 990 \text{ mm}$$

$$\gamma_c = \frac{1}{1 + \frac{\pi^2 E_c A_c s_{ef}}{K_f L^2}} = 0.317$$

$$\gamma_t = 1$$

$$h_c = 80 \text{ mm}$$

$$H = \frac{h_c}{2} + a_f + \frac{h_t}{2} = 255 \text{ mm}$$

$$a_c = \frac{\gamma_c E_t A_t H}{\gamma_c E_c A_c + \gamma_t E_t A_t} = 120 \text{ mm}$$

$$a_t = \frac{\gamma_c E_c A_c H}{\gamma_c E_c A_c + \gamma_t E_t A_t} = 135 \text{ mm}$$

$$(EI)_{ef} = E_c I_c + E_t I_t + \gamma_c E_c A_c a_c^2 + \gamma_t E_t A_t a_t^2 = 19.62 \times 10^{12} \text{ Nmm}^2$$

Bending strength limited by concrete capacity

$$h_c = 75.9 \text{ mm}$$

$$\phi = 0.65$$

$$\phi M_u = \phi f'_c \frac{2(EI)_{ef}}{\gamma_c E_c h_c} = 1010 \text{ kNm}$$

Concrete axial capacity

$$\sigma_{c.c} = \frac{\gamma_c E_c a_c M^*}{(EI)_{ef}} = 2.91 \text{ MPa}$$

$$N_c^* = \sigma_{c.c} \times A_c = 132.40 \text{ kN}$$

$$\phi N_u = \phi \times f'_c \times A_c = 874.37 \text{ kN}$$

Combined compression and bending check

$$\frac{N_c^*}{\phi N_u} + \frac{M^*}{\phi M_u} = 0.2; < 1 \quad ; \text{Therefore section is OK}$$

Bending strength limited by timber capacity

$$\phi = 0.9$$

$$k_1 = 0.8; \text{ (live load i.e. 5 month load duration)}$$

$$k_4 = 1.0; \text{ (equilibrium moisture content less than 15\%)}$$

$$k_6 = 1.0; \text{ (normal temperature range)}$$

$$k_9 = 1.0$$

$$k_{11} = (300 \text{ mm}/h_t)^{0.167} = 0.95$$

$$k_{12} = 1.0$$

$$\phi M_u = \phi k_1 k_4 k_6 k_9 k_{11} k_{12} f'_b \frac{2(EI)_{ef}}{\gamma_t E_t h_t} = 294 \text{ kNm}$$

Timber axial capacity

$$\sigma_{t.t} = \frac{\gamma_t E_t a_t M^*}{(EI)_{ef}} = 3.68 \text{ MPa}$$

$$N_t = \sigma_{t.t} \times A_t = 132.40 \text{ kN}$$

$$k_{11} = (150 \text{ mm}/h_t)^{0.167} = 0.85$$

$$\phi N_u = \phi \times k_1 \times k_4 \times k_6 \times k_{11} \times f'_t \times A_t = 660.11 \text{ kN}$$

Combined bending and tensile check

$$\frac{N_t^*}{\phi N_u} + \frac{M^*}{\phi M_u} = 0.37; < 1 \quad ; \text{Therefore section is OK}$$

Flexural shear strength

At the floor joist to bearer detail, a 125 mm deep notch is assumed. Therefore the notch geometry must be checked in accordance with AS 1720.1, Appendix E9 (not shown here) and the flexural shear strength must be checked for the net area, as shown below.

$$A_t = (h_t - 125 \text{ mm}) \times b_t = 24750 \text{ mm}^2$$

$$\phi V = \phi k_1 k_4 k_6 f'_s \frac{2A_t}{3} = 71.28 \text{ kN}$$

$$\phi V \geq V^* ; \text{Therefore section is OK}$$

Shear connector strength

Assuming 4 shear connectors per half span

From Figure 3.7 of the Design Guide:

$$Q_k = 85 \text{ kN}$$

Connector capacity

$$\phi = 0.9$$

$$k_1 = 0.8$$

$$\phi N_j = \phi k_1 k_4 k_6 Q_k = 54.4 \text{ kN}$$

Under a uniformly distributed load, the shear force can be taken in the centre of span respectively s_{max} , as shown in Figure A3.2.

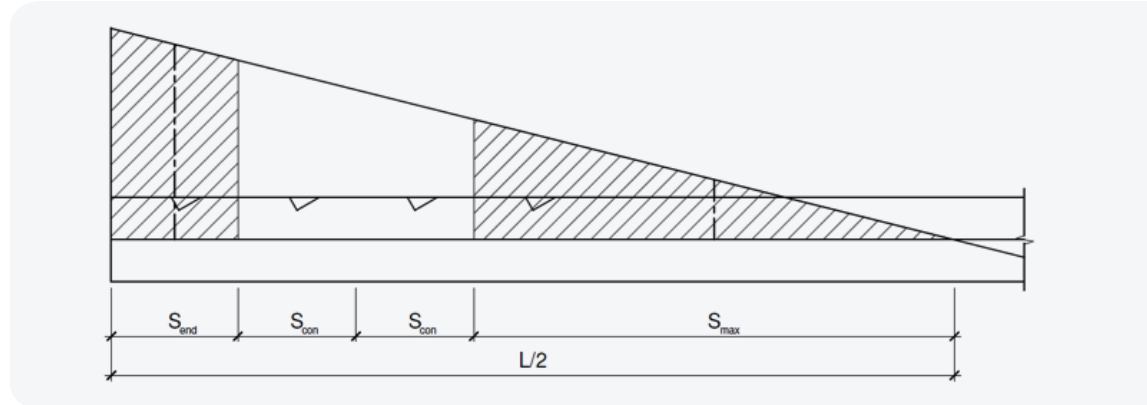


Figure A3.2: Shear force distribution along the beam.

At the support location:

$$V^* = (L/2 - s_{end}/2) \times S \times w^* = 22.4 \text{ kN}$$

$$Q^* = \frac{\gamma_c E_c A_c a_c s_{end}}{(EI)_{ef}} V = 36.7 \text{ kN}$$

At quarter point location:

$$V^* = s_{max}/2 \times S \times w^* = 6.9 \text{ kN}$$

$$Q^* = \frac{\gamma_c E_c A_c a_c s_{max}}{(EI)_{ef}} V = 43.0 \text{ kN}$$

$$\phi N_j \geq Q^* \quad \text{Therefore connectors are OK}$$

Tangential shear strength of the timber

Tangential Shear action in the area located between the support and the first connection:

$$\phi = 0.9$$

Generally:

$$\phi N_v = \phi k_1 k_4 k_6 f'_s (b_t l_s)$$

At the support location:

$$\phi N_v = \phi \times k_1 \times k_4 \times k_6 \times f'_s \times (b_t \times e) = 124.42 \text{ kN}$$

$$\phi N_v \geq Q^* \quad \text{Therefore connectors are OK}$$

A3.5.2 Serviceability Checks

This section outlines the serviceability checks that have been done on the floor joist. Table A3.1 lists the cases considered in the calculations.

Case	Load Applied	Criteria
1	Short-term (0.7Q)	Span /300
2	1 kN point Load	2 mm
3	Long-term (G+0.4Q)	Span/250
4	Long-term Only (G)	Span/300

Table A3.1: Deflection criteria for different load cases.

$$G = (B_w/b_c + C_w + F_w + SDL) \times S = 2.05 \text{ kN/m}$$

$$Q = LL \times S = 2.4 \text{ kN/m}$$

$h_c = 80 \text{ mm}$; (Assume all the concrete is in compression, therefore use the full thickness)

$$A_c = h_c \times b_c = 48000 \text{ mm}^2$$

$$A_t = h_t \times b_t = 36000 \text{ mm}^2$$

Case 1(Short-term)

Calculate the serviceability stiffness with short-term load parameters.

From Figure 3.3 of the Design Guide:

$$K_j = 140 \text{ kN/mm}$$

$$\gamma_c = \frac{1}{1 + \frac{\pi^2 E_c A_c s_{ef}}{K_j L^2}} = 0.38$$

$$\gamma_t = 1$$

$$H = \frac{h_c}{2} + a_f + \frac{h_t}{2} = 255 \text{ mm}$$

$$a_c = \frac{\gamma_t E_t A_t H}{\gamma_c E_c A_c + \gamma_t E_t A_t} = 105 \text{ mm}$$

$$a_t = \frac{\gamma_c E_c A_c H}{\gamma_c E_c A_c + \gamma_t E_t A_t} = 150 \text{ mm}$$

$$I_c = \frac{b_c h_c^3}{12} = 25.60 \times 10^6 \text{ mm}^4$$

$$I_t = \frac{b_t h_t^3}{12} = 480.0 \times 10^6 \text{ mm}^4$$

$$(EI)_{ef} = E_c I_c + E_t I_t + \gamma_c E_c A_c a_c^2 + \gamma_t E_t A_t a_t^2 = 21.24 \times 10^{12} \text{ Nmm}^2 \text{ Calculate the short-term deflection:}$$

$$\Delta = \frac{5(W^*)L^4}{384(EI)_{ef}} = \frac{5(0.7Q)L^4}{384(EI)_{ef}} = 4.2 \text{ mm}$$

$$L/\Delta = 1896 > 300$$

Section Meets Short-term Deflection Limits;

Case 2 (Point load)

$$\Delta = \frac{PL^3}{48(EI)_{ef}}$$

$$P = 1 \text{ kN}$$

$$\Delta = 0.5 \text{ mm} ; < 2 \text{ mm Therefore OK}$$

Case 3 (Short-term)

Calculate the serviceability stiffness with long-term load parameters.

$$\epsilon_{cs} = 880 \times 10^{-6}$$

$$\Phi_{cc} = 3.62 \quad ; \text{ (From Table 3.1.8.3 of AS 3600)}$$

$$E_{c,ls} = \frac{E_c}{(1 + \epsilon_{cs})(1 + \Phi_{cc})} = 6704 \text{ MPa}$$

$j_2 = 2$; (Adopt j_2 of 2 as currently developed by the UTS research work)

$$E_{t,ls} = \frac{E_t}{j_2} = 5500 \text{ MPa}$$

$$K_{i,ls} = \frac{K_i}{j_2} = 70 \text{ kN/mm}$$

$$\gamma_{c,ls} = \frac{1}{1 + \frac{\pi^2 E_{c,ls} A_{c,ef} s_{ef}^2}{K_{i,ls} L^2}} = 0.59$$

$$\gamma_t = 1$$

$$H = \frac{h_c}{2} + a_f + \frac{h_t}{2} = 255 \text{ mm}$$

$$a_c = \frac{\gamma_t E_{t,ls} A_t H}{\gamma_c E_{c,ls} A_c + \gamma_t E_{t,ls} A_t} = 130 \text{ mm}$$

$$a_t = \frac{\gamma_c E_{c,ls} A_c H}{\gamma_c E_{c,ls} A_c + \gamma_t E_{t,ls} A_t} = 125 \text{ mm}$$

$$(EI)_{ef} = E_{c,ls} I_c + E_{t,ls} I_t + \gamma_c E_{c,ls} A_c a_c^2 + \gamma_t E_{t,ls} A_t a_t^2 = 9.101 \times 10^{12} \text{ Nmm}^2$$

Calculate the long-term deflection under $G + 0.4Q$:

$$\Delta = \frac{5(W^*)L^4}{384(EI)_{ef}} = \frac{5(G + 0.4Q)L^4}{384(EI)_{ef}} = 17.6 \text{ mm}$$

$$L/\Delta = 453 > 250 \text{ Therefore OK}$$

Section Meets Long-term Deflection Limits;

Case 4 (Long-term)

Calculate the long-term deflection under G :

$$\Delta = \frac{5(W^*)L^4}{384(EI)_{ef}} = \frac{5(G)L^4}{384(EI)_{ef}} = 12.0 \text{ mm}$$

$$L/\Delta = 666 > 300 \text{ Therefore OK}$$

Section Meets Long-term Dead Load Only Deflection Limits;

If the initial deflection due to shrinkage of the concrete is not offset during construction, it should be included in the cases 1, 3 and 4. It can be calculated using:

$$\Delta_{mi} = \frac{\epsilon_{cs(28)} L^2}{8(h_c + a_f + h_t)}$$

A3.5.3 Bearer Design

The calculation below is for the internal bearer in the floor system. Although the edge bearer supports half the floor area of a typical bearer it may be required to carry facade loads. Given this, all the bearers have been assigned the same section in this design. This also simplifies fabrication. It would be expected that the design would be repeated for both elements.

Note that the testing done by UTS does not include members as thick as required for this bearer design and extrapolation of the available data does not give sufficient shear connector capacity to achieve composite action. At this stage the bearer has therefore been designed as non-composite, but with further testing or the use of a different connector system to achieve composite action the depth of the bearer could be reduced to approximately 600 mm.

The side pieces have been included to provide a bearing surface for joist connections.

Bearer ultimate strength checks

Geometric Input

Joist span;	$L = 6 \text{ m}$
Spacing;	$S = 8 \text{ m}$
Beam depth;	$h_t = 700 \text{ mm}$
Beam width;	$b_t = 180 \text{ mm}$
Side piece depth;	$h_s = 425 \text{ mm}$
Side piece width;	$b_s = 90 \text{ mm}$
Plywood thickness;	$a_f = 15 \text{ mm}$
Concrete thickness;	$h_c = 80 \text{ mm}$
Floor joists self-weight;	$J_w = 90 \text{ mm} \times 400 \text{ mm} \times g \times \rho / b_c = 0.37 \text{ kPa}$
Concrete self-weight;	$C_w = h_c \times \rho_c \times g = 1.96 \text{ kPa}$
Bearer self-weight;	$B_w = (b_t \times h_t + 2 \times b_s \times h_s) \times g \times \rho = 1.23 \text{ kN/m}$

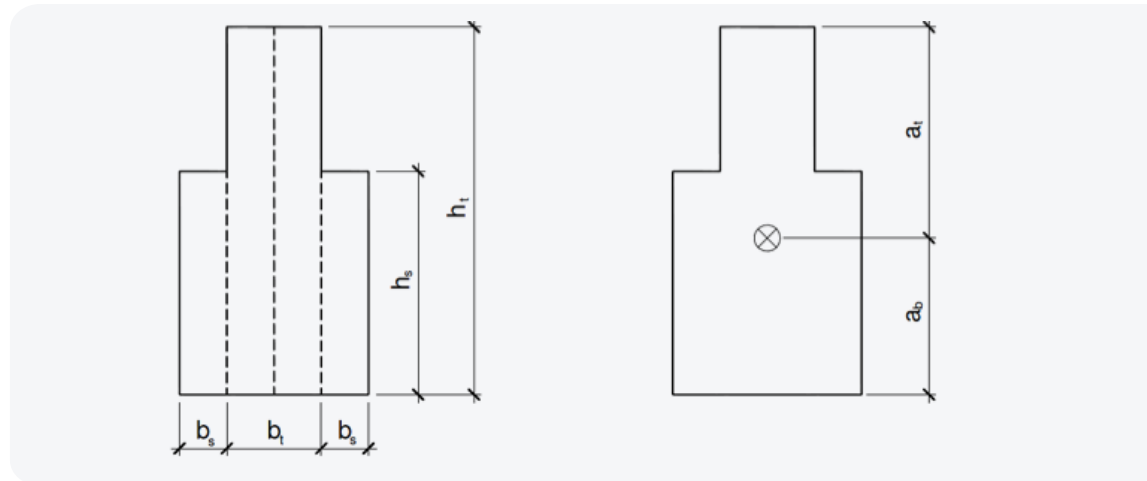


Figure A3.3: Bearer configuration.

Required Capacity

$\psi_a = 0.73$; (Area reduction factor, refer to AS 1170.1 clause 3.4.2)

$$w = 1.2 \times (B_w/S + C_w + F_w + J_w + SDL) + 1.5 \times LL \times \psi_a = 8.7 \text{ kPa}$$

$$M = w^* \times S \times L^2/8 = 312.6 \text{ kNm}$$

$$V = w^* \times S \times L/2 = 208.4 \text{ kN}$$

Section Properties

$$A_t = h_t \times b_t + 2 \times h_s \times b_s = 202500 \text{ mm}^2$$

$$a_b = (b_t \times h_t \times h_t/2 + 2 \times b_s \times h_s \times h_s/2) / A_t = 298 \text{ mm}$$

$$a_t = h_t - a_b = 402 \text{ mm}$$

$$I_t = \sum I_i + \sum ((a_i - a_b)^2 \times A_i) = 7.2 \times 10^9 \text{ mm}^4$$

$$Z_b = I_t / a_b = 24.14 \times 10^6 \text{ mm}^3$$

$$Z_t = I_t / a_t = 17.90 \times 10^6 \text{ mm}^3$$

$$(EI)_{ef} = E_t I_t = 79.16 \times 10^{12} \text{ Nmm}^2$$

Bending Strength Limited by Timber capacity

$$\phi M_u = \phi k_1 k_4 k_6 k_9 k_{11} k_{12} f'_b Z$$

$$\phi = 0.85$$

$$k_1 = 0.8; \text{ (live load i.e. 5 month load duration)}$$

$$k_4 = 1.0; \text{ (equilibrium moisture content less than 15\%)}$$

$$k_6 = 1.0; \text{ (normal temperature range)}$$

$$k_9 = 1;$$

$$k_{11} = (300 \text{ mm}/h_t)^{0.167} = 0.868$$

$$k_{12} = 1;$$

$$M_{d,b} = \phi \times k_1 \times k_4 \times k_6 \times k_9 \times k_{11} \times k_{12} \times f'_b \times Z_b = 724 \text{ kNm}$$

$$M_{d,t} = \phi \times k_1 \times k_4 \times k_6 \times k_9 \times k_{11} \times k_{12} \times f'_b \times Z_t = 537 \text{ kNm}$$

$$M_d \geq M^*$$

Capacity greater than M^* , Section OK;

Flexural Shear Strength

Check the flexural shear strength at the half height of the bearer (Section 1) and at the reduced width (Section 2).

$$\text{Beam depth; } h_t = 700 \text{ mm}$$

$$\text{Beam width; } b_t = 180 \text{ mm}$$

$$\text{Side piece depth; } h_s = 425 \text{ mm}$$

$$\text{Side piece width; } b_s = 90 \text{ mm}$$

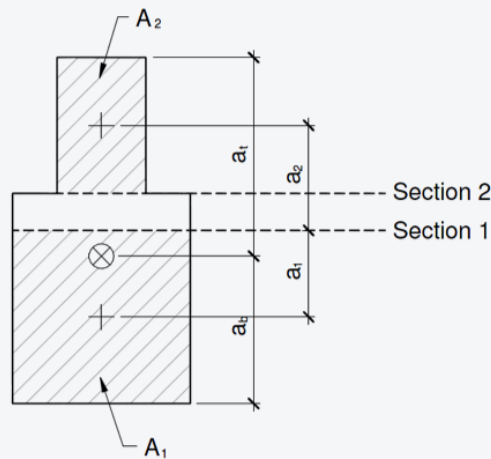


Figure A3.4: Bearer section.

$$A_1 = h_t/2 \times (b_t + 2 \times b_s) = 126000 \text{ mm}^2$$

$$a_1 = a_b - h_t/4 = 123.06 \text{ mm}$$

$$S_1 = A_1 \times a_1 = 15.50 \times 10^6 \text{ mm}^3$$

$$A_2 = (h_t - h_s) \times b_t = 49500 \text{ mm}^2$$

$$a_2 = a_t - (h_t - h_s)/2 = 264.44 \text{ mm}$$

$$S_2 = A_2 \times a_2 = 13.09 \times 10^6 \text{ mm}^3$$

Calculate the shear capacity

$$V_{d,1} = \frac{\phi k_1 k_4 k_6 f'_s I_t b_1}{S_1} = 721.8 \text{ kN}$$

$$V_{d,2} = \frac{\phi k_1 k_4 k_6 f'_s I_t b_2}{S_2} = 427.5 \text{ kN}$$

$$V_d \geq V^*$$

Capacity greater than V^* , Section OK;

Serviceability checks

Case	Load Applied	Criteria
1	Short-term (0.7Q)	Span /300
2	1 kN point Load	2 mm
3	Long-term (G+0.4Q)	Span/250
4	Long-term Only (G)	Span/300

Table A3.2: Deflection Criteria for different load cases.

$$G = (B_w/S + C_w + F_w + J_w + \text{SDL}) \times S = 28.6 \text{ kN/m}$$

$$Q = LL \times S = 32.0 \text{ kN/m}$$

Case 1: Short-term (0.7Q)

Calculate the short-term deflection:

$$(EI)_{ef} = E_t I_t = 79.16 \times 10^{12} \text{ Nmm}^2$$

$$\Delta = \frac{5(W^*)L^4}{384(EI)_{ef}} = \frac{5(0.7Q)L^4}{384(EI)_{ef}} = 4.8 \text{ mm}$$

$$L/\Delta = 1257 > 300$$

Section Meets Short-term Deflection Limits

Case 2: 1 kN point Load

$$\Delta = \frac{PL^3}{48(EI)_{ef}}$$

$$P = 1 \text{ kN}$$

$$\Delta = 0.1 \text{ mm} < 2 \text{ mm} \text{ Therefore OK}$$

Case 3: Long-term (G+0.4Q)

Calculate the long-term deflection under G + 0.4Q.

$$E_{t,lt} = \frac{E_t}{j_2}$$

$$j_2 = 2 \quad ; \quad (\text{From Table 2.1 of AS 1720.1 [2]})$$

$$E_{t,lt} = E_t/j_2 = 5500 \text{ MPa}$$

$$(EI)_{ef} = E_{t,lt} I_t = 39.58 \times 10^{12} \text{ Nmm}^2$$

$$\Delta = \frac{5(W^*)L^4}{384(EI)_{ef}} = \frac{5(G+0.4Q)L^4}{384(EI)_{ef}} = 17.6 \text{ mm}$$

$$L/\Delta = 340 > 250$$

Section Meets Long-term Deflection Limits

Case 4: Long-term Only (G)

Calculate the long-term deflection under G.

$$\Delta = \frac{5(W^*)L^4}{384(EI)_{ef}} = \frac{5(G)L^4}{384(EI)_{ef}} = 12.2 \text{ mm}$$

$$L/\Delta = 492 > 300$$

Section Meets Long-term Dead Load Only Deflection Limits

A3.5.4 Vibration Checks

The vibration performance can be assessed by various methods.

1. Deflection under a 1 kN point load
2. 8Hz frequency limit
3. Direct calculation of vibration performance in accordance with CCIP-016, A Design Guide for Footfall Induced Vibration of Structures by The Concrete Centre

For this structure checks using all three methods have been undertaken.

Deflection checks

The results of Method 1 are presented earlier in the deflection results section. The structure meets this requirement.

Frequency check

The design guide proposes a first fundamental frequency check is undertaken using the formula below.

Joists:

$$f = \frac{9.87}{2\pi} \left(\frac{9800EI_{ef}}{L^4G} \right)^{0.5}$$

Where G is the self-weight

$$G = (B_w/b_c + C_w + F_w) \times S = 1.45 \text{ kN/m}$$

$$f = \frac{9.87}{2\pi} \left(\frac{9800EI_{ef}}{L^4G} \right)^{0.5} = 9.3 \text{ Hz}$$

Where all values are expressed in Nmm.

Bearer:

Where G is the self-weight

$$G = (B_w/S + C_w + J_w + F_w) \times S = 20.6 \text{ kN/m}$$

$$f = \frac{9.87}{2\pi} \left(\frac{9800EI_{ef}}{L^4G} \right)^{0.5} = 8.5 \text{ Hz}$$

Where all values are expressed in Nmm.

The frequency calculated by this method is only appropriate if it is beams in one direction that are very active. For floors spanning in two directions this may not be correct. More accurate results can be obtained by hand calculation using methods proposed for steel composite floors or by using an analysis program to model natural frequencies.

It should also be noted that for a frequency prediction of the floor in service, the floor will have additional in-service mass due to fit out and the effective stiffness will be for small amplitude displacements.

The first vertical mode of the structure predicted using Finite Element modelling and the assumptions below is 8.3 Hz.

Direct Calculation of Vibration Performance

The vibration performance of the structure has been calculated in accordance to best practice document CCIP-016 referenced above. This method calculates the vibration response of the floor as a multiplier (or response factor) on the floor due to a single person walking. Acceptability criteria recommended for offices is $R = 4-8$ depending on the quality of office space required.

The following assumptions were made in the assessment:

- Under the small amplitude vibrations of footfall loading the joists were considered fully composite with the concrete floor.
- The bearers were assumed to have no composite action.
- Under the small amplitude vibrations of footfall loading the facade was assumed to provide a vertical restraint around the edge of the floor.
- The full super imposed dead load and 10% live load was assumed to be present.
- The floor plate was assumed to be an open plan office therefore a maximum walking frequency of 2.0 footfalls/second was assumed.
- 3% critical damping was assumed for all modes.

The resulting response factor was calculated to be below 6 throughout the office. Therefore, the floor meets the recommended performance target for a typical office, but not premium office space.

Considering a central corridor section along the long axis of the building – where walking up to 2.5 footfalls/second may occur – resulted in response factors up to $R = 11$ but these higher responses were limited to the corridor zone. The office areas on either side achieve a response factor of $R=4$ for this scenario.

Conclusions and Comments

The calculations for a simple TCC floor have been presented.

The depth of the joists could be reduced slightly and still work structurally, but this would cause wastage in the production of the joists out of the usual 1200 mm billets and therefore was not considered worthwhile. Reducing the joist depth would also result in a decrease in the vibration performance.

The research done by UTS does not include members as thick as required for the bearer design and extrapolation of the available data does not give sufficient shear connector capacity to achieve composite action. At this stage, the bearer has therefore been designed as non-composite, but with further testing or the use of a different connector system to achieve composite action the depth of the bearer could be reduced to approximately 600 mm.

The floor meets the recommended vibration performance target for a typical office, but not premium office space.

A3.6 Worked Example (b): 8 m Floor Using Cross SFS Screws

The serviceability and ultimate slip moduli below have been determined from the tested data utilising one pair of cross SFS screws inclined at 45° connection:

$$K_s = 70 \text{ kN/mm}$$

$$K_u = 44 \text{ kN/mm}$$

Refer to Section 3 of the Design Guide for further details.

The detail of screw spacing is as below:

A3.6.1 Ultimate Limit State Checks

Type of Connection:	L_1 (mm)	S_{min} (mm)	S_{max} (mm)	S_{ef} (mm)
SFSVB - 48-7.5 x 165	300	280	N/A	314

Table A3.3: Details of cross SFS screws.

$$ULS: \gamma_c = \frac{1}{1 + \frac{\pi^2 E_c A_c s_{ef}}{K_i L^2}} \quad 0.42$$

$$\gamma_t = 1$$

$$h_c = 80 \text{ mm}$$

$$H = \frac{h_c}{2} + a_f + \frac{h_t}{2} = 192 \text{ mm}$$

$$a_c = \frac{\gamma_t E_t A_t H}{\gamma_c E_c A_c + \gamma_t E_t A_t} = 71.9 \text{ mm}$$

$$a_t = \frac{\gamma_c E_c A_c H}{\gamma_c E_c A_c + \gamma_t E_t A_t} = 120.3 \text{ mm}$$

$$(EI)_{ef} = E_c I_c + E_t I_t + \gamma_c E_c A_c a_c^2 + \gamma_t E_t A_t a_t^2 = 9.74 \times 10^{12} \text{ Nmm}^2$$

Bending strength limited by concrete capacity

$h_c = 75.9 \text{ mm}$ (Reduced effective concrete thickness due to concrete tension at Ultimate Limit State, ULS)

$$\phi = 0.6$$

$$\phi M_u = \phi f_c' \frac{2(EI)_{ef}}{\gamma_c E_c h_c} = 378.8 \text{ kNm}$$

Concrete axial capacity

$$\sigma_{c.c} = \frac{\gamma_c E_c a_c M^*}{(EI)_{ef}} = 7.78 \text{ MPa}$$

$$N_{c}^* = \sigma_{c.c} \times A_c = 354.1 \text{ kN}$$

$$\phi N_u = \phi \times f_c' \times A_c = 874.37 \text{ kN}$$

Combined

$$\frac{N_c^*}{\phi N_u} + \frac{M^*}{\phi M_u} = 0.61 < 1 \quad ; \text{ Therefore section is OK}$$

Bending strength limited by timber capacity

$$\phi = 0.9$$

$$k_1 = 0.8; \text{ (live load i.e. 5 month load duration)}$$

$$k_4 = 1.0; \text{ (equilibrium moisture content less than 15\%)}$$

$$k_6 = 1.0; \text{ (normal temperature range)}$$

$$k_9 = 1.0$$

$$k_{11} = (300 \text{ mm}/h_t)^{0.167} = 0.95$$

$$k_{12} = 1.0$$

$$\phi M_u = \phi k_1 k_4 k_6 k_9 k_{11} k_{12} f_b' \frac{2(EI)_{eff}}{\gamma_t E_t h_t} = 145.8 \text{ kNm}$$

Timber axial capacity

$$\sigma_{t.t} = \frac{\gamma_t E_t a_t M^*}{(EI)_{eff}} = 10.44 \text{ MPa}$$

$$N_{t}^* = \sigma_{t.t} \times A_t = 376.16 \text{ kN}$$

$$k_{11} = (150 \text{ mm}/h_t)^{0.167} = 0.85$$

$$\phi N_u = \phi \times k_1 \times k_4 \times k_6 \times k_{11} \times f_t' \times A_t = 660.11 \text{ kN}$$

Combined

$$\frac{N_t^*}{\phi N_u} + \frac{M^*}{\phi M_u} \approx 1 < 1 \quad ; \text{ Therefore section is OK}$$

Flexural shear strength

At the floor joist to bearer detail, the flexural shear strength must be checked for the net area, as shown below.

$$A_t = (h_t) \times b_t = 36000 \text{ mm}^2$$

$$\phi V = \phi k_1 k_4 k_6 f'_s \frac{2A_t}{3} = 103.7 \text{ kN}$$

$$\phi V \geq V^* \quad 103.7 \text{ kN} > 39.2 \text{ kN} \quad \text{Therefore section is OK}$$

Shear connector strength

$$Q_k = 32 \text{ kN}$$

Connector capacity

$$\phi = 0.8$$

$$k_1 = 0.8$$

$$\phi N_j = \phi k_1 k_4 k_6 Q_k = 54.4 \text{ kN}$$

At the support location:

$$V^* = L/2 \times w^* = 3.7 \times 4 = 14.8 \text{ kN}$$

$$Q^* = \frac{\gamma_c E_c A_c a_c s_{end}}{(EI)_{ef}} V^* = 20.05 \text{ kN}$$

$$\phi N_j \geq Q^* \quad 54.4 \text{ kN} > 20.05 \text{ kN} \quad \text{Therefore connectors are OK}$$

Tangential shear strength of the timber

Tangential Shear action in the area located between the support and the first connection

$$\phi = 0.9$$

Generally:

$$\phi N_v = \phi k_1 k_4 k_6 f'_s (b_t l_s)$$

At the support location:

$$\phi N_v = \phi \times k_1 \times k_4 \times k_6 \times f'_s \times (b_t \times e) = 124.42 \text{ kN}$$

$$\phi N_v \geq Q^* \quad \text{Therefore connectors are OK}$$

A3.6.2 Serviceability checks

This section outlines the serviceability checks that have been done on the floor joist. Table A3.4 lists the cases considered in the calculations.

Case	Load Applied	Criteria
1	Short-term (0.7Q)	Span /300
2	1 kN point Load	2 mm
3	Long-term (G+0.4Q)	Span/250
4	Long-term Only (G)	Span/300

Table A3.4: Deflection Criteria for different load cases.

$$G = (B_w/b_c + C_w + F_w + \text{SDL}) \times S = 2.05 \text{ kN/m}$$

$$Q = LL \times S = 2.4 \text{ kN/m}$$

$$h_c = 80 \text{ mm}; \quad (\text{Assume all the concrete is in compression, therefore use the full thickness})$$

$$A_c = h_c \times b_c = 48000 \text{ mm}^2$$

$$A_t = h_t \times b_t = 36000 \text{ mm}^2$$

Case 1 (Short-term)

Calculate the serviceability stiffness with short-term load parameters.

From Figure 3.3 of the Design Guide:

$$K_i = 70 \text{ kN/mm}$$

$$\gamma_c = \frac{1}{1 + \frac{\pi^2 E_c A_c s_{ef}}{K_i L^2}} = 0.54$$

$$\gamma_t = 1$$

$$H = \frac{h_c}{2} + a_f + \frac{h_t}{2} = 189.95 \text{ mm}$$

$$a_c = \frac{\gamma_t E_t A_t H}{\gamma_c E_c A_c + \gamma_t E_t A_t} = 61.5 \text{ mm}$$

$$a_t = \frac{\gamma_c E_c A_c H}{\gamma_c E_c A_c + \gamma_t E_t A_t} = 128.5 \text{ mm}$$

$$I_c = \frac{b_c h_c^3}{12} = 25.60 \times 10^6 \text{ mm}^4$$

$$I_t = \frac{b_t h_t^3}{12} = 480.0 \times 10^6 \text{ mm}^4$$

$$(EI)_{eff} = E_c I_c + E_t I_t + \gamma_c E_c A_c a_c^2 + \gamma_t E_t A_t a_t^2 = 10.4 \times 10^{12} \text{ Nmm}^2$$

Calculate the short-term deflection:

$$\Delta = \frac{5(W^*)L^4}{384(EI)_{ef}} = \frac{5(0.7Q)L^4}{384(EI)_{ef}} = 8.6 \text{ mm}$$

$$L/\Delta = 930.3 > 300$$

Section Meets Short-term Deflection Limits

Case 2 (Point load)

$$\Delta = \frac{PL^3}{48(EI)_{ef}}$$

$$P = 1 \text{ kN}$$

$$\Delta = 1.03 \text{ mm} ; < 2 \text{ mm Therefore OK}$$

Case 3 (Long-term)

Calculate the serviceability stiffness with long-term load parameters.

$$\varepsilon_{cs} = 880 \times 10^{-6}$$

$$\Phi_{cc} = 3.62 \quad ; \text{ (From Table 3.1.8.3 of AS 3600)}$$

$$E_{c.lts} = \frac{E_c}{(1 + \varepsilon_{cs})(1 + \Phi_{cc})} = 6704 \text{ MPa}$$

$j_2 = 2$; (Adopt j_2 of 2 as currently developed by the UTS research work)

$$E_{t.lts} = \frac{E_t}{j_2} = 5500 \text{ MPa}$$

$$K_{i.lts} = \frac{K_i}{j_2} = 35 \text{ kN/mm}$$

$$\gamma_{c.lts} = \frac{1}{1 + \frac{\pi^2 E_{c.lts} A_c s_{eff}}{K_{i.lts} L^2}}$$

$$\gamma_t = 1$$

$$H = \frac{h_c}{2} + a_f + \frac{h_t}{2}$$

$$a_c = \frac{\gamma_t E_{t.lts} A_t H}{\gamma_c E_{c.lts} A_c + \gamma_t E_{t.lts} A_t}$$

$$a_t = \frac{\gamma_c E_{c,lts} A_c H}{\gamma_c E_{c,lts} A_c + \gamma_t E_{t,lts} A_t}$$

$$(EI)_{ef} = E_{c,lts} I_c + E_{t,lts} I_t + \gamma_c E_{c,lts} A_c a_c^2 + \gamma_t E_{t,lts} A_t a_t^2$$

the serviceability stiffness with short-term load parameters.

Calculate the long-term deflection under $G + 0.4Q$:

$$\Delta = \frac{5(W^*)L^4}{384(EI)_{ef}} = \frac{5(G+0.4Q)L^4}{384(EI)_{ef}} = 35.9 \text{ mm}$$

$$L/\Delta = 250 \approx 250 \text{ Therefore OK}$$

Section Meets Long-term Deflection Limits;

Case 4 (Long-term)

Calculate the long-term deflection under G :

$$\Delta = \frac{5(W^*)L^4}{384(EI)_{ef}} = \frac{5(G)L^4}{384(EI)_{ef}} = 24.4 \text{ mm}$$

$$L/\Delta = 330.6 > 300 \text{ Therefore OK}$$

Section Meets Long-term Dead Load Only Deflection Limits;

If the initial deflection due to shrinkage of the concrete is not offset during construction, it should be included in the cases 1, 3 and 4. It can be calculated using:

$$\Delta_{ini} = \frac{\epsilon_{cs(28)} L^2}{8(h_c + a_f + h_t)}$$

B

Appendix B – Notation

The symbols and letters used in the Guide are listed below:

A_c	cross-sectional area of the concrete member
A_t	cross-sectional area of the timber member
A_l	bearing area for loading parallel to the grain (timber)
A_p	bearing area for loading perpendicular to the grain (timber)
A_{sl}	shear plane area for shear action parallel to the grain (timber)
A_{st}	cross-sectional area of the coach screw (TCC only)
a	distance between points of zero bending moment
a_c	distance for the concrete member
a_f	thickness of the formwork
a_t	distance for the timber member
b_c	tributary width of the concrete member
b_{e1} & b_{e2}	measured effective on each side of the centre-line of the timber beam
b_{sf1}	effective width of composite beam top flange (0 for timber beam)
b_1, b_2	centre-to-centre spacing of adjacent beams or distance from centre of timber beam to edge of slab outstand
b_t	width (thickness) of the timber member
b_v	width of the notch (concrete)
d	timber density at a moisture content of 12% in kg/m ³
d_o	length of the notch (concrete)
D_c	overall depth of the concrete slab
E_c	value of the modulus of elasticity of the concrete member
$E_{c,lts}$	value of the modulus of elasticity of the concrete member for long-term serviceability
E_t	value of the modulus of elasticity of the timber member
$E_{t,lts}$	value of the modulus of elasticity of the timber member for long-term serviceability
El_{ef}	effective (apparent) stiffness of the TCC cross-section
f_n	axial (tensile or compressive) strength
f_b	bending strength
f'_b	characteristic strength in bending
f'_c	characteristic strength in compression
f'_l	characteristic strength in bearing parallel to the grain
f'_p	characteristic strength in bearing perpendicular to the grain
f'_s	characteristic strength in shear
f'_t	characteristic strength in tension
G^*	design self-weight
H	factor for the height of the TCC cross-section
h_c	thickness of the concrete member
h_t	depth (height) of the timber member
I_c	second moment of area (moment of inertia) of the concrete member

I_t	second moment of area (moment of inertia) of the timber member
j_2	stiffness modification factor – load duration
K_{eff}	connection (shear key) stiffness for design of the Service Limit State – long-term deflection
K_i	connection (shear key) stiffness
K_{ser}	connection (shear key) stiffness for design of the Service Limit State – short-term deflection
K_u	connection (shear key) stiffness for design of the Ultimate Limit State
k_1	duration of load (timber)
k_{c1}	shrinkage strain coefficient (concrete)
k_{c2}	creep factor coefficient (concrete)
k_{c3}	maturity coefficient (concrete)
k_4	moisture condition (timber)
k_6	temperature (timber)
k_7	length and position of bearing (timber)
k_9	strength sharing between parallel members (timber)
k_{11}	size factor (timber)
k_{12}	stability factor (timber)
L	span of the structure
L_{ef}	effective span of the beam calculated in accordance with Clause 5.3.3, AS 2327
l_s	length of the horizontal shear plane (timber)
M^*	design action effect in bending
ϕM_u	design capacity in bending (concrete)
(ϕM)	design capacity in bending (timber)
N^*	design action effect produced by axial force
N_p^*	design action effect in bearing produced by reaction at a support
ϕN_u	design capacity in axial stress (concrete)
(ϕN)	design capacity in axial stress (timber)
$(\phi N^{\parallel})$	design capacity of the connection in shear
$(\phi N_{\parallel})$	design capacity in bearing parallel to the grain (timber)
$(\phi N_{\perp})$	design capacity in bearing perpendicular to the grain (timber)
(ϕN_v)	design capacity in shear parallel to the grain (timber)
(ϕN_{θ})	design capacity in bearing at an angle to the grain (timber)
P^*	design action for point load action (Service Limit State)
$Q_{V_{L/4}}^*$	design action effect in shear in the connection
$Q_{V_{max}}^*$	design action effect in shear in the connection (at $L / 4$)
Q^*	design action effect in shear in the connection (at a support)
Q_k	design action for shear in the connection
	characteristic strength of the connection in shear
R_m	mean characteristic strength of the connection in shear (test data)
s_{ef}	factor for the connection spacing
s_{max}	distance of the first connector from mid-span
s_{min}	distance between the connectors (inside the external quarter-spans)
t	period of time, in minutes 90 mins
V^*	design action effect in flexural shear (also tangential shear)

$V_{L/4}^*$	design action effect in flexural shear (also tangential shear) at $L / 4$
V_{max}^*	design action effect in flexural shear (also tangential shear) at a support
(ϕV)	design capacity in flexural shear (timber)
ϕV_{uc}	design capacity in shear (concrete)
w_{imp}^*	imposed design load(s)
$\beta_{1,2,3}$	coefficients (concrete)
Δ	deflection at mid-span
γ_c	partial factor for material properties of the concrete member
$\gamma_{c,lts}$	partial factor for material properties of the concrete member – long-term serviceability
γ_t	partial factor for material properties of the timber member
$\gamma_{t,lts}$	partial factor for material properties of the timber member – long-term serviceability
ϵ_{cs}	design shrinkage strain (concrete)
$\epsilon_{cs,b}$	basic shrinkage strain (concrete)
$v_{0.4}$	mean slip of the connection measured at $0.4 R_m$ (test data)
$v_{0.6}$	mean slip of the connection measured at $0.6 R_m$ (test data)
ϕ	capacity factor
ϕ_{cc}	design creep factor (concrete)
$\phi_{cc,b}$	basic creep factor (concrete)
φ	creep coefficient (timber)
θ	angle of the notch facet under compression,
σ_b	effective bending stress
σ_c	effective compression stress
σ_t	effective tension stress
σ_n	effective axial stress
$\sigma_{t,N}$	axial stress of timber
$\sigma_{t,M}$	bending stress of timber
$\sigma_{b,t}$	tensile stress of timber
$\sigma_{b,c}$	bending stress of concrete

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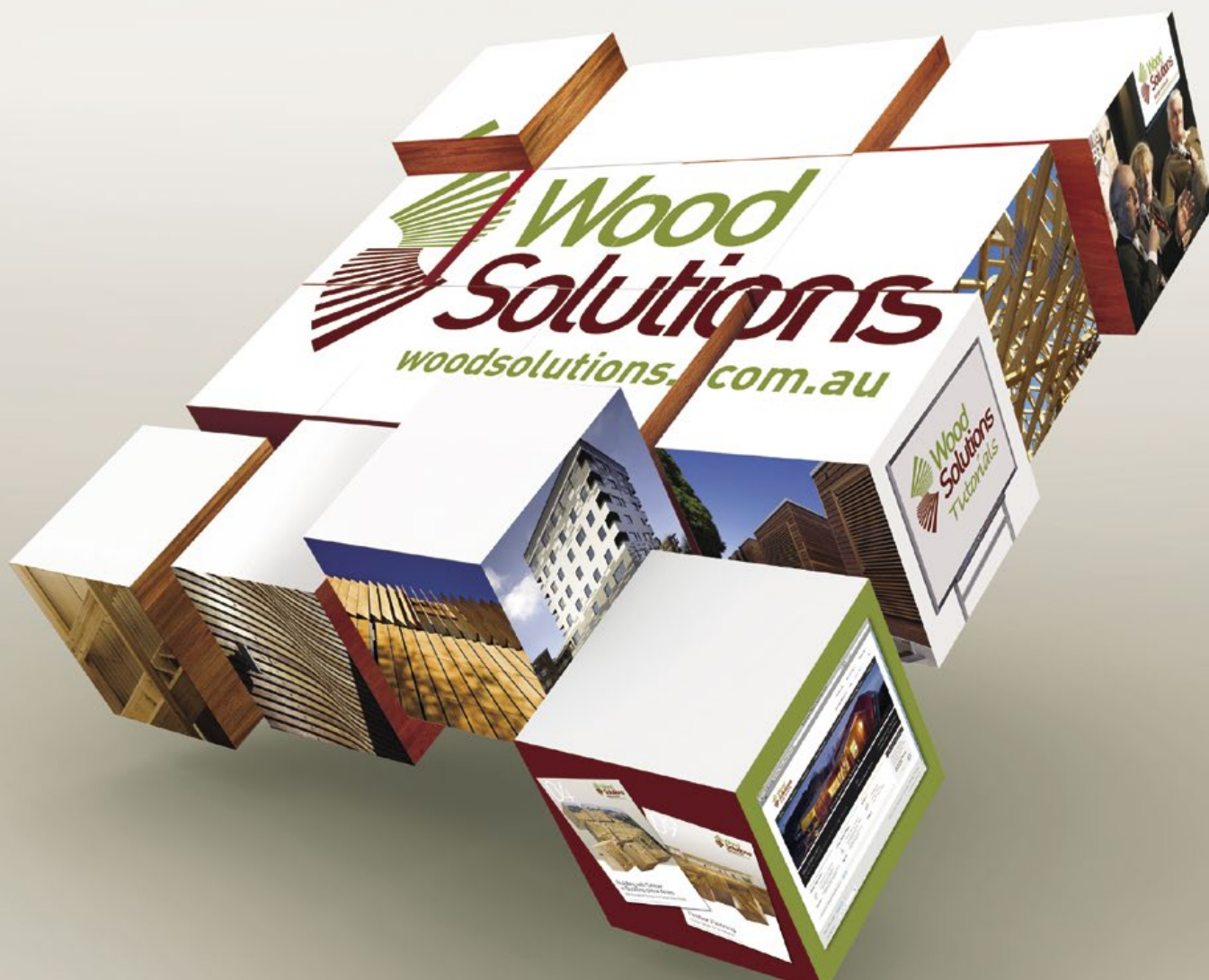
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